EXPERIMENTAL AND THEORETICAL INVESTIGATIONS OF THE MECHANICAL STRENGTH OF CLINCHING

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ABSTRACT

'Mechanical Clinching' or 'Press Joining' is a novel technique for applying a structural connection between two or more sheets of material. Commercial cold-formed steel framing systems have been developed using the clinch as the primary method of structural connection. Design guidance for the use of mechanical clinching in structural applications is currently limited to recommendations from the work of past research programmes.

In this research the database on clinching shear resistance is extended by shear testing of mechanical clinches using an Instron tensile testing machine. S-type and H-type clinches are tested in layers of two and three with variable steel thicknesses and at different angles of applied load. The cyclic shear resistance of single clinches is investigated by applying variable loading over 10,000 cycles. A clinch design factor of safety is proposed based on the static and cyclic clinch test results and analysis. Pop rivets, self-piercing rivets and self-tapping screws are also tested in shear and comparison is made with clinch shear resistance characteristics.

Rotational shear resistance of groups of clinches is investigated in experimental and finite element tests by applying in-plane moment to groups of clinches in a range of steel thicknesses and at different group spacings. Moment resistance of clinch groups is also investigated in cantilever and H-frame cross-beam full-scale tests where groups of 4, 6 and 8 clinches are applied to connect cold-formed steel components.

Full-scale tests are carried out on 10 truss beams up to 6m in length, joined by clinching struts and ties to folded parallel chords. Strain readings allow forces in the clinches over the course of each test to be recorded. Simplified clinch shear deformation characteristics are applied in finite element truss tests modelling the experimental truss tests. In a theoretical finite element model for each test a truss with no shear deformation at the connection nodes is analysed. Comparison is made with a finite element model allowing shear deformation at the connections to isolate the effect of clinch shear deformation on truss elastic stiffness and peak load.
DECLARATION

The work presented in this thesis was carried out by the Author at the departments of Architecture and Civil Engineering at the University of Edinburgh for the degree of Doctor of Philosophy. Where the work of another researcher or organisation is listed the authorship is stated and referenced.

Rory Lennon, April 2002.

[Signature]

Rory Lennon, April 2002.
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Special thanks to my Parents who made it possible for me to begin studying at the University of Edinburgh in 1993.
PUBLISHED PAPERS


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NOTATION

a  Constant in Wieck’s equation 4.8
A  Cross section area of compound section
A_g Gross cross-section area
A_g Gross area of cross section
A_n Net area of cross section
b  width of element
B  Width of flanges of compound section
b_1 Flange width
b_2 Web depth
b_eff Effective width of element under consideration
C  Constant in Wieck’s equation 4.8
c  Constant in Wieck’s equation 4.8
C_B Variable used to calculate M_B in BS5950-5 Section 5.6.2.1
C_T Variable used to calculate M_E in BS5950-5 Section 5.6.2.2
D  Depth of compound section
e  Eccentricity
E  Modulus of elasticity
F_0 Force in clinch at 0°
F_{90} Force in clinch at 90°
F_c Applied axial load
F_d Characteristic clinch design resistance
f_c Compressive stress on the effective element
F_{MAX} Maximum steel tearing resistance
F, F_P, F_m, P Peak clinch shear force
F_S Chord transverse shear
f_u Ultimate steel resistance
f_u Ultimate steel resistance
F_y, R_P Yield stress
f_y 0.2% proof stress
G  Shear modulus
I  Moment of Inertia
K  Strain gauge factor
K_1  Buckling coefficient of an element
L  Length between lateral restraints defining chord length
L_E  Effective length
M  Bending moment
M_b  Lateral buckling resistance moment about the x-axis
M_B  Buckling resistance moment
M_I  Moment in clinch group
M_cx  Section plastic moment capacity
M_cy  Y-axis bending moment capacity
M_E  Elastic lateral buckling resistance moment
M_P  Plastic moment capacity
M_S  Moment capacity of steel section
M_x  Applied bending moment about the x-axis
M_y  Applied bending moment about the y-axis
M_Y  Section Yield moment
N_t  Gross yielding failure
P_c  Buckling resistance under axial load
P_c  Compressive resistance
P_e  Buckling resistance under axial load
P_cr  Buckling resistance under axial load
P_crr  Buckling stress resistance under axial load
P cs  Short strut axial capacity
r_cy  Y-Y radius of gyration of one channel
r_1  X-X radius of gyration of compound section
r_y  Y-Y radius of gyration of compound section
s  Spacing of connections forming compound section
Stv  R_y/R_m
T, t, a_0'  Steel thickness
UTS, $R_m$  Steel ultimate tensile strength
$V_f$  End pull out
$w$  Width of mechanical clinch
$y$  Distance from neutral axis to extreme fibre
$\gamma_r$  Overall load factor
$\gamma_m$  Material strength factor
$\varepsilon$  Strain
$\sigma$  Bending stress
$\nu$  Poisson’s ratio
$\beta$  Ratio of smaller end moment to larger end moment
$\eta$  Perry coefficient
$\theta$  Angle of applied loading to short edge of mechanical clinch
$\phi_B$  Variable used to calculate $M_B$ in BS5950-5 Section 5.6.2.1
$\Delta e$  Differential voltage reading
1 INTRODUCTION

An investigation of the behaviour of clinch connections in load-bearing applications is presented in this thesis. Clinching is currently used extensively in process and manufacturing engineering systems in light load bearing applications such as the connection of auto panels to auto frames and the fabrication of air-ducts. Clinching in load-bearing building components however is used only in a small number of building systems. Clinching in many cases is a better cold-formed steel connection method over existing methods such as self-piercing riveting and self-tapping screws. For techniques such as mechanical clinching much of the need arises from the requirement to industrialise connections as much as possible by factory manufacture, off-site pre-fabrication and modular construction. At present cold-formed steel design codes do not include guidance on the use of mechanical clinching and because of this mechanical clinching is often overlooked in choosing methods of connecting cold-formed steel as there are many advantages in using the method.

Cold-formed steel building components are often used in favour of timber members in low rise building applications where cold-formed steel building systems are available. The use of cold-formed steel building frame systems is widespread in the USA where patented systems such as Tri-chord compete against timber alternatives.

Cold-formed steel building systems can provide advantages in:

- Reduced building cost
- Reduced energy usage
- Implementation of systematic fabrication and building procedures
- Reduced self weight
- Increased connection ductility
- Building response to cyclic earthquake loading with greater deformation capacity
- Increased building robustness
- Improved fire resistance
- Greater structural integrity
As the trend to use cold-formed steel continues, more advanced cold-formed steel joining techniques such as mechanical clinching are being used in cold-forming fabrication.

1.1 Connecting cold-formed steel

To give a direct comparison of clinch shear resistance against the shear resistance of other types of connection a range of commonly used cold-formed steel connectors are tested and analysed in Chapter 4 – self-piercing rivets, pop rivets and self-tapping screws. All types of connection have a connecting component with the exception of the mechanical clinches where the parent metal is interlocked to form the connection. Shear tests are carried out to establish shear resistance, shear deformation and shear failure mode characteristics of the connections for comparison with clinch shear resistance behaviour.

1.1.1 Pop rivets

Steel pop rivet connections were formed by applying a hand-held lazy tongues apparatus with the rivet inserted at the tip, to a pre-drilled hole in the plates. The rivet has a shaft diameter of 5mm and a 5mm diameter automatic drill is used to prepare a hole in the plates. An advantage of using pop rivets over other types of mechanical fastener is that a rivet can be applied to layers of steel where the rivet operator has access to only one face of the steel surface. A hole must be drilled in the parent metal before a rivet is applied.

1.1.2 Self-piercing rivets

Forming a self-piercing rivet connection involves driving a rivet component into the layers of the parent metal, piercing and clinching in a single operation. A hand held electrically powered tool is used to apply the connection and access to both sides of the surface being joined is required. Unlike the clinch the self-piercing rivet is circular on plan and does not have a particular orientation on plan. It has a separate rivet component driven into the layers of the parent metal to complete the join, as illustrated in Figure 1.1.

Figure 1.1, Cross-section of a self-piercing rivet
1.1.3 Self-tapping screws

Self-tapping screws are commonly used to sew steel roof sheeting to purlins and to join cold-formed steel structural components. Battery powered drills allow screws to be applied by handheld tools. In the shear tests screw samples were clamped and joined using an automatic drill with a single self-tapping and drilling screw.

1.1.4 Mechanical clinching

Mechanical clinches can be applied to a number of layers of steel with a maximum combined thickness of between 4mm and 6mm, depending on the mechanical clinching machinery being used. The type of material that can be joined with this method can be metallic or non-metallic or a combination of both. For example, a thermal break can be incorporated into the interface between two sheets of steel being joined, by including a third layer of insulating plastic sheet between two outer steel layers.

Typical applications for mechanical clinching include connections in white goods, fabrication of ducting for air conditioning and light load bearing applications in auto process manufacturing such as clinching an auto side panel to an auto frame. In this work the structural behaviour of mechanical clinches in load-bearing components were investigated to give a better understanding of the structural performance of components joined by mechanical clinches, such as a lattice roof truss or wall stud in a building.

The clinching process

A two-phase mechanical process is used to apply the clinch. A punch and die component are positioned on either side of the sheets to be joined. In the first phase a strip of the steel layers is punched through by the punch shearing against two raised parallel walls of the die part as shown in Figure 1.2. In the second phase the head of the punch contacts the lower base part of the die, between the raised edges, and the steel layers are forced to expand laterally, locking the connection.
Clinching tools

For rectangular mechanical clinches a different die tool part must be used for different combinations of steel thickness being clinched. The punch part generally remains unchanged. The clinching tool manufacturer supplies guidelines for choosing the correct die part, an information sheet supplied by Eckold Ltd. for use with their machinery is shown in Figure 1.3.

In Figure 1.3 the steel thickness on the punch side and the die side is matched on the sides of the die serial number grid to show which punch type should be used for a given combination of steels. If the die that is chosen is too large for the particular combination of thicknesses it will splinter the hardened steel tip of the punch and the base of the die part. If the punch tool is too small, the clinch will not deform the steel sufficiently in the second phase of clinching and a weak connection will result.

Hydraulic pressure setting was also adjusted, with the pressure required becoming larger for greater combined thicknesses of material. Several tests were carried out with varying pressures on different steel thicknesses to obtain the optimum joint shear resistance for each sample. Figure 1.4 shows a series of six shear tests on the same thickness arrangement – two layers of 1.5mm thick steel on the outside of a central 1.0mm single plate as shown in Figure 4.7 (a), connected by a single clinch.
Pressure used to apply the clinches was measured in bar pressure units. Six shear tests in Figure 1.4 can be grouped into three pairs of two identical tests, one pair clinched with a pressure setting of 400 Bar, the middle pair with a pressure setting of 450 Bar, and the last pair with a pressure setting of 500 Bar. It was clear from Figure 1.4 that the greatest initial stiffness and the highest peak load for this arrangement of thicknesses was achieved by setting the optimum pressure of 450 Bar. The 400 Bar pressure setting produced a lower peak load in the shear test and the 500 Bar pressure produced lower initial shear resistance in the sample.
Figure 1.4, Pressure sensitivity shear tests

The lateral spread of the deformed steel on the die side of the sample as shown in Figures 1.8 and 1.9 also gave an indication of the efficiency of the join. There is an optimum amount of spread that locks the connection. If the spread is less than the optimum amount the connection will be loose. If the spread is greater than the optimum amount the clinching force is too great and the punched part of the connection becomes thin, allowing failure to occur more easily by tearing and steel deformation. By measuring the width of the punched part of the connection and the force applied to form the clinch a high level of quality control can be achieved in the clinching process.

Applying clinches

The machinery needed to hold the punch and die parts and to react to the clinching force is typically a bench mounted machine shown in Figure 1.5, or a ceiling suspended tool that provides some tool manoeuvrability to the fabrication process, shown in Figure 1.6. Both types of clinching machine need to be connected to a hydraulic pressure pump to be operational. The pressure pump part is typically an electrically powered device on trolley wheels, which can be
connected to the clinching tool with hydraulic hoses, keeping it partly out of the way of the fabrication space.

![Bench mounted clinching tool](image1)

**Figure 1.5, Bench mounted clinching tool**

A hand held, mechanical tool, Figure 1.7, is also available using battery cell power to force the two stages of clinch production. The hand held tool is limited in the total thickness of steel it can be used to join, and is slower in applying mechanical clinches in comparison with hydraulic electrical units clinching in less than a second.

![Ceiling mounted clinching tool](image2)

**Figure 1.6, Ceiling mounted clinching tool**
Types of clinch

The rectangular clinch is commonly available in two similar types – the S-type clinch and the H-type clinch. Figure 1.8 shows the punch and die sides of an S-type clinch. The clinch type is defined by the punch and die sets used. The S-type clinch is sheared along the parallel rectangular edges, while the H-type clinch is only partially sheared in the fabrication process, producing an airtight connection, illustrated in Figure 1.9.
In addition to the rectangular type of clinch investigated here, a circular punch and die set is available for use with the same hydraulic tools. These parts produce an airtight axi-symmetric circular clinch. They are not used as commonly as the rectangular parts and the punch and die sets are more costly. Where the type of a clinch is not specified in this work it can be assumed that an S-type join is being investigated. The rosette clinch presented in [52, 53] is a larger circular connection that requires a pre-drilled hole to be prepared before being applied.

1.2 Advantages of mechanical clinching

The choice of the method of connection in the design of cold-formed steel building systems is influenced by many factors. The type of connecting systems already installed and used at the place of fabrication will often be given preference.

Clinching presents many advantages in cold-formed steel fabrication:

- There is no connecting component in the clinch – unlike rivets, screws and welding, the clinch is made from the parent metal with no external joining part.
- Cost savings can be made in fabrication projects requiring many thousands of cold-formed steel connections.
- Weight of cold-formed steel components is not increased by mechanical clinching.
- Clinching does not induce significant corrosion in galvanised coatings commonly used for corrosion resistance on mild sheet steel. Because there is no need to paint over mechanical clinches to re-seal the galvanised coating the problems of inhalation of fumes from galvanised steel in welding procedures are avoided. Where corrosion occurs at a clinch...
location experimental testing has shown that the shear resistance of the connection is increased by a small amount

- It is generally necessary to repaint welded areas of galvanised steel
- A stronger connection is provided by mechanical clinching in many cases
- Clinching is suitable for bench process manufacturing systems. A bench can be set up to fabricate multiple cold-formed steel components, carrying out cutting, folding, positioning and mechanical clinching operations, producing standardised cold-formed steel components.
- No pre-drilling of measured holes is necessary
- Tools can be assembled to apply more than one clinch in a single clinching operation
- The formation of a clinch under hydraulic pressure takes only a fraction of a second
- Circular and H-type mechanical clinches can be air and water tight
- Clinching can be carried out by semi-skilled operatives
- Energy efficiency is high in comparison with other joining methods
- Automatic quality controlling devices are available for use with mechanical clinching machines

1.3 Clinching in cold-formed steel building systems

Cold-formed steel components are used extensively in domestic and industrial building applications. Typical uses for cold-formed steel in small-span building designs are:

- Closed-box and c-sections for light load bearing lintels
- Wall stud stringers and beams
- Floor beam brackets
- Timber pitched roof truss gang-nail connectors

And in large industrial frame structures:

- Roof lattice truss beams
- Zed and channel sections for roof sheeting purlins
- Closed-box and c-sections for vertical columns and stringers
- Structural brackets supporting timber and cold-formed steel beams and columns
- Cladding panels and cladding components
- Support structures for curtain walling
Building systems have also been developed that use cold-formed steel for the entire structural frame and roof members. These systems often incorporate timber for floor and roof spans. In designing the frame system, the methods of off-site and on-site fabrication are integral to the design methodology of the building frame system. The most important aspects of cold-formed steel fabrication are the methods of cutting steel, folding steel and of joining folded components. The ‘Tri-chord’ cold-formed building system in Figure 1.10 uses clinched cold-formed steel folded triangular sections.

![Figure 1.10, Tri-chord shear panel](image)

A detail of Tri-chord truss struts and ties clinched onto stringers is shown in Figure 1.11. This arrangement of truss clinch pierces and connects four layers of steel. The sections are connected to form lattice truss beams, stud stringers, timber holding brackets etc. making up a structural frame system.
Tri-chord gave the following points as advantages of their framing system over standard steel structural framing systems:

- Clinching eliminates screws or welds
- Automated assembly saves on cost
- Easier attachment and connection
- Up to three times stronger triangular sections
- Compatible with timber framing
- Can be framed into a slotted steel track
- Matches or exceeds wood's thermal values

And advantages of Tri-chord over wooden framing systems include:

- Compatible connections and hardware
- Fire, rot, warp and insect proof
- Consistent quality
- Over 66% recycled content
- Electrical, plumbing and HVAC install easily in the open webs and stud cut-outs
1.4 Research objectives

Investigation of shear resistance characteristics of mechanical clinching in this thesis is an extension of established research work at the University of Edinburgh. Past research work gives a framework and reference for the continued investigation of mechanical clinching in load bearing building applications.

Objectives of the current clinching research are:

1. To understand established research data and methodology in the area of connection shear resistance and to provide more information to the current data on connection shear resistance, with an emphasis on the shear resistance of mechanical clinching in load-bearing applications

2. Investigate the structural behaviour of mechanical clinches with individual connections and groups of connections as the primary connecting elements of full scale cold-formed steel frames

3. Demonstrate the effect of non-linear clinch shear resistance characteristics on the structural behaviour of full-scale lattice trusses with numbers of clinches at connection nodes varied from under-strength to over-strength

4. Provide test and analysis data on the structural behaviour of clinches as single connecting elements, multiple connecting elements in groups, and as cold-formed steel frame connections

5. Investigate the moment-rotation behaviour of groups of clinches by testing groups of connections under loading conditions that apply moment to the connection group

6. Use test results from shear tests on individual connections from this research and from past research in cold-formed steel to produce a comprehensive data base for the shear resistance and deformation characteristics of connections

7. Compare the structural characteristics of clinches with other cold-formed steel joining methods

8. Use finite element analysis to model the behaviour of groups of connections, individual cold-formed steel components and components joined in a system to show how the individual connection and the group of connections affect the shear resistance and deformation characteristics of the cold-formed steel component and building system

9. Analyse experimental test results against current design codes
1.5 Research methodology

Testing connections in shear and connection groups under applied moment produces from each test. In the case of shear tests the data is applied load and corresponding shear displacement. For clinch group moment testing the data is applied moment and rotation. Shear test data were used in the analysis of moment rotation tests and in the analysis of the full-scale truss tests, giving the essential shear resistance characteristics of clinch connections.

1.5.1 Experimental testing

Shear resistance of connections was investigated by shear testing in an Instron testing machine. Two or more strips of steel are connected and pulled apart in shear causing the connection to fail. Load and differential movement of the steel strips were automatically recorded at frequent intervals giving a full data curve for analysis.

Consideration must be given to connections in components under cyclic loading where the connection can fail from fatigue while the peak design load is never reached. When a connection is considered individually in a controlled shear resistance test and then individually or in a group as a part of a component or system, the local elements that give global shear resistance to the system can more easily be separated and identified. Where cyclic tests are being carried out, load is applied in cycles of 10,000 between zero force and 50% of the peak force to cause failure of the connection in a static test.

In addition to static and cyclic connection testing, the Instron testing machine was used to apply force and measure displacement in moment-rotation tests. Full-scale trusses up to 6m in length were tested in a rigid testing frame. Components of the trusses were connected by one or more mechanical clinches at the connection nodes. Load was applied by a hand operated hydraulic loading ram and a load cell was used to measure applied load.

1.5.2 Finite element analysis

Finite element models of the moment rotation tests and the full-scale truss tests were created using the finite element pre-processing program HYPERMESH. Finite element solution was carried out using the finite element program ABAQUS. In post-processing and analysis of the finite element output data, the FORTRAN 77 programming compiler, Microsoft Excel and ABAQUS POST were used.
Clinch group moment-rotation experimental tests were carried out using two c-sections back to back, joined by the clinch group under investigation. In the finite element simulation of the clinch group moment-rotation tests, c-sections were modelled with eight noded shell elements. Elasto-plastic material properties were assigned to the elements to allow plastic buckling of the c-sections to occur at regions of stress concentration.

Shear deformation characteristics of the individual clinch connections were modelled in the finite element tests by using a connection element for each clinch with the stiffness properties established from the Instron shear tests. An orthotropic clinch stiffness model was applied in the finite element tests based on the orientation of the clinch in position on the c-sections. Force in the finite element tests at 0° to the short edge of the clinch was resisted by the stiffness characteristics from the clinch shear tests at 0° and force at 90° was resisted by the stiffness characteristics of the clinch at 90°. Finite element tests were non-linear - each test was carried out over several static increments, each increment applying more or less load to the system. This allowed full non-linear stiffness characteristics of the mechanical clinches to be applied in finite element tests.

In the finite element tests modelling the full-scale truss tests, top and bottom chord components of the trusses were modelled using beam elements. Beam elements were suitable for modelling components where the cross-section area and the moment of inertia of the sections are known. Internal diagonal tension and vertical compression components were modelled using rod axial displacement elements, where no transfer of bending moments from the ends is considered and no external forces act on the length of the elements. The effect of clinch shear resistance can be fully evaluated when there is no buckling of the truss cold-formed steel members and the trusses fail when a clinch connection node fails. The use of beam and rod elements that do not account for twisting and buckling in the finite element models was appropriate in modelling truss experimental tests where failure of the truss occurred by failure of the connections.

Special clinch connection elements were used for the truss finite element tests using the Instron shear test data to model the shear shear deformation at the connection nodes. A greatly simplified version of clinch stiffness using only four points from the clinch load displacement
curve was applied in the finite element tests, giving similar truss deflection readings to the experimental response. The simplified clinch load-displacement model was defined by:

- The point of elastic limit
- The point of peak load
- The point where plastic deformation capacity ended
- Maximum displacement at zero force.

Buckling of the chords occurred before failure of the connections in some cases in the experimental truss tests. A second set of truss finite element models were created with shell elements modelling the cold-formed steel surfaces. The limits of applied axial force and bending moments were established with a bending moment and axial force envelope for the two thicknesses of chord and for the central and end unrestrained truss chord lengths. Buckling checks to BS5950-5 [7] were also used to analyse the chord buckling strengths under combined axial force and bending moments.

1.6 Thesis overview

Following the introduction to the research in this section, Chapter 2 reviews research papers on the shear resistance of connections in structural applications. Points of interest are the shear resistance of clinch connections under static and cyclic loading and the behaviour of groups of connections under applied moment and axial force. Previous research carried out at the University of Edinburgh into mechanical clinching technology and applications is reviewed. Research papers of other research programmes presenting data from experimental tests on mechanical clinching and other methods of connection in cold-formed steel and timber are also reviewed and summarised.

In Chapter 3 properties of steel materials used in experimental tests are discussed. Material properties were obtained by testing steel samples in tension in the Instron testing machine. Steel material properties were used to define the material behaviour characteristics in the finite element tests where shell elements are applied. Chapter 3 also explains aspects of the Instron connection shear test set-up.

Results from shear tests on connections are presented and analysed in Chapter 4. Types of connections tested are mechanical clinches, self-tapping screws, self-piercing rivets and pop-
rivets. Shear resistance characteristics of mechanical clinches are presented and analysed with regard to:

- Types of mechanical clinch
- Response to cyclic loading
- Clinches joining two layers of steel
- Clinches joining three layers of steel
- Energy-displacement relationships
- Effect of the orientation of the clinch to the applied load
- The effect of varying the hydraulic pressure in the mechanical clinching tool
- Variability of clinch shear resistance
- Comparison with other types of mechanical connection

The existing data-base of clinch peak loads established at the University of Edinburgh is extended with current results. By normalising clinch peak loads against the highest theoretical clinch shear resistance and carrying out a regression analysis on the resulting data, an expression for clinch peak load was established. Variables in the expression were steel thickness, steel UTS and angle of applied loading to the short edge of the mechanical clinch.

The moment-rotation relationships of groups of four clinches were investigated in Chapter 5 with clinch spacing and steel thickness as variables. Finite element tests matching each experimental test were carried out and moment-rotation curves were presented and analysed for each experimental and finite element test. Forces in the individual clinch connections were analysed against applied moment. Comparison is made between the experimental test results and the finite element test results and the response of the finite element clinch stiffness model is discussed.

Experimental and finite element truss tests are presented in Chapter 6. Load, strain and displacement data from the truss experimental tests are analysed to investigate the effects of clinch shear resistance on truss deflection. 2D Finite element tests matching each experimental test were analysed with emphasis on the stiffness behaviour at the clinch nodes connecting the components of the lattice trusses. Buckling and twisting behaviour of the chords in the experimental tests was discussed. A second 3D finite element model was created to investigate
the limits of stability of the chord section unrestrained lengths under combined axial force and bending moment. Buckling checks to BS5950-5 [7] were also carried out.

Chapter 7 gives a summary and draws conclusions from the research. Experimental testing methodology is reviewed. Observations from the range of tests and conclusions from each chapter are discussed and summarised and suggestions for further research are made.
2 LITERATURE REVIEW

With new cold-formed steel framing systems being made available to the building trades and an upturn in the use of cold-formed steel in housing developments, greater efficiencies in the cold-formed steel design cycle are currently in demand. The relationship between the various groups that create a need for and supply the demand for cold-formed steel structures is complex. The end user is concerned with cost and functionality. The building trades need a system that is quick and easy for semi-skilled operatives to assemble and also a system that is cost effective.

For new framing systems to become successful, the housing designer, the framing systems engineers and the cold-formed steel manufacturer have to be aware of where competitive advantage is available to them in the form of new building technologies. All building designers need to be aware of their responsibilities to design safe and robust structures. When design guidance providing that assurance is not available for a new technology, the system is less likely to be used.

Cold-formed steel framing systems for low rise and residential applications are competing with timber framing alternatives for a large global market. Timber prices are unstable in comparison with the price of steel and this fact alone has encouraged the steel industry to recognise the demand for advanced framing systems and new cold-formed steel technologies. Clinching is a cold-formed steel joining technique that has many advantages over methods more commonly used to connect cold-formed steel such as screws and rivets.

Clinching also has great potential as an integrated part of new advanced light-weight cold-formed steel framing systems such as the Trichord [63]. Design guidance specific to mechanical clinching currently available to cold-formed steel designers is limited to research papers and reports. The following review of literature highlights the need for international design codes to give design guidance specific to mechanical clinching. This will have the effect of encouraging designers and builders to consider mechanical clinching and to apply the connection technique in new advanced framing systems.
2.1 Markets for cold-formed steel

Nancy Solomon describes how the variable price of wood has caused the global market in cold-formed steel to increase in [72], given the relatively stable price of steel. The focus of the steel industry in the past has been on promoting cold-formed steel framing systems in the residential housing market to compete directly with demand for timber framing in residential housing. This has been carried out by establishing a framework of building codes, training programs, tools and a standardised network of materials suppliers in the USA to support the technology.

The article outlines cold-formed steel housing beginning in the USA in the 1940s, with the first cold-formed steel design specification published by the American Iron and Steel Institute in 1946. With some increase in use in the 1970s the market for cold-formed steel has increased significantly in the 1990s with an increase in the price of timber. Two methods of building with cold-formed steel are described:

- Stick construction where cold-formed steel struts and beams are pre-fabricated and joined on site in housing units
- Panelisation where panels are constructed off site and assembled on-site

Advantages and disadvantages of cold-formed steel and timber construction are also discussed. The shape stability over time associated with cold-formed steel is an advantage over timber which can warp, shrink and decay. Cold-formed steel which has been galvanised has good resistance to corrosion. Parts of the galvanised coating that have been eroded or scratched away from cutting and connecting generally corrode very locally on the exposed area with the corrosion unable to spread over the galvanised area. The zinc coating has the corrosion resistant effect of being self-sacrificing - zinc will react with the water before it reacts with the steel. In severe cases however corrosion can occur. Such extreme situations are in marine coastal environments and instances where areas of cold-formed steel are left sitting in water. In buildings cold-formed steel elements are generally used in protected environments.

Solomon compares thermal transmittal effects of cold-formed steel to timber. Where cold-formed steel is used in wall panelling cold-formed steel wall studs create a thermal bridge between the divided spaces. Methods of insulating against this effect with polystyrene foam products are discussed.
Solomon concludes that to cut into the market for timber framed housing, cold-formed steel construction needs to become cheaper in comparison with the timber alternative. Methods of fast and easy erection of cold-formed steel housing need to be established and codes of practice and guidance for designers needs to be broadened. A prescriptive method of design for cold-formed steel framing was developed by the NAHB and the AISI [2] in 1996 to allow steel frame housing to be designed without the advice of a professional engineer, as is the practice with timber design. This applies to small domestic scale buildings. As a part of the prescriptive method of design for cold-formed steel framing, cold-formed steel manufacturers agreed to standardise the dimensions of the framing sections to the same dimensions as the equivalent timber rectangular sections. A standardised system of labelling cold-formed steel components has also been established by CABO (the Council of American Building Officials).

New innovations in cold-formed steel construction techniques are described by Solomon such as:
- New saw blades for cutting that are smoother and produce less noise
- Shot fired pins for connecting plaster board to steel frames
- *Hand held clinching devices that quickly join steel members without pins or screws*

Solomon also concludes in [72] that while cold-formed steel framing accounts for only a small percentage of housing being built in the USA, the market for cold-formed steel components has good potential for growth with new steel building techniques.

Ken Slattery describes an increase in the use of steel in residential housing in an article [71] due to:
- Increasing environmental awareness
- Accelerating rates of new technology development
- Evolving socio-economic conditions

The market for steel components in residential building is described as becoming globalised with steel component manufacturers competing to produce products that stand out on their own. The article also outlines a downturn in the use of steel in auto and industrial manufacturing markets due to increased use of plastics and cheaper high tech materials, creating a focus on expanding existing steel building markets.
New steel building system designs are described which utilise coated and galvanised steel sections. Mechanical clinching is particularly suitable for joining coated and galvanised steel sections as there is little damage to the surface coating layer in the clinching process. Mechanical clinching is a suitable for application to building system designs on its own or with other joining techniques such as screws or rivets as used in the Trichord building system. The difference in the typical characteristics of steel companies and housing builders is described: Steel companies are typically large corporate organisations with large capital investment capabilities while residential housing builders are smaller companies whose main assets are human skill resources.

Slattery suggests that for the steel industry which is used to dealing with large well organised industrial manufacturing companies to supply the smaller building contractors with the steel products and systems, a fresh approach is needed. Early attempts to make this link in the market have not produced good results and so new market research studies and a fresh approach to understanding the situation is needed.

The steel supply chain in residential building is broken down in the article into the following five groups:
1. Steel suppliers
2. Building product manufacturers
3. Building trades
4. Builders
5. Consumers

The steel supplier in this supply chain does not have a direct link with the builders because the steel components are supplied to the housing builder by the building product manufacturers.

Slattery describes how for the builder using traditional materials and building methods such as timber and brickwork to change to a modern steel framing system, the steel framing system must offer advantages in functionality, building efficiency and cost. The needs and issues specific to the steel framing builder are listed as:
The building trades companies are identified as the small sub-contracting organisations that implement the building systems. The efficiency of the building trades in erecting steel framing systems is important to the builders success in business and in the success of the framing system in general, and so the design of the framing system including the method of connection is critical.

Issues affecting the building trades are identified as:

- Technical support
- Quality
- Ease of use (speed)
- Cost
- Overheads
- Availability
- Adaptability

Slattery describes how the world steel industry is interested in expanding existing steel framing markets and sees market opportunities due to pressure to reduce the cost and time associated with traditional building methods such as timber and brickwork. The strategic response of the world steel industry to expanding the market for steel framing systems is to exploit the stability of the price of steel against the cost of timber. It is recommended that steel design codes and standards should be developed to adequately address needs of steel framing system designers. This will give assurance and clarity to those in steel supply chain dealing with steel framing systems.
Current design guidance on use of mechanical clinching in building framing applications is covered by general guidance on use of connections in cold-formed steel. Data and analysis in this thesis is an indication of suitability if mechanical clinching for joining of cold-formed steel framing systems.

The potential use of mechanical clinching in steel framing systems is overlooked because of lack of information in international design codes specific to mechanical clinching in load-bearing building applications. Mechanical clinching in steel framing is a technique that can benefit the international steel industry if the technique is suitably investigated, documented and presented.

2.2 Cold-formed steel technology

Methods of connection are central to the design of cold-formed framing systems. Understanding the potential of new advanced cold-formed steel framing systems can be achieved by shear resistance testing of connections and consideration of the advantages and disadvantages of one method of connection over another. The following research publications focus on various aspects of cold-formed steel design and applications. Papers reviewed in the following sections are only a small selection of the wide range of papers published on cold-formed steel connection technology, typical references are quoted and the main aim of the review is to consider research on clinching.

2.2.1 Work of Cramer, Shrestha and Mtenga (1993)

The structural shear resistance of metal plate connected wood trusses was investigated by Cramer et. al. in [11]. The analysis procedures being used in America at the time were referred to as 'The Current Design Analogue'. It recommended models to be formulated with frame members rigidly connected where wood members are continuous and pin connected where separate members meet.

A computer program called the 'Purdue Plane Structures Analyser' - a plane frame program with a built in design oriented post-processor - was given as an example of a popular program in America that used the 'Current Design Analogue'. Methods were suggested to model rotational and translational stiffnesses with metal plate connectors by specifying special $3 \times 3$ stiffness matrices in the computer model. Of these 3 degrees of freedom the first represented horizontal
translational stiffness, the second represented vertical translational stiffness and the third represented rotational stiffness. After implementing the method it was found that predicted values were still not reliable due to the effects of member eccentricity. It was therefore necessary to include more beam elements in the chord members, with more node positions, allowing a metal plate connector to be represented in the computer model by two or more nodes with the eccentricity between them.

The finite element program SADT (Structural Analysis of Diaphragms and Trusses) was used to apply connection eccentricities in the computer model. The non-linear metal plate connector stiffness behaviours that occur at large displacement were also introduced. Using this method the Foschi equation was developed and verified to define the rotational stiffness characteristics of metal plate connectors. This equation used three variables: initial stiffness, load, and tangent rotational stiffness, to describe the stiffness behaviour in terms of load and rotational movement.

Cramer et. al. described a series of experimental tests on metal plate connected pitched-chord and parallel-chord trusses. A new analysis method was used which was similar to the Foschi model with joint eccentricities introduced. Comparisons were made of mid-span deflections from experimental data, data calculated using the 'Current Design Analogue' and data calculated using the new model. It was shown that the new model gave considerably closer and more consistent predicted displacement values. Pitched chord trusses tested were approximately 8m in length and parallel chord trusses were approximately 4m in length. It was concluded that while deflections were predicted satisfactorily, member forces did not appear to be accurately calculated in the new model.

2.2.2 Work of Dubina and Zaharia (1997)

Bolted connections in cold-formed steel were analysed for rigidity and initial deformation in terms of resisting applied moment in by Dubina and Zaharia in [21]. The experimental arrangement in this research typically involved two cold-formed steel building components connected at a right angle by bolts. Inclinometers measured the change in orientation of the less stiff member as moment was applied to the system. The steel thicknesses used were 2, 3 and 4mm. Cold-formed steel sections were c-sections arranged back to back as are the components described in the chapters on moment-rotation behaviour of connection groups in this thesis.
With two bolts resisting all the applied moment the characteristic behaviour is an initial shear deformation of the steel due to the clearance between the bolt and the bolt hole in the plate, after which there is a linear elastic moment-rotation response to failure of the connection. To describe this behaviour in a simplified way requires only a small number of parameters such as the rotation or displacement magnitude of the initial shear deformation, the initial stiffness in moment-rotation or in force-displacement, and the failure load or moment capacity of the system. While the experimental work in the paper focused on a moment-rotation failure arrangement of a simple two component system, the work was geared towards providing insights into the behaviour of fabricated truss beam building components which were typically used for roofing applications.

Finite element analysis was carried out to establish the magnitude of local deformation at the bolt-hole local area in the steel. The analysis was in two dimensions using plate elements. The finite element mesh was a long rectangular plate with a circular bolt hole in the center. The bolt was represented by a rigid circular entity in the bolt hole that contacted the plate as the plate moved. Finite element analysis results show stress-power graphs that give close results to experimental work.

It is concluded from the finite element work that initial shear deformation is caused largely by the deformation of the steel around the contact area between the bolt and the bolt hole and only in part by the clearance between the bolt hole and the bolt. It is also concluded that the rotational behaviour is limited by the triangular closed arrangements of typical roof trusses and that therefore concentrated forces induced by rotational leverage will in practice be at a minimum.

2.2.3 Work of Toma, Sedlacek and Weynand (1997)

Toma et. al. discuss the reasons for a designer to choose a particular type of connection in cold-formed steel and the actual connecting techniques are discussed with respect to economy and efficiency in the overall design in [78]. Connections and connecting techniques are grouped into mechanical fasteners, welding and adhesive bonding. The group containing mechanical techniques is most relevant to the research in this thesis. Within the mechanical fasteners category are bolts with nuts, screws, blind rivets and shot pins. Mechanical clinching was not considered. M5-M16 bolts with property classes 8.8 or 8.9 are recommended for connecting thin
walled steel components. Applications for self-tapping and self-drilling screws and the appropriate type of screws for each type of application are described. Blind rivets are described for applications where only one side of two or more thin sheets of steel to be joined is accessible.

Current industrial usage of mechanical clinches in commercial framing systems is outlined in [78] and an increase in interest and usage of mechanical clinched building frame components is predicted. The increase is attributed to changes in the use of timber, which competes directly with cold-formed steel in many cases. Reference is made to Dedolph and Jaselskis [19] which describes a decline in the quality of timber, changing timber prices and limited potential for recycling timber as building components. Clinching technology is described being used in Tri-chord cold-formed steel framing with the mechanical clinches applied to close triangular box sections of chords on cold-formed steel trusses, as a part of the roll-forming process.

Several types of welding are considered. Under the heading of 'Open', welding, gas metal arc welding, manual arc welding, TIG welding and plasma welding are discussed. Spot welding, seam welding and projection welding are also suitable for cold-formed steel applications and come under the category of resistance welding. Adhesives to connect cold-formed steel components include epoxy adhesive and acrylic types, both types can have a brittle shear behaviour and poor peening resistance while resistance to cyclic loading can be higher than that of mechanical fasteners.

The sections 'Requirements and Selection Procedure', 'Mechanical Properties of Connections' and 'Background Studies' discuss the structural and non-structural aspects of connection techniques that will influence a designer in deciding which technique to use with reference to the old and the new Eurocode specifications [24]. In particular it is pointed out that EC3 [24] specified that the transition from thin walled steel to thicker members should be continuous. A procedure for obtaining a statistical evaluation of the behaviour of a specific method of joining cold-formed steel from experimental data using ECCS TC7 formulae [29] and a basic shear resistance function is outlined. A worked example follows for bolts connecting cold-formed steel where the statistical evaluation obtained from using the ECCS TC7 formulae is compared with the evaluation obtained from using the EC3 formulae. The EC3 formula produces a more conservative shear resistance evaluation.
This paper is concluded with the older and newer Eurocode procedures for evaluating the characteristic shear resistance behaviour of a connection technique under consideration. It is stated that with the newer EC3 recommendations, guidance for design in thin walled steel is closer to that for thicker steel components than it had been in the past.

2.2.4 Work of Kermani and Goh (1999)

While the materials and connection techniques being investigated in [35] by Kermani and Goh are different to those of this thesis, the methodology used to establish moment-rotation resistance characteristics for the ductile wooden truss system is relevant to the clinch connection group resistance investigations in this research.

Research in [35] sets out to establish effects of connection rigidity on the load-deformation characteristics, ultimate shear resistance and failure modes of the multi-nailed timber joints. The type of structure being investigated in the paper was two members of a wooden truss beam arrangement overlapped and connected at the single common node by a 100mm x 100mm flat steel plate with punched nails. The experimental rig consisted of a holding frame and a loading ram applying load eccentrically to a beam to apply a moment at the node. The effects of initial deformation in the connection are significant and consideration was given in the shear resistance formulae for this with an initial deformation modulus being established. Each nailed plate had a fixed layout of nail positions, however not all nail positions were utilised in each test giving a variable nail arrangement for different tests. Different types of timber were also used.

The effect of the number and position of the nails in the connecting plate on the moment resistance of the structure was analysed by calculating the polar moment of inertia of the nail group and plotting the value of inertia against the total number of nails used. The number and size of the nails was then considered. It was concluded that the stiffness of the joint increases as the number of nails increases at a constant rate up to 50% of the maximum number of nails used.

The failure modes of the joint were categorised into bearing failure of the timber member for lightly nailed connections, development of plastic hinges in the nails for larger diameter nails and shear failure of the nails in the case of joints with densely spaced small diameter nails. A shear resistance formula was established using as it's variables a nail parameter, a timber species parameter, a gusset plate parameter, a displacement parameter, a time parameter and an
environmental parameter. Worked examples were then given to demonstrate how formulas in the paper were used to determine an appropriate arrangement for nailed plate timber connections. It was concluded that further research was required incorporating different species of timber.

2.2.5 Work of Rogers and Hancock (1999)

18 shear tests were carried out on two steel sheets connected by bolts by Rogers and Hancock in [70]. Results of the tests were additional to a series of test results on bolted connections in cold-formed steel. The focus of investigation was on bolted connections in thin strength-enhanced steel of thickness less than 0.9mm. The strength of the thin steel sheets was increased by cold reduction, a rolling and squashing process that hardens the steel with four important effects:

- Grain of the steel was elongated in the rolling direction
- Consistency of the thickness of the steel was increased
- Yield stress of the steel was increased typically to 550N/mm²
- Steel became more brittle

Steel that was cold reduced to 0.42mm was referred to as grade G550 steel and steel that was reduced to 0.6mm is referred to as grade G300. The G300 sheet steels were annealed to a greater extent compared to the G550 sheets. Three types of cold reduced steels were used in the tests – G550 reduced from 1.0mm thickness to 0.42mm, G550 reduced from 0.8mm thickness to 0.42mm and G300 reduced from 0.8mm thickness to 0.60mm. Of the four design codes – European [27], Canadian [9], American [2] and Australia/New Zealand [73], it was stated that only the Canadian design code could be used to accurately predict the failure modes for the bolt samples in cold reduced steel shear test failure modes.

Shear test samples were tested in single and double bolt arrangements. The typical failure mode was piling of the sheet steel in front of the bolt. Tearing, cracking and necking of the steel around the bolts also occurred. Piling of the sheet steel in front of the bolt was observed in the shear tests on Henrob self piercing rivets in this research. Failure modes were categorised into three ultimate limit states – end pull out, bearing and net section fracture. All the G550 samples failed in bearing while the G300 samples failed by a mixture of bearing and pulling out.

A design provision for bolted connections in cold-reduced steel was proposed. It was stated that – ‘Significantly unconservative predictions of the load resistance obtained for certain bolted
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connection test specimens demonstrated a need for a gradated bearing coefficient which was
dependant on the stability of the edge of the bolt hole'. The recommended design equations
were:

Gross yielding failure: \( N_t = A_g f_y \)

Net section fracture: \( N_t = A_n f_u \)

End pull out: \( V_f = \frac{t e f_a}{1.2} \)

where:

\( A_g \) was the gross cross-section area

\( f_y \) was the yield or 0.2% proof stress

\( A_n \) was the net area of cross section

\( f_u \) was the ultimate steel strength

\( t \) was the steel thickness

\( e \) was the distance measured parallel to the direction of applied load from the center of a
standard hole to the nearest edge of an adjacent hole or to the end of the connected part.

It was concluded that the net section fracture of 0.42mm and 0.60mm, G550 and G300 sheet
steels at connections could be accurately and reliably predicted without the use of a stress
reduction factor based on the configuration of bolts and specimen width, as is commonly used in
current design codes. The net section fracture resistance of a bolted connection calculated
following the CSA-S136 design standard procedure, where the net cross-section area and the
ultimate material strength were used, was adequate. It was recommended that the gradated
bearing co-efficient formulation, the unreduced net section resistance, and the Eurocode method
for end pull out be used in the design of bolted connections.

2.3 Clinching

While there is currently no guidance in international design codes specific to designing strong
clinch connections in cold-formed steel, the following research papers investigate mechanical
clinching shear resistance. In many papers equations for predicting the shear resistance of
mechanical clinching were formulated based on experimental results and design considerations
for using mechanical clinches in new framing systems were discussed.
2.3.1 Work of Bober (1987)

Details of the mechanical process required to produce an ‘S type’ clinch of optimum shear resistance in different thicknesses of steel were investigated by Bober in [6]. By examining the depth of penetration of the punch part through the die and the thickness of the part of steel sheared through and the width of the same part it was possible to recommend particular punch and die parts for different steel thickness combinations. In the investigation the pressure that was used to clinch the parts for optimum connection shear resistance was also investigated. Results of Bober’s tests on mechanical clinches in different types of steel are listed in Table 4.15 in Chapter 4.

The minimum overlap of sheets of steel being joined by mechanical clinching was established as 15mm and the closest distance of a clinch to the edge of the steel was recommended as 5mm. When mechanical clinches were subjected to 10,000,000 cycles of loading it was found that a clinch could resist a third of the static peak load. A general relationship between mechanical clinches loaded at 0°, 90° and by pulling apart was given. The shear resistance of a clinch loaded at 90° was 0.7 times the shear resistance of a clinch loaded at 0° and the shear resistance of a clinch pulled apart by out of plane forces was 0.4 times the shear resistance of a clinch loaded at 0°. Clinches were also subjected to acidic corrosion and tested against control samples. It was found that there was no significant deterioration in shear resistance arising from corrosion of the connection.

2.3.2 Work of Wieck (1989)

In Wieck’s dissertation [80] the focus of investigation was on the shear resistance of H-type mechanical clinches. In this type of clinch one or more of the steel layers was deformed but not sheared through generally giving an air and water tight connection. Shear tests on mechanical clinches indicated that the H-type clinch was approximately twice as strong as the S-type join in two steel layers 0.5mm thick. However the shear resistance of the H-type clinch in two steel layers each 2.0mm thick was slightly less than the shear resistance of the S-type clinch. From an analysis of the data compiled in the clinch shear tests, Wieck presented a formula for predicting shear resistance of H-type clinches, Equation 4.8 in Section 4.6.2.
2.3.3 Work of Gopfert (1993)

Gopfert's research dissertation [30] described an investigation of the in-plane shear resistance of H-type, S-type and circular type mechanical clinches in steel thicknesses of 0.8mm in most test cases. The effects of different types of surface galvanising on the shear resistance of the connections was also considered. It was found that when zinc galvanised surfaces were in contact the shear resistance of the connections increased to a small degree.

Analysis of the clinch shear resistance test results showed that the H-type clinch tested at 0° showed higher shear resistance in comparison with the equivalent S-type clinch shear resistance tests. This was consistent with Wieck's shear resistance test results in the lesser steel thicknesses of less than 1.0mm. When connections were tested at 90° the shear resistance of the connection was greatest for the circular connection type, with the S-type connection giving the least resistance to loading. Cyclic tests were carried out as for Bober's research with circular mechanical clinches being tested over 10,000,000 cycles. The circular clinch was found to have good shear resistance characteristics after the cyclic loading tests.

The optimum tool part conditions for forming a strong circular clinch were investigated in depth. As for the other three dissertations from the Technischen Universitat Hamburg - [6, 55, 80], this aspect of clinch technology was important in developing mechanical clinching tools. Shear resistance from Gopfert's clinch tests are summarised in Table 4.17.

2.3.4 Work of Mutschler (1994)

The process of forming different proprietary types clinch was investigated in Mutschler's dissertation [55]. Three types of Eckold rectangular clinch – the R-type, Confix type and Tox-join were compared with the circular mechanical clinch. All connections were tested in 1.0mm thick steel showing that all types of clinch had similar shear resistances. With several combinations of punch and die part specifications used to test the connections it was found that the shear resistance from the tests was relatively insensitive to the types of parts being used.

Further tests were carried out to investigate the effects of the incorrect alignment of mechanical clinching tools on the shear resistance of the clinch connections. It was found that in general small misalignments had no significant effect on the shear resistance of the connections. When
the connections were greatly misaligned however the optimum shear resistance of the connections was not reached.

2.3.5 Work of Davies, Pedreschi and Sinha at the University of Edinburgh (1996-1997)

Shear resistance of clinch connections in two or more layers of steel with different thicknesses is influenced more by the thickness of the punch side steel than the die side steel thickness. Tests results show this in [15] (1996) with results presented by Davies et. al. and for a matrix of clinch shear tests with different steel thicknesses on the punch side and the die side of the connection. This paper reported on an initial project with the main aim of demonstrating the feasibility of mechanical clinching in buildings. Research presented in [15] began with a discussion on the advantages of mechanical clinching and the applications and trends in industry in cold-formed steel joining techniques. It was pointed out that the current British standard steel design code, BS5950, did not give design guidance for using mechanical clinches. Results from a series of shear tests on mechanical clinches were presented in addition to the results of tensile tests on samples of steel from the clinch shear tests. Structural behaviour of mechanical clinches in shear was characterised by four characteristics of deformation:

1. Loading stiffness
2. Peak shear load
3. Plastic limit
4. Unloading stiffness

Loading stiffness was the initial tangent stiffness of load divided by deformation beginning at zero load and zero deformation. Peak load was the maximum shear load over the full test. Plastic limit was the point of deformation where the line of peak load met the unloading stiffness. Unloading stiffness was the tangent stiffness of load divided by deformation of the clinch shear load losing shear resistance and falling to zero load, after the peak load.

Variability of the shear resistance of clinching was investigated by calculating the coefficient of variation of peak load from shear tests on clinch connections. Peak loads of clinch connections in steel thicknesses of 1.5mm and 2.0mm and with the angle of applied loading to the short edge of the connection at 0° and 90° were included in calculating the standard deviation. The standard deviation for clinch connections in 1.0mm steel thicknesses were higher in comparison with standard deviation for clinch connections in 2.0mm thick steel.
Shear tests described in [15] were carried out on an earlier version of the Instron testing machine used to carry out shear tests in this thesis. In the earlier machine a proving ring was used to measure load and a 100mm demec gauge was used to measure differential movement of the two strips of steel connected by the mechanical clinch. The speed of the test was set to create a rate of shear deformation at the clinch of 1.0mm/minute. The demec gauge was used to measure shear deformation on the sample in place of using measurements from the Instron cross-head because cross-head measurements included shear deformation in the Instron clamps. Using a demec gauge involved gluing small steel marking grooves to the test sample 100mm apart. The demec gauge was then pressed onto the marked points at intervals to allow displacement readings to be taken up to an accuracy of 0.01mm.

An Instron 100mm gauge length extensometer that connects directly to the Instron machine was used the shear tests presented in this thesis. The advantages of using the automatic extensometer are that readings are taken frequently with a corresponding reading of load and there is no physical interference with the test sample in the test.

Shear tests were carried out by Davies in [15] in steel thicknesses of between 1.4mm and 2.0mm, and with the angle of applied load to the short edge of the clinch of 0°, 30°, 45°, 60° and 90°. Modes of clinch failure were illustrated and investigated. A 'quasi plastic' response was described where at 90° applied load, one of the extended parts of the join was in tension and the other part was in compression. When the part in tension broke away the part in tension continued to resist shear load at a lower level until that part was also forced to break the connection. There was an approximately linear relationship between the angle of applied load to the clinch and the peak load achieved in each shear test.

Clinch shear deformation behaviour observed in the shear tests was further investigated under several sub headings. Initial stiffness behaviour and plastic limit were discussed under sub headings in relation to the influence of angle of applied load, steel thickness and ultimate tensile stress. It was concluded that the shear resistance and thickness of the parent steel and the angle of applied load directly affected the shear resistance of clinches and that the initial stiffness and plastic limit of the clinch is greatest when the load is applied at 0°.
Tests on ten pitched roof trusses were carried out in [61] (1997). Zed sections 120mm deep were used for the chord sections and the bottom stringer and pitched chords were joined by different arrangements of clinch groups. In two initial tests internal members were used between the outer triangle components. It was found in the tests that this arrangement greatly stiffened the triangular form, preventing rotation at the clinch groups, and so the internal members were removed in the remaining eight frames. Groups of between four and ten mechanical clinches were applied at the connection nodes. With ten mechanical clinches at each connection node, failure of the frame occurred by buckling of the truss sections rather than failure of the clinch group. By placing strain gauges near the clinch groups it was determined in the analysis that in the combination of axial force and bending moment, the axial component of load was more significant than moment in deforming the mechanical clinches.

Conclusions in [61] state that there was consistency in the clinch shear test results between different research programmes. The expressions for clinch peak shear resistance outlined in the paper were applicable to the test results obtained from research around the world. The highest standard deviation in a range of similar clinch shear tests was 4.45%.

Research presented by Davies in the Ph.D. thesis [16] (1997) was a comprehensive investigation of the shear resistance of mechanical clinching connection techniques in structural applications. Shear tests were carried out on clinch samples in different thickness arrangements and different layer arrangements of steel. Comparison was made against research results of previous research projects and against other connection techniques in cold-formed steel. A generalised mechanical shear resistance equation, Equation 4.1 in Section 4.6, was developed and discussed. The equation was formulated from 136 of Davies tests and from the work of five previous researchers. Results from Davies’s clinch shear tests in two layer of steel of equal thickness are listed in Table 4.12 in Chapter 4.

From the results of the clinch shear tests it was concluded that

- Shear resistance of mechanical clinches depended on the material strength, the sheet thickness and the angle of loading on the connection
Shear resistances were at a maximum when the load was applied perpendicular to the long axis of the join and at a minimum when the load was applied parallel to the long edge at 90°, typically reaching only 60% of the greatest peak load at 0°.

Peak load varied linearly between these two extremes.

Deformation capacity of the join increased as the angle of loading on the join increased.

When dissimilar thicknesses of steel were used to form a clinch a greater peak load was reached when the thicker material was on the punch side of the connection.

A series of moment-rotation tests were also carried out on mechanical clinches in groups of two and four and six in [16]. The moment-rotation testing set-up gave adequate lateral support to the clinch group samples and steel buckling occurred at approximately 60% of the capacity of the cold-formed steel sections only in cases where groups of six mechanical clinches were being tested. In the moment-rotation tests described in this thesis buckling of the sections was a greater problem as less stiff restraint was provided by the moment-rotation holding frame.

Full-scale tests were carried out on pitched chord roof trusses in [16] with spans of up to 6m with 'Z' and 'C' cold-formed steel sections. Large numbers of mechanical clinches at each connection node connected the sections. An increase of 38% of load capacity was reported for the clinched trusses in comparison with similar trusses with pin-jointed connections. By increasing the number of mechanical clinches at each connection node it was possible to change overall failure modes from failure of connections to failure of cold-formed steel sections. Bending moments and axial forces calculated from strain gauge readings gave a good comparison with forces calculated from deflections.

The focus of investigation in [17] (1997) was on the moment-rotation behaviour of groups of mechanical clinches. The experimental test set-up was a testing frame supporting two c-sections back to back in a simply supported arrangement. A screw jack applied load and a load cell measured applied load. A dial gauge extensometer was used to measure deflection. The clinch group under investigation joined the c-sections.

Clinches were arranged in rectangular groups of two, four and six joining the c-sections with spacings between mechanical clinches of 15mm, 25mm, 30mm, 35mm and 50mm. Steel
thickness of 1.6mm and 2.0mm were used in the c-sections. The moment-rotation behaviour was discussed with reference to the influence of steel thickness and the influence of clinch spacing. The expression for peak force in [15] was discussed and a regression analysis was carried out using the moment-rotation test data giving expressions for initial stiffness, unloading stiffness, peak load and plastic limit in the moment-rotation tests. The regression analysis was based on an incremental method developed by Colson and Louveau in [10].

In the clinch groups with two mechanical clinches, one clinch failed leaving a single clinch at the end of the test. Where there were four mechanical clinches again three mechanical clinches failed leaving a single join at the end, and where there were six mechanical clinches, four outer mechanical clinches failed leaving the central two which failed with further applied load. Peak moments in the clinch tests were predicted by multiplying the lever arm distance by the clinch forces. Predicted moments in [17] in most cases were within 10% to 15% of experimental test results. The conclusions state that the moment capacity of a group of mechanical clinches was influenced by the magnitude of spacing of the group, the steel thickness and the orientation of the individual mechanical clinches.

The paper [59] (1996) offers a review of the research results from Davies' Ph.D. thesis [16] summarising experimental work. In [59] a reference was initially made by Pedreschi et. al. to the current British standards in cold-formed steel design BS5950 parts 4 and 5 [7] for design guidance on cold-formed steel structures. The importance of methods of connection with mechanical interlock, welding and bonding in the design of cold-formed steel components in housing systems, single storey sheds and secondary support structures for cladding was outlined. A test programme was described with the aims of deriving basic data concerning the shear resistance of mechanical clinches, comparing the shear resistance of mechanical clinches with conventional mechanical fixing techniques and investigating the feasibility of making structural joints using standard cold-formed steel sections. The tests described in this thesis follow on from the work described in the paper [59].

The use of cold-formed steel components created on roll forming process lines for use in portal frame type buildings in the U.K. was outlined by Pedreschi et. al. in [59]. The use of zed and channel sections as purlins and sheeting rails was more common than timber and hot rolled sections in stud-framed housing, lattice beams, roof trusses and support structures for curtain
walling. The clinching process and advantages of clinching were described and the force against punch part movement of the clinching process was illustrated and explained. It was shown how automatic quality control was possible in mechanical clinching by using a device to monitor the force against punch part movement in the clinching process within a standard response envelope.

Testing of mechanical clinches in shear was also described in [59] and test results were presented and analysed. Shear tests were carried out on an Instron testing machine using a proving ring to measure load and a demec gauge directly on the shear test sample to measure differential movement of the two parts. Reference was made to the expression for clinch peak shear load established by Davies in [15].

Experimental tests in addition to shear tests on single clinch samples included shear tests on multiple clinch connection nodes, shear tests on mechanical clinches in multiple layers of steel, moment rotation tests on groups of mechanical clinches, pull out tests on mechanical clinches in gusset plate arrangements, and full-scale tests on pitched trusses joined by mechanical clinching. 6mm diameter pop rivets and 5.5mm diameter self tapping screws were also tested in shear, with screws bearing three times the applied shear loads in comparison with mechanical clinches and pop-rivets bearing approximately 50% greater shear. Shear tests on more than one clinch produced the equivalent shear resistance of a single clinch when the peak shear was divided by the number of clinches.

The test set-up for the first moment-rotation tests in [59] was a cantilever with a fixed upright cold-formed steel c-section and a second horizontal c-section clinched to the upright with a specific arrangement of 2, 4, 5, 7 and 10 mechanical clinches. Load was applied to the end of the cantilever, producing a moment at the clinch group. Deflection was measured by a dial-gauge extensometer. A second moment-rotation experimental set-up was a simply supported beam arrangement. The beam was in two parts, clinched at the central position by a third gusset plate component. Load was applied symmetrically at the central position by a spreader beam placing load on either side of the gusset connection. In this experimental set-up screws, rivets and mechanical clinches were tested. The moment-rotation test rig was a rigid steel frame providing simple supports to the test sample. Deflection of the c-sections was converted to rotation and applied load was converted to the equivalent applied moment.
From the clinch group moment-rotation tests in [59] it was concluded that the steel thickness and the number of mechanical clinches affected the moment-rotation resistance of the clinch group and that the orientation of mechanical clinches was critical to the moment-rotation behaviour. A mathematical model was described that was used to predict the peak moment in the moment-rotation tests. The model used an incremental process where the lever arm distance from the centroid of the clinch group to the position of the mechanical clinches was multiplied by the force in the mechanical clinches under applied moment to predict the rotation of the clinch group.

Four full-scale tests were reported in [59] on cold-formed steel pitched chord roof trusses 6.4m in length. Gusset plates connected the components of the trusses with three different connecting techniques in the four trusses: two with mechanical clinches, one with rivets and one with screws. Load was applied at four positions on the top sloping chords of the trusses at positions away from the gusset plate nodes. Trusses were restrained against lateral buckling and the number of joins at each gusset plate node was designed to allow the sloping chord to fail in bending away from the connections. Failure of the trusses occurred by torsional buckling of the bottom tension chord caused by eccentrically applied load from the truss members above. The two mechanical clinched trusses achieved peak loads of 12.0kN and 12.1kN. The truss connected by rivets achieved a peak load of 10.7kN and the truss connected by screws achieved a peak load of 14.1kN. Conclusions included the point that the strength of full-scale trusses was similar irrespective of the type of fastener used.

Full-scale beam tests using clinch group connections are also described and illustrated in [59]. The test set-up is a 'H' frame arrangement where a 3.35m horizontal cold-formed steel c-section was connected to two upright c-sections at either end by groups of four and eight mechanical clinches. The vertical c-sections are made semi-rigid by clamping blocks of wood to the inside of the channel. Strain gauges are placed at the top and bottom fibers of the ends of the horizontal beam near the clinch group and on the vertical supports to record the bending behaviour at those positions. Load is applied vertically downwards at the two third length positions along the beam. Substantial end moments in the clinch groups were recorded at the end positions of the beam. The same H-Frame tests were described in detail by Pedreschi et. al. in [62]. Finite element models of the H-frame test set-up are created and the procedures outlined in the paper [62] are discussed in Section 5.3.1 of this thesis.
2.3.6 Pedreschi's clinched light-weight steel beam (1999)

The light-weight steel beam system design presented in [64] was developed by Pedreschi in the Department of Architecture at the University of Edinburgh before the research described in this thesis was carried out. The beam system used mechanical clinching to connect the cold-formed steel components and the application of mechanical clinching is central to the design of the beam system. The light-weight steel beam was designed and patented in an environment of increasing use of cold-formed steel in building construction. A series of full-scale tests was carried out and a cost study was outlined in the paper.

The provision of a light-weight steel beam aimed to provide builders and designers with longer span cold-formed steel load-bearing components that could be safely used in floor and roof support positions. The beam was fabricated entirely from folded cold-formed steel sections with mechanical clinches joining the components. Savings in weight and cost by using mechanical clinches in place of screws or rivets in a connection intensive design were be significant.

The profile had an 'I' cross section with the top and bottom flanges formed from two folded sections back to back. The web had a series of rectangular corrugations 100mm in length repeated along the length giving resistance to buckling under vertical compression. The web and flange parts could be assigned different thicknesses for the optimum strength design.

Mechanical clinches were applied at two locations on the beam - joining the two parts of the flanges together and joining the clinched flanges to the corrugated web component. All these clinches were applied at one side of the steel sections oriented in one plane. Gaps in the flange parts joining onto the web allowed for insertion of the mechanical clinching tool.

In the experimental tests twelve full-scale beams 6 metres in length were fabricated and loaded to failure. Beams were simply supported in a rigid testing frame and loaded at the central position. Nine beams measured 300mm in depth, two were 450mm and one was 600mm deep. This gave a span to depth ratio of between 10 and 20 for the beams. Steel flange thicknesses of 1.5mm and 2.0mm and steel web thicknesses of 1.0mm and 0.7mm were used in the beams. Destructive tests set out to establish the structural behaviour of the beams with different span to
depth ratios, with different component thicknesses and with holes cut out of the web part representing service openings.

Experimental test observations showed that the 300mm deep beams failed by buckling of the compression flange at mid-span and the 450mm and 600mm deep beams failed by shearing of the mechanical clinches in the region of the end supports. The effects of reducing the steel thickness in the web and flange components was to weaken clinch connection between the components. By cutting a large hole in the beam web of approximately the size of two sections of web corrugations, the beam failed by buckling at the location of the hole and the strength of the beam was reduced by 35%. A smaller hole did not cause buckling near the hole but did reduce the strength of the beam by approximately 9%.

In another series of tests the beams were loaded to a span-deflection ratio of span/360, then the load was removed and the beams were cycled again over at least three cycles. Elastic recovery was approximately 95% in all cases. The proportion of deflection arising from shear displacement was estimated to be 25% to 30% for the 300mm deep beams and 48% for the 600mm deep beams.

Estimated force in the clinches in the beams at failure was less than the shear resistance established in shear lap tests on the clinches. Flexural strength of the beams estimated by a bending moment equation was greater than the experimentally measured strength due to shear deformation in the clinches. Details of a cost study comparing the lightweight steel beam design and two different proprietary beam systems applied in a building system indicated that the lightweight steel beam was an economic alternative to commercial systems available at the time.

The paper concluded that mechanical clinching was a suitable cost effective cold-formed steel joining method for modern construction. Failure of the larger 600mm deep beams was dictated by the shear resistance of the mechanical clinches and the possibility of introducing twice as many clinches at the same positions existed. The lightweight steel beam testing frame was used to test the lattice truss beams described in Section 6 of this thesis. Shear failure of the mechanical clinches in the lattice trusses near the upper and lower flanges and local buckling also occurred in the current tests. In the lattice truss beams overall geometric torsional warping and buckling is common and more comprehensive lateral and torsional restraint to the trusses
was required. Predicted shear failure loads of the mechanical clinches were closer to the measured shear failure loads in the lattice truss beams in comparison with data reported for the lightweight steel beams.

2.3.7 Trichord clinch tests (1997)

In [63] shear tests were carried out by Pedreschi and Sinha on clinched samples that were fabricated at the Trichord cold-formed steel factory in the USA. Samples were typical of the mechanical clinching arrangement in the Trichord framing system, with more than two layers of steel being clinched at each position. Two general arrangements of four clinched layers were tested. The first arrangement was composed of two central layers pulling in shear against two outer layers. The second arrangement was composed of four layers of steel, with each alternate layer pulling in the opposite direction as shown in Figure 2.1. Trichord clinch shear test peak loads are summarised in Table 2.1.

Factors influencing the shear resistance of clinch connections were listed and discussed in detail, with reference to other current papers by the authors. Shear tests carried out and documented by Mutschler [55] and Wieck in [80] were analysed with special observations on the influence of the strength of the parent steel, the steel thickness and the orientation of the applied load to the clinch on the shear resistance of the clinch connection. The expression for shear resistance of mechanical clinches developed by Davies in [15] was discussed incorporating the angle of applied load to the mechanical clinch, steel thickness and ultimate tensile shear resistance based on over 140 shear tests on mechanical clinches. It was shown that in 80% of test cases the predicted peak load was within 15% of the peak load established in the experimental tests. A second similar expression was given for the peak load of mechanical clinches in three layers of steel. All three layers of steel were of equal thickness, and the shear in the connection was applied by pulling the middle layer of steel in the plane of the steel away from the outside layers. The expression was re-written replacing the steel thickness variable with the combined thickness of the three layers. Pei and Kinney in Florida carried out further shear tests [65] showing a good correlation between their test results and the results presented in the paper.
Figure 2.1, Layer arrangement of Trichord clinch shear tests

<table>
<thead>
<tr>
<th>Steel Layer Arrangement</th>
<th>Steel Thickness (mm)</th>
<th>Angle of Applied Shear (degrees)</th>
<th>Peak Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.13</td>
<td>0</td>
<td>9.23</td>
</tr>
<tr>
<td>A</td>
<td>1.13</td>
<td>45</td>
<td>7.18</td>
</tr>
<tr>
<td>A</td>
<td>1.13</td>
<td>90</td>
<td>5.31</td>
</tr>
<tr>
<td>A</td>
<td>0.93</td>
<td>90</td>
<td>3.66</td>
</tr>
<tr>
<td>A</td>
<td>0.93</td>
<td>45</td>
<td>5.12</td>
</tr>
<tr>
<td>A</td>
<td>0.93</td>
<td>45</td>
<td>4.96</td>
</tr>
<tr>
<td>A</td>
<td>0.93</td>
<td>90</td>
<td>6.98</td>
</tr>
<tr>
<td>A</td>
<td>0.93</td>
<td>0</td>
<td>4.96</td>
</tr>
<tr>
<td>A</td>
<td>0.76</td>
<td>90</td>
<td>2.38</td>
</tr>
<tr>
<td>A</td>
<td>0.76</td>
<td>45</td>
<td>2.44</td>
</tr>
<tr>
<td>B</td>
<td>0.93</td>
<td>0</td>
<td>8.15</td>
</tr>
<tr>
<td>B</td>
<td>0.93</td>
<td>0</td>
<td>8.84</td>
</tr>
</tbody>
</table>

Table 2.1, Trichord clinch shear test results

Equations for normalised peak load were given by Pedreschi and Sinha in [63] (1997) and compared against results from Trichord’s own tests:

\[
P = \frac{\text{Peak load}}{(\text{UTS} \times t)}
\]

where:
- \(P\) was the normalised peak load
- \(\text{UTS}\) was the ultimate tensile strength
- \(t\) was the combined steel thicknesses.
From the research at the University of Edinburgh:

\[ P = (17.1 - (0.089 \cdot \theta)) \cdot \text{UTS} \cdot \text{t} \text{ (metric units)} \]

\[ P = (0.67 - (0.003 \cdot \theta)) \cdot \text{UTS} \cdot \text{t} \text{ (imperial units)} \]

where:

- \( P \) was the clinch peak shear capacity
- \( \theta \) was the angle of applied shear

From the shear tests at Trichord:

\[ P = 0.683 - (0.00288 \cdot \theta) \cdot \text{UTS} \cdot \text{t} \text{ (imperial units)} \]

It was concluded in [63] that the shear behaviour of mechanical clinches joining four layers of steel was similar to the shear behaviour of mechanical clinches in two layers of steel. There was a linear relationship between shear resistance and angle of applied loading and the peak load could be predicted by a linear equation. An increase in shear resistance of approximately 20% for layer arrangement A (Figure 2.1) over layer arrangement B samples is noted.

### 2.3.8 Work of British Steel

A series of connection tests were carried out by British Steel including shear tests on mechanical clinches in a range of steel thicknesses. Comparison was made with peak loads from spot welding, self-tapping screws, carbon steel rivets and Henrob self-piercing rivets. Spot welds produced the highest peak loads. A summary of test results is given in Table 2.3 and peak shear loads from the clinch tests are listed in Table 4.15.
Table 2.3, British Steel connection shear test peak loads

<table>
<thead>
<tr>
<th>Connection</th>
<th>Peak load Thickness 1.6mm</th>
<th>Peak load Thickness 2.0mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildex 22/7 steel, self-drilling screw, hex head</td>
<td>7.66</td>
<td>9.75</td>
</tr>
<tr>
<td>Special steel, self-drilling screw, hex head</td>
<td>9.33</td>
<td>13.00</td>
</tr>
<tr>
<td>Advel 2011 carbon steel rivet, brazier head</td>
<td>9.88</td>
<td>12.18</td>
</tr>
<tr>
<td>Resistance spot weld, (diameter 5 x root t)</td>
<td>14.74</td>
<td>19.91</td>
</tr>
<tr>
<td>Henrob high carbon steel, self piercing rivet</td>
<td>6.60</td>
<td>7.99</td>
</tr>
<tr>
<td>Eckold press-join at 0°</td>
<td>3.88</td>
<td>5.15</td>
</tr>
<tr>
<td>Eckold press-join at 90°</td>
<td>2.36</td>
<td>2.70</td>
</tr>
</tbody>
</table>

2.3.9 Work of Kolari and Hakka-Ronnholm (1998)

Kolari and Hakka-Ronnholm's research report [38] summarises and analyses the experimental results from a programme of testing on rivets, screws and rectangular and circular mechanical clinches in cold-formed steel. Static and cyclic shear tests were carried out.

It was stated that mechanical clinches were generally weaker than screws and therefore more than one clinch should be applied to the connection in place of a screw in a similar connection. The testing arrangement involved several mechanical clinches joining two steel samples. Mechanical clinches were arranged at even spacings in a line along the line of loading.

Mechanical clinches with the long edge parallel to the line of loading (90° clinch arrangement), clinches with the long edge normal to the line of loading (0° clinch arrangement), circular clinches, screws and rivets were all tested in shear and analysed. Superimposed load displacement paths from these tests in two layers of 1.5mm steel showed that clinches in the 90° arrangement had the lowest initial stiffness at approximately 2.5kN/mm per connection. The initial stiffnesses of all the other connections tested were similar at approximately 8kN/mm per connection in the elastic range.

Kolari's research [39] aimed to examine developments in techniques for connecting light gauge cold-formed steel used in the construction industry in Scandinavia, and to study the applicability
of new joining methods. Shear tests were carried out on Attexor circular and rectangular mechanical clinches, screws, and rivet connections in light gauge steel under static and cyclic loading conditions. Special consideration was given to the resistance to corrosion of all the types of connection tested, with samples being left in partially corrosive environments for extended periods of time. Shear tests were carried out on samples after extended corrosion had taken place with no obvious loss of shear resistance arising from corrosion. Shear tests on groups of three, four and five mechanical clinches in a line parallel with the line of loading showed that the equivalent shear resistance of a single clinch connection was maintained in a groups connections in a line. Circular clinches in 1.0mm thick steel had a higher peak shear load in comparison with rectangular mechanical clinches with the load applied at 0° to the short edge of the join. Circular mechanical clinches in 1.5mm thick steel showed a similar peak shear load in comparison with the same rectangular mechanical clinches in 1.5mm thick steel.

In the clinch cyclic loading tests the load levels were repeated and changed over three phases to simulate the estimated loading characteristics on a cold-formed steel frame. There are three cyclic loading levels, F₁ was the estimated force on the clinch in a framing system supporting self weight, wind and snow loads, F₂ represented self weight, wind and half the snow load, and F₃ was half of F₂. The samples were loaded through 5 cycles between zero force and F₁, 30 cycles to F₂ and 150 cycles to F₃. The F₁, F₂ and F₃ phases were repeated in series 100 times giving 18,500 cycles in total. Static shear tests were carried out after cyclic loading to obtain the shear resistance of the cycled clinch. Kolari concluded that ‘cyclic loading had no effect on the subsequent shear behaviour of screwed and press-joined connections’.

2.3.10 Work of Lu et. al. (1998)

Shear tests were carried out by Lu et.al. in [50] on circular and rectangular Attexor clinches in steel thicknesses of 1.0mm and 1.5mm, with the angle of applied load to the short edge of the clinch at 0° and 90°. Comparison was also made with the shear resistance characteristics of rivet and screw connections. Clinch failure modes were investigated and corroded samples were tested for comparison against samples with no corrosion. Shear deformation behaviour was plotted for all tests carried out and the thicknesses and peak loads were tabulated for comparison.

It was concluded that the punch side thickness of two layers of steel being joined greatly influenced shear resistance of connection, with the die side thickness having less influence.
Tests on two clinches along the line of loading gave lower peak loads due to the effect of one clinch failing before the other.

Research presented by Lu, et. al. in [52] is focused on investigating the shear resistance of the rosette joint. In the paper methods of joining not more than two layers of steel with a rosette were described. Cold-formed steel sheets being joined were prefabricated with a circular hole in one sheet and a collared hole in the other. The collared part was placed through the plain hole and a special expanding tool head was passed through the center of both holes. To clinch the connection the head of the tool part was expanded, crimping the collar. This was a similar method to riveting with access to only one side of the steel sheets required. The rosette joint was described in the context of a joining method for highly automated frame panel and roof truss assembly systems.

Shear tests were carried out on flat and folded sections with a single 20mm or 22mm rosette connection. A summary of the test results are shown in Table 2.2. Failure of the samples was initiated by local buckling of the sheet along the compressed edge of the hole and by failure of the collar. The rosette connections showed a higher peak load in comparison with clinches and other connection types described in this thesis. Additional shear resistance is offset however by size of the connection and effort prefabricating the hole and collar to the correct specifications in the steel sheets.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Shear Strength Maximum (kN)</th>
<th>Minimum (kN)</th>
<th>Average (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hole Part</td>
<td>Collar Part</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td>1.0</td>
<td>8.07</td>
<td>6.39</td>
<td>6.89</td>
</tr>
<tr>
<td>Flat</td>
<td>1.5</td>
<td>12.26</td>
<td>10.64</td>
<td>11.39</td>
</tr>
<tr>
<td>Folded</td>
<td>1.0</td>
<td>8.58</td>
<td>8.32</td>
<td>8.39</td>
</tr>
<tr>
<td>Folded</td>
<td>1.5</td>
<td>12.91</td>
<td>12.11</td>
<td>12.45</td>
</tr>
</tbody>
</table>

Table 2.2, Rosette joint shear resistance test results

An investigation of the static stress concentrations around the rosette connection in the shear tests was carried out using the finite element program NISA and the finite element post-processing program DISPLAY III. Eight noded three dimensional shell elements with six degrees of freedom were used to model the plates. Spring elements were used to model extra stiffness provided by the collar. Steel material properties were given a plastic limit and the tests
were static non-linear with the load displacement curves showing a decrease in stiffness as the tests progressed. Experimental and finite element test results showed a close match.

In a second series of tests samples of rosette connections in steel sheets were subjected to cyclic loading. In the first two tests the load was increased in steps of 1kN at the beginning and in steps of 0.2kN to 0.4kN at load levels close to failure. A failure load of 12.5kN was reached after 1240 cycles in the first test and 12.0kN in the second test. In the third test the load was applied at 70% of load estimated to cause failure. In this case no severe deformation was observed after 7000 cycles.

The method of analysis in the cyclic testing was to carry out experimental thermal measurements of the samples as they were being tested with a special AGMEA 900 thermovision thermal measurement system placed close to the rosette. Stresses in the samples were then calculated by establishing the stresses required to create the observed changes in temperature. A second set of finite element tests were carried out using the finite element program ABAQUS to establish the stress distribution around the rosette connections in the steel plates.

The paper concluded that the experimental and finite element cyclic test results showed a good match and that the thermal imaging method of analysis used was a useful method for the verification of finite element models.

In [53] the rosette clinching system investigated in [52] was implemented in a cold-formed steel pitched chord roof truss framing system. The truss framing system utilised modified top hat cold-formed steel sections for the flanges and nested channel sections for the web components. Web and chord sections were designed to be joined by two rosette connections in the framing system. The web and chord sections were initially tested as separate components under axial force. In the compressive tests on the web channel sections, the ends of the sections were cast in concrete to provide adequate end fixity and end rotational restraint. Channel sections were tested in single arrangements and in double nested arrangements. Web channel sections were approximately 38mm square with a steel thickness of 0.94mm. Lengths of web tested were between 660mm and 1060mm. When the double nested sections were tested the failure load
(average 75.6kN) was approximately equal to three times the failure load of a single section (average 25.0kN).

In a second series of web compression tests the web section was joined at the ends to two lengths of chord section. The two chord sections were fixed at the ends. This series of tests investigated the behaviour of the web section in the simulated position of the web sections in the truss frame. From the second series of tests a web buckling wave length of half the web length was observed. It was concluded from this observation that an effective buckling length reduction factor $K_b$ of 0.9 was appropriate for design. The maximum axial force recorded in these tests was an average of 12.9kN from three tests. Compression tests were also carried out on the chord sections with the ends set in concrete in lengths of 1255mm and 1755mm. 1255mm length sections reached an average maximum axial force of 48.0kN with a standard deviation of 1.6kN. 1755mm length sections reached an average axial force of 34.5kN with a standard deviation of 0.14kN. Torsional flexural buckling caused failure in all chord tests.

In the full scale truss tests two 10m span pitched chord trusses were tested. Trusses were fabricated from two chord sections, a bottom stringer section and 18 web sections of variable lengths – 75.5kG total in weight. The ends of the trusses below the eaves joint were simply supported. Web sections were joined to the chord sections with rosette connections. All steel thicknesses in the trusses were 0.95mm. Web sections were changed to an almost square 29mm open section design for the truss tests. The ends of the webs were positioned in contact with the chord sections in all cases to ensure that compressive force was transmitted directly from web to chord with minimal stress on the rosette connection. It was noted that the dimensions of the fabricated truss were within 5% of the design dimensions. Loading was applied at eighteen positions in the top pitched chords through hydraulic loading rams and distributed at each loading position over 420mm.

Reference was made to the testing procedure in the Eurocode [28] where a three stage testing regime was employed. The first stage was an ‘Acceptance Test’, the second stage was a ‘Strength Test’ and the third stage was a ‘Prototype Failure Test’. The first truss passed the acceptance test but failed in the strength test because of manufacturing difficulties and insufficient detail in the truss design. The second truss failed in the prototype failure test stage with the rosette connections joining the two top chords failing. It was concluded that the
behaviour of the trusses was linear and predictable through the full-scale tests. The safety factor used for the design of the truss components was 1.1 and the safety factor used for the rosette connections was 1.25. While the truss design passed the requirements set by the European design standard a more refined truss system design was required for high quality manufacture.

2.4 Review of existing clinch test data

For the designer of a cold formed steel building frame or framing system considering the use of mechanical clinching as the method of connection in the frames, design guidance on the shear resistance of mechanical clinching can be found in many of the research papers in this literature review:

The series of dissertations from the Technical University of Hamburg by Bober [6], Gopfert [30], Mutschler [55] and Wieck [80] investigate mechanical clinching as the clinching machinery is being developed, giving early results on clinching:

- Cyclic loading of rectangular ‘H’ type clinches over 10,000,000 cycles
- An equation for predicting the strength of ‘H’ type clinches
- Comparison of ‘S’ and ‘H’ type clinch shear resistance
- Analysis of the sensitivity of clinch shear resistance to machine settings

Kolari in [38, 39] investigated the shear resistance of clinching giving results for:

- Static tests on rectangular clinches at 0° and 90° in two layers of 1.0mm and 1.5mm thick steel
- Static tests on circular clinches in two layers of 1.0mm and 1.5mm thick steel
- Cyclic testing of rectangular clinches at 0° in two layers of 1.0mm and 1.5mm thick steel with a complex cyclic loading pattern

At the University of Edinburgh results from shear test and full scale experimental tests are reported in [15, 16, 59, 60, 61, 62] including:

- Shear resistance of rectangular clinches in similar and different thicknesses
- Shear resistance of rectangular clinches at 0°, 45° and 90°
- Moment rotation resistance of groups of 4 and 6 clinches
- Shear resistance of clinches in full-scale pitched chord roof trusses
2.5 Aims of this research

Following an examination of existing test data, this research aims to extend the range of test data currently available for clinching in different steel thicknesses and in multiple layer configurations. Data currently available on the shear resistance of clinches under cyclic loading is extended and testing of groups of clinches and full scale parallel chord trusses give new insights into how clinched framing systems are affected by the method of connection.

- The S-type and H-type type Eckold clinch are tested and shear resistance characteristics are compared
- S-type clinches are tested in two and three layers of steel
- Cyclic shear resistance is considered by applying loading to 50% of clinch shear capacity over 10,000 cycles.
- Equations for predicting shear resistance of clinches in [15, 16 and 63] are reviewed and a new equation incorporating results in the extended database is proposed
- The moment resistance and rotational deformation capacity of groups of four and six clinches is investigated in experimental and numerical tests
- A series of full-scale parallel chord clinched truss tests are carried out. The force in the truss clinches is monitored and analysed against the shear resistance characteristics established in the Instron shear tests. The effect of clinch shear deformation on truss deflection is examined by carrying out finite element tests modelling the experimental truss tests and comparing finite element models with and without clinch shear deformation.
3 EXPERIMENTAL METHODOLOGY

Steel shear resistance characteristics are applied in the analysis of the experimental tests and are used for input to numerical tests using the Finite Element program ABAQUS in the following Chapters. Tensile testing methods used to establish the shear resistance characteristics of the galvanised mild steel used in the moment-rotation and full scale truss tests are outlined in this section. Experimental testing procedures used in the Instron connection shear tests and the material tensile tests are described. The numerical methodology utilised in ABAQUS to model plastic steel straining and non-linear load-deformation control are also outlined.

3.1 Experimental shear and tensile tests

Mild steel galvanised sheets used for the cold-formed steel tests were supplied in four nominal thicknesses - 1.0, 1.2, 1.5, and 2.0mm. Steel used in the Instron shear tests was supplied in large sheets and was cut to size using an industrial guillotine. The thickness of all sheets was measured and measured thickness values were consistent with the nominal 1.0, 1.2, 1.5, and 2.0mm thicknesses.

In testing steel in tension samples were loaded to fracture and strain was automatically logged at regular intervals of approximately 5 per second during each test. Strain response was initially linear and after yielding fracture occurred at large strains of between 17% and 30%. Stress strain data from a tensile test of a 1.2mm thick mild steel sample is illustrated in Figure 3.2.
3.1.1 Test samples

Test samples were 250mm in length and 50mm in width. The Instron tensile testing machine's cross-head clamps that grip the sample at either end are 50mm length square surfaces, leaving 150mm clear length between grips to be strained. The 100mm gauge length extensometer was applied on the sample within the 150mm length between grips as shown in Figure 3.1.

3.1.2 Strain and load measurement

Cross-head displacement was used to calculate strain in the test sample in the initial tests. This method did not compensate for shear deformation between the grips and the steel sample and therefore did not give satisfactory test results. For the following tests a 100mm gauge length extensometer with a travel range of 50mm and an accuracy of 0.01mm was fixed along the 150mm clear length of the sample as illustrated in Figure 3.1. Strain readings were taken at a rate of 3 readings per second, as part of a data set with corresponding load readings from the Instron 100kN load cell. Strain was calculated using the differential of the displacement at points 100mm apart, at either end of the test sample. This procedure gave the average strain over the 100mm gauge length. All samples underwent considerable necking before fracture failure at
the center of the sample, between the grips of the extensometer. The 100mm gauge length extensometer proved to be the most suitable strain measuring device for the material testing application as it allowed enough clearance for the sample to narrow and fracture at the center, without damaging the extensometer.

![Graph](https://via.placeholder.com/150)

Figure 3.2, Material test on 1.2mm thick steel

3.1.3 Instron testing procedure and test results

The Instron load cell and 100mm gauge length extensometer were reset and automatically calibrated before each testing session. Each test was carried out at a controlled rate of strain in the sample of 0.05% per second.

3.1.4 Young's modulus

The modulus of elasticity of the samples was calculated by taking the value of stress at approximately 75% of yield stress in a sample and dividing by the corresponding experimental strain at that point. This was within the linear range of strain. The average value of Young’s modulus was 184GPa, as shown in table 3.1 - this value is used in analytical and numerical studies in the following chapters. BS 5950 Part 5 [7] Section 3.3.3 recommends using a modulus of elasticity of 205GPa for the design of cold-formed steel thin gauge sections.
3.1.5 Yield stress

The plastic limit, or yield stress, of the samples was taken as the point on the stress strain curve where the elastic range ends and plastic deformation begins. This was characterised by the change from a linear elastic relationship to a variable plastic one. Yield stress is also listed in table 3.1 for a range of steel thicknesses. The average yield stress was 320N/mm² - this corresponds to mild steel grade 'S 320 G, 'Continuous hot dip zinc coated carbon steel sheet of structural quality', in Table 4 of BS 5950 Part 5 [7].

3.1.6 Ultimate tensile stress

The Ultimate tensile stress was the highest stress recorded for a given test sample. This occurred after plastic deformation and in the steel hardening phase.

<table>
<thead>
<tr>
<th>Steel Thickness (mm)</th>
<th>Sample Number</th>
<th>Young's Modulus (GPa)</th>
<th>Yield Stress (N/mm²)</th>
<th>Ultimate Tensile Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1</td>
<td>190</td>
<td>316</td>
<td>341</td>
</tr>
<tr>
<td>2</td>
<td>182</td>
<td>326</td>
<td>348</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>193</td>
<td>332</td>
<td>334</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>178</td>
<td>311</td>
<td>335</td>
<td></td>
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<tr>
<td>5</td>
<td>186</td>
<td>314</td>
<td>343</td>
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<td><strong>Average</strong></td>
<td><strong>186</strong></td>
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<td><strong>340</strong></td>
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<tr>
<td>1.2</td>
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<td>179</td>
<td>346</td>
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<tr>
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<td><strong>345</strong></td>
<td><strong>353</strong></td>
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</tr>
<tr>
<td>5</td>
<td>168</td>
<td>303</td>
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</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>182</strong></td>
<td><strong>309</strong></td>
<td><strong>346</strong></td>
<td></td>
</tr>
<tr>
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<td>5</td>
<td>192</td>
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<td><strong>Average</strong></td>
<td><strong>188</strong></td>
<td><strong>299</strong></td>
<td><strong>328</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Overall Average</strong></td>
<td><strong>184</strong></td>
<td><strong>318</strong></td>
<td><strong>342</strong></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.1, Steel tensile test data
3.1.7 Stress-strain curves

Stress-strain diagrams for the material tests on steel thicknesses of 1.0mm, 1.2mm, 1.5mm and 2.0mm are illustrated in Figures A1.1.1 to A1.1.8 of Appendix 1. The stress-strain diagram for each thickness was plotted twice:

- Within the large strain range of 40% in Figures A1.1.1, A1.1.3, A1.1.5, A1.1.7 showing peak stress and steel hardening
- In the elastic strain range below 0.3% strain in Figures A1.1.2, A1.1.4, A1.1.6 and A1.1.8, showing characteristics of linear elastic strain and material yielding.

3.1.8 Variability of material properties

Six similar yield tests were carried out on steel samples in 1.0mm thickness and 2.0mm thickness, to establish the coefficient of variation with regard to the yield stress values of the steel used in the experimental work. Stress strain paths for six tests on the 1.0mm thickness mild steel samples are illustrated in Figure A1.1.1 in Appendix 1. Standard deviations are referenced in Chapter 4 as a comparison with variability of failure parameters in connection shear tests. Table 3.2 lists the coefficient of variation of yield stress for the 1.0mm thickness and 2.0mm thickness samples.

### Table 3.2, Standard deviation of yield stress

<table>
<thead>
<tr>
<th>Steel Thickness (mm)</th>
<th>Number of Samples in Population</th>
<th>Average Yield Stress (N/mm²)</th>
<th>Maximum Yield Stress (N/mm²)</th>
<th>Minimum Yield Stress (N/mm²)</th>
<th>Standard Deviation</th>
<th>Standard Deviation %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>6</td>
<td>315.5</td>
<td>332.3</td>
<td>311.6</td>
<td>8.6</td>
<td>2.73</td>
</tr>
<tr>
<td>2.0</td>
<td>6</td>
<td>306.5</td>
<td>321.5</td>
<td>279.8</td>
<td>8.5</td>
<td>2.77</td>
</tr>
</tbody>
</table>

3.1.9 Shear test set-up

Instron tensile testing set-up and data logging devices are shown in Figure3.4. Connection samples consisted of two strips of mild steel, 200mm long and 50mm wide, overlapped and joined by the connection under investigation, illustrated in Figures 3.1 and 3.4.
The overlapped length in the samples was 80mm, allowing 10mm clearance at each and for the 100mm gauge length extensometer to grip the two steel plates. With this arrangement the displacement reading from the extensometer gave the differential movement between the two plates, which was equal to the shear displacement at the connection, if the strain in the steel plates is not considered.

The strain in the plates between the 100mm gauge length of the extensometer was not considered because it is small at the low load levels required to cause failure in the connections. For example, deformation over a 100mm length of 1mm thick steel of 50mm width with a modulus of elasticity of 184GPa under an applied load of 5kN is 0.05mm.
3.2 Numerical modelling

In the following chapters finite element modelling of experimental tests was carried out. The finite element program ABAQUS was used. ABAQUS is published by Hibbitt, Karlsson & Sorensen, Inc. USA and is a popular finite element program for non-linear problems in particular.

Two methods of numerical modelling are explained in this section:
1. 3D Modelling of cold-formed steel shell sections
2. Modelling of non-linear orthotropic clinch stiffness in the plane of shear displacement of the clinch

These methodologies are referenced in four instances in the following parts of this work:

- **Section 5.1.2, ‘Moment-rotation finite element tests’:** C-sections in moment-rotation tests were modelled with shell elements and orthotropic clinch stiffness was modelled at four clinch locations
- **Section 5.2, ‘Six clinch cantilever model’:** C-sections in the cantilever tests are modelled with shell elements and orthotropic clinch stiffness was modelled at six clinch locations
Section 5.3, 'Clinched H-frame full-scale tests': A horizontal c-section component and a part of the upright supports of the H-frame tested experimentally by Davies in [16] were modelled with shell elements, orthotropic clinch stiffness was modelled at 4 and 8 clinch locations.

Section 6.3.2, 'Truss chord finite element buckling analysis': Lengths of unrestrained top and bottom truss chord sections were analysed under combined axial force and bending moment.

In the 2D finite element analysis of full-scale truss beams in Section 6.2 cubic beam bending elements were used to model the stiffness of chords and internal components. Clinch shear deformation was modelled by a 0° clinch connection element – all mechanical clinches in the truss beam tests were oriented at 0° to the applied loads transferring from internal members to top and bottom chords and the orthotropic stiffness model was not required. A simplified non-linear model of clinch shear deformation was used and is described in Section 6.2.4.

3.2.1 Steel material model in the finite element tests

Average values of Young's modulus, yield stress and peak stress in Table 3.1 were used as input data for finite element models in the following chapters. Yield in the material occurs at the strain value of the yield stress divided by Young's modulus, 0.173%. The Ultimate Tensile Stress (UTS) was estimated to occur at 10% strain, defining the steel hardening characteristics. The Steel model had perfect plasticity after straining past the UTS at 10%. Finite Element steel material behaviour characteristics are illustrated in a sketch of the stress-strain relationship in Figure 3.5.
3.2.2 ABAQUS shell elements

Eight noded shell elements with five integration layers through the shell thickness were used to model cold-formed steel shell sections. This type of shell element was capable of modelling local plastic buckling at stress concentrations. There were four stress integration locations on the element plan as shown in Figure 3.6 (a), with five integration layers through the thickness, Figure 3.6 (b). The two outer stress integration layers are positioned at the outer fibers. This allowed the Finite Element program to detect the onset of plastic buckling following plastic bending strain at the outer fibers. Plastic straining began when Von Mises' stress at any stress integration point exceeded material yield stress.

Figure 3.5, Sketch of Finite Element steel stress-strain input (not to scale)
3.2.3 The non-linear Rik's algorithm

The non-linear Rik's algorithm option in ABAQUS was used to control the non-linear application of load on the Finite Element models in the static non-linear tests. The Rik's algorithm applies load to the model in non-linear increments using an arc search technique, increasing the load in an increment if the model is stable and decreasing the load if geometric or plastic buckling are anticipated. By applying a total static load of 1kN downwards in the Finite Element models and controlling the 1kN load with the Rik's algorithm, it was possible to read the load factor output from the Rik's algorithm in kN units.

3.2.4 Orthotropic clinch connection elements

In moment-rotation and h-frame experimental tests in Chapters 5, groups of mechanical clinches were applied at each connection node. In corresponding Finite Element tests after cold-formed steel components had been meshed with shell elements, special orthotropic clinch elements were applied at each clinch location, joining components together and allowing shear deformation in the connections.
Clinch elements joining cold-formed steel components in finite element tests were assigned stiffness properties of clinches established in clinch shear tests in Chapter 4. Two sets of clinch shear test data were used for each clinch element - load-displacement data at 0° and load-displacement data at 90°. The full non-linear load-displacement response of the clinch - elastic, plastic and large displacement - were used to define stiffness in the clinch element, as shown in Figure 3.7. The clinch element was a non-linear orthotropic stiffness element and had a specific orientation in the same way that clinches in parent metal have a specific rectangular orientation.

Figure 3.7 shows 0° and 90° non-linear stiffness data for the 1.5/1.2mm steel thickness configuration clinch element as an example of non-linear orthotropic clinch stiffness input. Average values from two shear test data streams were used to define orthotropic stiffness characteristics of the clinch elements. The positive load-displacement data set was mirrored in the x and y axes to give a similar load-displacement response to negative force in the clinch groups.

![Figure 3.7, Clinch element stiffness input, 1.5/1.2mm punch/die arrangement](image)

Each clinch element in a clinch group at a connection node can have a different magnitude of shear deformation at a given loading increment, depending on the level of force at 0° or 90° resolved into the clinch element over each non-linear increment in the analysis.
Special nodes in the finite element models representing the positions of the clinches in the experimental tests were created using the auto-meshing facility in the HYPERMESH finite element pre-processing program, as shown in Figure 3.8. The two components in Figure 3.8 have clinch nodal positions specified and clinch elements were mapped between the two corresponding positions on each component.

Figure 3.8, Meshing clinch nodal positions
4 MECHANICAL STRENGTH OF CLINCHING

By experimental investigation of clinching shear resistance characteristics in this Chapter and by making comparisons with other connection techniques, the current understanding of the structural shear resistance of clinching is reinforced and widened to include more test results on cyclic shear resistance and multiple layer connections. The focus of investigation is on in-plane shear capacity of mechanical clinching in cold-formed steel. In commercial framing systems such as Trichord [63], clinch connections join in-plane internal components. When framing elements are loaded, clinch connections are forced in direct shear from vectors of tension and compression in the frame.

The current data-base of clinching research test results includes:

- Static clinch shear and pulling out tests in similar steel thicknesses and a non-dimensional equation for predicting the shear capacity of H-type mechanical clinches from Wieck and other researchers at the Technical University of Hamburg [6, 30, 40 - 45, 55, 80]
- Static and cyclic shear resistance from Kolari’s work including load-displacement data and shear resistance analysis [38, 39]
- Static shear capacity data from past work at the University of Edinburgh including the shear capacity of mechanical clinches in similar and different combined steel thicknesses, in two three and four layers of steel [15, 16, 59, 60, 61, 62, 63]

The range of clinch shear test results in this chapter was designed to extend the past research at the University of Edinburgh and from other researchers through investigation of:

- Clinches in two and three layers of steel
- Clinch shear resistance under cyclic loading
- Variability of clinch shear capacity
- Comparison with the variability of the clinch material and of screw connections
- Comparison with shear test results on pop-rivets, self-piercing rivets and self-tapping screws

Existing equations for clinch peak load from past researchers are analysed and applied. Equations for predicting the shear resistance of a clinch in two layers of steel have been developed by Wieck [80] and Davies [15] in past research. A new equation derived from new
shear test results and the existing data-base at the University of Edinburgh is presented. Test results are normalised against maximum possible shear resistance based on the resistance of the steel tension link in the short edge of the clinch. Width of the clinch is taken into account in the new equation to allow the size of different commercial rectangular clinch types to influence prediction of shear capacity. The new equation is investigated in section 4.5.2 and it is shown to give a standard deviation of 14% on predicted clinch shear capacity between limits of steel UTS, steel thickness and angle of applied load considered.

4.1 Shear resistance of connections

The Shear resistance of connections is obtained from shear test load displacement plots in Appendix 1 by establishing the first point at which a significant change in connection stiffness occurred in the shear test. An example of the point of shear resistance is shown in Figure 4.1.

The ductile peak load is the maximum force reached in the connection shear test. In many cases the shear resistance and ductile peak load were coincident. However, for press-joins tested at 90° in two layers of 1.0mm and 1.2mm thick steel, the ductile peak load was reached after the point of shear resistance, as shown in Figure 4.1.

The ductile behaviour of connections is characterised by the changing resistance of the connections with continued deformation after the point of shear resistance in Figure 4.1 is reached.
Figure 4.1, Shear resistance and deformation capacity

Deformation capacity is the total ductile shear deformation energy in the connection. It is the force-displacement area beneath the force-displacement paths up to maximum displacement, as shown in Figure 4.1. Simpson's rule was applied in the connection shear resistance spreadsheets to establish the deformation energy in each connection, discussed in Section 4.4.3.

In full scale tests of Chapter 6, chord and internal members of structural load bearing cold-formed steel trusses are connected by overlapping layers of steel and connecting them at discrete overlapped positions. A typical connection position is between a diagonal tie in tension and a top or bottom chord. It is this type of overlapped connection loaded in shear that is critical to the success of a cold-formed steel truss design. This is where clinches can be compared against other methods of connection in cold-formed steel.

4.1.1 Shear resistance of pop rivets

Although the shear behaviour of pop rivets was irregular between similar samples, peak load generally increased with increase in steel thickness as shown in Figure A1.4.1. Peak shear capacity in comparison with other types of connection tested are low at between 2.5kN and
3.75kN. Pop rivet peak loads are also shown in Table 4.1. Shear deformations at failure are small at less than 1.5mm.

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>STEEL THICKNESS (mm)</th>
<th>AVERAGE UTS (N/mm²)</th>
<th>PEAK LOAD (KN)</th>
<th>AVERAGE PEAK LOAD (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>POP RIVET</td>
<td>1.0 / 1.0</td>
<td>359</td>
<td>2.50</td>
<td>2.51</td>
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<td></td>
<td>1.2 / 1.2</td>
<td>351</td>
<td>2.74</td>
<td>2.83</td>
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<tr>
<td></td>
<td>1.6 / 1.6</td>
<td>360</td>
<td>3.20</td>
<td>3.23</td>
</tr>
<tr>
<td></td>
<td>2.0 / 2.0</td>
<td>356</td>
<td>3.59</td>
<td>3.64</td>
</tr>
<tr>
<td>SELF TAPPING SCREW</td>
<td>1.0 / 1.0</td>
<td>359</td>
<td>2.74</td>
<td>2.91</td>
</tr>
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<td></td>
<td>1.2 / 1.2</td>
<td>351</td>
<td>3.58</td>
<td>3.75</td>
</tr>
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<td></td>
<td>1.6 / 1.6</td>
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<td>6.00</td>
<td>6.28</td>
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<td></td>
<td>2.0 / 2.0</td>
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<td>HENROB SELF PIERCING RIVET</td>
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<td>5.28</td>
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<td></td>
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<td></td>
<td>1.6 / 1.6</td>
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<td>8.90</td>
<td>9.11</td>
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<tr>
<td></td>
<td>2.0 / 2.0</td>
<td>356</td>
<td>11.43</td>
<td>11.27</td>
</tr>
</tbody>
</table>

Table 4.1, Cold-formed steel connections in two layers - peak loads

In the pop rivet tests samples failed by the rivet head sliding down the rivet shaft after some local deformation, with the exception of the 2.0mm steel thickness where in some cases the rivet shaft failed in shear. The external work-displacement graph in Figure A1.4.2 shows similar levels of displacement energy being used to fail pop rivet connections in different steel thicknesses, indicating a failure mode of local rivet twisting and rivet deformation.

4.1.2 Shear resistance of self-piercing rivets

The Henrob self-piercing riveting system is an advanced fastening technology developed and patented by Henrob Ltd. The system is available in the UK from Henrob UK Ltd. Henrob 5mm nominal stem diameter self-piercing rivet samples were tested in various thicknesses of steel.
The load displacement graph from the shear test on self-piercing rivets is illustrated in Figure A1.3.1 of Appendix 1. In a shear test when the parent metal begins to deform, the rivet component acts as a link between the shearing strips of metal causing local deformation. High peak load and ductility are achieved in comparison with all other join types tested. Initial stiffness was variable. Load displacement characteristics of the connection, as illustrated in Figure A1.3.1, were an initial elastic response up to approximately 50% of peak load, after which there was a non-linear large deformation range where the interlocking rivet and parent metal parts were pulled apart.

In Figure A1.3.2 force values in Figure A1.3.1 are integrated with respect to displacement to give external work used in the test in kN-mm at corresponding displacement stages. There was a large variation in the load magnitudes of load displacement graphs and a large displacement range of up to 5mm. Energy required to deform the connections was similarly high with a large variation between thicknesses of steel joined.

### 4.1.3 Shear resistance of self-tapping screws

SFS Stadler self-drilling and tapping screws serial numbers SD3-T16-6.3x25 and SD5-T15-5.5x25 were tested in shear. Characteristic failure modes for a connection with a connecting component such as a screw or rivet as illustrated in Figure 4.2 were:

(a) Turning of the shaft component as the eccentricity ‘e’ between the layers of steel created a moment ‘M’ at the intersection of the shaft and the layers of steel
(b) Bearing failure – crushing and tearing of the steel around the shaft
(c) Shearing through the shaft of the screw or rivet

Typical failure modes for the self tapping screw samples begin with a rotation of the screw as the load is applied followed by an irregular pulling out action, where the steel in the plates around the screw was deformed locally and dragged over the screw threads. This type of load displacement behaviour created high ductility but low initial stiffness, as illustrated in Figure A1.5.1 of Appendix 1.

The greater thickness of the 2.0mm steel layers gave resistance to the turning action caused by the eccentricity of the plates, shown in the significantly higher initial stiffness value for that thickness sample. The 2.0mm steel thickness caused shear failure of the screw shaft in some
cases. The displacement range of self-tapping screws was large at between 6mm and 15mm. In Figure A1.5.2 a wide variation of external work was required to fail screw connections among different sample thicknesses.

**Figure 4.2, Screw shear test failure modes**

A series of six similar shear tests were carried out on screw samples in thicknesses of 1.0mm and 2.0mm, to establish the standard deviation for the peak load values. The 2.0mm thickness samples had a standard deviation of 3.81% as shown in Table 4.2, which was higher than the standard deviation of the 2.0mm thick steel material test samples at 2.77% from Table 3.2 in Chapter 3.

Peak loads of the screw shear tests in 1.0mm thick steel had a higher standard deviation of 8.05% in comparison with the standard deviation of 1.0mm thick material samples of 2.73%. Clinch standard deviation of 3.9% for the 0° samples and 3.49% for the 90° samples are also lower than
the self-tapping screw standard deviations in 1.0mm thick steel. In Figure 4.3, the load displacement paths of the screw shear tests became irregular after the initial elastic displacement range because the thinner 1.0mm thick steel tore more easily as the screw threads dragged along the parent metal. There was also less stiffness in the thinner parent metal to resist local distortions.

Figure 4.3, Screw variability test load displacement paths in 1.0mm thick steel

<table>
<thead>
<tr>
<th>STEEL THICKNESS (mm)</th>
<th>NUMBER OF SAMPLES PER POPULATION</th>
<th>AVERAGE PEAK LOAD (DEGREES)</th>
<th>STANDARD DEVIATION (KN)</th>
<th>STANDARD DEVIATION %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>6</td>
<td>4.91</td>
<td>0.40</td>
<td>8.05</td>
</tr>
<tr>
<td>2.0</td>
<td>6</td>
<td>8.88</td>
<td>0.34</td>
<td>3.81</td>
</tr>
</tbody>
</table>

Table 4.2, Standard deviation for screw shear test peak loads
4.2 Shear resistance of mechanical clinches

When a single clinch was applied to layers of steel the greatest shear resistance of the clinch was in direct in-plane shear. A single clinch provided little resistance to in plane moment about the center of the connection - if a single clinch was twisted by applying an in plane moment the clinched parts began to separate and the shear resistance of the clinch was greatly reduced. By applying two or more clinches the applied moment was resisted by the coupled forces in the individual clinches in the group separated by a lever arm distance. When clinches were applied in groups of at least two the effect of concentrated moments being applied to individual clinches was avoided. Applying more than one clinch at a cold-formed steel connection node did not require additional fastening components as was the case with screws and rivets. By clinching the overlapped steel at the critical load bearing nodes with the force at 0° to the clinch the optimum shear resistance characteristics of the clinch were utilised.

The highly non-linear two-step clinching process and the orthotropic state of clinches affected the response of the connection to shear loading. The shear resistance of the connection was also dependent on the materials being joined and on the degree of quality control applied in the process. These shear resistance characteristics were investigated extensively by Kolari at the VTT Building Technology Research Center in Finland [38, 39] and by Davies et.al. at the University of Edinburgh. [15, 16, 59, 60, 61, 62].

Analysis of clinch shear capacity characteristics in this chapter was applied in the following chapters analysing the shear resistance of clinches in cold-formed steel applications. The influence of individual clinch shear resistance behaviour was analysed in groups of clinches in Chapter 5 and the influence of shear deformation on the behaviour of full scale trusses was investigated in Chapter 6.

4.2.1 Effect of material thickness

Clinch shear capacities listed in Tables 4.4 and 4.5 showed that the peak shear load of a clinch increased with the thickness of steel being joined. The effect of steel thickness on clinch peak shear loads was investigated in detail in section 4.6, ‘Predicting the shear capacity of a mechanical clinch’. The thickness of steel was incorporated in Equation 4.6 to give an equation with correct dimensions to predict the shear capacity of a clinch.
4.2.2 Effect of steel Ultimate Tensile Stress

Davies' clinch peak shear load equation 4.1 in Chapter 4 raised the UTS term to the power of 0.98, suggesting that the relationship between steel UTS and clinch peak shear load was very close to linear. The Pedreschi Trichord equation 2.1 did not have a power term associated with the UTS which also suggested a direct linear relationship. The UTS to peak load relationship in Wieck's equation 4.8 was less clear with a different power term being applied to predict clinch peak shear loads at 0° and 90°. Constants in Davies' and Wieck's equations were derived statistically - they did not use correct dimensions or reflect the physical shearing process directly.

In Section 4.5 the steel UTS term was incorporated into Equation 4.6 as a part of the clinch capacity normalising steps by carrying out a linear regression on the normalised clinch capacity data over a wide range of results.

4.2.3 Effect of orientation of mechanical clinch

In-plane shear resistance of clinches was affected by orientation of force applied to the connection. When a force was applied to a clinch in the direction parallel to the long slits on the, the connection opened along the slits and failed in an in-plane pulling out mode.

![Graph showing shear tests on mechanical clinches in 1.2mm thick steel](image-url)
Greater shear resistance was achieved by applying a force to the connection in the direction perpendicular to the long slits on the sides of the rectangular join, where greater deformation of the steel was needed to cause failure of the connection. In this case the failure mode was typically distortion and out of plane twisting of the clinch before shearing distortion of the steel connecting the plates caused failure. Figure 4.4 shows the load displacement response in shear tests on mechanical clinches in two layers of 1.2mm thick steel.

**Figure 4.5. The effect of the clinch orientation on the shear test peak load**

The perpendicular orientation of applied load direction to the long side of the rectangular connection is referenced as 0°. When the applied load is parallel to the long side of the rectangular connection the arrangement is referred to as 90°. Arrangements of applied load direction to clinch orientation can also be interpolated to any value between 0° and 90°. Figure 4.5 shows linear regression lines from peak shear loads from shear tests on mechanical clinches in four thicknesses of steel at angle orientations of 0°, 45° and 90°.

**4.2.4 Clinch deformation capacity**

Figures A1.2.1 to A1.2.8 of Appendix 1 show load-displacement paths and external work-displacement paths for shear tests on mechanical clinches at angles of 0°, 45° and 90°, in steel sheet thicknesses of 1.0mm, 1.2mm, 1.5mm and 2.0mm. Two tests were carried out on each
configuration. The characteristic load displacement response from a shear test on a clinch was an initial elastic linear force displacement relationship followed by a non-linear response as the applied load increased towards the maximum value, and a failure response as the parent components of the connection were deformed, as shown in Figure 4.6.

Figure 4.6, Clinch shear test failure modes

4.2.5 Shear resistance of S-type mechanical clinches in two layers

Peak shear loads from clinch shear tests in two similar thicknesses of steel are shown in Table 4.3. Clinches at 90° gave a more regular deformation response in the large deformation range in comparison with mechanical clinches at 0°, as shown in Figures A1.2.1 to A1.2.8 of Appendix 1. This was particularly true for the thinner steel layers of 1.0mm and 1.2mm. The 90° arrangement pulled apart at a smaller displacement and left small deformed parts of the connection in contact to be pulled apart for complete failure.
The data presented in the load-displacement graphs of Figures A1.2.1 to A1.2.8 is integrated with respect to displacement, to give the relationship between external work in units of kN mm applied to shear the connection. Figures A1.2.2 and A1.2.4 show that at a displacement value of between 1.0mm and 1.5mm, the rate at which energy was used in shearing the connection apart was greatly reduced for the samples at 45° and 90° orientations. Connections can be considered to have failed at this change in stiffness.

Load displacement graphs and corresponding integrated external work graphs in Figures A1.2.1 to A1.2.8 illustrate that mechanical clinches in the greater combined thicknesses of between 3mm (2 x 1.5mm) and 4mm (2 x 2mm) had a more regular shear displacement response with greater deformation capacity. Applied load remained higher for a longer period over the displacement range and sloped off more gently towards failure at the end of the test.
<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>ARRANGEMENT OF LAYERS</th>
<th>ANGLE OF APPLIED SHEAR (DEGREES)</th>
<th>AVERAGE UTS (N/mm²)</th>
<th>PEAK LOAD (PUNCH / DIE) (KN)</th>
<th>AVERAGE PEAK LOAD (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>'S' TYPE PRESS-JOIN</td>
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<td>0</td>
<td>352</td>
<td>2.60</td>
<td>2.04</td>
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Table 4.3 continued, S-type clinch in two layers - peak loads

4.2.6 Shear resistance of S-type clinches in three layers

Triple layer clinch samples were tested in two different layer arrangements as illustrated in Figure 4.7. Layer arrangement 1 was the typical arrangement of three layers of steel in a truss beam tie to chord connection in Chapter 5, where the tie forced the middle layer of steel against the outer layers in tension and compression. Steel thickness arrangements for shear tests were chosen to provide clinch shear capacity data for analysis of truss beam tests in Chapter 5.

Failure modes for layer arrangement 1 three layer samples involved lateral separation of outside layers, '11' in Figure 4.7 (a), and gradual pulling out of the middle layer, giving greater deformation capacity in comparison with two layer samples. In Figure 4.7 (a) for layer arrangement 1 there was no eccentricity along the line of forces.
Figures A1.2.9 to A1.2.11 of Appendix 1 show the load displacement paths for shear tests on three layer samples, arranged as shown in Figure 4.7 (a). Three arrangements of steel thicknesses of 1.0/1.0/1.0mm, 1.2/1.0/1.2mm and 1.6/1.0/1.6mm were tested at clinch orientations of 0°, 45° and 90°. Large displacement behaviour of clinches in three layer arrangements can be seen in the peak load being sustained over a longer displacement range in comparison with that of two layer samples. The elastic range is not clearly defined with a gradual change from initial linear displacement to the curved plastic large deformation range. This effect is particularly evident for the 45° and 90° samples. Peak loads from shear tests on three layer samples are shown in Table 4.4.

Figure 4.7, Layer arrangement of steel thicknesses in triple layer samples
Layer arrangement 2 had an eccentricity equal to $t_1 + t_2$ in Figure 4.7 (b). Turning and peening in the Instron shear tests with Lay-up 2 was observed and the failure of the connection was initiated by local deformation from bending. This defines layer arrangement 2 as weaker in shear in comparison with layer arrangement 1. Steel thickness arrangements tested were chosen to give analysis data for moment rotation tests in Chapter 5, peak shear loads for the layer arrangement 2 test samples are listed in Table 4.4.

4.2.7 Shear resistance of mechanical clinches in different thicknesses

Shear capacity of connections with a connecting shaft such as a rivet in thin gauge steel is generally determined by the thinnest layer of steel being joined, assuming the shaft of the connector does not fail. In forming a clinch steel on the punch side is deformed into the steel on the die side. This gives steel on the punch side a more significant influence in determining shear capacity. Failure of the clinch is initiated on the punch side as the punch side steel pulls out or tears, as illustrated in Figure 4.6.

Table 4.3 shows shear capacity of mechanical clinches in similar and different combination layer arrangements. Shear capacity of clinches in different thickness was more significantly influenced by thickness of steel on the punch side in comparison with the influence from the thickness on the die side. There is also a significant difference between the shear capacities of clinches in similar and different thicknesses, demonstrating an influence from the die side thickness in the shear capacities also.
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Table 4.4, S-type clinch in three layers - peak loads
Table 4.4 continued, S-type clinch in three layers - peak loads

4.2.8 Shear resistance of H-type clinches in two layers

The H-type clinch is an airtight connection that is formed by a slightly narrower punch part in comparison with the S-type clinch. A cross-section of S-type and H-type clinches is shown in Figure 4.9. In the case of the H-type clinch only the die component of the parent metal is fully sheared through. For the S-type clinch both punch and die parts are sheared through. This arrangement generally gives the H-type clinch greater initial stiffness and peak load over the S-type clinch, but with less ductility.

Figure 4.8 shows linear regression lines for H-type and S-type clinches against angle (°) applied load in 1.0, 1.2 and 1.5mm steel thicknesses. H-type clinches showed a higher shear capacity in comparison with S-type clinches in 1.0mm and 1.2mm steel thicknesses and H-type clinches had a significant shear resistance advantage in 1.2mm thick steel. When a comparison was made in 2.0mm thick steel S-type clinches showed a higher shear capacity.

Regression lines in Figure 4.8 gave a nominally higher shear capacity for H-type clinches in 1.2mm thick steel. However this does not hold for the same comparison at 45° and 90°. The form of H-type clinches with no full shear through both layers in Figure 1.9 gave it the shear capacity advantage in 1.0mm and 1.2mm thick steel but not in 2.0mm thick steel. Ductility was also lower in general for H-type clinches in comparison with S-type.
Figures A1.2.12 to A1.2.32 in Appendix I show shear test load displacement paths for similar steel thickness configurations of S-type and H-type type mechanical clinches. In all cases where the punch side thickness was less than or equal to 1.2mm H-type mechanical clinches showed higher initial stiffness and higher peak load in comparison with S-type clinches. Where the punch side thickness was greater than 1.2mm or was greater than the die side thickness peak load was similar in magnitude for H-type and S-types. In all cases the length of displacement over which a load close to the peak load was sustained was less for H-type clinches in comparison with S-type, demonstrating greater deformation capacity in the S-type clinch in comparison with the H-type.

Shear capacities from tests on H-type clinches are shown in Table 4.5. In H-type clinches, shear capacity in different thickness was influenced primarily by thickness of steel on the punch side of the clinch, as for the S-type clinch. In 1.5/1.0mm thick punch/die steel layers H-type clinches showed higher shear capacity at 0°, 45° and 90°.
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Table 4.5, H-type clinch in two layers - peak loads
### Table 4.5 continued, H-type clinch in two layers - peak loads

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#### 4.2.9 Variability of clinch shear resistance characteristics

A series of clinch shear tests were carried out on clinches at 0° and clinches at 90° in 1.0mm and 2.0mm thick steel. Six of each configuration were tested to examine the variability of test results on similar configurations. Table 4.6 shows the average peak load and coefficient of variation for each set of six tests.

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#### Table 4.6, Standard deviation of clinch peak loads

A comparison of Table 4.6 and Table 3.1 of Chapter 3 shows that the standard deviation of peak loads obtained for mechanical clinches in 2.0mm thick steel was low in comparison with
material tests yield stress standard deviation. As steel thickness was decreased to 1.0mm, variation of clinch shear capacity became greater in comparison with variation of yield stress.

Clinch shear capacity was less variable in comparison with screw peak loads, this was evident when comparing Tables 4.2 for screw variability with Table 4.6 for clinch variability. Standard deviation of screws in 1.0mm thick steel was 8.05% while mechanical clinches standard deviation was 3.9% at 90° and 3.49% at 0° in 1.0mm thick steel. Standard deviation of shear capacity of screws in 2.0mm thick steel was 3.81% while standard deviation for mechanical clinches at 0° was 1.5% and standard deviation of mechanical clinches at 90° was 2.55%.

4.3 Cyclic shear resistance of mechanical clinches

Cyclic testing of mechanical clinches was carried out to determine whether deterioration in the shear resistance of clinched framing systems occurs at connection nodes over the structures life cycle under cyclic loading. Cyclic tests were carried out over 10,000 cycles between zero and 50% of clinch shear capacity recorded in the static experimental tests. Load was cycled to 50% based on the assumption that a minimum global factor of safety of two on clinch failure load would be acceptable in terms of strength. Following the cyclic tests, a static test was carried out on each sample where the clinch was sheared apart and the shear resistance after cycling was measured. Cyclic test results are listed in Table 4.7.

<table>
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<tr>
<th>Nominal Thickness (mm)</th>
<th>Number of Punch Cycles</th>
<th>Angle (°)</th>
<th>Average Peak Shear (kN)</th>
<th>Cycle Amplitude (kN)</th>
<th>Post Cyclic Shear Capacity Test 1 (kN)</th>
<th>Test 2 (kN)</th>
<th>Ave Post Cyclic Capacity (kN)</th>
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</tbody>
</table>

Table 4.7, Clinch cyclic test results
Kolari in [39] also investigated the effect of cyclic loading on the shear resistance of mechanical clinches in two layers of steel. Rectangular mechanical clinches at 0° and 90°, circular mechanical clinches and screw samples were tested. A complex pattern of cyclic loading was applied to the samples. In Kolari's cycling phase the low load level in the cycle was 1/6th of shear capacity. In the cyclic tests in this work the low load level was zero. Kolari cycled samples between the low load level and 2/3rd of estimated peak load for 500 cycles, 1/2 of estimated peak load for 3,000 cycles and 1/3rd of estimated peak load for a further 15,000 cycles. Kolari's complex cyclic loading pattern was intended to represent the typical loading history of a cold-formed steel frame over 100 years subjected to wind loading. Kolari concluded in [39] that cyclic loading had only a negligible effect on shear capacity of clinches when tested after cycling.

4.3.1 Cyclic resistance of S-type mechanical clinches in two layers

The effect of repeated loading on shear capacity of clinches was investigated using the Instron testing machine to cycle samples from zero load to 50% of shear capacity over 10,000 cycles. Samples were set up in the Instron testing machine in the same way the shear test samples were set up. Special software was used to apply cyclic loading in the Instron testing frame. When the cyclic loading phase was complete clinches were tested to failure with a similar testing procedure to the procedure applied in previous static shear tests. Figures A1.2.33 to A1.2.41 of Appendix 1 show a comparison of clinch samples tested to failure without cyclic loading and similar samples tested after 10,000 cycles to 50% of shear capacity.

The strengthening effect of cycling samples through 10,000 cycles at 50% shear capacity that occurred in most cyclic tests can be attributed to steel hardening at the interface between steel layers in the clinch. The join was formed under high pressure and was locked by plastic straining of steel in a controlled process, so that the interface parts of the connection were in tight contact with elastic and plastic residual stress in the deformed parts. Applying a force that causes straining close to the plastic limit of the connection over many cycles hardened the steel in the connection in discrete areas of the deformed parts, without causing the connection to fail. During cyclic testing there was a very small but noticeable amount of looseness in connections after tests and this was the result of squashing and hardening of steel in small areas. In test cases where static shear capacity of the samples decreased after cyclic loading, cyclic loading on
the clinch caused enough plastic straining at the material interfaces in the interlocked clinch steel to allow layers to be sheared apart with less force.

Figure A1.2.33 shows load displacement paths of clinch samples in steel thicknesses of 1.2/1.2mm at 0° orientation. Cycled samples showed a higher initial stiffness, a slightly higher peak load at 2.52kN, and a longer displacement range in comparison with the equivalent static shear tests, indicating slightly higher ductility. This shear resistance comparison was also applicable for the clinch cyclic tests at orientations of 45° and 90° in Figures A1.2.34 and A1.2.35, with the exception of one of two cycled samples in Figure A1.2.35, which were damaged in testing.

Test results in Figures A1.2.36 to A1.2.38 showed a similar pattern of shear strengthening in clinch connections in steel thicknesses of 1.5/1.5mm, except for the samples at 90° which did not show any gain in shear capacity. Test results in Figures A1.2.39 to A1.2.41 on clinches in steel thicknesses of 2.0/2.0mm again showed a higher initial stiffness and peak load. There was a small reduction in ductility, which was greatest in the 45° and 90° orientation samples.

4.3.2 Cyclic resistance of S-type mechanical clinches in three layers

Results of cyclic loading tests on three layer clinch samples is illustrated in Figures A1.2.42 to A1.2.48. In Figures A1.2.42 to A1.2.44 the three layer arrangement was 1.6/1.0/1.6mm, arranged as shown in Figure 4.12 (a). The characteristic higher initial stiffness and shear capacity for the cycled samples was apparent for the 0° and 45° connection orientations in Figures A1.2.42 and A1.2.43.

The 90° sample in Figure A1.2.44 showed a higher initial stiffness but a significantly lower peak load. Cycling of the middle layer against the two outer layers, when the force was parallel to the sheared slits of the mechanical clinch, caused plastic weakening of the connection below its potential peak load when it was tested to failure. The result of cyclic loading tests on the 1.2/1.2/1.2mm three layer arrangement is shown in Figures A1.2.45 to A1.2.47. Here the higher initial stiffness for cycled samples was present for all samples and a higher peak load for the 0° sample in Figure A1.2.45 was reached. Reduction in peak load for the 45° and 90° samples was small and overall ductility remained similar for cycled samples.
4.4 Comparison of connection techniques

Figure 4.10 shows three observed characteristic failure modes of screw and rivet thin gauge steel connections. The shaft of the connecting component was central to all three modes of failure. Clinches on the other hand did not have a connecting component. Factors giving a clinch shear resistance and the modes of failure associated with clinching were different to those of screws and rivets. Characteristic failure modes of clinches oriented at 0° and 90° are illustrated in Figure 4.15. Mechanical clinches were strong because steel in separate layers being connected was deformed and geometrically interlocked.

4.4.1 Elastic stiffness resistance characteristics

Figures 4.17 and 4.18 show the elastic response of connections in cold-formed steel of thickness 1.0mm and 2.0mm respectively. In both cases mechanical clinches applied at 0° showed a significantly higher elastic stiffness in comparison with other connection types tested. This shear resistance characteristic gave mechanical clinching a clear advantage as the method of connection in framing systems. If the framing system and fabrication process was designed to have rectangular mechanical clinches applied at 0° to applied load vectors in the cold-formed steel components, the framing system will have been designed for the minimum of elastic deformation under serviceability loading.

Elastic response of screws in 1.0mm thick steel in Figure 4.17 was particularly low, the elastic stiffnesses of mechanical clinches at 45° and 90° in 1.0mm thick steel were comparable to the elastic stiffnesses of Henrob self piercing rivets. In the thicker 2.0mm steel layers shown in Figure 4.18 the elastic response of mechanical clinches at 0° and 45° below 0.01mm deformation was higher in comparison with all other connection types. Only the screw sample below 0.01mm deformation showed a higher elastic stiffness in comparison with the clinch oriented at 0°. At 45° the elastic response below 0.1mm was similar to the elastic response of the screw sample and the elastic response of the clinch at 90° was similar to the elastic response of the pop-rivet.
4.4.2 Shear resistance

A direct comparison of clinch shear capacities and the shear capacities of Henrob self-piercing rivets, pop rivets and self-tapping screws in two layers steel is made was Figures 4.19 and 4.19. In Figure 4.19 connections were tested in 1.0mm thick steel. Clinches achieved slightly lower peak loads in comparison with pop rivets and self-tapping screws with clinch shear tests giving peak loads of 1.97kN, 1.69kN and 1.37kN at orientations of 0°, 45° and 90° respectively. Henrob self-piercing rivets achieved the highest peak load in two layers of 1.0mm thick steel at 5.38kN. Peak loads of self-tapping screws and of Henrob self piercing rivets were reached at displacements greater than 1mm while the peak loads at the end of the elastic response of clinch samples were reached below 1mm shear deformation.
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Figure 4.18, Elastic response of connections in 2.0mm thick steel

Load-displacement data from shear tests on connections in two layers of 2.0mm thick steel are shown in Figure 4.20. Shear capacities for mechanical clinches at 5.15kN, 4.15kN and 3.09kN for 0°, 45° and 90° orientations respectively were significantly lower than the peak load of Henrob self-piercing rivets and self-tapping screws at 11.43kN and 9.41kN. Again peak loads were reached at greater shear deformations in comparison with clinch samples. Peak load from the pop-rivet test sample at 3.59kN was low in comparison with clinch peak loads at 0° and 45° orientation.

The Shear resistance of connections in cold-formed steel framing systems affects limit state behaviour. In addition to direct axial forces, concentrated leverage forces develop in groups of connections when moment is applied to the cold-formed steel components. For frames connected by clinching to achieve the equivalent limit state shear resistance of frames connected by riveting techniques such as the Henrob self-piercing rivet and by self-tapping screws, approximately twice as many mechanical clinches need to be applied at each connection node. As no additional connecting component is required in applying additional clinches this is easily
achieved. Clinching equipment requires only a small offset movement to apply an additional connection.

4.4.3 Deformation capacity

Clinches form a connection by inter-locking the layers of the parent material and unlike screw and rivet connections there is no connecting component. This causes a difference in the deformation behaviour of mechanical clinches in comparison with other connectors—deformation capacity is obtained by the clinched parts of the connections being forced to separate by reversing plastic deformation that was applied to form the connection.

Rivets and screws showed deformation capacity because the material adjacent to the shaft of the connecting component deforms and the parent materials separate from the connecting shaft. The difference in deformation capacity between clinches, screws and rivets can be seen in load displacement graphs in Figures A1.2.1, A1.2.3, A1.2.5, A1.3.1, A1.4.1 and A1.5.1 and in the
external work graphs in Figures A1.2.2, A1.2.4, A1.2.6, A1.3.2, A1.4.2 and A1.5.2 in Appendix 1.

Figure 4.20, Load-displacement response of connections in 2.0mm thick steel

External work done by each type of connection in a shear test was read from the integrated displacement work energy graphs and is listed in Table 4.8 in kNmm. All clinches in 1.0mm thick steel used more deformation energy in comparison with the corresponding clinches in 1.2mm thick steel because of the tearing mode of failure associated with clinches in thinner material as shown in Figures A1.2.1 and A1.2.2. Extended deformation behaviour occurred after load-bearing failure. Clinches at 45° in Figures A1.2.7 and A1.2.8 used the greatest energy in 2.0mm thick steel because of the skew tearing mode of failure.

High levels of deformation energy used by screw samples in Figures A1.5.1 and A1.5.2 can be attributed to the pulling out mode of failure where the threads of the screw are dragged across the hole in the steel. Henrob self-piercing rivets in Figures A1.3.1 and A1.3.2 also show high
deformation energy usage because of the large deformation of steel required to unlock the rivet component from the connection.

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<th>STEEL THICKNESS (mm)</th>
<th>AVERAGE PEAK LOAD (kN)</th>
<th>AVERAGE EXTERNAL WORK (kN mm)</th>
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Table 4.8, Connection deformation capacity

4.5 Predicting the shear resistance of a clinch

Methods of predicting the shear resistance of connections are essential to the design of cold-formed steel framing systems. The number of connections at connection nodes and the general arrangement of the framing system can only be optimised when there is a method of predicting the shear resistance of connections and components. To produce a standardised method of shear capacity of a clinch using correct dimensional units, and incorporating the effect of steel thickness, shear resistance of the clinch must be related to actual steel thickness without incorporating a power term. Incorporating the width of the punched part of the clinch, it was
possible to produce a clinch shear capacity predictor equation generic across different sizes of commercial types of clinch.

Clinches are formed by plastic deformation of steel in shearing and press-locking actions. Non-linear plastic material behaviour locks the join. This causes non-linear load-displacement behaviour in the separation of the layers of steel when shear loading is applied. Shearing of two slits along the long parallel edges of a clinch and deformation of punch and die parts allows overlapping of the punch side layer or layers of material on top of the die side. A reversal of order in the layers of steel is imposed within the clinch rectangular boundary with the punch side layer of material locked below the die side. Shear resistance in the connection is achieved by locking after initial shearing. This creates a compression reaction between the steel layers along the sheared slits when external shear is applied, and a tension force at one or both short edges of the rectangle in the die side material, depending on the angle of applied loading.

With loading applied at 0° to the mechanical clinch, parallel to the short edge of the rectangle, both short edges of the material on the die side are in direct shear. When loading is being applied at 90°, parallel to the long edges of the clinch, only one edge of the die side material is significantly forced in tension. A shear deformation and pulling out mode at 90° also exists that is constrained when loading is applied at 0°. Shear deformation can occur in the long edges of the clinch when the load is applied at 90° because load is being applied parallel to the long edge.

The shear deformation mode of displacement increases the shear resistance of the clinch linearly from a minimum at 90° to a maximum at 0° when no long edge deformation is allowed. Highest shear capacity is achieved when load is applied at 0° and the two short edges of the rectangle on the punch side of the material are in shear. The two short edges in the punch side material are shown as dashed red lines in Figure 4.9. Lowest shear capacity is reached when the load is applied at 90° - shear deformation occurs along the long edges and only one short edge is forced in tension.

4.5.1 Normalised clinch shear capacity

Equation 4.4 was developed by establishing a dimensionless normalised shear resistance factor for each clinch tested, each experimentally measured clinch peak force was divided by the
maximum possible shear resistance over the range of results from the University of Edinburgh. A linear regression was then carried out on the dimensionless normalised shear resistance. Equation 4.6 is based on the normalised shear resistance and includes terms for:

- Peak clinch shear resistance: $F_p$ (N)
- Maximum steel tearing strength: $F_{MAX}$ (N) - based on the steel UTS
- Angle of applied loading: $\theta$ (degrees)
- Width of clinch: $w$ (mm)
- Punch side thickness: $t$ (mm)
- Steel ultimate tensile strength: UTS (N/mm$^2$)

Equation 4.1 was developed by Davies in [15] by normalising the peak shear clinch shear resistance in terms of steel thickness and steel UTS:

$$\text{Normalised shear resistance} = \text{shear resistance} + (\text{UTS}^m \cdot t^0)$$

A linear regression was carried out and power parameters were adjusted to fit the result of the linear regression. This gave a formula for predicting the peak shear resistance of the Eckold mechanical clinches of the type over the range of thickness and angle variables tested by Davies:

$$\text{Peak load} = (5.6309 - 0.0265 (0)) \times (\text{UTS}^{0.98} \times \text{thickness}^{1.45})$$

...Equation 4.1

There are different commercial mechanical clinching systems however and the main variable between different types of clinch that can be used as a measure of influence on shear capacity is the width of the short edge.

The maximum possible shear resistance of a clinch can be established by considering the greatest force sustainable by two strips of steel transferring clinch force between layers when the clinch is loaded at $0^\circ$. The width of each strip of steel is equal to the width of the mechanical clinch, as shown in Figure 4.9, and the thickness is equal to the thickness of the steel on the punch side. With two critical strips in each clinch the greatest possible shear resistance is equal to twice the thickness times the width times the material UTS:
By considering the maximum shear force sustainable by a clinch loaded at a specific angle between 0° and 90°, $F_p$, and comparing the peak shear resistance measured experimentally, the performance of the clinch can be measured by normalised peak shear resistance:

$$F_{\text{MAX}} = 2 \cdot t \cdot w \cdot UTS$$

...Equation 4.2

In the normalised peak shear resistance width, thickness, UTS and angle of loading are variables. The width of clinches tested in this work is 4mm in all cases. Applying a linear regression over the clinch peak shear results established by Davies, Pedreschi and Sinha [15, 16, 59, 60, 61, 62] and the clinch peak shears in this work produced the following clinch normalised shear resistance equation for $F_p/F_{\text{MAX}}$:

$$F_p/F_{\text{MAX}} = (0.816 - 0.0033 \cdot \theta), \text{ for } 0^\circ \leq \theta \leq 90^\circ$$

...Equation 4.4
The relationship between normalised peak load and angle of applied shear is illustrated in Figure 4.10. Rearranging and combining equations 4.2, 4.3 and 4.4 gave a general equation for clinch peak shear force:

$$F_p = (1.63 - 0.0067 \cdot \theta) \cdot w \cdot t \cdot \text{UTS}$$

...Equation 4.5

Substituting $w = 4\text{mm}$ based on the Eckold clinch investigated in this work for the width term in Equation 4.5 to establish a clinch shear capacity equation in angle of applied loading, steel thickness and ultimate tensile resistance gave:

$$F_p = (6.53 - 0.027 \cdot \theta) \cdot t \cdot \text{UTS}$$

...Equation 4.6
Increasing the data range from test results recorded at the University of Edinburgh to include data from Bober, Gopfert, Leibig and British Steel as listed in Tables 4.11, 4.12, 4.14, 4.15, 4.16 and 4.17, excluding Wiecks's H-type clinch tests in Table 4.13 gave an equation with similar constant parameters to Equation 4.6:

\[ F_p = (6.30 - 0.022 \theta) \cdot t \cdot UTS \]

...Equation 4.7

### 4.5.2 Analysis of equation 4.6

Distribution of shear capacity predicted with equation 4.6 from experimentally measured peak shear forces recorded by different researchers is plotted in Figure 4.11. Data plotted in Figure 4.11 is listed in Tables 4.11 to 4.17 for each researcher's results. Peak shear forces in two layers of steel are based on a wide range of clinch thicknesses and angles of applied loading. Constants in Equation 4.6 were generated using clinch shear capacity data from 'Davies' and 'Current' data in Figure 4.11.

![Figure 4.11, Equation 4.6 - predicting experimental results from a range of researchers](image-url)
Figure 4.11 shows the spread of forces predicted using Equation 4.6 against experimentally measured forces and Table 4.9 gave an overall standard deviation for the data in Figure 4.11 of 14%, excluding Wieck’s H-type clinch data. Values of standard deviation of experimental results and predicted results using equations 4.1, 4.6 and 4.8 are listed in Table 4.9 for the total range of results and also the range of results established by different researchers. The low standard deviations for predicted clinch resistance based on experimental results recorded by British Steel for all three equations in Table 4.9 are due to consistent experimental results being obtained over a small number of tests.

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Table 4.9, Standard deviation of predicted clinch capacities from experimental

Comparison with Davies' Equation 4.2

Table 4.9 shows an increase in accuracy in predicting clinch shear resistance for the current Equation 4.6 over Equation 4.1 in all cases. This pattern is reflected in Figure 4.12 – while there was overall variability in predicting clinch shear resistance, results predicted using equation 4.6 were closer to the ‘predicted = experimental’ line. The standard deviation of 96.2% in Table 4.9 in applying Davies’s S-type equation over Wieck’s H-type clinch results suggested that the use of power terms in Davies’ equation caused it to give less accurate results where input variables were at the upper or lower margins of the range.
Comparison with Wieck’s Equation 4.8

Wieck’s clinch shear resistance Equation uses constant power terms on steel thickness and steel yield stress and ultimate tensile stress to predict shear resistance of a H-type clinch. Constants C, a, b and were defined by the direction of applied loading and were given in [80] and are listed in Table 4.10 to predict the shear resistance of a clinch at 0° and 90°, and also the resistance in out of plane pulling apart.
\[ F_m = C a_0^a R_m^b \left( \frac{1}{Stv} \right)^c \]

where:

- \( C, a, b \) and \( c \) are constants
- \( a_0 \) = thickness
- \( R_m \) = ultimate tensile strength
- \( R_p \) = proof stress
- \( Stv = \frac{R_p}{R_m} \)

...Equation 4.8

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<th>C</th>
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<th>b</th>
<th>c</th>
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Table 4.10, Constants in Wieck's Equation 4.8

Clinch shear capacities predicted by Wieck's Equation 4.8 are plotted in Figure 4.12. Where the steel proof stress data was not available, the \((1/Stv)\) part of equation 4.8 was taken as 1.2. Using Wieck's Equation, predicted shear resistance in Figure 4.12 vary more widely from experimentally measured values in comparison with shear resistance predicted with Equations 4.1 and 4.6. Table 4.9 gives Wieck's Equation 4.8 an overall standard deviation of 27%, compared to 14% for the current Equation 4.6. Wieck's Equation was effective however in predicting results by Gopfert and British Steel. The fact that Wieck was working to establish parameters for H-type mechanical clinches accounts for the variability of the results when the equation is applied over the current range of experimentally measured shear capacities.

Wieck's predicted clinch capacities are plotted in Figure 4.11 and 4.12 but excluded from the calculations of overall standard deviation in Table 4.9. Standard deviations were distorted by Wieck's predictions based on H-type clinches and Davies' overall standard deviation was significantly increased when Wieck's results were included.
Variability of clinch shear resistance in Equation 4.6

The standard deviations of clinch resistance listed in Table 4.6 for clinches at 0° and 90° in 1.0mm thick and 2.0mm thick steel shows that the variability of clinch resistance for clinches in 1.0mm thick steel was higher than the variability of clinch resistance in 2.0mm thick steel. The standard deviation of 1.5% for clinches in 2.0mm thick steel at 0° was particularly low. This is also evident in Figure 4.13, where the variation from the ‘predicted = experimental’ line for the highest clinch resistance results at 0° is lowest.

![Figure 4.13, Clinches at 0° and 90° predicted by Equation 4.6](image)
Characteristic clinch resistance

From Table 4.6 the average standard deviation of clinch resistance at 90° was 3.225% and the average standard deviation of clinch resistance at 0° was 2.495%. Taking the characteristic value of clinch resistance as the average resistance less two standard deviations for both the 0° and 90° orientations, the following characteristic resistance equation is a modification of the average resistance Equation 4.6:

\[ F_p = (6.11 - 0.028 \cdot 0) \cdot t \cdot UTS \]

...Equation 4.9
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<th>Thickness (mm)</th>
<th>Angle (°)</th>
<th>Max shear strength $F_{max}$ (KN)</th>
<th>Experiment shear strength $F_p$ (KN)</th>
<th>$F_p/F_{max}$</th>
<th>Predicted Current Equation 4.6</th>
<th>Predicted Davies' Equation 4.1</th>
<th>Predicted Weick's Equation 4.8</th>
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Table 4.11, Current experimental clinch shear capacity test results
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<th>Angle (°)</th>
<th>Max shear strength $F_{\text{MAX}}$ (kN)</th>
<th>Experimental $F_p/F_{\text{MAX}}$</th>
<th>Predicted $F_p/F_{\text{MAX}}$</th>
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Table 4.12, Davies' experimental clinch shear capacity test results
### Table 4.13, Wieck's experimental clinch shear capacity test results

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### Table 4.14, British Steel's experimental clinch shear capacity test results

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Table 4.15, Bober's experimental clinch shear capacity test results

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Table 4.16, Leibig's experimental clinch shear capacity test results
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#### 4.6 Chapter summary and conclusions

Clinching machinery and the clinching technique have been researched and developed extensively in the 1980’s and 1990’s by a series of PhD. researchers at the Technical University of Hamburg. In recent years research into mechanical clinching in cold-formed steel framing systems has also been carried out at the VTT Building Technology center in Finland and at the University of Edinburgh, including this research.

Shear resistance characteristics of the Eckold S-type clinch are at the center of investigation in this research. The S-type clinch is a suitable type of clinch for general application in cold-formed steel framing systems showing good shear capacity, ductility, cyclic response and...
corrosion resistance. The H-type clinch is similar to the S-type clinch except that only the die side layer of steel is sheared through, leaving an air-tight seal on the punch side.

Elastic stiffness characteristic of clinching in cold-formed steel affects the response of a framing system to serviceability loading. Clinches in 1.0mm and 2.0mm thick steel showed high elastic stiffness in comparison with pop-rivets, self-piercing rivets and screws. Elastic stiffness of clinches was considerably greater than elastic stiffness of screws in all test cases. While clinch peak loads were generally low in comparison with the other connection techniques investigated, a distinct advantage of clinching was the ease of applying multiple clinches at a node.

Equation 4.6 has been developed using as input current clinch shear capacity test results and past research results from the University of Edinburgh. The equation was developed by establishing a normalised clinch shear capacity using steel UTS, thickness and width as variables. The dimensions used in arranging the Equation (N/mm^2 + (mm x mm)) are consistent with the output shear capacity units (N). A linear regression was carried out on the normalised shear capacity giving an Equation for clinch shear capacity.

The rosette clinch is a circular clinch connection larger in size compared to a rectangular mechanical clinch. Steel sheets to be connected by a rosette clinch connection must be prepared with a die-side hole and punch-side sleeve before the sleeve is folded back through the hole to form the clinch.

The following conclusions were drawn from the clinch shear resistance investigation:

1. Variability of clinch shear resistance was small across samples. Average standard deviation of clinch peak loads in 1.0mm thick steel was 3.7% and standard deviation of clinch peak loads in 2.0mm thick steel was 2.0%. Standard deviation of steel UTS used in connection tests was 2.7% and standard deviation of screw connection peak loads in 1.0mm thick steel was 8.1%.

2. Peak shear resistance of a clinch varied linearly from a maximum under 0° applied loading to a minimum at 90° applied loading.

3. Shear tests on H-type clinches showed higher initial stiffness and peak load in comparison with the S-type, but with less ductility. Peak shear force of H-type mechanical clinches was
high in comparison with the S-type in 1.0mm and 1.2mm steel thicknesses but less in the 1.5mm steel thickness. At 90° a higher shear capacity was observed in H-type clinches in 1.2mm thick steel in comparison with peak shear forces of H-type and S-type clinches in 1.5mm thick steel.

4. To apply additional clinches no additional connecting component was needed such as a rivet or screw. The suitability of clinching to automated process fabrication allows multiple clinches to be applied at a connection node. By an adjustment of process machinery the shear capacity of a clinch connection node in thin gauge steel can be increased significantly.

5. The database of clinch shear capacities has been extended to include clinches in two layers of different steel thicknesses. Shear capacity of clinches in two layers of different thicknesses was influenced greatly by the thickness of the steel layer on the punch side and to a lesser extent by the thickness of steel on the die side of the clinch.

6. Clinches in three layers of steel gave similar shear capacities to clinches in two layers for a given combined thickness. Different failure modes in the clinch were observed in different arrangements of applied shear in the layer arrangements in three layer samples. Where two outer layers were sheared in the opposite direction to a central layer, no in-plane eccentricity in the sum of the line of forces existed. Where two outer layers were sheared in opposite directions and the middle layer was held by outer layers, eccentricity of in-plane shear forces caused additional local deformation of the steel and turning of the clinch, reducing shear capacity.

7. Test results on fatigue resistance of clinches are less common in past research investigations in comparison with static tests. By cycling clinch shear test samples to 50% of shear capacity over 10,000 cycles, it has been demonstrated that there was no loss of shear resistance in the connection. Strain hardening of the interlocked clinch steel caused a small increase in initial stiffness and also a small increase in peak load. Ductility of the clinch was not significantly affected by cyclic loading at 50% shear capacity.

8. Equation 4.6 gave a standard deviation of 14% when applied over a wide range of clinch shear capacity results from different researchers, nearly twice as low as predictions made with equations from past research.

9. The series of shear tests carried out on clinch samples in two and three layers demonstrated how well clinching can compete in terms of shear resistance and ductility with methods of connection used more commonly such as rivets and screws.
5 MOMENT CAPACITY OF CLINCH GROUPS

Load-displacement behaviour of mechanical clinches was developed in this chapter applying the direct relationship between applied load and shear deformation in Chapter 4 to the non-linear behaviour of symmetrical groups of mechanical clinches subjected to in-plane moments. In [17 and 61] Davies et al. investigated moment rotation behaviour in groups of clinches in cold-formed steel. Samples were tested in groups of two, four and six, in 1.6 and 2.0mm thick steel. A selection of Davies' test results are listed in Table 5.1. An incremental computer method was developed to analyse the moment rotation behaviour of groups of mechanical clinches. Increments of rotation were applied to the connection group. The corresponding shear deformation in the appropriate orientation of each clinch was determined at each increment. Forces corresponding to the deformation were obtained from an idealised clinch force-deformation relationship allowing non-linear stiffness of each clinch to change in response to the force being applied to the clinch through leverage in the connection group. Shifting of the center of rotation and redistribution of forces were taken into account. Moment was obtained at each increment by multiplying the force by the distance from each clinch to the centroid of the group and summing for all the clinches in the group.

Davies' clinch non-linear stiffness was defined by four parameters:

1. Initial stiffness
2. Plastic limit
3. Peak load
4. Unloading stiffness

The incremental procedure was carried out until the clinches reached the plastic limit of displacement and began the unloading phase. Davies concluded in [17] that a good prediction of moment rotation behaviour was achieved by considering the contribution of the shear deformation of individual clinches in the clinch group. Davies also concluded that predicted peak moments became increasingly smaller in comparison with experimentally recorded peak moments as the number of clinches in the group was increased. In this work the range of Davies moment-rotation tests listed in Table 5.1 were tested and a new set of tests were carried out on clinch groups in three layers of steel.
In a series of full-scale experimental tests, Davies connected a horizontal c-section to two upright posts with groups of four and six symmetrically arranged clinches. Finite element tests predicting experimental results of Davies’ H-frame tests were carried out in Section 5.2.

A greater number of uncertainties exist in cold-formed steel design in comparison with design of thicker steel sections in framing systems. In thin gauge steel the number of failure modes is greater — parts of flat sections buckle locally in a greater number of modes and require a higher degree of restraint against buckling. Connection groups subjected to moment and axial force in cold-formed steel are more likely fail by local deformation and tearing of the steel around the connections. For thicker steel sections modes of connector failure can be bounded to failure in shear if steel surface interfaces at the connection group remain flat.

In BS5950 Part 5 [7] Section 8.1, design guidance for the effectiveness of groups of connections resisting moment between cold-formed steel elements is given in two sections. The designer decides whether a connection group is intended to perform as a rigid or semi-rigid node:

BS5950 Part 5 [7] Section 8.1.4 ‘Joints in Rigid Construction’ states:

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<th>CLINCH SPACING</th>
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</table>

Table 5.1, Selection from Davies’ moment-rotation test results
'Joints between members in rigid construction should be capable of transmitting the forces and moments calculated from the design method. For elastic design the rigidity of the joint should not be less than that of the members. For plastic design the moment capacity of a joint at a plastic hinge location should not be less than that of the member. In addition the joint should possess sufficient plastic rotation capacity'.

And BS5950 Part 5 [7] Section 8.1.5, 'Joints in Semi-Rigid Construction' states:

'Joints between members in semi-rigid construction should provide a predictable degree of interaction between members, as described in 2.1.2.4 [of BS5950 Part 5]. They should be capable of transmitting the restraint moments in addition to the other forces and moments at the joints. It is important that the connection is neither too rigid nor too flexible to fulfil accurately the assumptions made in the design. If the design strength of the connection is less than that of the connected members, it should be demonstrated that the deformation capacity of the connection is sufficient for full redistribution of load up to the relevant limit state to take place'.

In the experimental moment rotation-tests in Section 5.2 connection groups were intended to be semi-rigid. The aim of the moment testing was to investigate rotational behaviour of groups of clinches under applied moment. Moment-rotation tests were designed with lateral restraint to the cold-formed steel c-sections to fail the connection groups before the c-sections buckled. In the majority of tests failure of the clinch group was achieved without significant warping of the cold-formed steel. In a small number of tests listed in Table 5.2 clinches sustained moment while the sections warped and the lateral restraint to the c-sections in the testing frame splayed apart, giving an effectively rigid connection group response.
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<th>EXPERIMENTAL MOMENT CAPACITY</th>
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Table 5.2, Moment-rotation experimental test results

Shear resistance of clinching has been investigated in Chapter 4 of this work and by the other researchers [15, 16, 38, 39, 59, 60, 61, 62]. Section 5.1 aims to extend the established data-base of clinch shear resistance into an investigation of multiple clinch behaviour by analysing rotational behaviour of groups of clinches subjected to in-plane moment. Cold-formed steel framing systems use groups of clinches at the connection nodes to transfer bending moments between components.

Non-linear behaviour of a group of six clinches was analysed in detail in Section 5.2 in an ABAQUS cantilever finite element model including the non-linear clinch shear resistance input. The investigation provided information on the non-linear response of groups of clinches. Two types of non-linearity arising from clinch shear resistance characteristics were observed. The
first type of non-linearity was the non-linear curved shear-displacement characteristics of the individual clinch that directly affected axial and moment response. The second was redistribution of forces and the shift in the center of rotation when one or more clinches in the group failed ahead of the remaining mechanical clinches, leaving a connection group with a lower number of effective connections.

In section 5.3 experimental tests carried out by Davies in [16] were investigated with finite element tests incorporating the clinch shear resistance model in c-sections connected with clinches to form a H-frame.

5.1 Moment-rotation tests

In the experimental clinch moment-rotation tests four clinches were applied to two c-sections back to back in 16 tests with a third layer in between in 8 of the tests. In-plane moment was applied to the clinch group as illustrated in Figure 5.1.

![Figure 5.1, Moment-rotation test set-up](image)

Buckling of the c-sections in the moment-rotation tests occurred in many cases because the moment sustained by the clinch group was greater than the buckling moment capacity of the c-sections in the simply supported clinch group test set-up.
To predict peak moment in the experimental tests in this work two methods were used: an analytical method and a finite element method. By analysing the moment-rotation test data it has been demonstrated how much bending capacity in the elastic and plastic ranges of response can be generated at such a connection node, and how spacing of the connections affected structural behaviour.

Points of special interest in the moment-rotation tests were:

- The stiffness relationship of applied moment to rotation of the clinch group
- Shear resistance behaviour of different steel thickness combinations of clinch groups
- Effect of different spacing arrangements
- The extent to which the experimental test reflected the intended theoretical testing arrangement
- The magnitude of principal, 0° and 90° force components in the clinches at the peak applied moment in comparison with potential peak force components
- Comparison of experimental and finite element moment-rotation tests

5.1.1 Test apparatus and procedure

A 1m length rolled channel of approximately 10mm thickness was modified to provide the base of the testing frame as shown in Figure 5.1. The channel section was bolted securely to the base of the Instron tensile testing machine.

Clinch group spacing

Cold-formed steel c-sections used for moment-rotation tests were cut from steel sheets of different thicknesses and folded into c-sections 250mm in length and 100mm in height with 50mm flange width. Two c-sections were placed back to back with an overlap of 100mm and clinches were applied within the 100mm overlap region, as shown in Figure 5.2. In 8 tests a third layer of steel 100mm in length and 100mm wide was sandwiched between the two c-sections in the overlap region giving a triple layer clinch group test.
Figure 5.2, Clinch group before testing

Testing frame

The 1m length steel channel was bolted down with the web to the bottom on the base of the Instron testing fame as shown in Figure 5.3. Adjustable upright supports were secured at the ends of the channel with a welded nut and bolt vice arrangement to provide simple supports for the clinched c-sections. Two steel tubes were used as the bearing for the c-sections, one of which was a roller providing simple support and ensuring that there was no axial force along the length of the mechanical clinched c-sections.
Small vertical steel angle sections, shown in Figure 5.3, acted as lateral and torsional restraints to the mechanical clinched c-sections. These components were adjustable and were secured to the frame uprights with nut and bolt ties.

**Applied loading**

A loading stem with two steel contacting tubes was fabricated to fit onto the Instron cross-head as shown in Figure 5.3. The contacting tubes were 100mm in length and 100mm apart.
Applied load was directed downwards and was measured by the Instron 100kN load cell. Automatic calibration and balancing of the load cell was carried out before each test. Force readings from the downward movement of the loading stem, $\delta$, were converted to bending moment, $M$, in the clinch group with equation 5.1 in Figure 5.4. Cross-head deflection, $\delta$ in Figure 5.4, was converted to rotation, $\phi$, on a spreadsheet with equation 5.2. Force and corresponding deflection data streams at half second intervals were logged.

5.1.2 Moment-rotation finite element tests

A 3D finite element model of the moment-rotation test was created to investigate the behaviour of the clinch group and to model the experimental set-up numerically. In the finite element analysis of clinch connection groups:

- The finite element program ABAQUS was used to carry out non-linear elasto-plastic finite element tests modelling the cold-formed steel c-sections and the clinch connection groups in the experimental tests.
• C-sections were modelled with shell elements that have plastic steel material properties, allowing steel buckling to occur

• Clinch connection elements were modelled with special orthotropic elements that were assigned the stiffness properties of the mechanical clinches at 0° and 90° from the shear test load-displacement data.

Material properties, shell elements and the Rik’s method of controlling the loading in the moment-rotation finite element tests are described in Section 3.2. Figure 5.5 shows an elevation of the moment-rotation Finite Element model, showing the distribution of vertical stress with the clinches in the elastic range of response. Flanges of the c-sections are perpendicular to the plane of the page.

![Figure 5.5, Direct vertical stress in the moment-rotation finite element model](image)

Small concentrated regions of vertical compression are shown in blue at the points of support and loading. The system of resistance against rotation in the clinch group created compression on the bottom, shown in blue, and tension on the top, in red, for the two clinches to the left of the group. Conversely there was concentrated compression on the top and tension on the bottom of the two clinches to the right.

**Boundary conditions**

The c-sections in the moment-rotation experimental tests were supported at the ends on two steel pipes, one a roller and one fixed, as shown in Figure 5.3. Lines of support in the finite element tests were represented by grouping a line of nodes in the exact position of the support in experimental tests, as shown in Figure 5.6, allowing a simple support boundary condition to be applied.
The two points marked 'A' in Figure 5.6 were fixed against lateral movement to simulate the lateral restraint provided by the uprights in the experimental tests shown in Figure 5.3. Colour-coded contours in Figure 5.6 represent membrane shear stress in the shell elements of the finite element model in the elastic range.

**Loading**

Load from the Instron loading ram was applied to two steel rods positioned near the center of the c-section clinch arrangement. This was represented in the finite element tests by grouping two similar lines of nodes on the top of the c-section, shown in Figure 5.6. The proportion of load distributed to each node was factored to apply the load evenly along the line, and to provide a total downward static force between the two lines, of 1kN. The non-linear Rik's algorithm was used to control the application of load on the moment-rotation finite element model. The Rik's algorithm is described in Section 3.2.3. Figure 5.7 shows a buckled laterally unrestrained moment-rotation model.

*Figure 5.6, Moment-rotation model boundary conditions*
5.1.3 Orthotropic clinch behaviour

The two c-sections in the finite element tests were joined by clinch elements as outlined in section 3.2.4 to reflect the shear resistance effect of clinching the c-sections in the experimental tests. The clinch load-displacement stiffness data established in the connection shear tests in Chapter 4 was used as input data for a clinch element.

All the clinches in the moment-rotation tests were aligned horizontally. When moment, ‘M’ in Figure 5.8, was applied to the clinch group, each clinch was subjected to an in plane force, ‘F’, at an angle, \( \theta \), to the short edge of the clinch. The angle ‘\( \theta \)’ can be established from the horizontal and vertical spacings, ‘x’ and ‘y’, with equation 5.3 in Figure 5.8. Section 4.3.3 ‘Effect of orientation of clinch’ describes how the response to loading of a rectangular clinch changes depending on the angle of the applied load to the mechanical clinch.

The analytical method used to calculate the clinch group capacity was as follows:

- The angle of applied loading to the clinch as shown in Figure 5.8 was established from the spacing geometry
- The principal force in the clinch at failure was linearly interpolated from experimentally recorded values between 0° and 90°
Shear capacity of each clinch was multiplied by the lever arm in Figure 5.8 to give a peak clinch group moment.

\[ \theta = \tan^{-1} \frac{y}{x} \quad \ldots 5.3 \]

\[ F_{90} = F \sin \theta \]

\[ F_0 = F \cos \theta \]

\[ M = 4dF \quad \ldots 5.4 \]

Figure 5.8, Clinch group forces

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Table 5.3, Analytical clinch group moment capacities

Peak forces resisted at 0° and 90° in the clinch configurations used in the moment-rotation tests, \( F_0 \) and \( F_{90} \) in Figure 5.8, are listed in Table 5.3. Peak forces at angles between 0° and 90° in
Table 5.3 were interpolated linearly between 0° and 90°. Equation 5.4 in Figure 5.8 was used to establish the analytical clinch group moment capacity, also listed in Table 5.2.

5.1.4 Analysis of moment-rotation test results

Moment-rotation graphs from experimental tests and corresponding finite element tests are shown in Figures A2.1.1 to A2.4.4 of Appendix 2. Applied load and deflection data output from the Instron testing machine were converted to applied moment in units of kNm and rotation in radians with equations 5.1 and 5.2 in Figure 5.4. Peak moments recorded in the experimental and finite element tests are listed in Table 5.4. Where three layers were listed, the middle thickness refers to a third flat square plate clinched between the c-sections, giving the layer arrangement and applied load arrangement shown in Figure 4.15 (b) in Chapter 4. The horizontal spacing of the mechanical clinches in the group is referenced on each graph as 'x' and the vertical spacing as 'y' shown in Figure 5.8.

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Table 5.4, Moment-rotation experimental, finite element and analytical test results
1.5/1.5mm thickness arrangement

Moment-rotation graphs for four spacing arrangements of the 1.5/1.5mm steel thickness layer arrangement are shown in Figures A2.1.1 to A2.1.4 of Appendix 2. The x=15mm and y=25mm spacing arrangement in Figure A2.1.1 showed a clearly defined peak moment for ‘Instron tests 1’ and ‘Instron test 2’ experimental tests. The initial response was elastic with behaviour becoming non-linear as load was increased towards the peak value. As there was no significant warping or buckling of the c-sections in the 1.5/1.5mm tests and considering that the c-sections are relatively stiff in longitudinal bending, the rotation of the c-sections in the test can be attributed to the shear deformation of the four clinches in the group and the in-plane deformation of steel between clinches. Shear deformation characteristics of the 1.5/1.5mm steel thickness configuration S-type clinch are shown in Appendix 1 in Figure A1.2.30 for 0° and in Figure A1.2.32 for 90°.

In the x=15mm and y=25mm clinch spacing arrangement, the lever arm distance ‘d’ in Figure 5.8 is 14.6mm and the angle ‘θ’ is 59.0°. Average peak load of the S-type clinch at 0° was 3.91kN and average peak load of the 90° clinch was 2.79kN. By linear interpolation between these values the peak moment at 59.0° was 3.18kN. Applying Equation 5.4 to multiply the lever arm distance by the peak force in each clinch in Equation 5.4 gave an analytical moment capacity of 0.19kNm. This was close to the experimental and finite element moment capacities both of 0.18kNm.

When the x=15mm, y=25mm spacing was reversed in the following test to x=25mm, y=15mm, the clinch group was stronger and more efficient, with the ratio of 0° to 90° force components and the ratio of 0° and 90° clinch shear resistance more closely matched. Using Equation 5.4 to calculate moment capacity gave 0.21kNm, just below peak moments of 0.22kNm and 0.21kNm for experimental and finite element tests respectively.

Figure A2.1.3 shows moment-rotation curves for experimental and finite element tests on the x=25mm, y=50mm spacing arrangement. With clinches aligned horizontally in the c-sections, the x=25mm, y=50mm spacing arrangement was less efficient in comparison with the x=50mm, y=25mm spacing in the following test. The angle θ in Figure 5.8 was 63.4° and the lever arm distance ‘d’ is 28.0mm. The analytical moment capacity was 0.35kNm. This was slightly low in
comparison with the finite element test giving a peak moment of 0.36kNm and higher than the experimentally measured peak moment of 0.32kNm. The experimentally measured moment of 0.32kNm was low in comparison with predicted moments due to torsional twisting of c-sections observed in the experimental moment-rotation test.

The spacing arrangement was reversed from the previous x=25mm, y=50mm arrangement, to the more efficient x=50mm, y=25mm arrangement. This caused the higher vertical moment-force component to be directed at the 0° clinch orientation. Some warping of the c-section occurred in the experimental test although the clinch group rotated until the connections had failed. The angle $\theta$ in Figure 5.8 is 26.6° and the lever arm distance ‘d’ again is 28mm. Experimental moment capacity in Figure A2.1.4 was 0.39kNm and finite element moment capacity was 0.39kNm. Predicted moment capacity was 0.40kNm.

2.0/2.0mm thickness arrangement

Clinch load deformation paths at 0° and 90° from Instron shear tests for the 2.0/2.0mm steel thickness combination is illustrated in Figures A1.2.7 in Appendix 1. Figure A2.2.1 of Appendix 2 shows the moment-rotation paths for the x=15mm, y=25mm spacing arrangement of the 2.0/2.0mm c-section thicknesses. Average experimental moment capacity was 0.25kNm, analytical moment capacity was 0.23kNm and finite element moment capacity was 0.22kNm.

Warping and buckling of c-sections was observed in the experimental tests and is shown in Figure 5.7. Lateral support struts were pushed apart by torsional warping forces in the c-sections and in the first test torsional buckling occurred at 0.22kNm. Wooden blocks were placed between the c-section flanges and the lateral restraints were adjusted in a second experimental test, the moment-rotation path is plotted in Figure A2.2.1. In the restrained experimental model a moment capacity of 0.28kNm is reached.
Two additional finite element models were analysed, the first simulating lateral restraints in the experimental tests and a second without lateral restraint shown as point 'A' in Figure 5.6. A peak finite element moment capacity of 0.23kNm was reached in the laterally restrained model. In the unrestrained finite element model buckling started near the peak moment of 0.22kNm, the buckled shape of the unrestrained finite element model is shown in Figure 5.7.

In Figure A2.2.2 the moment-rotation paths for the x=25mm, y=15mm spacing arrangements are shown. Experimental moment capacity was high in comparison with the finite element moment capacity, while the general pattern of moment-rotation was similar. Figure A2.2.3 shows the moment-rotation paths for the x=25mm, y=50mm spacing arrangement. There was a good agreement in this case between 'Instron test 2' and the finite element test with peak moments of 0.45kNm and 0.46kNm respectively. Moment capacity based on the clinch shear resistance in equation 5.4 was 0.43kNm.

The moment-rotation relationship for the x=50mm, y=25mm clinch spacing arrangement is shown in Figure A2.2.4. This test also showed agreement between experimental and finite
element peak moments with peak moments of 0.50kNm and 0.49kNm respectively and a predicted peak moment of 0.52kNm.

1.0/1.0/1.0mm thickness arrangement

Figure A2.3.1 of Appendix 2 shows moment-rotation results of the x=15mm and y=25mm spacing arrangement experimental and finite element tests for the 1.0/1.0/1.0mm thickness combination and with layer arrangement 1 in Figure 4.15 (b). Applying the interpolated peak clinch shear force in equation 5.4 gives 0.14kNm, a similar to value as the experimentally measured moment capacity and the finite element test moment capacity of 0.13kNm. In Figure A2.3.2 the spacing arrangement of x=25mm and y=15mm was used. Experimentally measured moment capacity was 0.14kNm and the peak moment using equation 5.13 was also 0.14kNm. Finite element test peak moment was 0.13kNm.

Spacing in the moment-rotation test in Figure A2.3.3 was larger at x=25mm and y=50mm and a higher experimental moment capacity of 0.25kNm was achieved. The more efficient x=50mm and y=25mm spacing arrangement in Figure A2.3.4 yielded an experimental moment capacity of 0.30kNm. The finite element test in this case reached 0.25kNm applied moment and the analytical moment capacity based on the clinch capacity was 0.30kNm.

1.0/1.2/1.0mm thickness arrangement

Moment-rotation paths for the x=15mm, y=25mm spacing arrangement are shown in Figure A2.4.1 of Appendix 2. A moment capacity of 0.18kNm and 0.19kNm was reached in finite element and experimental tests respectively. Analytical moment capacity was 0.18kNm. Figure A2.3.2 shows similar moment capacities from moment-rotation paths for the x=25mm, y=15mm spacing arrangement.

In Figure A2.4.3 moment-rotation paths for the larger x=25mm, y=50mm spacing arrangement are illustrated. A peak moment of 0.30kNm was reached in the experimental and finite element tests and in each case overall steel buckling of the c-sections occurred at the peak moment. Lateral buckling was allowed in the finite element test by removing the lateral restraint boundary condition. The analytical moment capacity in Table 5.4 was also 0.30kNm.
In the x=50mm, y=25mm spacing arrangement of Figure A2.4.4, 0.39kNm experimental moment capacity was reached. This moment was close to the analytical moment capacity of 0.35kNm in Table 5.4. In the finite element test overall buckling of the unrestrained c-sections occurred at the slightly lower peak moment of 0.36kNm.

5.1.5 Discussion of moment-rotation test results

Behaviour of clinches in rectangular groups of four has been investigated with experimental and finite element methods. Clinch groups have been subjected to concentrated applied moment in experimental and finite element tests and have responded to loading with strong rotational behaviour, causing the c-sections to buckle in some cases.

Moment-rotation resistance

A comparison of the experimental and finite element moment capacities against predicted moment capacities in Table 5.5 shows that the method of predicting the peak moments is effective with a average difference of predicted peak moments from average experimentally measured peak moments of 2.9%.

Finite element analysis

Moment-rotation responses from the finite element analysis of the clinch groups showed a close match with experimental tests in many cases, indicating a realistic connection shear resistance model. Where clinch group arrangements used thick sections, buckling and displacement occurred at high applied moments in finite element and experimental tests. Finite element analysis can be improved on if sections being modelled are confined to the region directly around the clinch group, taking away buckling and twisting response of c-sections under high applied moments. This will be an effective improvement if the experimental set-up is changed to provide the same response.

Finite element methods are also available in ABAQUS to provide rigid restraint and rigid relationships between chosen degrees of freedom between nodes and sets of nodes. Restraints and equations can be applied between translational degrees of freedom such as those represented by '1, 2 and 3' in Figure 5.6, or between rotational degrees of freedom that rotate about the '1, 2 and 3' axes. In this way longitudinal bending of the c-section can be included without buckling
behave, by restraining a region of nodes representing the wooden block insert in Figure 5.10. The wooden stiffening supports can then be applied in all experimental and finite element tests.

Two finite element tests were carried out, one restrained laterally at points ‘A’ on Figure 5.6 and one unrestrained. Figure A2.2.1 shows that the unrestrained finite element model buckled close to the peak load while the restrained model continued unbuckled past the peak moment. The resistance of the clinch group in resisting applied moment in some tests caused c-sections to buckle and displace the lateral restraints. Wooden blocks as shown in Figure 5.10 were fitted between flanges of c-sections to prevent buckling where it was could be predicted.

In the finite element test in Figure A2.1.1 when the lower peak shear resistance of the 90° clinch model had been passed, the 0° clinch deformation at the same clinch remained in the elastic range until a large enough vertical shear deformation occurred to push the clinch past the 0° elastic limit. In the experimental test when the clinch has lost shear resistance in any orientation, the connection failed and overall shear resistance was lost. This was evident in the experimental curve where a maximum was reached at 0.19kNm and the moment-rotation path turned downwards, indicating a loss of moment-rotation resistance. The finite element model after loss of resistance in the clinch at 90° continued to give constant resistance from the 0° shear resistance in the orthotropic clinch shear resistance model, until that failed after a further small rotation of the group. The finite element model can be considered effective in simulating the response of the experimental test up to the limit of the clinch shear resistance model, and not after that point.

**Experimental test set-up**

The moment-rotation frame was successful in most cases in applying moment to the connection group to cause full connection failure. Computer logged load and cross-head test data were available at frequent intervals after the tests for processing and analysis.

Recommendations to prevent twisting and buckling of the c-sections are:

- Wooden inserts can be used to stiffen the sections against buckling. The wooden inserts can be blocks as illustrated in Figure 5.10 or longer wooden box sections to fit along the length of the c-sections, and clamped or screwed into position.
Lateral restraints in Figure 5.9 can be welded into position. The restraint rods should be stronger and the c-sections should be folded to fit flush between them.

Roller and fixed supports in Figure 5.3 can be changed to two roller supports, both of which are connected rigidly to the c-sections at two or more points on the line of the support, with a nut and bolt screw-down set-up.

By fabricating a moment-rotation frame to fit onto the base of the Instron testing frame for the moment-rotation tests reliable computer generated load-displacement data from the Instron control system was available for analysis.

5.2 Six clinch cantilever model

A cantilever finite element model with six mechanical clinches connecting the components was used to investigate non-linear behaviour in a clinch group and to monitor effectiveness of the group in redistributing forces when one of more clinches in the group fails. The method of applying the non-linear clinch shear resistance model in special connection elements is described in section 3.2.4.
The point load was applied directly above the connection in the moment rotation tests in Section 5.1 to ensure that no direct axial force was sustained by the connection group, only moment. By using a cantilever or beam system, direct downward shear force was applied into the connection group in addition to in-plane moment. This loading condition is more likely to occur at a connection node in a cold-formed steel framing system.

5.2.1 Cantilever finite element model

In Figure 5.11 a 1.5mm thick steel post connects at the center with a 1.5mm thick steel cantilever beam loaded at the end. The post was fixed at the top and bottom. The connection group was formed by six clinches arranged symmetrically and aligned with the long edge of the clinch horizontal as shown in Figure 5.13 (a). Spacing in the clinch group was 40mm horizontal and 70mm vertical. The cantilever length was 600mm and the post width was 120mm.

The horizontal direction stress plot in Figure 5.11 demonstrates that because of the flexible nature of thin gauge cold-formed steel, the distribution of the pattern of stress in the material is highly irregular. Variation in stress is greatest around the clinches and at the support and reaction positions of the components. Variation in stress and stress concentrations often give rise to local buckling in cold-formed steel. At the interface region between components where layers
of steel are connected with clinches, two or more layers of steel provide increased thickness in the layers coupled by the clinches to resist out of plane warping. Variation in direct vertical membrane stress in the clinch group resisting moment applied through the cantilever is illustrated in Figure 5.14 (a). A pattern of tension and compression points built up in the steel in the regions directly above and below the active clinches as moment and downward force was applied through the clinch group load path.

The cantilever applied less downward force and greater moment into the connection group in comparison with the H-frame beam set-up in Section 5.3. The beam restrained rotation at the end connections while the cantilever allowed free rotation. In the cantilever the end force was transferred in a load path through the horizontal c-section and the mechanical clinches into the post, which has fixed reactions at the ends. The cantilever clinch group will fail under applied moment more easily in comparison with the H-frame set-up - leverage forces in the clinches arising from moments in the cantilever model were high in comparison with direct downward loading forces reacted by bending along the length of the beam in the H-frame. This ensures the clinches in the cantilever test model displace far enough to yield in the ABAQUS clinch shear resistance model, shown in Figure 5.13.

5.2.2 Analysis of the six clinch group

Figure 5.14 shows the non-linear relationship between applied downward load at the end of the cantilever and the rotation of the cantilever arm. Initially the clinches were stretched in the elastic range and the downward force was greatest on the two clinches closest to the end point load. As the end load was increased the peak 90° clinch force-displacement is passed (Figure 5.12), this was where the peak moment of 0.67kNm was reached. Rotation increased to 0.065 radians, where the 0° clinch shear capacity was passed in the two clinches closest to the end load and unloading by the non-linear Rik’s algorithm was started.
A shift in the center of rotation of the clinch group occurred (Figure 5.13 (b)) and the clinch group now sustained an applied moment of 0.43kNm (Phase 2 in Figure 5.14), resisted by the four clinches to the left of the group. Rotation continued until the two centrally positioned clinches failed at an estimated moment of 0.16kNm, phase 3 in Table 5.5.
The moment capacity of the clinch group was estimated using the clinch peak force method outlined in section 5.1.3. The first two clinch group moments in Table 5.5 correspond to four outer and two inner clinches in the six clinch group cantilever test set-up. Moments can be added to give a total predicted moment capacity of 0.78kNm for the first phase of rotation. This is high in comparison with the finite element test moment capacity. The effect of the downward force from the end load acting at 0° through the clinches can also be taken into account as a reducing influence on the analytical calculation. Predicted moment capacity in the clinch group can also be read as 0.62kNm in comparison with the finite element moment of 0.67kNm as the two central clinches being closer to the center of rotation had a less significant effect in resisting moment in comparison with the four corner clinches. The third moment of 0.43kNm in Table 5.5, Six clinch group analytical moment capacities
5.5 was the moment sustained by four clinches remaining when the two clinches nearest to the end load had failed. After this point the center of rotation shifted and moment was redistributed to the four clinches to the left of the group. This analytical moment capacity was close to the finite element test peak moment of 0.44kNm in Figure 5.14.

5.3 Clinched H-frame full-scale tests

In [16 and 62] Davies et. al. describe a set of full scale experimental tests on cold-formed steel H-frames connected with clinch groups. Davies explains in [16] how the H-frame tests were carried out to investigate the effect of end fixity of clinch connections in a simple full-scale structure.

A horizontal c-section 80mm web depth and 40mm flange width, 1.58mm in thickness spanning 3.27m was clinched at the ends onto two upright 300 x 100mm 1.58mm thick c-sections as shown in Figure 5.15. Load was applied equally at two third points along the beam. Clinches were applied with the long edge vertically aligned - the direction of the applied vertical load was
parallel to the long edge of the clinch. Three test cases with different end connection details were considered as illustrated in Figure 5.16:

(a) Pinned with a single bolt allowing free rotation  
(b) 4 vertically aligned clinches in a square spacing arrangement  
(c) 8 vertically aligned clinches in two horizontal rows with four clinches in each row

![Figure 5.16, Davies' H-frame connection spacing arrangements (mm)](image)

5.3.1 H-frame finite element model

In this work ABAQUS finite element simulations of Davies’ H-frame experimental tests were carried out to give an additional analytical examination of the structural behaviour observed in the H-frame tests, with emphasis on the influence of the clinch resistance characteristics on overall modes of deflection and failure. With a similar method described in the cantilever tests in the previous section and also described in section 3.2.4, the clinch shear resistance model was introduced to the cantilever finite element tests. Shear resistance characteristics from the 1.5/1.5mm clinch thickness combination in Figure 5.12 were scaled up by a factor of 1.05 to be consistent with Davies’ 1.58/1.58mm clinch thickness arrangement. The orthotropic clinch shear resistance model was rotated through 90° as clinches were applied in the H-frame tests at 90° to the orientation in the cantilever tests.

ABAQUS shell elements with 8 nodes were used to model c-sections, the element types are described in section 3.2.2. A yield stress of 300N/mm² from Davies’ material tests was applied in the shell elements. End posts were relatively stiff in the experimental tests. As no warping of the end posts was reported by Davies in [16], only the rectangular section of the post around the
clinched group was modelled. The rectangular outline of this section was restrained by reducing all rotations and displacements to a single control node. The control node was restrained against in-plane rotation, out of plane rotation and vertical deflection giving a fixed support effect. The steel within the rectangle was free to strain in the plane of the rectangle, allowing deformation of the steel in the post adjacent to the clinches in the clinch group.

Point loading in the finite element tests was applied as a surface pressure over a 50mm x 40mm patch on the top flange of the beam c-section to avoid local stress concentrations that could arise if the load was lumped onto a single node. Central symmetry was applied in the finite element models at mid-span.

5.3.2 Analysis of H-frame test results

In [16] Davies reported all H-frame beam tests ending by local buckling failure of the beam section under a point load. Davies' observed modes of failure were supported by load cell and strain gauge measurements. Davies' load deflection measurements are plotted in Figure 5.17 with the finite element test results from this work. In Figure 5.17 linear elastic response to loading in the experimental tests up to 3.69kN for the 4 clinch model and 4.25kN for the 8 clinch model was distorted at higher loads by local buckling under the point loads. Beams lost stiffness after local buckling occurs. Elastic mid-span deflection stiffness in the pinned model was approximately 7.24kN/mm while in the clinched models it was increased to 9.57kN/mm for 4 and 8 clinch models, an increase from the pinned condition of 32%. In Davies work failure or shear deformation in the clinches were not observed.
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Figure 5.17, H-frame load-deflection paths

Figure 5.17 shows load-deflection paths for the H-frame finite element tests, with similar stiffness gradients to the experimentally measured results. In the pinned test torsional warping in the beam increased stiffness at higher loads. In the clinched models a small decrease in stiffness in the elastic range at higher applied loads was observed. The yield stress of 300N/mm² was reached in the flanges of the beam at approximately 4.25kN in the pinned and 4 clinch models and at 4.75 in the 8 clinch model, causing the beams to twist and lose shear resistance. Figure 5.18 shows the distribution of longitudinal bending stress in the beam at this point in the 4 clinch model. Tension in the bottom flange and compression in the top flange at the center are both approximately 300N/mm².
Figure 5.19 shows Von Mises' stress. Von Mises stress was used by ABAQUS to initiate yield in the material – the top and bottom flanges at the center in Figure 5.19 have reached the yield value of 300N/mm². Von Mises' stress in the steel around the clinch group was approximately 200N/mm². The maximum force in the clinch connection elements reached a maximum of 0.56 times the potential 90° peak force in the 4 clinch model and 0.35 times the potential 90° peak force in the 8 clinch model.
The following equation is used in [62] to establish the experimental c-section bending moments at the connection and mid-span strain gauge positions:

\[ M = \left(\frac{\varepsilon_T - \varepsilon_B}{2}\right) E Z \]

where:

- \( M \) is the section bending moment
- \( \varepsilon_T \) is the strain in the section top fibre
- \( \varepsilon_B \) is the strain in the section bottom fibre
- \( E \) is the modulus of elasticity in the steel (220GPa)
- \( Z \) is the section elastic modulus (5675mm\(^3\) experimentally measured value)

Assuming a fully plastic condition in the clinch groups at the supports the failure loads of the test beams listed in Table 5.6 were predicted by observations of load-rotation relationships from the experimental tests:

- Total internal work: \( 1.5 \theta (M_I + M_S) \)
- Total external work: \( 1.5 \theta \)
- External work = Internal work: \( 1.5 \theta (M_I + M_S) = 1.5 \theta \)
- \( P = (M_I + M_S) / 1.117 \)

where:

- \( M_I \) was the moment capacity of the clinch group
- \( M_S \) was the moment capacity of the c-section
- \( P \) was the force from one loading ram, the total load was 2P

<table>
<thead>
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<th>Tset</th>
<th>Moment capacity</th>
<th>Failure load</th>
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<tbody>
<tr>
<td></td>
<td>Clinch group</td>
<td>C-section</td>
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<tr>
<td></td>
<td>(kNm)</td>
<td>(kNm)</td>
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<tr>
<td>4 clinches</td>
<td>0.266</td>
<td>1.71</td>
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<tr>
<td></td>
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<td></td>
<td>0.550</td>
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Table 5.6, Results of analysis in [62] extended with finite element failure loads from this work

5.4 Chapter summary and conclusions

The moment-rotation resistance of groups of four clinches in four different spacing arrangements and in four different combinations of steel thicknesses was investigated. Clinch groups in three layers of steel were investigated for the first time. Experimental tests were carried out by
fabricating a moment-rotation testing frame and attaching it to the base of the Instron testing frame. The downward force of the Instron loading ram was used to apply moment to the clinch group. Finite element tests modelling the experimental set-up were carried out using the finite element program ABAQUS. A method for predicting the moment capacity of clinch group connections based on the shear capacity of the individual clinches was developed and applied.

In the experimental moment-rotation tests in section 5.1 buckling of the cold-formed steel c-sections was observed in tests where the moment capacity of the clinch group was greater than the moment capacity of the c-sections. This occurred when the spacing of the mechanical clinches was large and when 2mm thick steel was used in the c-sections. Lateral deformation of vertical lateral restraining supports allowed the c-sections to twist out of plane. With adjustment of the boundary conditions to allow lateral displacement buckling also occurred in the finite element moment-rotation tests. Applying the non-linear orthotropic clinch stiffness in the finite element model gave the moment resistance in the clinch group required to cause buckling in the c-sections.

The cantilever six clinch moment-rotation model in Section 5.2 was the only model in the research with no experimental counterpart. The pattern of stress caused by in-plane steel straining in the region of steel around the mechanical clinches was analysed. Buckling of the sections did not occur as the concentrated moment applied to the clinch group through free rotation of the cantilever against the fixed post caused failure of the clinch group before failure of the chord sections through buckling.

A finite element cantilever model connected with six clinches was developed to investigate non-linear behaviour of the clinch group under axial force and bending moment. Non-linear orthotropic shear resistance characteristics of the clinches at 0° and 90° were applied at clinch locations. Two stages of moment resistance were observed. In the first stage the six clinches resisted moment. In the second stage the two most heavily loaded clinches failed and redistribution of stresses to the remaining four clinches occurred, shifting the center of rotation of the clinch group and sustaining a reduced moment with increasing rotation of the clinch group.
Full-scale clinched H-frame tests carried out by Davies et. al. [16, 62] were simulated in ABAQUS finite element tests. In the finite element tests a horizontal beam c-section was modelled with shell elements and clinches were applied in groups at the end connections. A non-linear clinch stiffness model was applied to the individual clinches to allow the possibility of failure at the connections. In Davies' experimental tests the mode of failure of the H-frame beams was buckling of the beam flanges. This was also the mode of failure observed in the finite element tests.

Analysis of the experimental test results and comparison with the finite element and theoretical moment predictions allowed the following conclusions to be developed:

1. Significant moment resistance can be developed at clinch groups in cold-formed steel. In 5 out of 32 test where the combined thickness of clinched steel was high, the moment resistance of the four clinches was high in comparison with the moment resistance of the insufficiently laterally restrained c-sections and the c-sections warped and failed. These gave a semi-rigid moment-rotation response and were interpreted as 'Joints in Rigid Construction' as described in BS5950 Part 5 [7] Section 8.1.4. Connection groups that failed before the c-sections buckled behaved as 'Joints in Rigid Construction', discussed in BS5950 Part 5 [7] Section 8.1.5.

2. Arranging the clinches in a group with the spacing parallel to the long edge of the clinches greater than the spacing perpendicular to the long edge of the clinches gives the most efficient use of the clinch orthotropic shear resistance characteristics under in-plane moment loading.

3. Using the non-linear orthotropic 0° and 90° shear resistance characteristics of a single clinch to create special clinch connection elements in a finite element model allows clinches to be modelled effectively in non-linear finite element tests.

4. Shear capacity of a single clinch can be used to obtain the analytical moment capacity of a clinch group. The direction of forces in the clinch group was resolved giving a lever arm distance from the center of rotation to the center of each clinch. The direction of force applied to the clinch in the clinch group was used to establish the clinch shear capacity based on linear interpolation between 0° and 90° shear capacities. Shear capacities of the clinches were then multiplied by lever arm distances to give an analytical clinch group moment
capacity. Analytical and finite element peak moment predictions in Table 5.4 were closely matched.

5. The average difference of analytical moment capacities from experimentally measured moment capacities over the 16 moment rotation tests was 5.8%.

6. Comparison of Davies’ experimental clinch group test results in Table 5.1 and the current experimental test results in Table 5.2 showed an average difference of 1.2% with a standard deviation of 17%. The smaller spacing arrangement of 15 x 25mm and 25 x 15mm gave higher moment capacities in the current work in comparison with Davies’ past results.

7. Providing adequate lateral restraint to the c-sections in the experimental tests is necessary to obtain consistent moment capacities in the clinch groups. Lateral displacement, torsional warping and steel buckling distorted experimental test results in a small number of tests. When the restraint provided in the experimental test was applied to the finite element models buckling also occurred in the finite element tests.

8. When one or more clinches in a clinch group fail, the remaining clinches continue to resist a reduced moment after a shift in the center of rotation of the clinch group.

9. In the analysis of Davies’ full scale H-frame tests, failure in full-scale cold-formed steel components connected with clinch groups was governed by limits of buckling in the H-frame crossbeam. Clinch connection groups in Davies’ tests provided an increase in elastic mid-span deflection stiffness of 32% in comparison with the H-frame with pin bolted end connections. Analysis of the forces in the clinches in the ABAQUS finite element tests showed that clinches were at a maximum of 0.56 times the full shear capacity when the crossbeam buckled. Current Finite element failure load test results in Table 5.6 showed a close correlation with past predicted and experimentally measured failure loads.
6 FULL SCALE TRUSS TESTS

Following from the investigation of the shear deformation characteristics of mechanical clinches in Chapter 4 and the analysis of groups of mechanical clinches in Chapter 5, the influence of clinch shear resistance characteristics on the structural behaviour of full-scale trusses is investigated in this chapter by experimental and numerical methods.

In [16] Davies carried out full-scale loading tests on pitched chord roof trusses up to 6.3m in length with pitch angles of between $30^\circ$ and $45^\circ$, connected with groups of mechanical clinches. Components of the roof trusses were c-sections in the outer chords and z-sections in the internal members. Load, strain and deflection were recorded during the tests. Failure modes reported by Davies included:

- Buckling of the eves connection by twisting
- Local buckling of z-section compression flange under point loads
- Buckling of chord members with secondary rotational joint failure
- Shear failure of joints
Davies reported shear failure at the apex joint of the roof truss. As the trusses were tested the truss arrangement was modified by removing the internal components. Modifications were carried out to encourage in-plane moment in the corner clinch groups and to reduce the number of load paths within the structure, giving greater certainty to estimate of axial force and bending moment in the chords and corner connections.

The apex detail in Davies' roof truss tests was a plate connecting the z-sections of the pitched chord with approximately 20 clinches on either side. Identifying the state of force in the clinches was made difficult by the pitched chords butting together in compression, partly relieving the clinch bearing load. The large group of clinches also built up a moment in the apex plate under concentrated applied moment and distortion of the plate around the clinches occurred. Pitched chords were reinforced by clinching a second z-section on to the existing members.

Davies reported difficulty in measuring large non-linear deflections. Trusses typically failed by buckling of pitched chords under point loads, with gusset plates connecting chords with clinches buckling rather than the actual clinches in the group failing. Member failures occurred with as few as four clinches in the apex plate. A capacity increase of 38% was reported using clinch groups in place of pinned bolt connections, the increase in strength arising from increased moment capacity in the corner joints. Downward applied loads in a triangular pitched chord truss in Davies thesis [16] were effectively resisted by the triangular form, giving the truss system little flexibility to induce failure in connections between components. Davies approach was to encourage bending in the pitched chords and corner connections by removing the internal components.

Following from Davies research, the truss type in this research was changed from pitched chord to parallel chord. Slender parallel chords in truss beam tests were stiff in comparison with internal short struts and diagonal ties. While there was enough longitudinal beam bending flexibility to induce failure at the clinch connection nodes, deflections were not high enough to transfer significant in-plane bending between chords and internal members through one, two or three clinches in series. This allowed the force in the clinches to be resolved and analysed using the test data from the strain gauges on the internal members. Using a small number of clinches at each connection node also encouraged failure of the clinches and gave greater certainty to the estimate of force in each clinch.
Parallel chord tests were carried out to establish how clinch shear resistance influenced:

- Elastic response
- Failure mode
- Peak load
- Flexure
- Buckling behaviour
- Distribution of forces within the truss

And how clinch failure mode changed with:

- Number of mechanical clinches in a connection node
- Thickness of steel being connected
- Local buckling

10 lattice type truss beams up to 6m in length were tested, each fabricated from cold-formed steel sections and connected with clinching. Clinches were used to connect tension ties and compression struts to top and bottom chords. Trusses were placed in a reaction frame with lateral restraints and loaded equally from a central loading ram through a spreader beam to two points as shown in Figure 6.1. Strain gauges were attached to the truss at twelve positions, allowing force in the clinches and the bending moment and axial force in truss chords to be determined.

Finite element tests simulating the experimental tests were carried out using the program ABAQUS. In 2D plane-frame finite element tests the clinch shear resistance model established in Chapter 4 is simplified and applied at the connection nodes. 3D shell element buckling finite element tests were carried out to establish the buckling limits of axial force and bending moment in the chord sections.

6.1 Truss experimental tests

The focus of investigation was the effect of clinch shear resistance on the structural behaviour of each truss. The trusses were loaded at two points along their length to the point of either failure of the clinch connections or buckling of the cold-formed steel components. Table 6.1 gives a
summary of the experimental and finite element failure loads. Table 6.2 gives the modes of failure for the 10 truss tests.

<table>
<thead>
<tr>
<th>Truss number</th>
<th>Length (m)</th>
<th>Number of clinches</th>
<th>Clinch layer arrangement</th>
<th>Experimental failure load (kN)</th>
<th>Finite element failure load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.8</td>
<td>3</td>
<td>1.5/1.0/1.5</td>
<td>1.5</td>
<td>3.1</td>
</tr>
<tr>
<td>2</td>
<td>5.2</td>
<td>2</td>
<td>1.5/1.0/1.5</td>
<td>11.6</td>
<td>12.4</td>
</tr>
<tr>
<td>3</td>
<td>6.0</td>
<td>1</td>
<td>1.2/1.0/1.2</td>
<td>5.6</td>
<td>5.2</td>
</tr>
<tr>
<td>4</td>
<td>6.0</td>
<td>3</td>
<td>1.5/1.0/1.5</td>
<td>12.0</td>
<td>13.7</td>
</tr>
<tr>
<td>5</td>
<td>6.0</td>
<td>3</td>
<td>1.2/1.2/1.2</td>
<td>9.8</td>
<td>11.8</td>
</tr>
<tr>
<td>6</td>
<td>6.0</td>
<td>1</td>
<td>1.2/1.2/1.2</td>
<td>6.0</td>
<td>6.3</td>
</tr>
<tr>
<td>7</td>
<td>6.0</td>
<td>1</td>
<td>1.5/1.0/1.5</td>
<td>6.1</td>
<td>6.0</td>
</tr>
<tr>
<td>8</td>
<td>6.0</td>
<td>2</td>
<td>1.5/1.0/1.5</td>
<td>10.5</td>
<td>12.3</td>
</tr>
<tr>
<td>9</td>
<td>3.0</td>
<td>1</td>
<td>1.2/1.0/1.2</td>
<td>13.2</td>
<td>13.2</td>
</tr>
<tr>
<td>10</td>
<td>3.0</td>
<td>1</td>
<td>1.2/1.2/1.2</td>
<td>9.5</td>
<td>9.5</td>
</tr>
</tbody>
</table>

Table 6.1, Truss test peak loads

<table>
<thead>
<tr>
<th>Truss Number</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Compression strut buckling</td>
</tr>
<tr>
<td>2</td>
<td>Failure caused by flawed clinch detail</td>
</tr>
<tr>
<td>3</td>
<td>Top chord end span buckling</td>
</tr>
<tr>
<td>4</td>
<td>Press-join failure</td>
</tr>
<tr>
<td>5</td>
<td>Top chord end span buckling</td>
</tr>
<tr>
<td>6</td>
<td>Press-join failure</td>
</tr>
<tr>
<td>7</td>
<td>Press-join failure</td>
</tr>
<tr>
<td>8</td>
<td>Press-join failure / buckling</td>
</tr>
<tr>
<td>9</td>
<td>Centre span buckling</td>
</tr>
<tr>
<td>10</td>
<td>Press-join failure</td>
</tr>
</tbody>
</table>

Table 6.2, Truss modes of failure
In the truss tests it was important to provide adequate lateral and torsional restraints to the top and bottom chord members. In cases where there were three clinches applied at each connection node, trusses buckled at the chord or strut members. Where there were two clinches at connection nodes, truss strength was approximately balanced between clinch shear resistance and...
buckling resistance. Additional restraint mechanisms and restraint positions were added to the test rig as more beams were tested.

Figure 6.3, Structural failure at a clinch connection node

6.1.1 Truss general arrangements

Two folded steel sections 0.4m apart formed parallel chords in the trusses as illustrated in Figure A3.0.1. Vertical z-section compression struts were clinched beneath the points of loading and at the end reactions. Struts were also positioned at horizontal intervals similar in length to the depth of the truss, between positions of loading and end reactions. Flat diagonal tension ties were clinched between compression struts to resist shear in the beam and to effect a moment lever arm between the top and bottom chords. Vertical struts in the zero shear span between points of loading were nominal strut members required to keep the top and bottom chords from separating vertically. No diagonals were required in the central span. Specifications for each truss are listed in Table 6.3.

The form of the components in each mechanical clinched truss was determined by a need for a structural capacity to meet a loading demand. Shear resistance of clinching affected the capacity
of the trusses against the loading demand. Understanding of the influence of the mechanical clinches in the trusses was gained by making the shear resistance of the clinch connections variable between trusses.

Demand

Allowable loading demand on a cold-formed steel structure in a building application is determined by a pattern of live and dead loads factored to a magnitude that provides a level of safety and performance. In the experimental truss tests:

- The magnitude of loading demand was predicted by finite element tests and met the clinch capacity and buckling capacity of the truss
- One central point load or two equal point loads applied vertically downward on a simply supported beam framework represented the pattern of loading demand. Applied to a simply supported long-span lattice truss, the pattern of loading influenced deflection flexibility of the trusses

Capacity

The strength capacity of a cold-formed steel frame is in the shear resistance capacity of the connections, resistance of the material and resistance in buckling. Two parameters in the truss configurations were varied to influence the shear resistance capacity of each truss:

- Number of clinches at each connection node
- Thickness of steel parts – this affected shear resistance characteristics of clinching in addition to elastic and buckling strength of the components

By changing these two variables in the full-scale tests, the factor governing failure capacity of the truss was controlled between clinch failure and buckling. The number of mechanical clinches at each connection node proportionally affected elastic stiffness and peak load at the connection node as illustrated in Figure A3.0.8. Thickness of steel being connected, strength of the steel being connected, number of layers of steel and orientation of applied load affected the shear resistance of each clinch in the connection nodes. Clinches connected internal struts and ties to chords, the two outer layers were the chord webs and the inner layer was the strut or tie. The inner layer reacted against the outer layers giving a shear force in the connection. All clinches in full-scale tests were applied at 0° to applied loads and the direction of applied load
was determined by orientation of the internal part being connected, diagonal tie or vertical strut. Clinch elastic stiffness at connection nodes directly contributed to elastic mid-span deflection of the truss.

Steel buckling affected clinch behaviour. When buckling was initiated the stiffness of the components in the truss changed. Forces were redistributed to meet the loading demand and connection forces were increased or decreased at connections. Local buckling capacity was greatly affected by thickness of steel - thin flat parts buckled under compression if the width to thickness ratio was high. If lengths between lateral and torsional restraint in the chords were too great lateral and torsional buckling determined failure capacity of the truss. Truss buckling capacity was also affected by elastic stiffness. When truss elastic deflection was high local buckling and geometric warping were induced, this effect was greatest in compression components.
Chords

Bench mounted and a ceiling mounted clinching tools were used to connect chords to internal components of the trusses and pop rivets were applied in non-load bearing positions on the ends of the vertical struts, as illustrated in 6.8 to strengthen the compression strut end bearing.

The general arrangement of horizontal chords for trusses 1 to 8 was similar - two chord members were positioned back to back and clinched with tie and strut flat components between them as shown in Figure 6.5 (a). This was carried out for top and bottom chords, using four half chord sections in two chords for each truss. For trusses 1 to 8 the top and bottom double chord pairs
were positioned in an upright ‘T’ arrangement to resist longitudinal bending. In trusses 9 and 10 the top chord is arranged in an upright ‘T’ shape while the bottom chord is oriented in an inverted ‘T’ shape as shown in Figure 6.5 (b).

An automated guillotine machine with a cutting length range of 2m was used to cut the cold-formed steel chord and internal member components for the trusses 1 and 2. A hand operated cold-formed steel folding machine was used to fold top and bottom chord sections. Chord sections in trusses 3 to 10 were folded from flat sections in 2m lengths and but welded into 6m lengths by Metsec PLC.

Struts and tie ends were clinched between chord webs in a triple layer arrangement in the trusses. Triple layer clinches also had the function of connecting the two chord parts back to back along the length of the chords. The maximum distance between triple layer mechanical clinches was 319mm between end supports and load points and 379mm between load points as shown in Figure A3.0.1. BS5950-5 [7] Section 8.6.2 gives the maximum pitch for connections in two channels connected to form an I-section. For members in compression and members in flexure:

\[ s \leq 50r_{cy} \]
where:

- \( s \) is the spacing between connections along the length of the compound member
- \( r_{cy} \) is the minimum radius of gyration of one channel

The minimum radius of gyration of a half-chord in Table 6.4, \( r_{cy} \), was 11.5mm and 50 times \( r_{cy} \) is equal to 575mm. The maximum applied spacing of 379mm was less than 50 times \( r_{cy} \) and separation of half-chord parts was not observed in the experimental tests. Longitudinal shear force

![Diagram of truss component cross-section dimensions (mm)](image)

Figure 6.6, Truss component cross-section dimensions (mm)
Figure 6.7, Truss component section geometries

<table>
<thead>
<tr>
<th>Truss beam component</th>
<th>Component thickness</th>
<th>Cross section area (mm²)</th>
<th>Depth to X-X elastic neutral axis (mm)</th>
<th>Depth to Y elastic neutral axis (mm)</th>
<th>X-X moment of inertia (mm⁴)</th>
<th>Y-Y moment of inertia (mm⁴)</th>
<th>X-X radius of gyration (mm)</th>
<th>Y-Y radius of gyration (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compound</td>
<td>1.2</td>
<td>304.8</td>
<td>29.8</td>
<td>B/2</td>
<td>198256</td>
<td>71729</td>
<td>25.5</td>
<td>15.3</td>
</tr>
<tr>
<td>Chord</td>
<td>1.5</td>
<td>381.0</td>
<td>26.6</td>
<td>B/2</td>
<td>241594</td>
<td>87007</td>
<td>25.2</td>
<td>15.1</td>
</tr>
<tr>
<td>Half</td>
<td>1.2</td>
<td>152.4</td>
<td>D/2</td>
<td>8.3</td>
<td>99128</td>
<td>23288</td>
<td>25.5</td>
<td>12.4</td>
</tr>
<tr>
<td>Chord</td>
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<td>190.5</td>
<td>D/2</td>
<td>8.2</td>
<td>120797</td>
<td>25365</td>
<td>25.2</td>
<td>11.5</td>
</tr>
<tr>
<td>Strut</td>
<td>1.0</td>
<td>85.0</td>
<td>D/2</td>
<td>B/2</td>
<td>44012</td>
<td>583</td>
<td>22.8</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>127.5</td>
<td>D/2</td>
<td>B/2</td>
<td>52815</td>
<td>700</td>
<td>20.4</td>
<td>2.3</td>
</tr>
<tr>
<td>Tie</td>
<td>1.0</td>
<td>50.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>60.0</td>
<td></td>
<td></td>
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</table>

Table 6.4, Truss component elastic section properties

<table>
<thead>
<tr>
<th>Truss beam component</th>
<th>Component thickness</th>
<th>X-X plastic section modulus (GPa)</th>
<th>Depth to plastic neutral axis (mm)</th>
<th>Plastic moment capacity (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compound</td>
<td>1.2</td>
<td>6819</td>
<td>17.3</td>
<td>1.96</td>
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<tr>
<td>Chord</td>
<td>1.5</td>
<td>8431</td>
<td>17.5</td>
<td>2.42</td>
</tr>
</tbody>
</table>

Table 6.5, Truss chord plastic section properties
In section 8.6.2 in BS5950 Part 5 [7], connections between half-chords in compound members in compression must be able to resist the longitudinal shear force between half-chords arising from lateral curvature of:

\[ F_S = 0.25Q(s/r_{cy}) \]

where ‘Q’ is not less than 2.5% of the design axial force. In Section 6.4, ‘Analysis of test results’ in this work this design requirement was analysed in Trusses 6 and 8 and it was shown that longitudinal shear in the chords at the level of \( F_S \) will have influenced the capacity of clinch connections.

Ties

All diagonal tie members were flat plates, 50 mm in width and were arranged and clinched as shown in the truss general arrangement and dimensions drawing in Figure A3.0.1 of Appendix 3. As diagonal ties were designed to resist tension only in the experimental tests they had no stiffening return details.

Struts

Vertical compression struts for trusses 1 to 8 were Z-sections, fabricated from flat plate members 85mm in width. Dimensions of Z-section were 10mm flange length and 65mm web length (Figure 6.6). In trusses 1 to 8, one Z-section was positioned centrally at each compression strut location. There was a length of approximately 50mm at either end of the compression strut where flanges were not bent into a Z-section (Figure 6.28). This allowed compression struts to be connected to top and bottom chords by forming the central layer of three steel layers, with the two outer chord components forming the out layers in a triple layer arrangement at each connection location.
Trusses 9 and 10 used a different compression strut arrangement with two C-sections positioned at the outside of the chord flanges, as illustrated in Figure 6.5, in place of one central compression component. As the top and bottom chord members of trusses 9 and 10 were oriented differently, it was possible to arrange the C-section compression struts of trusses 9 and 10 to sit in the flanges of the chord members and to clinch them onto the chord flanges.

**Splice details**

Chord sections of trusses 1 and 2 were fabricated from lengths of 2m chord section spliced to give full transfer of moment and axial force across the splice. The splice was made from an inner sleeve 400mm in length, slightly smaller than the inside surface of the chord profile in the trusses, placed inside the two sections to be joined and secured in place. The splice in truss 1 was secured with bolts. Splice details in truss 2 used arrangements of 16 clinches, as shown in Figure 6.9. Chord sections used in trusses 3 to 10 arrived in 6m pre-formed and welded sections, splicing was not required.
6.1.2 The testing frame

The testing frame was constructed from hot rolled ‘I’ and ‘C’ sections as shown in Figure 6.11, the testing frame had far greater stiffness in comparison with the cold-formed steel trusses being tested. Rigid uprights forming reaction positions for the trusses allowed trusses to be raised to suit varying depths. Internal uprights at the loading and central positions allowed lateral restraints to be applied between supports.

6.1.3 End bearings

The ends of all trusses were positioned on rocker type plates as illustrated in Figure 6.10. Steel rocker plates were positioned directly beneath the central axis of the end vertical struts at the end supports of each truss providing a simple support.
6.1.4 Lateral restraints

The trusses were slender cold-formed steel fabricated structures, buckling between lateral restraints under concentrated compression and bending in the components, if an unrestrained length was too great. The provision of lateral restraints was critical in the testing arrangement and in particular for the longest trusses 2 to 8,6m in length.

Lateral restraint was provided by securing specially cut blocks of wood, Figure 6.12, to the upright parts of the testing rig, at positions along the length of the trusses, Figure 6.11. Two of the lateral restraint positions were at the end supports. Approximately 1mm clearance was left between the lateral restraints and the chords of the trusses to allow free vertical deflection. The effect of friction between the lateral restraints and the chords of the trusses was neglected in the analysis of the truss tests, as it could not be considered significant in comparison with the high loads applied in testing the beams.
Trusses 1 and 2 had lateral restraint at the end uprights and on the upper chords at the loading positions shown in Figure A3.0.4 of Appendix 3. Trusses 3, 4, 5 and 6 had lateral restraint at the end uprights and on the upper and lower chords at the loading position, also shown in Figure A3.0.4. Trusses 7 to 10 had lateral buckling restraints provided at the end points, the loading points and the truss center position to both the upper and lower chords.
6.1.5 Torsional restraint

The top chords of trusses 7 to 10 were restrained at the ends, load points and central positions against rotation about the long axis of the chord. Torsional restraint was achieved by inserting two blocks to fit under the flanges of the chord, fitting squarely against the vertical lateral restraints, as shown in Figure 6.13. Torsional restraint was also provided to the central and loading positions of the bottom chord of trusses 9 and 10 as shown in Figure A3.0.5 of Appendix 3.
6.1.6 Deflection measurement

Central deflection of the bottom chord was measured with a mechanical dial gauge to an accuracy of 0.01mm and was recorded with the simultaneous load cell reading.

6.1.7 Data logging system

Strain and load readings were recorded at 2 second intervals to a Squirrel 1600 data logger. After each test logged data was transferred from the data recorder to a PC computer for processing and analysis.

6.1.8 Strain measurement

Strain measurements were taken at 12 points on the surface of the beams when testing trusses 3 to 10. Figure 6.14 shows strain gauges on the surface of the end span of truss 8.
Strain measurement locations

Positions of strain measurement were chosen to allow axial strain and bending moment data to be recorded in the top and bottom chord members and also to record axial straining and buckling of the internal members. Locations of each strain measurement point are illustrated in Figure A3.0.2 in Appendix 3. Strain measurement points 1 and 2 are located at the center of the left end diagonal, on either side of the flat plate. Points 3 and 4 are at corresponding positions on the right end diagonal tie. Strain measurement points 5 and 6 were located at either side of the center of the second right end vertical compression strut. Strain gauges 9 and 10 were located at the bottom and top surfaces respectively of the top chord to allow bending strains at the outer fibers of the chords to be recorded. Gauges 11 and 12 were located at the bottom and top surfaces respectively of the bottom chord of each truss.

Type of strain gauge

The 120Ω resistance strain gauges used were 3mm in length, type FLA-3-11 manufactured by the Tokyo Sokki Kenkyujo Co.Ltd. The gauge factor specified by the manufacturers was 2.14 ±1%. The manufacturers also specified a 0.0% transverse sensitivity and $11 \times 10^{-6}/{°C}$ temperature compensation. The effect of temperature compensation was not great enough to have any influence on strain divisions greater than 0.001%.
Strain gauge application

The steel surface area where the strain gauge was to be applied was brushed with fine sandpaper and cleaned with white spirit before strain gauges were glued to the surface with cyanoacrylate adhesive.

Processing voltage to strain readings

The following formula was used to convert voltage output readings from the strain gauges to strain:

\[
\varepsilon = \frac{4 \Delta \varepsilon}{KE}
\]

Where:

- \( \varepsilon \) = Strain
- \( \Delta \varepsilon \) = Differential voltage reading
- \( K \) = Gauge factor from manufacturer = 2.14
- \( E \) = Exciting voltage = 2.4v

This formula was taken from the strain gauge manufacturer's product literature. As raw data from strain gauges was in voltage output it was necessary to calculate the differential voltage on a spreadsheet and multiply it by 0.7788 or \((4 / K \times E)\), to produce strain data. The strain data was processed to provide force and moment data for analysis.

Strain gauge verification

The accuracy of the strain gauges and the value given by the manufacturers for the gauge factor were verified against the Instron tensile testing machine. Strips of steel 50mm x 250mm, similar in dimensions to those used in the material tensile tests were prepared with one strain gauge attached to either side of the center point of the steel strip. Steel samples were then positioned in the Instron testing frame and the 100mm gauge length extensometer was attached to the sample, straddling the strain gauges. This arrangement allowed straining of one piece of steel to be measured with two strain gauges and with one Instron extensometer simultaneously.

The Squirrel data logging system was placed beside the Instron testing machine to allow comparison to be made between the three strain readings. The load on the samples was applied
over several cycles between zero and 50% of force to cause yield in the sample. Readings from the three sources were taken by hand.

Initially the cycle rate was too fast and this caused the time of the readings taken from the Instron to be a short time before the strain readings taken from the Squirrel data logger. When the test was slowed down it was found that division of the Instron strain readings and the calculated Squirrel strain gauge readings were close to unity as shown in Figure 6.15

![Figure 6.15, Comparison of Instron and strain gauge strain readings](image)

In the truss tests when two strain gauges were placed on either side of the same central position on an internal truss member, in particular on diagonal tension components, the strain and calculated stress values at a particular time increment were similar. Figures A3.3.2 to A3.3.5 of Appendix 3 show the tension force in the connection nodes of truss 3 based on strain recorded in the diagonal ties. The force in the tie was divided by the number of clinches to give the force resisted by each clinch at a connection node.

6.1.9 Loading

Load was applied by a hydraulic ram connected to a hand-operated pump. The hydraulic ram was applied to the central position of a spreading beam to provide two equal point loads on the truss, as shown in Figure 6.1. Load was transferred from the ends of the spreading beam to the
truss through short wooden loading blocks 80mm wide, 50mm long and 50mm in height, as shown in Figure 6.12. Overall dimensions of trusses and positions of loading are illustrated in Figure A3.0.1 of Appendix 3.

To reduce inaccuracy of data readings caused by the truss system creaking and bedding in, load was initially applied to the trusses in three cycles up to 50% of force to cause failure at the connections. This procedure also helped to establish the load-deflection characteristics of the truss. Load was then increased in a single phase from zero force to failure of the trusses. Load, force and displacement data are presented in Appendix 3 from the final loading phase from zero to failure.

Load measurement
A 30kN load cell was placed at the point of reaction between the loading ram and the testing frame as shown in Figure 6.11. The load cell was monitored by a digital indicator, which was connected to the Squirrel data logger.

Load cell verification
The 30kN load cell with digital indicator was calibrated by placing it in the Instron testing machine and applying load up to 20kN. The calibration factor on the digital indicator was adjusted to make the load reading on the load cell digital indicator equal to the load reading on the Instron display.

6.1.10 Processing logged test data
Load and displacement data that is plotted in truss load-displacement graphs was recorded by hand from the load cell digital indicator and from the dial gauge displacement extensometer and is less frequent in comparison with the strain data, which was recorded automatically during the tests. While electronic displacement transducers giving readings to 1mm accuracy were available, a mechanical dial gauge giving readings to 0.01mm was used instead of the electronic displacement transducer to gain adequate accuracy in the deflection readings.

Conversion of strain to stress values was carried out by multiplying strain data values by the modulus of elasticity of the steel. Chord axial force values were calculated by multiplying the calculated stress by the cross-section area of the truss component under consideration.
Struts and ties

Strain data from strain gauges at locations 1 to 8 on the truss drawing in Figure A3.0.2 in Appendix 3 were converted to force and plotted for each truss in Appendix 3. Strain gauges 1 to 8 were located on the three diagonal members and one vertical strut shown in Figure A3.0.2, marked with gauge locations 1 to 8. For each internal component a gauge was located on either side of the central point and data readings were expected to be similar on either side. It was also expected however that the compression Z-section with strain gauges 5 and 6 would bend slightly from top to bottom, as compressive load was applied, while diagonal tie members would remain straight in tension.

As internal components were connected to the chord members by clinching at either end, the force in the internal members could be considered to be equal to the total force at each connection node, in the direction of the component being considered.

Top and bottom chords

Strain gauges at locations 9 to 12 on the truss drawing in Figure A3.0.2 recorded strain in the upper and lower fibers of the top and bottom truss chords at the center of the truss. This allowed bending behaviour of the chord members to be analysed. Axial stress in chord members at any time during the tests was calculated indirectly as a factor of applied load by applying the following steps:

1. The reaction at the supports was calculated by dividing the load cell reading by two
2. Truss bending moment at and between the load positions was calculated by multiplying the support reaction by the distance between the load position and the support
3. The Axial force in the top and bottom chords was calculated as plus and minus the bending moment from 2. above respectively, divided by the lever arm distance between the two parallel chords
4. The compressive and tensile axial stress values for top and bottom chords were calculated by dividing the axial force by the cross-section area of the chord

Stress at these positions was a combination of axial stress and longitudinal bending stress. The effects of torsional distortion and local buckling were small in the elastic range of displacement. Axial stress was distributed evenly over the cross section from top to bottom, as shown in Figure
6.16. Stresses arising from longitudinal bending varied linearly in a triangular distribution from top to bottom.

Total Strain = Axial Strain + Bending Strain

Figure 6.16, Stress components in the chord sections

Bending strain should have opposite signs at the top and bottom fibers as the neutral axis lies within the depth of the section. Bending stresses at the center of the trusses at the top and bottom fibers of the top and bottom chords, were calculated by subtracting the calculated axial stress component from the total stress from the strain readings.

Bending moment in the chords was calculated from bending stresses with the following formula:

\[
\sigma = - \frac{M \cdot y}{I}
\]

or

\[
M = - \frac{\sigma \cdot I}{y}
\]

Where:

\[ M \] = Bending Moment
\[ \sigma \] = Bending Stress
\[ y \] = Distance from neutral axis to extreme fibre, positive upwards
\[ I \] = Moment of Inertia of the chord section about the longitudinal bending axis

Bending moments calculated separately from bending stress values at the same position on a chord should give the same result. Bending moments for top and bottom chords at the central
position are plotted with applied load on the y-axis in Appendix 3 for trusses 3 to 10 in Figures A3.3.10, A3.4.6, A3.5.6, A3.6.6, A3.7.6, A3.8.6, A3.9.6 and A3.10.6.

**Load displacement data**

The displacement variable in the load-displacement graphs in this chapter is vertical downward deflection of the lower surface of the bottom chord at mid-span, measured by a mechanical dial gauge extensometer. Readings of deflection and load at the load cell were taken at 0.5kN loading increments over the course of each test.

**6.2 Truss 2D finite element tests**

Finite element simulations of the truss tests were carried out to provide an analytical reference for the comparison of structural behaviour of the trusses. The following features of the experimental set-up put an analytical solution beyond the scope of hand calculations used to solve pin jointed trusses:

- Top and bottom chords were continuous along the length of the truss, transferring moment across truss nodes
- A small eccentricity of 60mm between horizontal positions along chords where diagonal and vertical members were clinched to horizontal chords created additional bending in chords
- Clinch shear deformation at the connection nodes was non-linear, as established in Chapter 4
- Plastic buckling occurred in chord sections under concentrated axial force and bending moment

Clinches were applied in groups of between 1 and 3 in the experimental truss tests to join the internal members to the horizontal chords, and to clinch a steel sleeve at the center of the chords in truss 2. The clinch shear tests in Chapter 4 investigate the magnitude of shear deformation in clinches connecting cold-formed steel. The effect of shear deformation in the clinch on the overall truss stiffness behaviour is illustrated in the 10 truss load-displacement graphs in Section 6.4. Experimental and finite element load-displacement slopes were compared with a theoretical finite element model with no shear deformation at the connection nodes where connections in the internal struts and ties were pinned to the top and bottom chords.

Two modes of truss failure were observed in the experimental tests:

- Failure of the clinch or clinch group at a connection node
Buckling of the cold-formed steel components

Truss buckling was influenced by large deflections causing geometric distortion along the length of the chords. Large deflections can be analysed by reading vertical deflection at mid-span in the load-deflection graphs for each truss. Stiffness of chord sections and internal components influence deflection behaviour. Shear deformation in truss clinches also contributed to mid-span deflection. Shear deformation at one clinch node had an influence on shear deformation behaviour at other clinch nodes in the truss.

A combination of shear deformation in clinches and axial and bending strength of the cold-formed steel components therefore controlled buckling capacity of the truss. The truss could also reach its capacity by failure of a clinch or clinches at connection nodes. Mode of failure was determined by a state of balance between clinch capacity and axial, flexural and buckling capacity of flat and folded cold-formed steel parts.

The 2D finite element analysis in this Section was carried out to establish the influence of clinch shear resistance on the capacity of the truss when overall failure was initiated by the failure of the clinch or clinches at a connection node. Results from the 3D chord section finite element analysis in Section 6.3 were compared against axial forces and bending moments from 2D finite element tests to investigate buckling strength limits of chord sections in 3D non-linear elasto-plastic buckling finite element tests.

The finite element program ABAQUS was used for 2D numerical calculations. 2D truss modelling was carried out by establishing nodal co-ordinates in a text input file, and by mapping elements onto the nodes. The post-processing program ABAQUS POST was used to create displacement and colour coded line force and stress diagrams.

6.2.1 2D beam elements

The truss finite element analysis was carried out in a 2 dimensional plane, all displacements were in the plane of the rectangular outline of the truss. All steel components were represented by rod or beam type elements.
6.2.2 Struts and ties

Diagonal tension ties and vertical compression struts were represented by axial rod elements in two dimensions, as shown in Figure 6.17. Each element was defined by two nodes and was effectively a linear stiffness relationship in two dimensions between two nodes, the stiffness representing the axial straining of the components. As bending resistance was not significant in internal members, single elements pinned at the ends were used to represent each component, giving two active degrees of freedom at each node. Each rod element's physical properties were defined by the steel material properties established in Section 3 and the cross-section area of the component.

![Axial expansion and contraction](image)

Figure 6.17, Axial rod type element used for internal members

6.2.3 Chord members

Beam elements with cubic displacement modes, as shown in Figure 6.18, were used to represent top and bottom chord members in the finite element models. Chord members have continuous bending resistance along their length, with the internal members pinned to the chords. Cubic beam elements modelled axial straining and shear deformation along the length of the element in addition to bending, with three active degrees of freedom at each node. No bending moment was transferred to the chords from the internal members in the finite element models. Beam elements were spaced along the length of the chords in the finite element models at spacings of three between pairs of vertical struts, as shown in Figure A3.0.5 of Appendix 3.
Parameters required to define a chord member with the type of beam element used were:

1. Modulus of elasticity, E, 184GPa from Section 3.1.4
2. Shear modulus G=70GPa:

\[ G = \frac{E}{2(1 + \nu)} \]

where

- \( G \) = Shear modulus
- \( E \) = Modulus of elasticity
- \( \nu \) = Poisson's ratio, taken as 0.3 for mild steel

3. Cross-section area, calculated from the width of the chord flat before it was folded, multiplied by the steel thickness

4. Moment of inertia of the chord cross-section, calculated by dividing the half cross-section into four parts – web, flange and two return lips as shown in Figure 6.7, finding the vertical location of the neutral axis from the top of the section, and taking second moments of area of each part about the neutral axis. This value was then doubled to account for two chord sections positioned back to back. The small radius of curvature at the corners of the cross-section shown in Figure 6.7 were not considered in calculating the moment of inertia.

BS5950 Part 5 [7] Section 3.5.1 states that when calculating section properties for design purposes, 'the actual round corners are replaced by intersections of the flat elements'. Table 6.4 lists the values used for the truss components in the truss finite element tests, including moments of inertia.

6.2.4 Clinch shear deformation

Shear deformation of single clinches and clinch groups joining components in the truss was taken into account in finite element models by using connection elements with the non-linear stiffness characteristics of clinches, established in the experimental Instron shear tests in Chapter 4. Clinch connection elements behaved in a similar way to the axial rod type elements shown in Figure 6.17, as a displacement stiffness relationship between two nodes in 2 dimensional space. The two nodes that define the connection elements were located at the same position in space,
and the stiffness relationship between the two nodes was defined by a simplified version of the non-linear load-displacement characteristics of the clinch. 

In the truss finite element models, at each location where an internal component was connected to a chord member, a clinch connection element was applied to represent shear deformation of the connection at that point, as shown in Figure 6.19.

![Figure 6.19, Clinch connection elements](image)

Stiffness behaviour of the clinch at a particular orientation and thicknesses of steel being clinched were taken into account in defining the clinch connection in the finite element models. Experimental load-displacement data used to define the simplified models was taken from Chapter 4. Clinch simplified stiffness models for the thickness combinations clinched in fabricating trusses, and used in the finite element tests, are plotted in Figs.A3.0.6 to A3.0.8 of Appendix 3.

6.2.5 Boundary conditions

Half the length of the trusses were modelled in the finite element tests, with boundary conditions at the mid-span end applied to simulate central symmetry, as shown in Figure 6.20. The vertical support end of the finite element model was restrained against vertical translation at the bottom node. Lateral and torsional restraints in the truss finite element models were not required in the two dimensional analysis.
6.2.6 Loading

Load was applied vertically downwards at a single node point in the finite element truss models, corresponding to the position of loading in the experimental tests. The central symmetry arrangement in the finite element models allowed the spreader beam effect of two equal load positions in each half of the truss to be modelled. The finite element tests were non-linear static tests, and the load was controlled by the non-linear Rik’s algorithm facility, as described in Section 3.2.3. A static load of 0.5kN was applied in the finite element tests at the loading position. With central symmetry, the factor applied by the Rik’s algorithm to the loads can be compared against the load measured by the load cell in the experimental work.

Sources of non-linearity in the truss tests were the non-linear stiffness model in the clinch connection elements, and large displacement non-linearity. The analysis was carried out over several non-linear increments, with the Rik’s algorithm controlling the magnitude of applied load in response to deform in the model. This method of load application was similar to the method of loading in the experimental work, where load was increased in steps with consideration of the structural behaviour of the truss.

6.2.7 Truss geometry

The nodal co-ordinates and node numbers of trusses 2 to 7 finite element models are shown on Figure A3.0.5 of Appendix 3. In Figure A3.0.5, nodes 1 and 100, 2 and 101, 4 and 102 etc. are
in the same position and are the two nodes defining the clinch connection element at that point. Results from the 2D truss tests are discussed and analysed in Section 6.4.

6.3 Truss chord buckling analysis

In the truss experimental tests trusses often failed by local plastic buckling, or geometric elastic buckling of the chord sections between points of lateral and torsional restraint as noted in Table 6.2. The chord sections failed under a combination of high axial force and bending moment. By analysing the chords in the context of the current British Standard design guidelines and finite element analysis in the following sub-sections, the buckling behaviour that was observed in the experimental tests was investigated. Four design cases of chords were chosen for analysis:

1. \( t = 1.2\text{mm}, \text{center span} \)
2. \( t = 1.2\text{mm}, \text{end span} \)
3. \( t = 1.5\text{mm}, \text{center span} \)
4. \( t = 1.5\text{mm}, \text{end span} \)

Loading was considered in axial compression and sagging bending - the loading pattern characteristic of the top chord in the lattice trusses. The end span was considered pinned at the free end and continuous where it joined with the center span. The center span was considered continuous at both ends. Torsion was not applied to the finite element models or included in the British Standard buckling capacity checks as the wooden insert blocks (Figure 6.13) that were cut to fit between lateral restraints in the experimental tests provided torsional fixity at the ends of the chord spans of many of the trusses. Also with no moments being applied out of the plane of the truss on the experimental tests, out of plane bending was not considered. In the finite element tests moment was applied to one end of the end span and to both ends of the central span (Figure 6.22).

Buckling modes that were observed in the experimental tests in the top chord under concentrated combined axial force and bending moment were:

- Warping of the flanges, lateral displacement and elastic geometric buckling over the central 2.48m span as shown in Figure A3.0.9 in Appendix 3
- Splaying of the t section flanges and local plastic buckling at approximately 3/4 span from the end in the 1.76m end span as shown in Figure 6.21
Buckling was also initiated by the failure of the clinch or clinches in the connection node adjacent to the buckled region of the truss chord.

6.3.1 BS 5950 Part 5 buckling checks

Buckling behaviour observed in the experimental tests was initiated by splaying of the chord section flanges and local buckling. In some cases on the center span the chord twisted at the center under concentrated axial force and bending moment. Central torsional restraints were added to trusses 7 to 10. BS 5950 local capacity and lateral buckling checks were carried out in the following sections to establish a current design guidance estimate for combined loading limits of the sections that buckled before and after the clinch connection nodes failed in the experimental tests.

Short strut buckling

BS5950 Part 5 [7] Section 4 gives guidance for calculating the effective widths of elements in cold-formed steel sections resisting local buckling under compression. With no consideration for lateral buckling this gave the short strut buckling axial limit. This limit was applied in the combined axial force and bending moment local capacity buckling check. For the top 35mm compression element in a 1.2mm thick chord, both sections 4.3 ‘Basic effective width’ and 4.4, ‘Effective width of plates with both edges stiffened (stiffened elements)’ of BS5950 Part 5 [7] gave an effective width of 0.99 times the actual width, practically no reduction for effective widths in local buckling.

The 70mm length web however was subjected to a stress gradient as shown in Figure 6.12 and was twice as long as the flange with the same thickness. Section 4.4.2 of [7] uses the average stress across the element as the stress parameter $f_{cm}$ to calculate the effective width of the element in Section 4.3 of [7]. Figure A3.3.6 in Appendix 3 shows the total strain measured at the top and bottom fibers of the top chord in truss 3 with 1.2mm thick chord sections. The average compressive stress $f_{cm}$ was 60.1% of the top fibre compressive stress and so the compressive stress value $f_c$ substituted back into Section 4.3 of [7] was 0.601 x $F_y$. 
With reference to BS5950 Part 5 [3]:

Fig B.2: $b_1 = 35$, $b_2 = 70$, Curve 2 - $K_1 = 4.16$

$$P_{cr} = 0.904 E.K. \left(\frac{t}{b}\right)^2 = 203 \text{ N/mm}^2$$

$$f_c = 0.601 \times 287 = 172 \text{ N/mm}^2$$

$$\frac{f_c}{P_{cr}} = \frac{172}{203} = 0.845$$

$$\frac{b_{\text{eff}}}{b} = \left(1 + 14 \left(\frac{f_c}{P_{cr}}\right)^0.5 - 0.35\right)^4 \times 0.2 = 0.83$$

where:

- $b = \text{width of element under consideration}$
- $b_{\text{eff}} = \text{effective width of element under consideration}$
- $b_1 = \text{flange width}$
- $b_2 = \text{web depth}$
- $K_1 = \text{buckling coefficient of an element}$
- $P_{cr} = \text{buckling stress resistance under axial load}$
- $f_c = \text{compressive stress on the effective element}$

This calculation suggested that the web of the 1.2mm thick chord section was 83% effective in resisting axial compression in comparison with a fully restrained element. The flange elements were fully effective and so the overall cross section omitting the stiffening lips was generally strong in compression when short strut buckling was being considered. A similar calculation for the web of the 1.5mm thick chord section gave $b_{\text{eff}}/b=0.95$. Short strut buckling calcs are summarised in table 6.6.

<table>
<thead>
<tr>
<th>Chord thickness (mm)</th>
<th>Web $b_{\text{eff}}/b$</th>
<th>Effective area (mm$^2$)</th>
<th>$f_c$ (N/mm$^2$)</th>
<th>$P_c$ (kN)</th>
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</thead>
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<tr>
<td>1.2</td>
<td>0.83</td>
<td>223.4</td>
<td>172</td>
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<td>1.5</td>
<td>0.95</td>
<td>304.5</td>
<td>172</td>
<td>52.37</td>
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Table 6.6, Chord short strut buckling loads
Buckling resistance under axial load

Section 6.2.3 and Table 10 both of BS5950 Part 5 [7] were used to calculate the buckling resistance of the chords under concentrated axial load. A factor, $L_E/r_y$, equal to the value of the effective buckling half wave length of the chord section divided by the out of plane radius of gyration was entered in Table 10 of [7], and a reduced compressive stress, $p_e$ was obtained. Reduced compressive stress was multiplied by the full cross-section area of the compound chord to obtain the buckling resistance under axial load, $P_c$. $L_E/r_y$, $p_e$ and $P_c$ for the four chord test cases are described in Table 6.7 (a) and values are listed in Table 6.7 (b). $P_c$ was included in the overall buckling check.

Local capacity check

The local capacity check in Section 6.4.2 of [7] takes account of the effects of combined short strut axial capacity and fully plastic moment capacity in the equation:

$$(F_c / P_{cs}) + (M_x / M_{cx}) + (M_y / M_{cy}) \leq 1$$

where:

- $F_c$ is the applied axial load
- $P_{cs}$ is the short strut axial capacity
- $M_x$ is the applied bending moment about the x-axis
- $M_{cx}$ is the section plastic moment capacity (Table 6.5)
- $M_y$ is the applied bending moment about the y-axis
- $M_{cy}$ is the y-axis bending moment capacity

The y terms were not used in the calculation as there were no externally applied minor bending moments on the trusses. Local capacity check axial force and moment envelope lines are plotted in Figures 6.23 and 6.24 for the 1.2mm and 1.5mm thick chord sections.

Overall buckling check

Section 6.4.3 of [7] gives the following equation for the calculation of the buckling axial force and bending moment combined limits, neglecting y terms:

$$(F_c / P_c) + (M_x / M_b) \leq 1$$

where:

- $F_c$ is the applied axial load
- $P_c$ is the buckling resistance under axial load
\( M_x \) is the applied bending moment about the x-axis
\( M_b \) is the lateral buckling resistance moment about the x-axis

Parameters and formulas used to calculate \( M_b \) are described and listed in Table 6.7. Overall buckling axial force and moment envelope lines are plotted in Figures 6.23 and 6.24 for the end and center spans of the 1.2mm and 1.5mm thick chord sections. The overall buckling envelope was similar to the local capacity envelope for the end span. The center span was slender enough to buckle geometrically and achieved far less axial moment strength in comparison with local capacity limits.

6.3.2 Truss chord finite element buckling analysis

The application of two and three clinches at a connection node caused concentrated bending and axial forces to develop in the chord sections, leading to buckling for trusses 1, 3, 5, 8 and 9. A 3D finite element analysis of chord sections was carried out to establish failure criteria of the two thicknesses of chord sections tested, at the end and central spans of the top chord, with combined compressive axial force and sagging bending moment as loading variables. The material properties, shell element type and non-linear loading control used in the 3D chord section finite element tests are described in Section 3.2.
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<th>Description/formula</th>
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<td>Steel thickness</td>
<td></td>
</tr>
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<td>L</td>
<td>Length between lateral restraints defining chord length</td>
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<td>Width of flanges of compound section</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>Depth of compound section</td>
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</tr>
<tr>
<td>A</td>
<td>Cross-section area of compound section</td>
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<td>E</td>
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<td>(r_{cy})</td>
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<td>X-X radius of gyration of compound section</td>
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<td>(P_c)</td>
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<td>(\beta)</td>
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</tr>
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<td>(C_B)</td>
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<td>(C_T)</td>
<td>((1 + 1.5B/D - 0.25(B/D)^2)/(1 + 2B/D))</td>
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<tr>
<td>(M_E)</td>
<td>Elastic lateral buckling resistance moment:</td>
<td>5.6.2.2 c)</td>
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\[
\frac{\pi^2 AED}{2 [L_E/r_y]^2} C_b C_T \left[ \frac{1}{1 + \frac{1}{20} \left( \frac{L_E t}{C_T r_y D} \right)^2} \right]^{1/2} + 1
\]
| \(\eta\) | Perry coefficient: 0.002 \((L_E/r_y) - 40 C_B)\) | 5.6.2.1 |
| \(M_Y\) | Section Yield moment: \(I_x P_y / y\) | 5.6.2.1 |
| \(\phi_B\) | \((M_Y + (1 + \eta) M_E) / 2\) | 5.6.2.1 |
| \(M_B\) | Buckling resistance moment \(\leq M_F\): | 5.6.2.1 |
\[
\frac{M_E M_Y}{\phi_B + \sqrt{\phi_B^2 - M_E M_Y}}
\]
| \(M_P\) | Plastic moment capacity \(S_x P_y\) from Table 6.5 |          |

Table 6.7 a), Definition of parameters from BS5950 Part 5
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<th>End</th>
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Table 6.7 b), BS5950 Part 5 Buckling checks
Figure 6.21, Von Mises' stress (N/mm²) - 1.5mm end span chord

An example of a finite element stress contour plot of a 1.5mm thickness end span chord section buckled under combined axial force and bending moment is illustrated in Figure 6.21. This was a central chord span with boundary conditions as shown in Figure 6.22. Under concentrated axial and bending forces the stresses in the top flanges and in the bottom stiffening return have reached the plastic limit. Plastic buckling has occurred by splaying of the top flanges and local bending at the stress concentration at approximately 1/5 of the chord length from the center span.

Geometry

The two symmetrical parts of the chord section back to back were meshed with the finite element pre-processing program, HyperMesh. Two similar shell models were created each representing a length of the truss top chord between lateral restraints:
- End span - from the end of the chord to the position of loading, 1756mm in length
- Center span - between the position of loading and the central restraint, with central symmetry, 1244mm half length, 2488mm full length

The two cross section halves of the chord sections were rigidly connected between webs at node points along the length of the chords, represented by the clinch connection locations in Figure A3.0.1 in Appendix 3, spaced at intervals of 60mm and 379mm.

**Boundary conditions**

All nodes on the surface of the flat ‘T’ ends of the chord sections were restrained to move as though joined by a fully rigid ‘T’ shape. Rotations and translations of this rigid ‘T’ plane in 3D space was reduced to the rotation and translation of a single control node, as shown in Figure 6.22, at the vertical position of the neutral axis. The end span chord section was given simple support boundary conditions. The center span section had central symmetry at one end and a simple support at the other. Both models had lateral restraint and torsional restraint at both ends.

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**Figure 6.22, Chord section simple support and loading arrangement**
Loading

The arrangement of boundary conditions allowed axial force and bending moments to be applied to the model through control nodes with force and rotational loading. Load was controlled by the Rik's algorithm, described in Section 3.2.3. Local plastic buckling and plastic material characteristics caused non-linearity in the model.

Figures A3.0.11 to A3.0.14 of Appendix 3 show horizontal displacement at the point of axial loading on the graph x-axis for the 1.2 and 1.5mm thickness center and end span chord sections. The y-axis shows the factor applied by the Rik's algorithm to both the applied axial load and the applied bending moment simultaneously, to cause the chord section to fail.

The fixed ratio factor dictates that at all times the applied bending moment and the applied axial force are in a fixed ratio - the magnitudes of applied bending moment and axial force can increase and decrease but they always remain proportional to each other in the given ratio. Several tests are needed to establish an axial-bending envelope curve - each test gave a single limit for bending moment and axial force in a fixed ratio. The ratio of axial force to bending moment was set to values between 30 and 1000 to establish combined axial force and bending moment envelopes, Figures 6.23, 6.24, for the axial and bending capacity of the chord sections.

6.3.3 Analysis of chord buckling tests

Axial and bending local capacity and local buckling limits from BS5950 Part 5 [7] are plotted in Figure 6.23 for 1.2mm thick chords and Figure 6.24 for 1.5mm thick chords. These design envelopes were used for general guidance and indicative performance. Corresponding chord buckling finite element tests were carried out specifically to model chord sections in the experimental tests and showed slightly lower buckling limits in comparison. Figs.A3.0.9 and A3.0.10 of Appendix 3 show shell bending moment and Von Mises’ stress contour plots for the 1.2mm and 1.5mm thickness chord section finite element models respectively.
Figure 6.23, 1.2mm chord - bending moment and axial force envelope

Figure 6.24, 1.5mm chord - bending moment and axial force envelope
Bending moment - axial force envelopes in Figures 6.23 and 6.24 were upper limits of stability of chord sections, established from maximum loading values in Figures A3.0.11 to A3.0.14. There were significant changes in stiffness before these loads were reached, indicating that failure in experimental tests could occur at loads less than the load combinations shown in Figures 6.23 and 6.24. Failure in all the chord section finite element tests was by local plastic buckling which initiated overall buckling of chord sections, as shown in Figure 6.21.

6.4 Analysis of truss tests

Structural behaviour of trusses was recorded in load-displacement data, strain data, chord axial force and bending moment values, beam bending moment data and specific observations. Understanding the structural behaviour of the trusses was aided by analysing the stiffness characteristics of the clinches, truss finite element analysis, chord section finite element analysis and experimental test results.

6.4.1 Truss 1

The first truss to be fabricated and tested was 4.8m in length with similar fabrication details to trusses 2 to 8, except for the absence of return details at the ends of the vertical struts as shown in Figure 6.28, and the shorter overall length. Struts in truss 1 were plain ‘Z’ sections. In trusses 2 to 8 struts were given additional return details providing restraint in the struts against buckling at the ends and preventing local buckling observed in truss 1. Chord members were 1.5mm thick and ties and internal members were 1.0mm thick. Chord members were fabricated from two 1.5m chord lengths, spliced together at the truss center position with bolts through a steel sleeve. Three clinches were applied at each connection of internal component to chord section as shown in Figure 6.25, providing a strong strut/tie to chord connection node.
Using the BS 5950 Part 5 [7] Section 4 local buckling procedure the buckling resistance of the strut under axial load was calculated:

\[ K = 4 \]

\[ B = 65 \text{mm (Figure 6.6)} \]

\[ t = 1.0 \text{mm} \]

\[ \frac{b}{t} = 65 \]

BS 5950-5 Section 4 Table 5: \[ \frac{b_{\text{eff}}}{b} = 0.621 \]

\[ b_{\text{eff}} = 40.4 \text{mm} \]

\[ P_c = 40.4 \text{mm} \times 287 \text{N/mm}^2 = 11.6 \text{kN} \]

As the strut stiffener buckled at 0.62kN compression, the local buckling capacity, \( P_c \), of the compression strut was not reached. Buckling behaviour is marked on the experimental load displacement path in Figure 6.26 at 1.45kN applied load. Early buckling of the compression strut below the central loading position, at 0.62kN compression in the strut occurred because total compression in the strut was being transferred through a single stiffening return detail that was cut slightly too long, as shown in Figure 6.25. The buckle was a local buckle of the unstiffened plate element connecting strut to chord. A small degree of twisting was observed in the top chord in the region above the buckled strut.

The experimental test was continued after local buckling of the strut to an applied load of 3.11kN and the strut continued to resist buckling under compression. In Figure 6.26, the slope of the load-displacement paths decreased after local buckling of the compression strut, giving a small
change in overall stiffness. Overall failure of the truss occurred by buckling of the top chord at mid-span by bending in a 'V' shape under the single load point. Axial force and bending moment in the chord at failure are plotted in Figure 6.24, showing that the chord buckled close to the predicted local capacity and lateral buckling limits. Bending moment and axial force in the chord was interpolated from the finite element analysis output in Figures A3.1.1 and A3.1.2 in Appendix 3 at an applied load of 3.11 kN – the failure load of the truss in the experimental tests.

![Figure 6.26, Truss 1 - load-displacement paths](image)

The arrangement of continuous and pinned links between elements in Figure 6.27 was applied to the truss 1 finite element model. Clinch shear deformation was allowed at connection nodes at locations indicated by the ‘Clinch shear deformation’ symbol. This arrangement was used in all 2D truss finite element tests.
In Figure 6.26 the non-linear characteristics of the clinch shear deformations were included in the finite element model and the finite element stiffness gradient remained straight over the experimental displacement range. Three clinches were applied through a thickness arrangement of 1.5/1.0/1.5mm. Peak load for a clinch in this thickness configuration at 0° applied load from Table 4.4 was 7.13kN. With three clinches at 0° in series the peak force in the end diagonal tie to chord connection node was 21.39kN. Figure A3.1.1 in Appendix 3 shows an axial force of 0.36kN in the end diagonal component under 1kN applied load. Scaling this axial force by the peak applied load of 3.11kN gives 1.12kN at the connection node, considerably less than the capacity of the 3 clinches, 21.39kN. This shows that in the arrangement of three clinches at a connection node, connections remained in the elastic range and trusses buckled before failure of the connections occurred.

**Truss 1 summary**

- 1.5/1.0/1.5mm clinch layer arrangement
- Three clinches/connection node
- 1.12kN in clinch at truss buckling failure
- 6.3kN clinch peak load
• 3.11 kN truss failure load
• Local buckling of strut at half truss capacity
• ‘V’ shape buckling of top chord at section moment/axial capacity
• No observed plastic deformation or failure of clinches

6.4.2 Truss 2

An extra end return detail was added to the vertical struts of trusses 2 to 10 as illustrated in Figure 6.28 (b), giving additional stability to the vertical compression members. Two 3m chord top and bottom lengths were spliced together with clinches in a double sleeve splice arrangement to form a 6m long truss. A mistake in placing a diagonal component in fabrication resulted in the two end bays being cut off, reducing the overall length to 5.2m. The thickness of chord members of truss 2 was 1.5mm and the thickness of internal members was 1.0mm. Two clinches were applied at each connection node.

![Local buckle at 1.45 kN](image)

(a) Truss 1
(b) Trusses 2 to 10

Figure 6.28, Modified compression strut detail
The load displacement graph for truss 2 is shown in Figure 6.29. In the loading phase, the bottom splice weakened and opened as the clinches joining the splice deformed in shear at an applied load of approximately 7kN, 12kN tension in the bottom chord. The splice in the top chord did not deform as the chords were butted together in compression.

In the truss 2 finite element model, the clinch stiffness characteristics of the bottom splice connection were simulated at the position of the splice. The splice stiffness characteristics are shown in Figure A3.2.1 in Appendix 3. Initial stiffness in the weakened finite element model approximately matches the experimental result in Figure 6.29 when the splice began to open. In the finite element model the clinch splice began to weaken at an applied load of 5.5kN.

In the experimental test, when it was clear that the clinches would not resist any increases in applied load, loading was removed and the splice was fixed in position with 16 No. 5mm diameter bolts through the steel sleeve. Displacement had occurred in the clinches in the bottom splice, causing a permanent vertical displacement of approximately 7mm at the center of the truss. When load was applied again the additional bolts in the splice prevented further shear deformation. Load was applied to approximately 11.5kN when the double clinch in the top of
the left end diagonal failed. After the test the top half chords were separated. One of the two clinches was positioned half over the edge of the diagonal tie member, as shown in Figure 6.30, causing early failure of the connection node.

![Incorrect positioning of mechanical clinches of truss 2](image)

**Figure 6.30, Incorrect positioning of mechanical clinches of truss 2**

The shear behaviour of the flawed configuration and of the unflawed connection were re-created in an Instron shear test. Linear behaviour stopped in the flawed configuration at 6.5kN while in the shear test on the unflawed sample a change in linear behaviour occurred at approximately 12kN, as shown in Figure A3.2.2. For large shear displacements of 1mm and greater in the connection, the flawed configuration reduced in stiffness while the unflawed sample increased in stiffness to a mid-span deflection of approximately 4mm at a peak load of 15kN.

In a second truss 2 finite element test, the weak splice detail was removed and the flawed connection stiffness shown in Figure A3.2.2 was applied at the position connecting the left end diagonal to the top chord. In the truss 2 load displacement graph of Figure 6.29, displacement of the second finite element test was adjusted by adding 7mm to allow for permanent displacement caused by permanent shear deformation at the splice detail in the first loading cycle. The adjusted finite element load-displacement path matched the experimentally measured load-displacement path closely with a marginally higher load at peak applied loads. By applying non-linear stiffness of clinches in finite element tests, the experimental response of clinches in the trusses was predicted.

**Truss 2 summary**

- Strengthened compression strut detail
- 1.5/1.0/1.5mm clinch layer arrangement
- Two clinches/connection node
- 7.13kN predicted clinch peak load
- 11.6kN truss failure load
- Clinch splice not strong enough under tension in the bottom chord
- Splice bolted – test restarted
- Failure at misaligned clinch detail

### 6.4.3 Truss 3

Chord sections of truss 3 are shown in Figure 6.31 and dimensioned in Figure A3.0.1. Chord sections for this truss and the following trusses arrived prefabricated in 6m lengths, specially manufactured by Metsc PLC. Chord sections were formed from 2No. 3m long sections, brake pressed and butt-welded. Chord thickness for truss 3 was 1.2mm and internal struts and ties were 1.0mm thick with a single clinch at each connection node. The 0° peak load for the 1.2/1.0/1.2mm clinch was 4.9kN as illustrated in Figure A3.0.6.

![Figure 6.31, Truss 3 - welded truss chords](image)
Ellis was the first truss in the series for which strain and load readings were recorded automatically by the data logger. The clinch force and moment graphs are plotted against applied load in Figures A3.3.1 to A3.3.10 in Appendix 3. Figures A3.3.1 to A3.3.3 show the force in the clinch in the diagonal ties. One strain gauge was placed on either side of the same location on the internal members and the average strain was used to calculate the force in a single clinch. Figure A3.3.4 shows the force in the second right end compression strut. Unlike the force in the diagonal ties the force in the compression strut was not considered to be equal to the force in the connection node as there was a bearing contact between the ends of the strut and the chord sections as shown in Figure 6.28.

Figure A3.3.9 shows axial force in the top and bottom chords from experimental tests and from finite element tests, calculated by the method described in Section 6.3.2. For truss 3 all recorded strains are plotted and presented in Appendix 3. For trusses 4 to 10 only clinch force, chord axial force and chord moments are presented.

Recorded strain on the top and bottom surfaces of the top chord is plotted in Figure A3.3.5. Bending strain in Figure A3.3.6 on the top and bottom surfaces of the top chord was calculated.
by subtracting the axial component of strain from the measured strain. Figures A3.3.7 and A3.3.8 show similar readings corresponding to the bottom chord section. Experimental and finite element axial force was plotted in Figure A3.3.9 and bending moment was plotted in Figure A3.3.10.

![Truss 3 - Chord twisting at the top end span](image)

**Figure 6.33, Truss 3 – Chord twisting at the top end span**

Stiffness of the truss increased in the first stages of loading, this was because of slack created in the diagonal tension members during fabrication, which was noted and did not appear in the following trusses. The corresponding change in stiffness in the finite element test at 5.2kN was caused by failure of the single clinch connection joining the diagonal tension tie to the top chord close to the load point.

At an applied load of approximately 5.6kN in the experimental test, lateral movement and twisting of the top chord occurred between the load point and the support at the left end as shown in Figure 6.33, which caused a sudden deflection. Immediately after twisting the bottom clinch
on the left end diagonal failed by pulling out of the chord at 6.2kN applied load. In the truss 3 load displacement graph in Figure 6.32, the chord failure can be seen as a decrease in stiffness at 5.6kN applied load.

Shear deformation was observed in the clinches at that point and the lowest recorded clinch peak force was 4.1kN the left end diagonal (Figure A3.3.1). Transverse shear from lateral curvature leading to lateral buckling in the top end span could have reduced the clinch shear capacity from the predicted 4.9kN to the measured 4.1kN.

In Figure A3.3.10, the change in bending moment to applied load slope at approximately 5.25kN applied load and 0.13kNm sagging moment marks the point where clinches started to deform, initiating top chord twisting and buckling. The corresponding axial force in the top chord in Figure A3.3.10 was 12.0kN compression at 5.25kN applied load. This axial force and bending moment combination was far below the level predicted to cause buckling in the end span of the 1.2mm top chord in Figure 6.23. The twisting mode of failure in this case can be attributed to a lack of torsional restraint in the experimental set-up, which was present in the finite element tests carried out to establish the envelope. Torsional restraint was added to the chords of trusses 6 to 10.

An additional finite element test was carried out on the truss 3 arrangement with internal components pinned to chords. The pinned arrangement did not allow shear deformation in the clinched connections. Figure 6.32 shows the load displacement relationship of original finite element test compared to the modified finite element test in addition to the experimental result. The reduction in load-displacement slope in the pinned and clinched finite element models of 28% was caused by the shear deformation of the clinches between the internal members and the chords. This demonstrates the significant effect of the clinch shear deformation on elastic behaviour of the truss.

Truss 3 summary

- Metsec prefabricated sections with a welded splice details
- 1.2/1.0/1.2mm clinch layer arrangement
- One clinch/connection node
- 4.9kN predicted clinch peak load
- 4.1kN measured truss clinch peak load
- 5.6kN truss failure load
- Lateral/torsional buckling of top chord end span
- Clinch connection failure near predicted load

### 6.4.4 Truss 4

The thickness of the chord members of truss 4 was 1.5mm and the thickness of the internal members was 1.0mm. Three clinches were applied at each connection joining internal members to top and bottom chords. The initial displacement response to loading was linear and a failure load of 12kN was reached.

![Graph](image)

**Figure 6.34, Truss 4 – load-displacement paths**

Shear deformation was observed in the right end diagonal tie clinches at 10kN applied load. At this point the mid-span deflection was 10mm and the force in the right end diagonal tie clinches was 4.2kN as shown in Figure A3.4.2. As loading was increased, shear deformation in the clinches increased and deflection also increased. Bending curvature in the chords increased rapidly with applied load as shown in Figure A3.4.6. The point of failure of the right end diagonal connection node is illustrated in Figure A3.4.2 where the maximum tension in the tie per clinch was 4.45kN. In the left end diagonal in Figure A3.4.1 the clinch connection does not
follow the same behaviour and there is no change in clinch force at 10kN. After failure of the clinch in the left end diagonal the bottom chord began to move laterally, as shown in Figure 6.35.

Figure 6.35, Truss 4 - lateral displacement of the bottom chord

Clinch stiffness for this thickness arrangement is shown in Figure A3.0.8 of Appendix 3. Clinch peak load was 6.3kN and the clinch began to lose shear resistance at 4.5kN. In the truss test the force in the clinch was at 4.2kN when failure was observed in the left end diagonal and the remaining clinches reached a peak load of 4.45kN. Lateral curvature was observed in the right end top chord before failure of the clinch. Transverse shear between half-chords in the compound section applied shear into the clinch connections in addition to direct shear from ties and struts. This had a reducing effect on the clinch peak shear measured from strain gauges on diagonal ties.

Figures A3.4.7 to A3.4.9 show colour coded stress line plots of axial stress in internal components, axial force in chords and bending moment in chords for the truss 4 finite element test. Each of the three finite element plots are at the final increment of loading at 12kN at the end of the load-displacement path in Figure 6.34 labelled 'Finite element test with clinch connections'. Diagonals colour coded red are at an axial stress of 259N/mm², giving an axial force of 259N/mm² (stress) x 50mm (width) x 1mm (thickness) ÷ 3No. = 4.31kN/ clinch. Clinches in the diagonal colour coded orange are at an axial force of 3.31kN.
Figures A3.4.8 and A3.4.9 show an irregular distribution of forces in the chords. Bending moment distribution in the chords of Figure A3.5.9 were distorted by the eccentricity of 60mm between vertical and diagonal internal components meeting chords, illustrated in the truss general arrangement drawing in Figure A3.0.1.

The second finite element test carried out on truss 4 had pinned connections in place of clinch shear deformation connections between the internal members and chords. This gave a method of comparison of overall truss behaviour with and without shear deformation at the clinch connection nodes. The pinned model was a theoretical model – all connections in cold-formed steel including bolts deform under applied load. The 13% reducing effect of clinch shear deformation on central truss deflection for truss 4 with three clinches in Figure 6.34 was less than the 28% reducing effect of clinch shear deformation for truss 3 in Figure 6.32, with one clinch. Increasing the number of clinches increased stiffness in the truss. Where clinches were shown to be close to failure near the end of the test the effect of shear deformation in the clinches on mid-span deflection was greatest.

**Truss 4 summary**

- 1.5/1.0/1.5mm clinch layer arrangement
- Three clinches/connection node
- 6.3kN predicted clinch peak load
- 4.45kN measured truss clinch peak load
- 12.0kN test failure load
- Visible shear deformation in clinches
- Transverse shear between half-chords affected clinch capacity
- Failure of triple clinch connection node close to predicted load
- Buckling of chord section after clinch failure

**6.4.5 Truss 5**

Chords and internal components of truss 5 were all 1.2mm thick and three clinches were applied at each connection node. In the test the top chord over the right end span started twisting at 8kN and the test was stopped. Wooden struts were placed between the top and bottom chords to prevent further buckling. Torsional restraints applied in later tests to restrain torsional rotation at
specific points along the length of the beam were not applied to truss 5. This made the top chord weak under compression and bending.

![Graph](image)

**Figure 6.36, Truss 5 - load-displacement paths**

The test was restarted and at an applied load of 9.8kN the wooden struts snapped out of their positions causing the bottom right span of the truss chord to buckle by bending over on one side as shown in Figure 6.37. This mode of buckling was initiated by the strut that eventually snapped out transferring force directly from the top chord to the bottom chord, in the final phase of loading. The final failure load was 11.8kN applied load and the mode of failure was torsional buckling of the right end span bottom chord.

There was no indication of failure at the connections when the chords buckled. Three clinches combined in a truss connection node were predicted to fail at approximately 20.25kN - the clinch shear resistance model in Figure A3.0.7 shows that each clinch has a peak load of 6.75kN. Force in the clinches varied linearly with applied load up to the point of buckling of the chord sections. Force in the clinches at this point in Figures A3.5.1 to A3.5.3 was approximately 2.5kN. Load was applied to the truss past buckling of the chord sections and the clinches in the left end diagonal in Figure A3.5.1 reach a peak load of 6kN, close to the maximum individual peak shear capacity of 6.75kN.
Figures A3.5.1 to A3.5.6 of Appendix 3 show applied load against axial force and bending moment for truss 5. Deviations from the linear relationship between axial forces and applied loads in the internal components in Figures A3.5.1 to A3.5.4 above 9.8kN marked twisting buckling behaviour of the truss top chord at that load level. The difference in strain readings on either side of the compression strut in Figures A3.5.4 shows that twisting of the top chord transferred moment out of the plane of the truss into the internal components as load was applied. The top chord buckled in a twisting mode above 10kN applied load. The top chord center span 40kN compressive axial force and 0.55kNm sagging bending moment combination for truss 5 is plotted in Figure 6.24. The truss 5 axial and bending limits were marginally above the finite element and lateral buckling check envelopes for the center span.

Axial force and bending patterns in Figures A3.5.5 and A3.5.6 were linear below an applied load of 9.8kN. This was because the strain gauges for those central locations were positioned away from the buckled end span. The finite element load–displacement graph for the case with clinch connection stiffness in Figure 6.36 shows how the truss displacement behaviour would have progressed if buckling and twisting had been prevented, reaching a peak applied load of 14.4kN.
The finite element test with pin jointed connections gave a comparison of how shear deformation in the double clinch connections affected deflection of the truss. There was a reduction of approximately 8% in truss stiffness arising from the shear deformation in the clinches.

Finite element output Figures A3.5.7 to A3.5.9 show axial forces and bending moments in the truss at 8kN applied load, at a mid-span displacement of 13.7mm in Figure 6.36. This was the point in the test when the chord section started to buckle. The force in the diagonal tie clinches from Figure A3.5.7 was $100\text{N/mm}^2$ (stress) $\times 50\text{mm}$ (width) $\times 1.2\text{mm}$ (thickness) + 3 mechanical clinches = 20kN, within the clinch linear range below 5.75kN in Figure A3.0.7. This corresponded to the experimentally measured clinch force at 8kN applied load of approximately 2kN in the left end diagonal in Figure A3.5.1.

While variation of chord axial forces in Figure A3.5.8 was regular along the length, variation of chord bending moment in Figure A3.5.9 was irregular. This was caused by the eccentricity of forces framing into chords from internal members. Bending moments around the positions of eccentric connection were higher than actual mid-span bending moments.

**Truss 5 summary**

- 1.2/1.2/1.2mm clinch layer arrangement
- Three clinches/connection node
- Failure load of clinches not reached
- 6.79kN predicted clinch peak load
- 6kN measured truss clinch peak load
- 9.8kN test failure load
- Torsional buckling of top chord and lateral buckling of bottom chord
- Attempt to restrain buckling with wooden supports

**6.4.6 Truss 6**

The general arrangement of truss 6 was similar to the general arrangement of truss 5, with chords and internal components 1.2mm thick and with one clinch per connection in truss 6 compared with two in truss 5. This arrangement was intended to make the connection sufficiently weak to cause failure at the connection node at 6.75kN, before buckling occurred in the chords between positions of restraint.
The load-displacement graph in Figure 6.38 shows the linear phase of load-displacement response in the experimental test ending at an applied load of approximately 6kN. At this point the single clinch in the right end diagonal to top chord node failed. The force in this clinch was plotted against applied load in Figure A3.6.2. There was a linear relationship between applied load and clinch force up to 4.5kN in the clinch. This was less than the experimentally measured peak load of 6.79kN illustrated in Figure A3.0.7 and is also below the end of the linear clinch response in the simplified clinch model at 5.75kN.

When the clinch lost shear resistance in the truss test, the top chord at the center span buckled laterally. Figs A3.6.5 and A3.6.6 show that the top chord at the center of the truss does not build up any further sagging bending moment greater than 0.07kNm. The bottom chord at the same time reaches sagging bending moment of 0.40kNm.

The test was continued past the failure of the clinch and at larger mid-span displacements of 30mm and greater there was visible shear deformation of approximately 0.5mm in the clinches at the diagonal tie to chord member connections. The remaining monitored clinch forces at the left end diagonal in Figure A3.6.1 and the second right end diagonal in Figure A3.6.3 reach peak
loads of approximately 5kN. The dial gauge displacement extensometer was removed from beneath the truss at a displacement of 40mm, load was again applied to 7.5kN when the truss had lost resistance to loading. When the dial gauges were removed and the mechanical clinches had lost shear resistance, the bottom chord section buckled outward at the center.

In the load-displacement graph in Figure 6.38 the finite element test with pinned connections had a higher initial stiffness. There was also a higher peak load as was expected because shear deformation and failure of clinches in the pinned model was not in effect. Variation of axial tension and compression straining of the struts and ties in Figures A3.6.1 to A3.6.4 was linear with applied load below 6.3kN. The non-linear relationship between clinch force and applied loads higher than 6.3kN in Figures A3.6.1 to A3.6.3 suggests the shear forces in the clinches of 4.5kN and 5.0kN were below the predicted clinch failure load of 6.75kN in Figure A3.0.7. Failure of the truss is initiated as the non-linear clinch stiffness response becomes significant at approximately 5 kN, \( \frac{3}{4} \) of peak clinch load.

A pattern of behaviour was observed in many of the truss tests: clinch shear deformation became visible at the end of the linear clinch load-displacement characteristics. The shear deformation in the clinch caused downward deflection of the truss and additional deflection accelerated clinch non-linear response up to peak clinch force. When the clinch peak forces were reached, shear resistance in the connections was lost and deflection was increased, causing the chord sections to buckle.

Section 8.6.2 of BS5950-7 [7] gives the minimum connection shear resistance to resist the longitudinal shear force between half-chords:

\[
F_s = 0.25Q(s/r_{cy})
\]

where:

- \( Q \) is not less than 2.5% of the design axial force,
- \( s \) is the spacing between connections along the length of the compound member,
- \( r_{cy} \) is the minimum radius of gyration of one channel.

Measured clinch shear capacity was approximately \( \frac{3}{4} \) of Instron measured shear capacity.
In Figure A3.6.5 axial force measurement at the strain gauges in the top chord was distorted by warping of the chord section at mid-span. To balance axial tension in the bottom chord, the top chord required a maximum axial compression at mid-span of 60kN. This gave:

\[
F_s = 0.25 \times (2.5\% \text{ of } 60\text{kN}) \times (379\text{mm} + 11.5\text{mm}) = 12.35\text{kN},
\]

greater than twice the 45\(^\circ\) single 1.2/1.2/1.2mm clinch capacity of 5.66kN (Table 4.4), indicating that clinch capacities in the truss were affected by transverse shear between half-chords in truss 6.

**Truss 6 summary**

- 1.2/1.2/1.2mm clinch layer arrangement
- One clinch/connection node
- 6.79kN predicted clinch peak load
- 4.5kN measured truss clinch peak load
- Transverse shear between half-chords affected clinch capacity
- 6.0kN truss failure load
- Visible shear deformation in clinches
- Lateral buckling of bottom chord after clinch failure

**6.4.7 Truss 7**

Two torsional restraining blocks described in Section 6.1.5 were inserted at positions of lateral restraint and at mid-span under the flanges of the top and bottom chords of truss 7 and following trusses. Positions of lateral restraint on the trusses are illustrated in Figure A3.0.4, positions of torsional restraint are illustrated in Figure A3.0.5.
In truss 7 chord members were 1.5mm thick and internal struts and ties were 1.0mm thick. A single clinch connected internal members to chord members. During the test, the torsional restraining blocks at the ends, load positions, and center of the top chord were in contact with the vertical uprights, resisting torsional rotation. Twisting of chord members did not occur. The test was carried out beyond the point of failure of the clinches at 6.1kN applied load.

Experimental and finite element axial forces and bending moments in Figures A3.7.1 to A3.7.12 showed a close match up to and beyond the failure of single clinch connection nodes. Figure A3.0.8 shows clinches in the 1.5/1.0/1.5mm thickness combination beginning to deform at 4.5kN and reaching a peak load of 6.3kN in the Instron shear test.

Shear forces in clinches in Figures A3.7.1 to A3.7.3 showed the linear relationship between applied load and clinch force ending at approximately 7kN applied load and 5kN clinch force. In Figure A3.7.2, the force in the right end diagonal clinch at 7kN applied load was 5.2kN. The non-linear clinch load-displacement relationship above 5kN shear force was reflected in the truss 7 load-deflection graph Figure 6.39. Clinches started deforming above 4.5kN at 6kN applied load. The truss was in the non-linear range of load-deflection behaviour at 7kN applied load.

Figure 6.39, Truss 7 - load-displacement paths
With increased applied load above 7kN the clinches were in the low stiffness peak shear range between 0.4mm and 5mm shear deformation in Figure A3.0.8.

Figures A3.7.7 to A3.7.9 show the finite element axial stress, axial force and bending moment colour coded line diagrams for truss 7 at an applied load of 1kN. At 1kN the truss was in the elastic range and axial stress and forces were uniformly distributed. In FigA3.7.7 the two vertical compression struts towards the center of the truss had a small compression value of 10N/mm² carried over by bending resistance in the chords. The bending moment line diagram in Figure A3.7.9 showed hogging and sagging bending moments where diagonal ties and vertical struts met chords with an eccentricity of 60mm. Bending moment in chords of the central span was uniform sagging.

Corresponding axial stress, axial force and bending moment ABAQUS line diagrams at an applied load of 6.0kN, are shown in Figures A3.7.10 to A3.7.12. The applied load of 6kN marked the end of truss linear load-deflection response. Axial stress in internal components and axial force in chords was evenly distributed as before but at a higher magnitude. At 6.0kN there was a redistribution of forces through axial straining in internal members and shear deformation in clinch connections. The distribution of bending moment along the chords in Figure A3.7.15 changes, with considerable hogging moment at the left end of the bottom chord and sagging towards the center, and less variation in bending around points of eccentricity between diagonal and vertical internal components meeting chords.

The load-deflection graph in Figure 6.39 shows a close correlation between experimental and finite element truss tests. The initial stiffness of the simplified clinch shear resistance model in Figure A3.0.8 was higher than experimentally measured response and in Figure 6.39 the experimental test had a slightly higher initial stiffness than the finite element model predicted. The simplified clinch model caused the clinch elements in the finite element model to deform and influence non-linear deflection of the truss at the same point recorded in the experimental test. This behaviour in the finite element and experimental tests was marked in Figure 6.39 by a change in truss stiffness at 6kN applied load.
Truss 7 summary

- 1.5/1.0/1.5mm clinch layer arrangement
- One clinch/connection node
- 7.13kN predicted clinch peak load
- 5.5 to 6.0kN measured truss clinch peak load
- 6.1kN truss failure load
- Additional torsional restraint provided in the top chord
- Torsional restraint effective
- Failure of clinch at predicted load

6.4.8 Truss 8

Truss 8 had a similar configuration to truss 7 with 1.5mm thick chords and 1.0mm thick internal components. There were two clinches at each connection in truss 8 compared to a single clinch in truss 7. Truss 7 failed in the single clinch connections. Additional torsional restraints were added to trusses 7 and 8. The failure mode of truss 8 was predicted to be more closely balanced between failure at the double clinch connection nodes and buckling of the chords.

Figure 6.40, Truss 8 – left end span top chord buckling
A small sudden buckle occurred at the top of the right end support strut at 9kN applied load. Twisting forces at the end of the top chord induced the buckle, causing a jump in deflection readings at 9kN applied load as shown in Figure 6.41. Deformation was not large enough to buckle the strut, however continued deformation with applied load as shown in Figure 6.40 appeared to initiate lateral buckling of the right end span chord section with increased applied load. The additional shear resistance of two clinches in the connections caused them to retain shear resistance while compressive force built up in the top chord, buckling laterally between restraints at the left end span at an applied load of 10.5kN.

In the finite element test load-displacement graph in Figure 6.41, the applied load to cause failure at the two clinch connections if the chord members had not buckled was approximately 12.3kN. Clinch shear deformation was observed at the double clinch connection nodes before buckling of chords occurred. This behaviour was consistent with chord buckling being initiated by loss of shear resistance in clinches.

![Figure 6.41, Truss 8 - load-displacement paths](image)

The distance between lateral and torsional restraints at the end span where the top chord buckled was 1756mm. Figure 6.24 shows mid-span axial force and bending moments in the top chord at
failure close to the values obtained from the 3D finite element plastic buckling tests and also to the BS 5950 local capacity and lateral buckling limits.

Force diagrams for the clinches in Figures A3.8.1 to A3.8.4 showed a linear relationship between applied load and force in the internal components. Individual clinches in the left and right end diagonals in Figures A3.8.1 and A3.8.2 reached a maximum force of approximately 5kN while the clinch in the second right end diagonal in Figure A3.8.3 reached a maximum force of 4.5kN. In Table 4.4 the 1.5/1.0/1.5mm 0° peak load was 7.13kN. Figure A3.0.8 showed a significant loss of stiffness above 5kN.

In Section 8.6.2 of BS5950-7 [7], connections between half-chords in compound members in compression must be able to resist the longitudinal shear force between half-chords: \( F_s = 0.25Q(s/r_y) \) where ‘Q’ is not less than 2.5% of the design axial force, ‘s’ is the spacing between connections along the length of the compound member and ‘r_y’ is the minimum radius of gyration of one channel. The maximum axial force recorded in the top chord in the experimental tests in Truss 8 (Figure A3.8.5) was 70kN giving an estimate of the magnitudes of the effect of transverse shear on clinch capacity:

\[ F_s = 0.25 \times (2.5\% \text{ of } 70\text{kN}) \times (379\text{mm} + 11.5\text{mm}) = 14.4\text{kN}. \]

Tension tie triple layer clinches were oriented at 45° to the axial force in the chords (Figure A3.0.1), and two 1.5/1.0/1.5mm clinches with a peak 45° shear resistance of 5.97kN (Table 4.4) were applied at each connection node in truss 8. The predicted shear resistance of each connection node was \( 2 \times 5.97\text{kN} = 11.94\text{kN} \), below \( F_s = 14.4\text{kN} \).

Lateral curvature displacement of the end span and lateral buckling of the chord was observed in the experimental test in the region of the clinch failing below capacity at 4.5kN, suggesting 0° clinch shear capacity in direct tension from the diagonal tension tie was reduced by longitudinal chord transverse shear force oriented at 45° to the double clinch connection node.
Truss 8 summary

- 1.5/1.0/1.5mm clinch layer arrangement
- Two clinches/connection node
- 7.13kN predicted clinch peak load
- 4.5kN measured truss clinch peak load
- Transverse shear between half-chords possibly reduced clinch capacity
- 10.5kN test failure load
- Top chord end span lateral buckling initiated by clinch shear deformation
- Failure of double clinch connection node on buckling of top chord

6.4.9 Truss 9

Thickness of chord members of truss 9 was 1.2mm, thickness of diagonals was 1.0mm and thickness of vertical struts was 1.0mm. The loading frame for the truss 9 test was redesigned to resist torsional buckling in the top chord more effectively under concentrated axial force and bending moment. The overall length was reduced to 3m and the vertical upright arrangement was changed from a single central z-section to a double arrangement of c-sections back to back as shown in Figure 6.43 (b). The bottom chord section orientation was inverted to accommodate struts within chord flanges. The change to truss section design was also intended to give torsional stability to chords – the double compression strut arrangement connected top and bottom chord sections between flanges.
Three loading cycles to 3kN were initially applied to truss 9 to settle the truss into the supports. Initial load applied to truss 9 was resisted by longitudinal bending in the top and bottom chords until observed slack in the diagonals appeared to be taken up. To remove tie slack a further load increment was applied to the truss and the strain gauge readings were reset to zero.

When load was applied to the truss in the experimental test the chord flanges splayed vertically under direct compression from the struts as shown in Figure 6.43 (b) and the truss did not gain the intended stiffness. In the load-deflection graph in Figure 6.42, the stiffness measured in the experimental test was significantly less than the finite element predicted stiffness. Mid-span deflections above 8.2mm in truss 9 were not recorded because of lateral deflection of the bottom chord.

Figure 6.42, Truss 9 - load-displacement paths
In the finite element load-deflection path in Figure 6.42 the frame response was not distorted by loss of compression stiffness at the ends of the struts. Chord sections were modelled by beam elements defined by a moment of inertia and a cross section area – the shape of the cross section was not used. The lattice system applied force into the clinches and the non-linear shear resistance characteristics of the clinches affected the load – mid-span deflection response of the truss. In the experimental test the lattice system was weakened by bending of the chord flanges and the loss of compression stiffness at the ends of the struts. Higher applied loads and higher mid-span deflections were required to bend the chords to obtain the equivalent levels of force in diagonals and clinches.
Connections in truss 9 were relatively weak with only one clinch per node. Shear deformation was observed in the clinches at the end of the test. The maximum recorded clinch force of 5.21 kN in the left end diagonal in Figure A3.9.1 is 6% greater than the predicted clinch peak force of 4.9 kN. At an applied load of 13.2 kN the bottom chord began to twist and buckle, causing the truss to fail. The mid-span dial gauge was removed at 6.5 kN applied load because of sideways deflection of the bottom chord.

Because the bottom chord was a tension member no restraint against twisting was provided. In the context of overall truss bending behaviour the orientation of the bottom chords of truss 9 and 10 were inverted from the previous design as shown in Figure 6.43 and were less able to resist buckling under sagging moment. This caused the bottom chord to buckle at the central position at an applied load of 13.2 kN. Lateral buckling was also initiated in the top chord. Buckling behaviour at 13.2 kN is clear on the bending moment and axial force diagrams for truss 9 in Figures A3.9.5 and A3.9.6.

**Truss 9 summary**

- New double outer compression strut detail
- Inverted bottom chord
- Shorter 3m length
- 1.2/1.0/1.2 mm clinch layer arrangement
- One clinch/connection node
- Clinch shear deformation observed
- 4.9kN predicted clinch peak load
- 5.21kN measured truss clinch peak load
- 13.2kN truss failure load
- Compression strut bearing caused chord flange bending and loss of strut compression resistance
- Lateral torsional buckling of bottom tension chord

6.4.10 Truss 10

Chord members of truss 10 were 1.2mm thick, compression struts were 1.0mm thick and diagonals were 1.2mm thick. An additional lateral and torsional restraint was applied to the bottom chord at the central position of truss 10, to avoid the bottom chord lateral torsional buckling behaviour occurring in truss test 9. Slack in the internal components appears in Figures A3.10.1 to A3.10.3 as a difference in the rate of change in measured force as load was applied, and as the components straighten from the initial slightly curved slack position. Figure A3.10.4 shows different measured force patterns on either side of the compression strut as it bent slightly when compression was applied.

Truss 10 was fabricated at the same time as truss 9 and had the same problem with bending of flanges under compression bearing from double outer compression struts (Figure 6.43). This is illustrated in the low levels of force monitored in the compression strut in Figure A3.10.4, showing a maximum of 1.6kN. Loss of stiffness from deformation of chord flanges under compression from double struts was also illustrated in Figure 6.45 by comparing the low experimentally measured load-deflection stiffness with the finite element predicted load-deflection stiffness.
Truss 10 gave a linear response up to the point of failure, as shown in the load-displacement diagram of Figure 6.45. Failure of the truss occurred at an applied load of 9.5kN when the clinch at the top of the right end diagonal failed at 4.9kN in the connection as shown in Figure A3.10.2. Additional torsional restraint at mid-span prevented chords twisting about the long axis of the chords.

The clinch stiffness diagram in Figure A3.0.7 shows a peak load for the 1.2/1.2/1.2mm thickness combination of 6.79kN, and a significant reduction in clinch stiffness starting at 5.8kN. Predicted clinch failure load was 39% greater than measured load at 4.9kN. Elastic lateral displacement of 4mm and torsional rotation about the long axis of 2.5° were measured in the top chord end span between restraints before failure of the clinch. This distortion could have contributed to the reduction in shear capacity of the clinch by introducing transverse shear into the double half-chord interface.

In Figure A3.10.6 there is greater bending moment in the bottom chord at mid-span towards the end of the test, in comparison with the top chord. The axial force diagram for the bottom chord in Figures A3.10.5 shows a more even distribution of axial force between bottom and top chords.
Truss 10 summary

- 1.2/1.2/1.2mm clinch layer arrangement
- One clinch/connection node
- 6.79kN predicted clinch peak load
- 4.9kN measured truss clinch peak load
- 9.5kN test failure load
- Torsional restraint added to mid-span bottom chord
- Elastic lateral and torsional warping of the end span
- Compression strut bearing on chord flange caused loss of intended truss stiffness
- Initial slack in ties

6.5 Discussion of test results

Truss mid-span elastic stiffness for the experimental, finite element pinned and finite element clinch shear deformation models are listed in Table 6.8. Experimental and finite element failure loads and peak loads for trusses 1 to 10 are summarised in table 6.1. Failure loads were loads causing clinch connections to fail or chord sections to buckle.

6.5.1 Influence of number of clinches on truss strength

The number of clinches applied at a connection node was significant in preventing failure at connections and raising the load at which truss failure occurred. An increased number of clinches determined mode of failure of a truss between chord buckling or clinch failure by increasing the limit of force in a connection and also by increasing elastic truss stiffness.

Shear deformation in clinches of approximately 0.5mm was visible in connection nodes of truss tests 3, 4, 6, 7 and 8. Experimental and finite element elastic stiffnesses of each truss are listed in Table 6.8. By comparing elastic stiffness in trusses allowing shear deformation at the connections with the theoretical pinned model that did not allow clinch shear deformation, it was possible to establish the effect of clinch shear deformation on overall deflection behaviour in each truss. The chord thickness and internal component arrangement of trusses 4, 7 and 8 was 1.5/1.0/1.5mm at the clinch connections. In table 6.8 the reduction in elastic stiffness of truss 7 with 1 clinch/connection node was 24%. The same value for truss 8 with 2 clinches/connection node was 16% and for truss 4 with 3 clinches/connection node, 10%. As the number of clinches was increased the elastic stiffness margin governed by shear deformation at connection nodes
decreased. With three clinches in trusses 4 and 5 the effect of reduction in truss stiffness from clinch shear deformation was as low as 10 and 11%.

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**Table 6.8, Truss Beam elastic mid-span deflection stiffness**

Truss 7 has 1 clinch/connection node. A comparison of pinned and clinched shear deformation finite element truss 7 models in Figure 6.39 showed that the truss peak load was governed by non-linear shear deformation in the clinches beginning at 6kN applied load. Two clinches were applied at each connection node in Truss 8. In Figure 6.41 non-linear behaviour contributed by clinching was less pronounced with no distinct mid-span deflection when the clinch simplified shear resistance model in Figure A3.0.8 reached the end of the linear response. Truss 8 reached an experimental failure load of 10.5kN. Similarly the three clinches in truss 4 gave the connection nodes additional shear resistance and the clinch shear deformation stiffness slope was marginally less than the pinned model as illustrated in Figure 6.34, with a peak experimental applied load of 12kN.

The steel thickness lay-up at the clinch connections for trusses 5 and 6 was 1.2/1.2/1.2mm. As for truss 7 in Figure 6.39 with one clinch/connection node, truss 6 in Figure 6.38 showed a marked change in stiffness behaviour at 5.5kN applied load when the single clinches reached the end of the linear response to loading in Figure A3.0.7. The effect of clinch non-linear stiffness in Truss 5 with three clinches per connection node in Figure 6.36 was less pronounced than truss 6. Truss 6 with one clinch reaches an experimental failure load of 6.0kN and truss 5 with three clinches reaches an experimental failure load of 9.8kN listed in Table 6.1.
6.5.2 Influence of chord thickness on truss strength

An increase in thickness in the chord sections from 1.2mm to 1.5mm gave the chords greater resistance against buckling under axial force and bending moment. With the increase in combined thickness the shear resistance of clinching was also increased, raising levels of forces in the chords before clinch failure occurred. Cutting and folding was more easily achieved in thinner chord sections, however the thinner chord sections demonstrated greater sensitivity to local buckling and lateral and torsional distortion.

Trusses 4 and 5 had three clinches per connection node, in truss 4 clinches had a lower shear capacity of 6.3kN compared to 6.79kN and chords were thicker in comparison with truss 5. Truss 4 failed by failure of the clinch connections and Truss 5 failed by buckling in the top end span. In Table 6.8 the experimentally measured elastic stiffness of truss 4 was higher than truss 5 and in Table 6.1 the peak load for truss 4 was higher than for truss 5. This demonstrates the dominant influence of chord thickness in truss strength when enough clinches were applied at the connections.

6.5.3 Experimental testing

Measured forces in the chords were used to help to understand the conditioning and behaviour of the truss and to determine whether the trusses were behaving as expected. Consistency in experimental testing was important for the generation of reliable and consistent analysis data.

Cold-formed steel buckling distortion

Cold-formed steel sections can be made light and strong by choosing the optimum folded form. Rectangular flat regions on the surface of cold-formed steel buckle under in-plane compression. Breaking up flat regions with folding adds strength to sections. Long slender sections also buckle between points of restraints under applied axial and bending forces.

Buckling of chord members under combined axial force and bending moment was a feature of five out of ten of the full-scale truss tests. Lateral restraints and torsional restraints described in sections 6.1.4 and 6.1.5 of Chapter 6 were successful in providing lateral and torsional restraints to the chords. In the truss tests these restraints were applied at positions where it was thought that lateral and torsional restraint would be required. The use of up to three clinches at each connection node caused concentrated axial forces and bending moments to build up in the
chords. This high loading combination induced lateral displacement and torsional buckling between the positions of restraint.

The truss beam chord sections can be made stronger in resisting lateral and torsional buckling if both types of restraint are applied to all available positions in all tests along the length of the truss. These positions are top and bottom nodes at the ends, loading positions and central positions, 10 locations in total.

**Effects of tie slack and eccentric compression strut details on truss behaviour**

Tie slack and the effect of bending of the chord flanges under direct eccentric compression between top and bottom chords greatly reduced the initial stiffness of trusses 9 and 10. The potential stiffness of the truss was reached at large displacement values when the slack in the diagonals was taken up and there was significant axial force and bending in the chords, affecting the suitability of the test data for analysis.

Slack was introduced in trusses 9 and 10 with double outside compression struts by clinching the diagonal tension struts to the parallel chords in one phase and clinching the vertical compression struts to the parallel chords in a second phase. The chords were then brought together creating the slack in the diagonals.

In the truss 1 to 8 method of fabrication all internal members are joined to either chord in series, allowing slack to be taken out of the truss. Emphasis should be placed on removing slack from the internal components in all cases. The arrangement of outside compression struts bearing on the top and bottom chord flanges was also unsuitable, causing a loss of compression resistance at the ends of the compression struts. Slack in the diagonals in trusses 1 to 8 was avoided by:

- Clinching all the internal members to one of either top or bottom chords
- Opposite ends of the internal components were mechanical clinched to the remaining top or bottom chord parts

Finite element tests for the second of type of truss in trusses 9 and 10 did not show the close load – mid-span deflection relationships with the experimental tests in comparison with the longer trusses 2 to 8. The double outside arrangement of compression struts in the experimental tests were unable to resist structural compression loads effectively, bearing on and bending chord
flanges—only a small elastic vertical displacement of the chord flanges was necessary to weaken the reaction to the ends of the struts. Using a 3m half-length beam for trusses 9 and 10 did reduce the axial force and bending moment combination in the chords and allowed force to be concentrated in connections and diagonal members in addition to chord central spans. However initial slack in the diagonal members and the weakness of the double outer compression struts bearing on the chord flanges in the experimental tests provided an unsatisfactory truss beam design for experimental investigation.

6.5.4 Finite element modelling

ABAQUS finite element models were used to model the experimental tests, giving a source of data for predicting experimental behaviour and also for comparison when irregularities such as steel buckling and fabrication errors occurred in the experimental tests.

2D beam elements

Comparison of experimental and finite element mid-span deflection in trusses 1 to 8 gives a good correlation over the full non-linear range. Struts and ties were modelled with axial rod elements. Chord section moment of inertia was established in a hand calculation using the parallel axis theorem; chords section finite element properties were defined with a moment of inertia and a cross section area.

The design of trusses 9 and 10 was flawed—double outside compression struts lost compression resistance through splaying of the chord flanges. Truss 9 and 10 finite element models did not model this localised flange behaviour, giving a method of contrasting experimental truss deflection and the intended truss deflection.

3D shell elements

Shell elements modelled cold-formed steel parts in several simulations of experimental tests, allowing elastic and plastic deformation and buckling in the experimental tests to be analysed. Buckling behaviour was simulated in the finite element tests by modelling the cold-formed steel with shell elements allowing plastic yield to be determined at five section layers through the thickness of the shell.
Combined axial force and bending moment envelopes were generated for the center and end spans of the chords in the truss tests in 1.5 and 2.0mm thicknesses in Section 6.3.2. Moment and axial force were applied in the chord shell sections through special boundary constraints at the ends. Chord sections failed by local buckling in the flanges, a mode of failure observed in the truss tests. Axial-bending envelopes were plotted on graphs with buckling limits established from the local capacity and overall buckling checks in BS5950-5 and experimentally recorded axial-moment limits. There was a good correlation between experimental, finite element and BS5950-5 buckling limits, especially for the end span where moment was applied at one end only. In the longer center span the finite element tests gave lower axial-moment limits in comparison with BS5950-5 buckling checks. Large displacement geometric warping and splaying of the chord flanges was observed in the center spans in the experimental tests that did not occur to a great extent in the end spans.

6.6 Chapter summary and conclusions

10 lattice trusses were fabricated from cold-formed steel sections and connected with clinches to investigate the behaviour of the connections in full-scale tests. 8 trusses were fitted with strain gauges and a load cell to monitor the response to loading up to failure. Analysis of the computer logged test data and comparison against finite element tests gave insights into the influence of clinch shear deformation on deflection behaviour in the trusses.

2D finite element models were generated that included the effects of shear deformation at the connection nodes. A second set of theoretical 2D finite element models were also generated using pinned connections in place of clinch deformation elements at the connection nodes. In these second finite element models two of three sources of non-linearity in the experimental truss tests were eliminated in the finite element tests – clinch shear deformation and buckling (the third source of non-linear behaviour was large displacement geometric effects). This gave an idealised load-deflection path for each truss that was used to estimate the effect of elastic clinch shear deformation on truss deflection by comparison with models including clinch shear deformation.

In the Truss 2 experimental test the shear resistance of the connection node at the top left end was weakened by misalignment of one of two clinches. By recreating the misaligned connection and testing in shear in the Instron testing machine the stiffness of the misaligned connection node
was established. The connection stiffness model was then simplified and applied in the 2D finite element model. Similarly when the clinch splice deformed in shear in the experimental test the same behaviour was observed in the 2D finite element test when the simplified clinch splice stiffness was applied. This demonstrated the robustness of the analysis method in modelling non-standard experimental behaviour.

Axial force and bending moment buckling limits in the top and bottom parallel chords were estimated by reference to BS5950 Part 5 and by carrying out 3D finite element tests on shell element models of chord lengths between positions of lateral and torsional restraint. Local buckling, lateral buckling and finite element axial force and bending moment envelopes for the 1.2mm and 1.5mm thick chord sections were plotted in Figures 6.23 and 6.24. Forces in the chords at failure of the trusses were superimposed for comparison.

Conclusions

1. The number of clinches at each connection node in similar trusses had a significant effect on failure load. By applying a single clinch at each connection of internal strut or tie to chord section, stresses in components of the truss remained in the elastic range and truss peak load was determined by the non-linear shear resistance of the clinch

2. When two clinches were applied at each connection, the connection node shear resistance was doubled. At load levels close to the peak applied load the truss had three non-linear influences — steel yield strength inducing local buckling, large displacement geometric buckling, and non-linear shear resistance characteristics of clinches

3. By increasing the number of clinches to three per connection node, the influence of material yield strength and large displacement geometric effects were more significant in determining the peak load. However the change in response from 1. to 2. above had a greater influence on peak load, causing a 'phase change' type effect in the influence of connection shear resistance on truss behaviour

4. Increasing the number of clinches above three will push the connection node shear resistance towards the idealised pinned connection model which was investigated in theoretical finite element tests, giving an increasingly reliable and predictable method of connection

5. In truss 3, 4, 6 and 10 experimental tests the recorded force in the clinch at failure of the clinch connection node was less than the predicted failure force. Taking the non-linear clinch load-displacement response in Figure A3.0.7 as an example, at the end of the
approximately linear response, near the first point on the simplified clinch path, clinch shear deformation begins to affect the deflection response of the truss. As the truss deflects buckling is induced in the chords and the truss fails. This effect is particularly pronounced in trusses 3, 6 and 10, where a single clinch was applied at each connection node and the peak force recorded in the clinches was approximately \( \frac{3}{4} \) of the predicted peak force.

6. Shear capacity of clinches can also be affected by transverse shear between half chord sections in the compound chords. This shear was introduced when lateral and torsional warping occurred in the chords. Chord warping can be in the elastic range and can also lead to lateral torsional buckling.

7. By applying lateral and torsional restraints it was possible to restrain lateral and torsional buckling of the top and bottom chords in the trusses. This induced failure of clinches at connection nodes.

8. Shear deformation in the connection nodes reduced the elastic stiffness of the trusses by 11 to 28% in trusses 3 to 8 from the idealised finite element pinned models. The number of clinches at each connection node affected the reduction in elastic stiffness.

9. Load-deflection paths in 2D finite element models showed a close match to experimentally measured load-deflection paths for truss tests 1, 2, 3, 4, 6, 7 and 8. This was achieved by applying a simplified set of the clinch load-displacement data at connection nodes. 2D finite element models did not allow lateral, torsional or local buckling which affected truss tests 5, 9 and 10 significantly.

10. Load-displacement response of clinches in shear was non-linear at loads past \( \frac{1}{4} \) of peak load. The initial response was idealised as linear in the simplified clinch stiffness model. By simplifying the clinch stiffness curves it was possible to include a simplified clinch shear resistance model in finite element tests giving an equivalent non-linear response to models including a detailed clinch model.

11. The design of the truss must carefully match the relative strength of the chord, tie and connection.

12. Chord sections of trusses 1, 4, 5, 8 and 9 showed axial force and bending moment load combinations within the range of limits established in the BS5950 checks and 3D finite element tests.
7 CONCLUSIONS

An extensive experimental and numerical testing programme was carried out to investigate clinch connections in cold-formed steel load bearing trusses. Clinching was demonstrated to be consistent in shear resistance and a reliable method of connecting cold-formed steel providing connections with high stiffness and deformation characteristics. Clinching is particularly suitable for use in cold-formed framing systems where clinching tools can be automated in the cold-formed steel assembly process.

Static and cyclic loading shear tests were carried out on clinches in different steel thicknesses in two and three layers of mild galvanised steel. Results of the shear tests add to existing test results on the shear resistance characteristics of clinch connections and provide a reference for cold-formed steel designers and fabricators.

7.1 Shear resistance of clinching

The characteristic shear resistance of clinches can be calculated using a simple, dimensionally correct equation presented in this thesis. The Instron testing machine was used to carry out shear tests with an electronic extensometer attached directly to the sample accurately measuring shear deformation and producing results to extended the current data-base of clinch shear resistance.

The following conclusions on clinching in cold-formed steel have been made:

- Shear resistance and deformation capacity of clinches increases with increased thickness of steel being joined. Both initial stiffness and peak load increase in with thicker steel layers

- In two layers of different thicknesses shear capacity of clinches is influenced greatly by the thickness of the steel layer on the punch side and to a lesser extent by the thickness of steel on the die side of the clinch. Steel on the punch side is deformed into the steel on the die side when a clinch is formed. Failure of the clinch is initiated on the punch side when the punch side steel pulls out or tears. This gives steel on the punch side a more significant influence in determining shear capacity.

- Shear resistance characteristics of rectangular clinches are greatly affected by the orientation of the applied load to the short edge of the join. At the 0° orientation the maximum shear resistance is achieved while at 90° with the applied load perpendicular to the short edge of
the join, the least shear resistance is obtained from the join. Peak loads at 90° are approximately 2/3\textsuperscript{rd} of peak loads at 0° for the same clinch configuration.

- Peak load varies linearly at angle between the maximum peak load at 0° and the minimum peak load at 90°
- Clinch peak loads were normalised and a linear regression was carried out on the data-base of results from the University of Edinburgh. Equation 4.6 was developed from a linear regression to predict the peak load of an S-type clinch with steel thickness, steel UTS and angle of applied loading as variables. Comparison was made between Equation 4.6 and equations developed by other researchers by applying clinch variables in the equations.
- The characteristic resistance Equation 4.9 was developed from the average resistance equation 4.6, taking into account the different standard deviations observed between the 0° and 90° clinch shear resistance sample populations. The characteristic design resistance Equation 7.1 is developed in Section 7.8 by applying a partial factor of safety of 0.8 to Equation 4.9.
- H-type clinches achieve a higher initial stiffness and peak load in comparison with the S-type clinch. The H-type clinch has less deformation capacity and reaches the peak load at a smaller displacement, giving less shear resistance in flexibility.

7.2 Simplified clinch model

The behaviour of cold-formed steel trusses using mechanical clinches can be accurately predicted using FE techniques provided an appropriate model for the connection is used. Clinch shear deformation was included as an essential part of the finite element models in special clinch stiffness elements. Each clinch element joined in-plane displacement degrees of freedom at two nodes with the non-linear clinch stiffness characteristics. The two nodes being joined corresponded to the location of the clinch connection in the layers of steel.

In the moment-rotation tests in Section 5.1 the clinch stiffness was modelled with the exact clinch load-displacement response established from the Instron clinch shear tests. This gave a complex non-linear moment-rotation response in the finite element tests that corresponded well to the experimental results where buckling of cold-formed steel did not occur. In the six clinch cantilever test in Section 5.2 the non-linear behaviour of the clinch group was analysed where
two clinches in the clinch group failed, transferring moment to the remaining four clinches and shifting the center of rotation.

Buckling of the cross beam occurred before the clinch group non-linear response was utilised in the experimental and finite element H-frame tests in Section 5.3 in this work and in [16 and 62]. The clinch stiffness model in both experimental and finite element tests affected elastic mid-span deflection stiffness of the H-frame crossbeam.

All forces were applied in the strongest direction at 0° to clinches in the truss tests in Chapter 6, an orthotropic stiffness model was not required. A simplified model of the clinch stiffness was applied in 2D finite element truss tests. Four points determined the non-linear response over three phases – elastic, yield plateau and unloading deformation. The close correlation between experimental and finite element truss mid-span deflection in Chapter 6 demonstrates the suitability of using a simplified clinch model to simulate clinch shear deformation in experimental tests. This also indicates that a simplified method of shear resistance prediction such as application of the clinch shear capacity equation 4.6 in a hand calculation can be applied to estimate the shear resistance of a clinched frame.

7.3 Cyclic shear tests

Shear resistance of clinches was not greatly influenced by cyclic loading up to 10,000 cycles with the cycle load less than 50% of clinch peak load. Cyclic tests were carried out using the Instron testing machine on clinches in two and three layers of steel. Connection samples were cycled 10,000 times between zero load and 50% of peak load at one full cycle every 2 seconds. Following the cyclic loading phase samples were tested in shear to failure. These final static tests on cycled samples were compared to shear tests on similar connection samples with no cyclic loading. Cyclic loading affected the shear resistance of the clinches causing some plastic deformation over the 10,000 cycles and creating a marked difference in the load-displacement paths.

The process of forming a clinch is non-linear - the area of steel local to the connection is forced and interlocked by plastic straining from pushing and shearing of clinch tool parts in two controlled forming phases. The material stress-strain curves in Appendix 1 show that there is little additional stress generated in the steel from plastic deformation and steel hardening in
comparison with the stress generated in the elastic range of straining. Residual stresses and forces in the connection are caused by the interlocked steel parts pushing against each other after formation of the connection. This restrains the parts of the connection in a static elastic state, giving a tight clinch with no play or shear deformation between the parent sheets of steel when the clinch is properly formed.

Cyclic loading to 50% of peak load caused localised plastic straining of the interlocked join, pushing small regions of the steel into the steel hardening range of strain and altering the shear resistance characteristics of the connection. Altered shear resistance characteristics can be seen in the cyclic clinch test load-displacement graphs of Appendix 1. In all cases the initial stiffness gradient of the clinches were increased. In most cases a similar or slightly higher peak load was achieved and ductility was not significantly affected.

7.4 Variability of clinch shear resistance characteristics

Mechanical clinches demonstrated a low variability of shear resistance characteristics between similar tests, with variability not much greater than the variability in the material. In Table 3.2 the standard deviation of the Ultimate Tensile Strength (UTS) of six 1.0mm thick steel samples was 2.72%. Table 4.6 lists standard deviations of peak load of four types of clinch connection samples. The six 1.0mm thickness test samples at 0° have a standard deviation of 3.49% and the 90° samples have a standard deviation of 3.90%. The difference of standard deviations between material samples and clinch samples was approximately 1% in both cases. Taking the assumption that the variation in steel UTS in the clinch variation tests accounts for 2.7% of the coefficient of variation of clinch peak loads, the remaining 1% can be attributed to variability inherent in the clinch connections after formation.

The coefficient of variation of UTS of six 2.0mm material samples was 2.77%, similar in magnitude to the 2.72% achieved with the 1.0mm thick material samples. Table 4.6 shows the standard deviation of peak loads of six clinch samples in 2.0mm steel thickness to be 1.5% at 0° and 2.55% at 90°. As for the 1.0mm equivalent samples the standard deviation of UTS is higher at 90° in comparison with the stronger 0° applied load orientation. The difference in standard deviations between 0° and 90° orientations has been taken into account in developing the characteristic clinch resistance equation 4.9 in Section 4.5.2.
7.5 Clinches compared to other connections

Mechanical clinches also demonstrated a low variability of shear resistance characteristics when compared with screw tests in similar thickness arrangements. Variation analysis of screw shear tests in Section 4.2.3 of Chapter 4 show a coefficient of variation in the screw samples in 1.0mm thick steel of 8.05%, and in 2.0mm thick steel 3.81%. Coefficients of variation of the screw test samples are significantly higher in comparison with the equivalent clinch peak loads joins as the peak is reached when the screws are being pulled along the treads in the large displacement range of response. Variation in material properties for the screw samples can account for a small proportion of the variation in peak loads for the 1.0mm and a high proportion of variation for the 2.0mm steel thickness tests.

While self-tapping screw peak loads are variable in comparison with mechanical clinches, the initial stiffnesses are regular but weaker in comparison with clinching. This weakness can be attributed to the mode of elastic deformation in a screw shear test – the screw rotates from the moment applied by the small eccentricity between the sheets being pulled apart. The eccentricity is equal to the average thickness of the sheets of steel being joined.

Henrob self-piercing rivets achieve higher peak loads in comparison with clinches – typically 2 to 2.5 times greater than the equivalent clinch at 0° peak shear resistance. Initial stiffnesses gradients measured from the shear tests for self-piercing rivets are less than those of the clinches, especially in the less thick 1.0mm steel samples. In the mode of shear displacement and failure of the Henrob self-piercing rivet, the rivet part acts as a structural link between the layers of steel, causing deformation and tearing of the parent material close to the connection until the connection is pulled apart. In the initial elastic stage of shear there is some twisting of the rivet component to a lesser extent than the self-tapping screw samples because of the squat curved shape of the rivet.

Shear test on pop-rivets give peak loads between the 0° and 90° peak loads of equivalent clinch tests. In 1.0mm steel thickness the elastic stiffness gradient was similar to the clinch sample at 90° and in 2.0mm thick steel the elastic stiffness was slightly less than the clinch at 0°. Pop rivets show less ductility in comparison with clinches with the connections failing in shear at a shear deflection of 1.0mm.
7.6 Rotational resistance of clinch groups

Clinch groups showed strong rotational behaviour with rotational deformation capacity under applied moment. The rectangular arrangement of four clinches subjected to moment and no applied axial force was a severe loading arrangement because of the direct leverage of forces evenly to each join and also because of the small distances between the clinches. Moment-rotation paths were similar to clinch load-displacement paths where the clinch group inherits shear resistance characteristics of the individual clinches.

In the clinch group moment-rotation tests where the clinches were spaced at the wider 25mm and 50mm distances, a higher moment was generated in the clinch group in comparison with the 15mm and 25mm spacing combinations. Because of a lack of restraint from the testing frame and the flexibility of the cold-formed steel c-sections, buckling and twisting of the c-sections occurred in some tests at higher applied moments. In the finite element moment-rotation tests the cold-formed steel c-sections in the experimental set-up were modelled with shell elements. Special orthotropic clinch connection elements were developed using the clinch load-displacement data from the Instron clinch shear tests. In several of the finite element moment rotation tests the non-linear orthotropic clinch connection elements gave a similar moment-rotation response in the finite element tests in comparison with the experimental moment-rotation tests. This was especially true for the experimental tests where there was no significant twisting of c-sections.

The non-linear moment-rotation behaviour of a group of six symmetrically arranged clinches was analysed in a cantilever finite element test in section 5.2.1. The non-linear orthotropic clinch shear resistance model established in Chapter 4 was applied in each clinch connection element in the finite element model.

The peak moment in the finite element test was compared against an analytical solution. In the analytical solution the angle of force applied to the clinches in the group was established. The force angle was then used to determine the clinch peak resistance by linear interpolation between $0^\circ$ and $90^\circ$ clinch shear resistances. To predict the peak moment the clinch group would generate, the peak force was multiplied by the lever arm distance between the centre or rotation and each clinch.
While there were six clinches in the group, the closest match between analytical and finite element peak moments was obtained by considering only the four outer clinches in the group in the analytical solution. The lever arm to the four outer clinches was greater than the lever arm to the two central clinches. This suggests that the outer clinches will had greater shear deformation in comparison with the inner clinches for a given rotation of the group, and that the outer clinches reached their peak shear resistance before the two inner clinches. However the below peak resistance of the two inner clinches were excluded in the analytical solution and further experimental and numerical investigation of the rotational behaviour of larger groups of clinches is recommended.

Following the peak moment, two of the clinches in the group of six failed before the remaining four, transferring load to the remaining four clinches. The center of rotation of the clinch group shifted to the center of the remaining four clinches and the moment sustained by the clinch group reduced to approximately 2/3rd of the peak moment sustained by the six clinches.

Full-scale clinched H-frame experimental tests carried out by Davies in [16] were modelled with finite element tests in this work. There was a close agreement between experimental and finite element load to mid-span deflection paths and peak loads. The clinch shear resistance model was applied to the clinch connection groups at the ends of the cross-beam in the H-Frame. Stiffness increase from moment resistance at the clinch connection groups was evident in the experimental and finite element tests. Failure of all H-frame experimental and finite element tests occurred through plastic failure in the H-frame cross-beams.

7.7 Clinch connections in full scale trusses
Lattice type parallel chord truss beams up to 6m in length were fabricated into cold-formed steel and connected by clinching in groups of 1, 2 and 3 clinches at connection nodes. Trusses were simply supported in a rigid testing frame with lateral and torsional restraints along their length. Vertical downward loading was applied equally through a spreader beam to two positions on the trusses and testing was carried out to structural failure. Strain gauges were applied at twelve positions on the trusses and strain was recorded to a data logger with corresponding measurements of applied load. Readings of vertical deflection at mid-span and corresponding applied load were taken by hand in each truss test.
Clinches were applied singly and in groups of two and three at connection nodes connecting internal vertical struts and diagonal ties to the top and bottom chords of the truss. At each connection node clinches were oriented with the short edge of the join parallel to the applied load - the stronger 0° clinch orientation. In the truss tests the clinching method proved to be a strong method of connection with good deformation capacity. In many cases trusses buckled between lateral and torsional restraints while clinch connections held truss components together.

By measuring strain on the external surfaces of the internal struts and ties it was possible to obtain the magnitude of force in the internal components. The internal component force is equivalent to the shear force in the connections at either end of the component. Analysis of graphs of internal component strain against applied load in the truss tests showed how shear force in the clinch connections varied with applied load:

- Clinch failure initiated overall failure of the trusses in trusses 2, 4, 6, 7, 8, and 10
- When clinch failure was the primary cause of the truss structural failure there was a linear relationship between the applied load on the truss and the force in the clinches during the tests, up to a point close to failure of the connections
- In tests with one and two clinches/connection node the point of clinch connection failure was balanced with buckling failure of the chords

Experimental tests were set-up with special emphasis on analysing forces at the connections. There was little static indeterminacy in the trusses apart from bending resistance in the chords and so in the truss tests forces are generally directed into the connection nodes. In an industrial parallel chord or pitched chord truss application it is likely that there would be more redundancy in the frame arrangement, and that there would be several clinches at each connection node. This would make the contribution of clinch shear deformation in reducing the stiffness of the structure less than the 15% estimated from the finite element truss tests.

The measured resistance of the clinches in the truss tests was approximately 0.8 times the predicted clinch resistance in the truss tests. This can be explained by two influences:

- Eccentricity between the line of the diagonal ties and the vertical struts meeting at the chords caused additional bending in the chords in the region of the clinch connection nodes.
Lateral bending in the chords caused differential displacement between the two half chords in each member, creating shear forces in the cinches joining the chords.

In half of the experimental truss tests carried out, buckling of the top and bottom parallel chords caused the structural failure of the trusses. Methods of lateral and torsional restraint against buckling were devised and applied as the tests were carried out. The positioning of strain gauges on the upper and lower fibers of the top and bottom chords gave strain data from which the bending moment and axial forces in the chords could be established. Graphs of applied load against bending moment and applied load against axial force in the top and bottom chords were analysed with reference to buckling behaviour noted in the experimental tests.

The general lattice arrangement of trusses 1 to 8 gave a satisfactory response under concentrated loading, partly because internal members all connected with chords along the central plane of the truss, as shown in Figure 6.43, avoiding out of plane eccentricities. Lateral and torsional buckling of chords occurred in many tests. Lateral and torsional restraints were added and improvised. By providing lateral and torsional restraint at supports and at regular lengths along trusses, buckling behaviour was restrained. Buckling limits of the trusses gave an additional dimension to the experimental investigation by allowing the behaviour of cinches to be analysed in a steel structure with realistic shear resistance limits.

ABAQUS Finite element models of the trusses were generated using beam elements with bending and axial force resistance for the chords. This first set of finite element tests were effectively in two dimensions with no consideration of lateral and torsional buckling of chords. Rod type axial displacement elements were used to represent internal components. The shear deformation of the clinch connections in the truss finite element tests was represented by a special clinch shear deformation connection element, joining all the internal components to chords. A simplified version of the clinch load-deflection data from the Instron clinch shear tests was used to define the stiffness characteristics of the clinch elements. There was a good agreement between the applied load to mid-span deflection paths of the experimental and finite element tests in the cases where there was no significant buckling of the chords in the experimental tests.
A second run of each of the two dimensional truss finite element tests was carried out with each of the clinch connection elements joining internal members to chord members being replaced by a pinned connection. This gave the theoretical effect of a truss with no shear deformation at the connections. By comparing the mid-span deflection to applied load stiffness relationships of the tests with clinch shear deformation against the tests without connection shear deformation, it was possible to estimate the effect of clinch shear deformation on overall stiffness of the trusses. The shear deformation at the clinch connections reduced the initial stiffness tangents of the trusses by approximately 15%, with a lesser reducing effect for trusses with greater numbers of clinches at each connection node.

A further set of three dimensional finite element tests were carried out modelling two unrestrained lengths of chord sections under combined axial force and bending moment. Cold-formed steel chord sections were modelled with shell elements. Plastic and steel hardening material properties established in the Instron material tests were applied to the shell elements, allowing plastic steel buckling to occur. A bending moment and axial force resistance envelope was established for the two unrestrained lengths of chord and for the two steel thicknesses used in the chord sections. BS5950 buckling checks and finite element chord strength envelopes were used in the analysis of the bending moments and axial forces in the chords measured in the experimental tests.

A lot has been learned about design of cold-formed steel trusses that can be applied to cold-formed steel trusses with many types of primary connection:

- The connection of internal components transferring compression to external components was sensitive to buckling in Truss 1
- In Trusses 9 and 10 compression struts were arranged to press down on the outside of the chord flanges where there was little stiffness
- Connections in cold-formed steel can be subjected to out of plane forces caused by section warping and buckling in addition to shear from direct tension and compression in the connected truss parts

7.8 Design considerations

In designing cold-formed steel framing systems it would not normally be acceptable for a truss to fail at the connections rather than through buckling. Failure of chords in the truss tests
represented the maximum possible load - the application of three or more clinches was sufficient
to ensure failure at connections was unlikely.

BS 5950 Part 5 Section A1.4, 'Calculation of shear capacity in tilting and bearing' for screws,
blind rivets and powder actuated fasteners gives the shear capacity of a connection in two equal
steel thicknesses as the lesser of:

\[ P_s = 2.1 \ t_3 \ d \ p_y \]

and

\[ P_s = 3.2 \ (t_3^3 d)^{1/2} p_y \]

where:

- \( P_s \) is the shear capacity of the connection
- \( t_3 \) is the thickness of the steel in contact with the connection head
- \( d \) is the diameter of the connection shaft
- \( p_y \) is the design strength of the steel

A design factor of 2.1 is being applied to the bearing strength of the steel at the connection shaft.
It is also recommended that the shear capacity of the fastener itself should be greater than 1.25\( P_s \),
in clinching there is no fastener with the connection being formed from the parent metal.

In the full-scale truss tests local major axis bending moments were induced at connection nodes
by the eccentricity between the diagonal ties and vertical struts meeting at a connection node.
Lateral bending in the chords occurred under concentrated compression and major axis bending.
With distortion of the layers of steel forming the clinches, measured clinch resistance in the truss
tests was typically 0.8 times the predicted average clinch resistance in tests where clinch failure
initiated the failure of the truss. By applying a partial factor of safety of 0.8 to the characteristic
clinch resistance equation in Equation 4.9, the following characteristic design equation is
obtained:

\[ F_c = (4.89 - 0.023 \cdot 0) \cdot t \cdot UTS \]

...Equation 7.1

Where \( F_c \) is the characteristic normalised design resistance of a clinch.
The overall load factor, $\gamma_f$, applied in cold-formed steel design is typically 1.55, and the material strength factor, $\gamma_m$, is 1.25. In Equation 7.1 the characteristic design clinch resistance is based on the average clinch resistance over the sample population less two standard deviations in Equation 4.9. Equation 4.9 also takes into account different standard deviations obtained from clinch tests at 0° and 90°.

In a cold-formed steel frame design where $\gamma_m = 1.25$ and $\gamma_f = 1.55$ are applied, clinch resistance can be obtained from Equation 7.1 and applied directly as the variability in clinch resistance is taken into account.

7.9 Suggestions for further research

Designers responsible for innovation in commercial framing systems need to have the analysed results of tests on clinched frames available to have enough confidence in clinching shear resistance to apply it in industry. Further clinch testing research will push the range of clinch design guidance from research publications into international design codes. Industrial partners can benefit from investigations in how the clinching system can be applied in the manufacturing process to give efficiencies over other connection techniques.

The current database of clinch shear resistance can be verified and extended with further shear tests covering:

- Clinches in two, three and four layers
- Different layer/load direction arrangements
- Different thickness combinations of steel
- Upper and lower ranges of steel UTS
- Combinations of steel and sandwiched plastic thermal breaks
- Different commercial clinch types
- Cyclic performance of the extended range
- Corrosion resistance
- Combination of clinching with adhesives to improve stiffness without loss of ductility
- Effect of out of plane forces on shear capacity
More data is also required on the non-linear moment resistance behaviour of clinches in groups greater than 4 and in unsymmetrical layouts. A simple analytical method for the determination of the moment resistance of groups of more than 4 clinches has yet to be established.

Shear resistance of clinches is the core influence affecting the suitability of the technique for use in framing systems compared with other more commonly used connections. This can be investigated in:

- Testing and numerical modelling of other cold-formed steel connections
- Full-scale frame tests taking account of bracing effects in framing systems
- Cyclic performance of clinching in braced frames
- Sensitivity of clinched trusses to design details
## APPENDIX 1 – INSTRON MATERIAL AND SHEAR TESTS

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**STEP 2 INCREMENT 12**

44 -`-, -1 TIME COMPLETED IN THIS STEP ^_. 76EE-02 TOTAL ACCUMULATED TIME 1.03

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- Simplified model
- Simplified modal force x 8
- Simplified modal force x 8 disp x 2

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- Finite element stiffness input
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APPENDIX 4 – PUBLISHED PAPERS


Comparative study of some mechanical connections in cold formed steel

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Abstract

In this paper a comparative investigation carried out on the shear behaviour of mechanical connections in thin gauge steel is described. The connection techniques considered are press joining, Henrob fasteners, pop rivets and self-tapping screws. Samples joining two strips of steel of 1.0-, 1.2-, 1.6- and 2.0-mm thickness are reported. Press joins are applied at orientations of 0°, 45°, and 90°. Peak loads of all types of join tested are tabulated. The variability of the peak loads of press joins and screw samples in 1.0-mm and 2.0-mm thick steel are established with six tests on each configuration. A variability analysis is also carried out on peak stresses obtained from elasto-plastic material tests on 1.0-mm and 2.0-mm steel samples. The load displacement responses of the four types of connection in the small displacement range of < 0.1 mm are plotted and compared. © 1999 Elsevier Science Ltd. All rights reserved.

Keywords: Mechanical connections; Steel; Cold formed

1. Introduction

There is tremendous pressure on the construction industry to reduce cost, improve quality, and build faster. Cold-formed steel elements exhibit a combination of low cost, quality assured material, which utilises the benefits of mass construction. With proper design highly efficient structural forms can be produced. To date most of the benefits of cold-formed steel have lain with the production of simple elements such as purlins and sheeting rails. Recently there has been a growth in the use of cold-formed steel in fabricated structures such as trusses and stud frames. An essential aspect in the efficient fabrication of cold-formed steel structures is the type of connection used.

The choice of which connecting technique should be most suitable for a particular cold-formed steel application will be influenced by several factors including:

- The strength of the connection or connection group at a discreet location in terms of
  1. initial small displacement response and
  2. ductility.
- The availability of a generic analysis and design method.
- The reliability of the join and availability of quality control methods.
- The cost of the connection technique.
- Maneuverability of the joining equipment.
- Local environmental effects associated with the joining process, e.g. fumes from welding galvanised steel.

UK design guidance for connections in cold-formed steel [1] specifies that the join should be capable of
transmitting the forces and moments calculated in the design. In this paper, the initial displacement and plastic shear behaviour of four types of mechanical connection will be investigated. The load response characteristics of connections defined by mechanical interlock, welding and bonding has previously been researched [2] and press joining and mechanical clinching have been compared to spot welding connection techniques [3].

2. Types of fasteners considered

2.1. Mechanical clinching

Fig. 1 shows three stages of forming a press join. All three views are a half vertical section of the die part (below), the layers of steel being connected (centre) and the punch part (above). To begin the connection process two or more layers of steel are arranged and positioned between the punch and die parts of the press joining tool. In Fig. 1a the punch is sheared through the steel sheets. Note that the sides of the punch are aligned with the sides of the die parts to produce a concentrated shear reaction when the punch is initially brought down. The sides of the die parts are also sprung laterally (Fig. 1b) to allow lateral expansion of the steel between the punch and die under concentrated pressure in the second phase of joining. In Fig. 1c the vertical force is removed from the punch, the steel sheets are removed from the die, and the sides of the die part return to their vertical positions.

The pressure used to deform the layers of steel in the second stage of mechanical clinching influences the strength and behaviour of the join loaded in shear. Several tests were carried out with varying pressures on different steel thickness to obtain the optimum join strength for each sample. The lateral spread of the deformed steel on the die side of the sample gives an indication of the efficiency of the join. The orientation of a press join is defined by the direction of the load applied to the connection.

The orthogonal strength and deformation characteristics of press joins have been demonstrated previously by Pedreschi et al. [5,6]. The potential of press joining in cold-formed steel structures has been investigated [4], and some advantages of using mechanical clinching for connecting light gauge steel have been highlighted [7].

The press join configurations tested are joins at 0° and 90° in two strips of steel of equal thickness and for four different thickness.

2.2. Self piercing rivets

Henrob 5-mm nominal stem diameter self piercing rivet samples were tested in four different thickness of steel. The process of forming a Henrob self-piercing connection involves driving a separate rivet component into the layers of the parent metal, piercing and clinching in a single operation. A cross section view of a formed Henrob connection is illustrated in Fig. 2.

2.3. Pop rivets

The pop rivet joins were formed by applying a hand held lazy tongues apparatus with the rivet inserted at the tip to a pre-drilled hole in the plates. The rivet has a shaft diameter of 5 mm and a 5-mm diameter automatic drill is used to prepare the hole. An advantage of using pop rivets over other types of mechanical fastener is that a rivet can be applied to layers of steel where the rivet operator has access to only one face of the steel surface. A hole must be drilled in the parent metal before a rivet is applied.

2.4. Self tapping screws

SFS Stadler Self drilling and tapping screws SD3-T16-6.3×25 and SD5-T15-5.5×25 were tested. Screws are commonly used to sew steel roof sheeting to purlins and to join cold-formed steel structural components. In the shear tests screw samples are clamped and joined using an automatic drill with a single self tapping and drilling screw.

Fig. 1. Forming a press join.

Fig. 2. Cross section of a Henrob join.
3. Experimental procedures

3.1. Materials used in the tests

Galvanised mild steel was used in all samples. Each type of connection was tested in 1.0-mm, 1.2-mm, 1.6-mm and 2.0-mm thick steel. Material tests were carried out on all steel thickness used in the shear tests. In the material tests, the test sample is a single steel strip. The extensometer provides a travel distance of 50 mm allowing the extensometer to measure the elastic, plastic and failure behaviour of a sample. There is considerable narrowing of the sample in the central region at failure.

An additional set of six tests were carried out on the 1.0-mm and 2.0-mm steel thickness to establish the variability of the parent metal material properties used in the mechanical connection tests. Table 1 shows average peak stress levels and standard deviations of yield stresses over each set of the six samples for the 1.0-mm and 2.0-mm steel thickness. The stress strain curves for the six tests of 2.0-mm thickness are illustrated in Fig. 3.

3.2. Test procedure for mechanical connections

A series of shear tests were carried out on each connection type. The test samples consist of two strips of mild steel, 200 mm long x 50 mm wide, with an overlap length of 80 mm (Fig. 4). The connected samples were clamped into the grips of the Instron testing machine. A 100-mm gauge length extensometer with a travel distance of 50 mm was attached to the samples with wire clips that provide a knife edge connection to one side of the sample. The extensometer and load cell were calibrated and balanced.

The test was started by applying a tensile force to the sample in the direction of the bold arrows in Fig. 4. The force was regulated by the tensile testing machine to produce a rate of displacement specified by the machine operator. When mechanical fasteners were being tested, a rate of displacement of 1 mm/sec was specified. Values of displacement at the 100-mm gauge points on the sample were recorded at rate of five data samples per second and measurements of applied load were recorded simultaneously from a load cell. The test was carried out until the join had failed. In the connection sample tests, it is assumed that all displacement takes place at the connection, and the elastic deformation of the steel strips is negligible.

4. Results and analysis

4.1. Experimental results

Average peak loads for all connection types considered are listed in Table 2, the corresponding load

Table 1

<table>
<thead>
<tr>
<th>Steel thickness (mm)</th>
<th>Number of samples in population</th>
<th>Average yield stress (N/mm²)</th>
<th>Standard deviation</th>
<th>Coefficient of variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>6</td>
<td>315.5</td>
<td>8.6</td>
<td>2.72</td>
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<tr>
<td>2.0</td>
<td>6</td>
<td>306.5</td>
<td>8.5</td>
<td>2.77</td>
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</table>
displacement paths are illustrated in Figs. 5–8. Initial displacement responses of <0.1 mm for all connection types in steel thickness of 1.0 mm and 2.0 mm are illustrated in Figs. 9 and 10.

4.2. Mechanical clinching

The effects of the angle of orientation of the join and the thickness of steel joined on the shear capacity of the press join can be seen in Fig. 11. The join is strongest in shear at 0° and weakest at 90°. There is an approximately linear relationship between the angle of orientation and shear peak load, as shown in Fig. 11. Peak load also increases with increased thickness of steel being joined.

The failure mode of a press join is greatly affected by the orientation of the applied load to the parallel slits that are sheared into the connection parent metal in the first stage of forming the join (Fig. 1). If the applied load is parallel to the slits (90° orientation), the parent metal components slip away from each other in a squeezing action. Any resistance to displacement is provided by lateral deformation against the slits, there is little plastic deformation of the connection parent metal and consequently a lower peak load is achieved in comparison with press joins at 0° and self-piercing rivets.

In the case of press joins at 0° orientation, the applied load is perpendicular to the sheared slits in the connection parent metal. When the load is increased, the two parent metal components lock and failure is caused by local bending, tearing and pulling out. Con-

Table 2
Shear test peak loads

<table>
<thead>
<tr>
<th>Type of Join</th>
<th>Peak load (kN) steel thickness 1.0 mm</th>
<th>Peak load (kN) steel thickness 1.2 mm</th>
<th>Peak load (kN) steel thickness 1.6 mm</th>
<th>Peak load (kN) steel thickness 2.0 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test value</td>
<td>Average</td>
<td>Test value</td>
<td>Average</td>
</tr>
<tr>
<td>Press join at 0°</td>
<td>2.06</td>
<td>2.04</td>
<td>2.65</td>
<td>2.68</td>
</tr>
<tr>
<td>Press join at 45°</td>
<td>1.69</td>
<td>1.64</td>
<td>2.17</td>
<td>2.13</td>
</tr>
<tr>
<td>Press join at 90°</td>
<td>1.38</td>
<td>1.37</td>
<td>1.95</td>
<td>1.92</td>
</tr>
<tr>
<td>Pop rivet</td>
<td>2.50</td>
<td>2.51</td>
<td>2.74</td>
<td>2.83</td>
</tr>
<tr>
<td>Self-piercing rivet</td>
<td>5.38</td>
<td>5.28</td>
<td>6.04</td>
<td>6.17</td>
</tr>
<tr>
<td>Self-tapping screw</td>
<td>2.74</td>
<td>2.91</td>
<td>3.58</td>
<td>3.75</td>
</tr>
</tbody>
</table>
sequently greater energy is required to displace the joined steel plates and higher initial stiffness and ductility are achieved in comparison with press joins at 90°. The failure mode of a press join at 45° is a combination of modes for 0° and 90°.

Davies et al. developed equations to predict the initial stiffness, unloading stiffness, peak load and plastic limit of a press join in 1.6- and 2.0-mm thick steel for any angle of applied loading [6]. An equation [5] that uses the ultimate tensile strength of the steel, the angle of applied loading in degrees and the steel thickness to estimate the peak load of a press join is:

\[
\text{Peak load} = \left(6.34 - 0.022 \times \theta\right) \times \text{U.T.S.} \times \text{thickness},
\]

where \(0° \leq \theta \leq 90°\), the U.T.S. is in N/mm² and thickness is in mm. The press join peak loads listed in Table 2 show a close correlation to the peak loads predicted using this formula. The initial displacement responses of press join connections illustrated in Figs. 9 and 10 show a high initial stiffness.
4.3. Self-piercing rivets

Peak loads for all self-piercing rivet samples tested are listed in Table 2 and the load displacement paths are included in Figs. 5–8. Unlike the press join the self-piercing rivet is circular and does not have a particular orientation. It has a separate rivet component driven into the layers of the parent metal to complete the join, as illustrated in Fig. 2. In a shear test when the parent metal begins to deform, the rivet component acts as a link between the shearing strips of metal causing local deformation and tearing. High peak load and ductility are achieved in comparison with all other join types tested.

4.4. Pop rivets

In the pop rivet tests although the shear behaviour was irregular between similar samples, peak load generally increased with increase in steel thickness (Table
2). The samples failed by the rivet head sliding down the rivet shaft after some local deformation except for the 2.0-mm steel sheet thickness where in some cases the rivet shaft failed in shear (Figs. 5–8).

4.5. Self-tapping screws

The characteristic failure mode for self-tapping screws in shear tests begins with a rotation of the screw as the load is applied. This is followed by an irregular pulling out mode when the steel in the plates around the screw is deformed locally and dragged over the screw threads. This type of load displacement behaviour creates high ductility but low initial stiffness (Figs. 5–8). Fig. 9 shows that where screws are applied in 1.0-mm thick steel the load displacement path shows a low initial stiffness.

4.6. Variability (screws and press joins)

An additional series of mechanical clinching shear tests were carried out on press joins at 0°, press joins at

Table 3
Variation of peak loads for selected samples

<table>
<thead>
<tr>
<th>Type of join</th>
<th>Steel thickness (mm)</th>
<th>Number of samples in population</th>
<th>Average peak load (kN)</th>
<th>Standard deviation</th>
<th>Coefficient of variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Press join at 90°</td>
<td>1.0</td>
<td>6</td>
<td>1.41</td>
<td>0.055</td>
<td>3.90</td>
</tr>
<tr>
<td>Press join at 0°</td>
<td>1.0</td>
<td>6</td>
<td>2.09</td>
<td>0.073</td>
<td>3.49</td>
</tr>
<tr>
<td>Press join at 90°</td>
<td>2.0</td>
<td>6</td>
<td>2.98</td>
<td>0.076</td>
<td>2.55</td>
</tr>
<tr>
<td>Press join at 0°</td>
<td>2.0</td>
<td>6</td>
<td>5.12</td>
<td>0.077</td>
<td>1.50</td>
</tr>
<tr>
<td>Screw</td>
<td>1.0</td>
<td>6</td>
<td>4.905</td>
<td>0.395</td>
<td>8.05</td>
</tr>
<tr>
<td>Screw</td>
<td>2.0</td>
<td>6</td>
<td>8.875</td>
<td>0.338</td>
<td>3.81</td>
</tr>
</tbody>
</table>
90° and self tapping screws, in 1.0-mm and 2.0-mm thick steel. Six of each configuration were tested, to examine the variability of test results on similar configurations. Table 3 shows the average peak load and coefficient of variation for each set of six tests.

A comparison of Table 3 and Table 1 shows that the coefficient of variation of peak loads obtained for press joins in 2.0-mm thick steel was lower than the material tests yield stress coefficient of variation for that thickness. As the steel thickness is decreased to 1.0 mm, the variation of press join peak loads becomes greater than the variation of yield stress. Screw samples show high peak load variability, approximately twice that of a press join for a given steel thickness.

5. Conclusions

Shear tests have been carried out on rectangular press join mechanical clinching connections, self-piercing rivet connections, pop rivet connections and screw connections in similar thickness of two layers of steel 1.0, 1.2, 1.6 and 2.0 mm thick. The following conclusions can be drawn from the results:

- There is a large variation in the responses to loading of the various connection types tested. In the small displacement range of loading press joins gave a non-linear load–displacement response while self-piercing rivets gave an approximately linear response. Press joins at 0° consistently showed a higher peak load in comparison with press joins at 90° in different thickness of steel tested.
- Self-piercing rivets and press joins at 0° orientation showed a high initial stiffness in comparison with the other types of connection tested.
- Self-tapping screws showed a low initial stiffness in 1.0-mm thick steel. The ductility of all self-tapping screws samples tested was high due to the parent metal dragging on the screw threads in the large displacement range of response.
- The initial stiffness and peak loads of pop rivets in comparison with the other connections tested was highest in the 1.0-mm and 1.2-mm steel thickness.
- Self-piercing rivets show a high peak load and high ductility as the rivet part of the connection links the parent metal components through the large displacement range of response.
- The variability of yield stress of the mild steel used in the shear tests cannot account directly for the variability of peak loads from press join or self-tapping screw shear tests.

Acknowledgements

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References

Factors Influencing the Strength of Mechanical Clinching

by

Remo Pedreschi, Braj Sinha, Russell Davies and Rory Lennon

Abstract

The use of cold-formed steel in fabricated structures is increasing. A major factor in the efficiency and economics of fabricated cold-formed steel structures is the nature and design of the connections. Mechanical clinching, also known as press joining, uses the parent metal of the sections to form a structural connection and has advantages over conventional connection techniques. The paper describes the key characteristics of mechanical clinching, illustrated using some typical, practical applications. The strength of the connection is influenced by a number of factors which include: Ultimate tensile strength, thickness of steel, number of layers of steel connected and where, dis-similar layers of steel are connected, the pattern of lay-up of the steel in relation to the joining tools.

Introduction:

During the last two or three years there has been a rapid increase in the use of cold-formed steel in residential applications. Builders are replacing traditional timber studs with cold formed steel C sections in the construction of both stud wall panels and roof trusses. Dedolph and Jaselkis (1997) attribute this change to the following:

• A general decline in the quality of timber
• Fluctuating timber prices
• Limited reusability of timber studs.

The nature of steel production and supply tends to avoid these problems and hence becomes a more attractive alternative to the contractor. The transition from timber to steel does not seem to present great difficulties to the carpenter. Appropriate documentation on the structural design and construction of steel stud framing is becoming available with the publication of the Prescriptive Method for Residential Buildings (NAHB 1996).

A key factor in the efficient fabrication of cold-formed steel structures is the type of connection used.

Self tapping screws are most often specified for site assembled stud frames and their design and application is covered in the Prescriptive Method. Other forms of connection such as rivets, bolts and welding are also used, mostly in pre-engineered stud frames.

Conventional techniques such as these have disadvantages which may contribute to the overall inefficiency in the fabrication process

• rivets and self tapping screws are sensitive to operator error and require accurate fit of the components before installation.
• the strength of bolted connections is generally governed by bearing failure in the relatively thin cold-formed sections and therefore the bolt is never used to its full capacity.

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Elliot and Co Structural Engineers
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welding destroys galvanised steel coatings, producing fumes and by-products which may contaminate welds and also requires skilled labour.

Mechanical clinching techniques have been used in general manufacturing processes, such as automobiles and air conditioning for a number of years. These techniques offer a number of potential benefits over the conventional methods for cold-formed steel structures that are likely to lead to improved fabrication efficiency. There are a few variations in the detailed configurations of mechanical clinching, also known as press-joining, but the key aspects are the use of pressure and controlled cutting and deformation to push one layer of steel through another and form a strong mechanical clinch between the steel layers. The process of forming a press join is illustrated in figure 1 and described in more detail elsewhere (Pedreschi et al 1996) and (Davies 1996). The advantages offered by these methods over conventional techniques can be summarised as follows:

- galvanised and painted coatings are left intact.
- the process uses very little energy, approximately 10 percent of an equivalent spot weld.
- the process can be readily automated
- it requires only semi-skilled operatives
- it uses only the parent metal of the elements to be connected and therefore eliminates the need for consumables such as screws or rivets.
- it can be easily checked for quality by non-destructive methods.

Mechanical clinching is now being used in buildings in a number of different applications. Figure 2 illustrates the use of press joining in a fabricated steel truss. The truss is manufactured off site and uses a semi automatic production process for all the node point connections. Figure 3 shows the use of mechanical clinching to produce a more efficient form of steel stud with closed triangular boxed flanges. Clinching is applied during the roll-forming process. Figure 4 illustrates a fabricated beam which uses clinching to connect the flange to the web. The beam can span up to 50 feet (15 metres). The web uses 0.039 ins (1.0 mm) thick steel whilst the flange uses 0.078 ins (2.0 mm) thick steel. Tests have been carried out on depths of the beam between 11.8 and 23.6 ins (300 and 600 mm). To date most applications of press joining have concentrated on factory produced elements however at least two tool makers have developed small tools specifically intended for in the field fabrication of steel stud frames, figure 5.

The original applications of press joins were essentially either non-structural or semi-structural and consequently the attention of earlier researchers such as Leibig (1987) and Bober (1987) and others was focused on production parameters rather than a detailed understanding of the structural behaviour of the press joins. Understanding the structural behaviour and the development of predictive methods for calculating the strength of press joins are essential for the further application of the technique. There is an ongoing programme of research into the structural behaviour of mechanical clinching in cold-formed steel structures at Edinburgh University, details of this research have been reported elsewhere (Pedreschi et al, 1996, Davies et al, 1996, Davies 1996). This paper discusses the some key factors that influence the strength of mechanical clinching in cold formed steel structures. These include:

- steel strength, thickness and orientation of applied shear
- multi-layer joints
- dis-similar thicknesses
- variability of press join strength

Although considerable progress has been made in understanding the structural behaviour of mechanical clinching there are areas where further research is in progress or needed and these are identified.

Steel strength, thickness and orientation of applied shear

The most common form of connection is between two layers of steel of the same thickness and ultimate tensile strength. A considerable amount of research has been carried out on this particular arrangement and has been reported elsewhere, (Davies et al 1996, and Davies 1996). This work has shown, as would be
expected, that the shear strength of the connection increases as the ultimate tensile strength and thickness of the steel increases. The most typical form of press join, figure 6, displays orthotropic behaviour and the strength is significantly influenced by the orientation of the applied shear to the connection itself. The normal convention for defining the angle of shear to the orientation of the connection is also shown in figure 6. As the angle changes from 0 to 90 degrees there is a progressive reduction in shear strength. The mode of failure changes as the angle of applied shear changes. When the direction of applied shear is 0 degrees failure occurs by shearing across the two protruding parts of the punch side layer as they pass through the die side layer. When load is applied at 90 degrees failure occurs by deformation of the punch side steel leading to tearing and eventual pull-out from the enclosing die side layer. Typical results showing the influence of angle of applied shear are presented in figure 7. Various expressions have been developed to predict the shear strength of press joins taking into account the influence of ultimate tensile strength (UTS), thickness and angle of applied shear. These are reviewed in more detailed elsewhere (Davies 1996). Recent work by (Pedreschi et al 1998), reviewing results of a number of researchers and combining these results with the research carried out at Edinburgh University has shown that there is consistent behaviour across all the available data that indicates a linear relationship between shear strength, UTS, steel thickness and angle of applied shear. The following expression was shown to provide an accurate prediction of the shear strength of press joins for two layers of steel with equal thickness and steel strength.

$$F_{p2} = (0.25 - 0.00086 \cdot \phi) \cdot UTS \cdot t \quad \text{for} \quad 0 \leq \phi \leq 90$$

where UTS and t are in psi and inches.

$$F_{p2} = (6.34 - 0.022 \cdot \phi) \cdot UTS \cdot t \quad \text{(UTS in N/mm}^2\text{ and t in mm})$$

Equation 1 is based on over 140 tests results covering a broad range of materials including thicknesses ranging from 0.0236 - 0.0787 ins, (0.6 - 2.0 mm) and ultimate tensile strengths ranging from 640287-99280 psi, (280 - 690 N/mm$^2$). Figure 8 compares the predicted and experimental results and good agreement can be seen. A statistical analysis of the results (Pedreschi et al 1998) indicated that over 80% of the predicted results were within 15% of their corresponding test result.

### Multi-layer connections

In the design of some structural systems advantages can be obtained by connecting through more than two layers of steel. The trusses shown in figure 2 incorporate a triangular shaped closed section. This is made possible by the simultaneous clinching of four layers of steel. Research into this particular form of connection (Pedreschi and Sinha 1997) has shown that the same basic relationships between UTS, steel thickness and angle of applied shear, previously demonstrated in two layer joints, also apply in multi-layer connections. The results are presented graphically in figure 9.

An expression for predicting the shear strength of the connections was developed and it can be seen that it takes the same form as equation 1.

$$F_{p4} = (0.67 - 0.003 \cdot \phi) \cdot UTS \cdot t \quad \text{for} \quad 0 \leq \phi \leq 90$$

$$F_{p4} = (17.1 - 0.089 \cdot \phi) \cdot UTS \cdot t \quad \text{in SI units}$$

If equation 2 is re-written in terms of the combined thickness of steel on either side of the connection it becomes:

$$F_{p4} = (0.335 - 0.0015 \cdot \phi) \cdot UTS \cdot t \quad \text{for} \quad 0 \leq \phi \leq 90$$

$$F_{p4} = (8.55 - 0.045 \cdot \phi) \cdot UTS \cdot t \quad \text{in SI Units}$$

The coefficients of the equations now more closely resemble those of equation 1. Equation 3 implies that a stronger connection can be obtained if the total combined steel thickness in the connection consists of four
layers rather than two. This has yet to be proven conclusively as the mechanical clinch used to develop equations 2 and 3 is a modified version of that used in the development of equation 1, to provide improved penetration and spread of the steel.

Equation 2 was used to predict the shear strengths of a separate series of tests, conducted by Pei and Kinney (1998). The results are compared in table 1. The experimental figures are the average of between 5 and 7 tests for each of the variables considered. It can be seen that, although equation 2 uses statistically derived coefficients, based on a separate data set and tested in a different test laboratory, there is good correlation between the experimental and predicted failure load.

<table>
<thead>
<tr>
<th>angle of applied shear</th>
<th>Ultimate tensile strength psi (N/mm2)</th>
<th>thickness inches (mm)</th>
<th>average experimental failure load lbs (N)</th>
<th>predicted failure load lbs (N)</th>
<th>experimental predicted</th>
</tr>
</thead>
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<tr>
<td>0</td>
<td>65091 (448)</td>
<td>0.027 (0.685)</td>
<td>1196 (5319)</td>
<td>1177 (5293)</td>
<td>0.99</td>
</tr>
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<tr>
<td>60</td>
<td>68000 (468)</td>
<td>0.035 (0.889)</td>
<td>1180 (5248)</td>
<td>1166 (5220)</td>
<td>1.01</td>
</tr>
</tbody>
</table>

Table 1
Comparison between experimental and predicted results using equation 2
to predict the results of Wei et al (1998)

Further work on the behaviour of multi-layer connections is currently in progress.

Dis-similar materials

In many applications the layers of steel to be connected may be of differing thicknesses. Using conventional mechanical connections the strength is generally determined by the thinner of the layers. In mechanical clinching the orientation of the layers relative to the punch and die of the forming tool has a significant influence on the strength of the connection, figure 10. Some test results are presented in table 2. The values presented are the average of at least two test results for each configuration. Table 2 is divided into three separate sections. For a combination of two dis-similar materials the greatest strengths are obtained when the thicker material is placed on the punch side, irrespective of the direction of applied shear. Failure of the mechanical clinch tends to initiate on the punch side layer and therefore the strength is influenced more by the punch side material than the die side material. Also shown in table 2 is the corresponding shear strength for press joins formed of two equal layers of the thinner material. With the exception of one result the strength of press joins formed with two layers of dis-similar materials is stronger than the corresponding press join formed with two uniform layers of the thinner steel.

<table>
<thead>
<tr>
<th>Angle of applied shear</th>
<th>Thicker steel on punch side</th>
<th>Thinner material on punch side</th>
<th>Equal steel thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>punch side (ins)</td>
<td>die side (ins)</td>
<td>peak load (lbs (kN))</td>
</tr>
<tr>
<td></td>
<td>thickness</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.078(2.0)</td>
<td>0.059(1.5)</td>
<td>1013 (4.51)</td>
</tr>
<tr>
<td></td>
<td>0.059(1.5)</td>
<td></td>
<td>0.059(1.5)</td>
</tr>
<tr>
<td>90</td>
<td>0.078(2.0)</td>
<td>0.059(1.5)</td>
<td>638 (3.04)</td>
</tr>
<tr>
<td></td>
<td>0.059(1.5)</td>
<td></td>
<td>0.059(1.5)</td>
</tr>
<tr>
<td>90</td>
<td>0.059(1.5)</td>
<td>0.039(1.0)</td>
<td>717 (3.19)</td>
</tr>
<tr>
<td></td>
<td>0.039(1.0)</td>
<td></td>
<td>0.039(1.0)</td>
</tr>
<tr>
<td>90</td>
<td>0.059(1.5)</td>
<td>0.039(1.0)</td>
<td>398 (1.64)</td>
</tr>
<tr>
<td></td>
<td>0.039(1.0)</td>
<td></td>
<td>0.039(1.0)</td>
</tr>
<tr>
<td>0</td>
<td>0.078(2.0)</td>
<td>0.039(1.0)</td>
<td>813 (3.62)</td>
</tr>
<tr>
<td></td>
<td>0.039(1.0)</td>
<td></td>
<td>0.039(1.0)</td>
</tr>
<tr>
<td>90</td>
<td>0.078(2.0)</td>
<td>0.039(1.0)</td>
<td>438 (1.95)</td>
</tr>
<tr>
<td></td>
<td>0.039(1.0)</td>
<td></td>
<td>0.039(1.0)</td>
</tr>
</tbody>
</table>

Table 2 Strength of mechanical clinching in multi-layer joints
Variability of the strength of press joins

An important aspect in safe structural design is the inherent variability of the process. In order to study the variability of mechanical clinching ten samples of two different steels were tested with the load applied at both zero and ninety degrees. The results are summarised in table 3.

<table>
<thead>
<tr>
<th>Steel no.1</th>
<th>Steel no.2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>UTS</strong></td>
<td><strong>UTS</strong></td>
</tr>
<tr>
<td>(69470 psi)</td>
<td>(54288 psi)</td>
</tr>
<tr>
<td>(479 N/mm²)</td>
<td>(378 N/mm²)</td>
</tr>
<tr>
<td><strong>Thickness</strong></td>
<td><strong>Thickness</strong></td>
</tr>
<tr>
<td>0.059 ins</td>
<td>0.079 ins</td>
</tr>
<tr>
<td>(1.5 mm)</td>
<td>(2.0 mm)</td>
</tr>
<tr>
<td><strong>0 degrees peak load</strong></td>
<td><strong>90 degrees peak load</strong></td>
</tr>
<tr>
<td>lbs (kN)</td>
<td>lbs (kN)</td>
</tr>
<tr>
<td>926 (4.12)</td>
<td>1014 (4.51)</td>
</tr>
<tr>
<td>1014 (4.51)</td>
<td>1036 (4.61)</td>
</tr>
<tr>
<td>982 (4.37)</td>
<td>1005 (4.47)</td>
</tr>
<tr>
<td>959 (4.27)</td>
<td>1014 (4.51)</td>
</tr>
<tr>
<td>1056 (4.70)</td>
<td>1014 (4.51)</td>
</tr>
<tr>
<td>959 (4.27)</td>
<td>991 (4.41)</td>
</tr>
<tr>
<td>977 (4.35)</td>
<td>1013 (4.51)</td>
</tr>
<tr>
<td>1016 (4.52)</td>
<td>1036 (4.61)</td>
</tr>
<tr>
<td>926 (4.12)</td>
<td>1016 (4.52)</td>
</tr>
<tr>
<td>959 (4.27)</td>
<td>1036 (4.61)</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td><strong>Mean</strong></td>
</tr>
<tr>
<td>964 (4.29)</td>
<td>1018 (4.53)</td>
</tr>
<tr>
<td>557 (2.48)</td>
<td>663 (2.95)</td>
</tr>
<tr>
<td><strong>Standard deviation</strong></td>
<td><strong>Standard deviation</strong></td>
</tr>
<tr>
<td>30.3 (0.1347)</td>
<td>24.7 (0.1102)</td>
</tr>
<tr>
<td><strong>Variance %</strong></td>
<td><strong>Variance %</strong></td>
</tr>
<tr>
<td>3.14</td>
<td>4.45</td>
</tr>
<tr>
<td><strong>Characteristic strength</strong></td>
<td><strong>Characteristic strength</strong></td>
</tr>
<tr>
<td>915 (4.074)</td>
<td>996 (4.43)</td>
</tr>
<tr>
<td>517 (2.30)</td>
<td>629 (2.80)</td>
</tr>
</tbody>
</table>

Table 3 Variability of press join strength.

The results indicate consistent behaviour. The standard deviation was calculated in accordance with British Standards (BSI 1987). The standard deviation is low, a maximum of 4.45% variance. During the forming process the steel layers are clamped in position and then subjected to an applied force and controlled deformation. The maximum deformation is limited by the shape and depth of the die. Thus if sufficient pressure is applied a consistent and repeatable join will be formed. Also presented in table 3 is the characteristic design strength of the press join calculated in accordance with British Standards and is within 7% of the mean strength of the connection.

Summary and conclusions

The research to date on the strength of press joins indicates the following:

- The strength of mechanical clinching is primarily influenced by the thickness and Ultimate tensile strength of the steel being connected and the angle of applied shear.
- In connections consisting of four layers of steel comparable structural behaviour to two layer connections was observed.
- It has been shown that the strength of press joins in shear can be accurately predicted using expressions that assume a linear relationship between ultimate tensile strength, thickness and angle of applied shear.
Consistent structural behaviour has been observed across the work of a number of researchers and the expressions developed to predict the strength of press joins have been shown to accurately predict the results of tests reported by other researchers.

In connections using dis-similar thicknesses of steel, the steel layer on the punch side of the connection is more critical. The connection is strongest with the thicker layer positioned on the punch side. The strength of connections using dis-similar materials is generally stronger than the corresponding connection using two equal layers of the thinner steel.

Mechanical clinching demonstrates consistent behaviour. The maximum coefficient of variance over a range of different samples was 4.45%.

Research into the strength of mechanical clinching in cold-formed structures forms part of an ongoing research programme into the structural behaviour of cold-formed steel structures at the University of Edinburgh. The broad aim of the current research is to develop a consistent and reliable method for the structural design of mechanical clinching. The scope of the investigation includes:

- Further study of multi-layer joints including three and four layers of steel
- Further testing of dis-similar materials to include a wider range of materials and thicknesses
- A comparative study of alternative forms of mechanical clinching
- An experimental study of mechanical clinching subjected to cyclic loading

As part of the programme a series of full scale test structures will be constructed to verify and develop the design methodology.

Acknowledgements

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Appendix - References

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Pedreschi R., Sinha, B.P. and Davies, R.J. (1996), 'Advanced connection techniques for cold-formed steel structures', Journal of the Structural Division, ASCE, 123(2), 138-144.

Research Report, The Department of Architecture, University of Edinburgh.


Appendix - Notation

$F_p^2$ the shear strength of press joints formed using two layers of steel
$F_p^4$ the shear strength of press joints formed using four layers of steel
$t$ thickness of steel
$\theta$ angle of applied shear
$UTS$ ultimate tensile strength of steel
The use of mechanical clinching in cold-formed steel trusses.
Figure 3
The use of mechanical clinching in steel studs

Figure 4
The use of mechanical clinching in fabricated steel beams
Figure 5
In the field tool for steel stud framing

Figure 6
Typical press join showing orientation of applied shear
Figure 7
Typical relationship between angle of applied shear and strength
Figure 8
Comparison between experimental and predicted results using equation 1

$F_p = (6.34 - 0.22\sigma) \cdot \text{UTS}$
Figure 9
Typical relationship between angle of applied shear and failure load for multi-layered mechanical clinching

Figure 10
Dis-similar steel thicknesses - arrangement of steel layers