# Evaluating the design strength of sheathing connection in CFS shear wall

By

Mariam Usman

(Registration No: 00000359048)

Department of Structural Engineering

NUST Institute of Civil Engineering

School of Civil and Environmental Engineering

National University of Sciences & Technology (NUST)

Islamabad, Pakistan (2020)

# Evaluating the design strength of sheathing connection in CFS shear wall

By

Mariam Usman

(Registration No: 00000359048)

A thesis submitted to the National University of Sciences and Technology, Islamabad,

in partial fulfillment of the requirements for the degree of

Masters of Science in

Civil Engineering

Supervisor: Dr. Azam Khan

Co Supervisor: Dr. Sarmad Shakeel

NUST Institute Civil Engineering

National University of Sciences and Technology (NUST)

Islamabad, Pakistan

(2024)

# THESIS ACCEPTANCE CERTIFICATE

It is certified that final copy of MS thesis written by **Ms. Mariam Usman**, Registration No. **00000359048**, of Nust Institute of Civil and Environmental, (NICE), has been vetted by undersigned, found completed in all respects as per NUST Statutes/Regulations, is free of plagiarism, errors, and mistakes and is accepted as partial fulfilment for award of MS degree.

Signature: ____________
Supervisor: Dr. Azam Khan
Date: 5/6/2024

Signature (HoD): Structural Engineering
NUST Institute of Civil Engineering
School of Civil & Environmental Engineering
National University of Sciences and Technology
Date: 5.06.2024

Dr. S. Muhammad Jamil
Associate Dean
NICE, SCEE, NUST
Signature (Associate Dean, NICE) ____________
Date: 13.06.24

Signature (Principal & Dean): ----------------
PROF DR MUHAMMAD IRFAN
Principal & Dean
SCEE, NUST
Date: 13 JUN 2024

Form TH-4

# National University of Sciences and Technology

## MASTER'S THESIS WORK

We hereby recommend that the dissertation prepared under our Supervision by: **Mariam Usman, Regn No.00000359048**

Titled: **Evaluating the Design Strength of Screw Connection in CFS Shear Walls**

be accepted in partial fulfillment of the requirements for the award of **Master of Science** degree with ( B Grade).

### Examination Committee Members

1. Name: Dr. Muhammad Usman Signature:

2. Name: Dr. Junaid Ahmad Signature:

2. Co-Supervisor: Dr. Sarmad Shakeel Signature: Sarmad

Supervisor's name: Dr. Azam Khan Signature:

Date: 5/6/2024

Head of Department
Dated: 5/6/2024

HoD Civil Engineering
NUST Institute of Civil Engineering
School of Civil & Environmental Engineering
National University of Sciences and Technology

Associate Dean
Dated 13.06.24

Dr. S. Muhammad Jamil
Associate Dean
NICE, SCEE, NUST

**COUNTERSIGNED**

Date: 13 JUN 2024

Principal & Dean SCEE

PROF DR MUHAMMAD IRFAN
Principal & Dean
SCEE, NUST

# Certificate of Approval

This is to certify that the research work presented in this thesis, entitled **Evaluating the Design Strength of Sheathing Connection in CFS Shear Wall"** was conducted by Mr./Ms. Mariam Usman under the supervision of Dr. Azam Khan.

No part of this thesis has been submitted anywhere else for any other degree. This thesis is submitted to the Department of Structural Engineering in partial fulfillment of the requirements for the Masters in Field of Structural Engineering, National University of Sciences and Technology (NUST).

Sudent Name: Mariam Usman Signature: Mariam

Examination Committee:

(a) Dr. Muhammad Usman Signature:
Assoc Prof, SCEE (NICE)

(b) Dr. Junaid Ahmad Signature:
Asst Prof, SCEE (NICE)

(c) Dr. Sarmad Shakeel, Co-Supervisor Signature: for
Asst Prof, SCEE (NICE)

(d) Dr. Azam Khan, Supervisor Signature:
Assoc Prof, SCEE (NICE)

HoD Str Engineering Signature:

HoD Structural Engineering
NUST Institute of Civil Engineering
School of Civil & Environmental Engineering
National University of Sciences and Technology

Dr. S. Muhammad Jamil, Associate Dean, SCEE (NICE) Signature:

Dr. S. Muhammad Jamil
Associate Dean
NICE, SCEE, NUST

Dr. Muhammad Irfan, Principal & Dean SCEE Signature:

PROF DR MUHAMMAD IRFAN
Principal & Dean
SCEE, NUST

*“Dedicated to my resilient mother and supportive husband”*

# ACKNOWLEDGEMENTS

I am very thankful to Almighty Allah, the most Merciful for bestowing upon me the dedication to complete this research work. I express my deepest gratitude for the guidance of my supervisor, Dr. Sarmad Shakeel and Dr. Azam Khan who continuously and convincingly conveyed a spirit of hard work and steadfastness to complete the project.

I am forever indebted to my Mother and husband and children for their unconditional Love, motivation and the endless patience.

# ABSTRACT

Construction using light gauge steel (LGS) is similar to the construction of structural steel frames. The steel used in the construction of light gauge is mostly cold form steel (CFS) which is molded into shape at room temperature. For LGS, two basic steel fram- ing components called studs and tracks are used. Tracks are vertical support members that function as top and bottom plates. Tracks run between horizontal framing members called plates (Fig. 1c). The studs are vertical support members running horizontally be- tween framing members called tracks. (Fig. 1b) The screws examined in this research proposal are used between stud and tracks.

In cold form steel (CFS) buildings, seismic resistance is provided by sheathing braced & strap braced shear walls, which resist horizontal in-plane action. The walls are de- signed to withstand loads laterally via in-plane shear. The shear strength and performance of such panels are determined by the sheathing connection between the sheathing panel (OSB, GWB, gypsum, wood etc.) and CFS frame. Fasteners (nails and screws) are used between the sheathing panel and CFS frame to develop resistance against lateral loads. The current design method by the American Iron and Steel Institute (AISI) is unable in as it should be predicting the design shear strength of the sheathing braced shear walls through analytical design formula.

The purpose of this research proposal is to suggest the design criteria to evaluate the ul- timate shear strength of the connection between the panel and steel frame utilizing the available experimental studies on them. The development of accurate design criteria will enable better design guidelines for structural engineers thereby leading to efficient use of CFS building systems.

The multiple experimental studies examined in this research include self-drilling screws (SDS), screw diameter, the frame thickness, as well as also assessing the strength of the sheathing braced connections between cold formed steel systems and panels under monotonic and cyclic shear loads. This research proposal also has taken into consideration the study and failure of CFS panel sheathing connections; effect of the edge distance between the sheathing and the thickness of the

boards on behavior and strength. In the realm of Cold-Formed Steel (CFS) structures, we can categorize connections into two main types: strap- bracing and sheathing braced connections. Strap-braced connections are used to join various parts of CFS skeletal systems, such as connecting beams to studs, trusses, and attaching tracks to studs. Research in the field of steel-to-steel connections, as documented in studies [3,4], delves into elements like connection pat- terns, how forces are transmitted, and the characteristics of fasteners.

On the other hand, steel-to-sheathing connections serve the purpose of creating a

”stressed skin” effect between the structural framework and the sheathing material. These connections significantly influence the way wall panels behave, introducing nonlinear characteristics. Additionally, the sheathing boards provide essential bracing at specific fastener points, thereby enhancing the overall strength of the wall studs against buckling failure.

# Contents

# List of Tables

# List of Figures

# LIST OF NOTATIONS

| Symbol | Description |
|---|---|
| $d_p$ | Screw diameter |
| $d_h$ | Screw head diameter |
| $d_w$ | Larger of the screw head diameter or the washer diameter and shall be taken not larger than ½-in. (152-mm) |
| E | Young's modulus of elasticity |
| $F_{u1}$ | Tens-ile strength of the mem-ber in contact with the scr-ew head |
| $F_{u2}$ | Tens-ile strength of the mem-ber not in contact with the scr-ew head |
| $t_1$ | Thickness of the member in contact with the scr-ew head |
| $t_2$ | Thickness of the member not in contact with the sc-rew head |
| $P_{not}$ | Pull- out; where tc is the lessor of the depth of the penetration and the thickness t2. |
| $P_{nov}$ | Pull- over; where dw is the larger of the scr-ew head diameter or the washer diameter and shall be taken not larger than ½-in. (152-mm) |
| $P_{ns}$ | Nom-inal shear stren-gth for scr-ew connections |

# LIST OF ABBREVIATIONS

| **Abbreviation** | **Description** |
|---|---|
| AISI | American Iron Steel Institute |
| ASTM | American Society for Testing and Materials |
| B | Bearing |
| CUREE | Consortium of Universities for Research in Earthquake Engineering |
| CFS | Cold-Formed Steel |
| CFSP | Cold-Formed Steel Panel |
| D | Depth |
| E | Edge tearing |
| ECCS | European Convention for Constructional Steelwork |
| EWM | Effective Width Method |
| FTS | Fine Threaded Screws |
| GSCFSP | Gyp-sum board Shea-thed Cold-Formed Steel Panel |
| ISO | International Standard-ization Organization |
| LGS | Light guage steel |
| LB | Local buckling |
| LDT | Lin-ear Displace-ment Trans-ducer |
| LVDT | Lin-ear Vari-able Diff--erential Transformer |
| M | Monotonic |
| OSB | Ori-ented Standard B-oard |
| PO | Pull Out |
| PT | Pull through |
| R | Response modification factor |
| S | Single layer panel |
| SDS | Self-drilling screws |
| SBCFSP | Stra-p-Brac-ed Co-ld-For-med Ste-el Pa-nel |
| T | Tilting |
| XSCFSP | X-Strap-braced-Cold-Formed Steel Panel |

# Chapter 1: INTRODUCTION

## 1.1 Introduction

Construction using light gauge steel (LGS) is similar to the construction of structural steel frames. The steel used in the construction of light gauge is mostly cold formed steel (CFS) which is molded into shape at room temperature. For LGS, two basic steel framing components called studs and tracks are used. Tracks are vertical support members that function as top and bottom plates. Tracks run between horizontal framing members called plates (Fig. 1c). The studs are vertical support members running horizontally between framing members called tracks. (Fig. 1b) The screws examined in this research proposal are used between stud and tracks.

In cold formed steel (CFS) buildings, seismic resistance is provided by sheathing braced & strap braced shear walls, which resist horizontal in-plane action. The walls are de- signed to resist loads laterally via in-plane shear. The shear strength and performance of such panels are determined by the sheathing connection between the sheathing panel (Orient Strand Board, Gypsum Wall Board, Plywood etc.) and CFS frame. Fasteners (nails, screws & steel pins) are used between the sheathing panel and CFS frame to develop resistance against lateral loads.

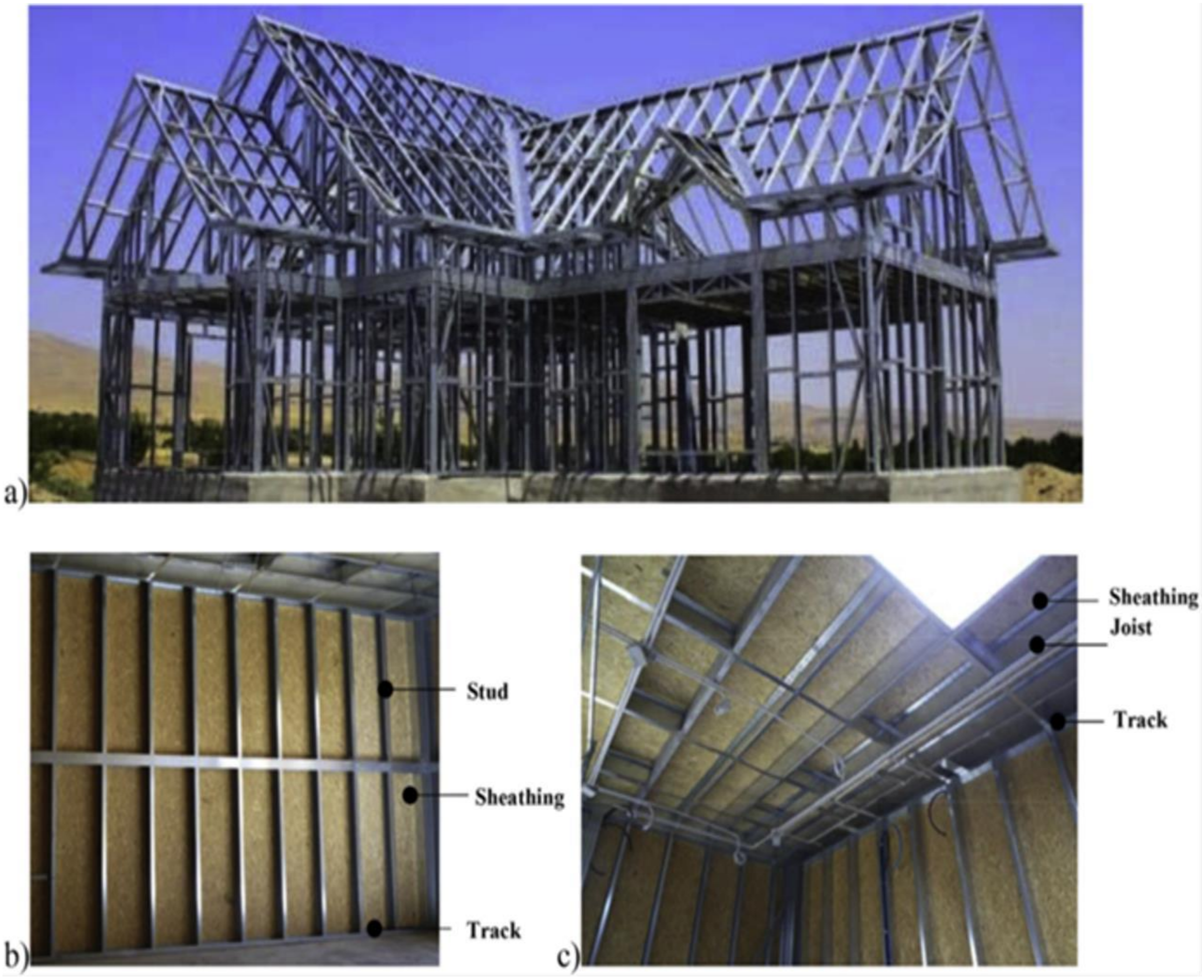


Figure 1.1: Typical application of lightweight steel elements in structural systems. a) Typical lightweight steel construction; b) Wall framing; c) Floor framing.

## 1.2 Statement of Problem

The design method by the American Iron and Steel Institute (AISI) is unable to predict the design shear strength of the sheathing braced shear walls through analytical design formula.

To attain serviceability and required safety codes. The shear strength and performance of such panels are determined by the sheathing connection between the sheathing panel and CFS frame. The design method by the American Iron and Steel Institute (AISI) fails to accurately predict the design shear strength of the sheathing braced shear walls through analytical design formula. The current design scope for sheathed bracing is performed using AISI –S400. The design guidelines can be used under lim- ited parameters mentioned in E2.3.1.1.1 AISI S400-15.

### 1.3 Objectives

Using available experimental studies, this research proposes design criteria for evaluating the ultimate shear strength of the connection between the panel and steel frame. The development of accurate design criteria will enable better design guidelines for structural engineers thereby leading to efficient use of CFS building systems.

The multiple experimental studies examined in this research include self-drilling screws (SDS), screw diameter, the frame thickness, as well as also assessing Under monotonic and cyclic shear loads, the strength of cold formed steel sheathing connections. There are many types of experiments on sheathing materials, sheathing connections, and wall geometry, as well as panel types, thickness profiles, screw diameters, and panel layers, etc. being con- structed in various parts of the world. This research also has taken into consideration the behavior and failure of CFS panel sheathing connections; effect of the edge distance between the sheathing and the thickness of the boards on behavior and strength. Development of accurate design criteria will enable better design guidelines for struc- tural engineers by eliminating experimental studies currently being done.

In response to the current short comings in the current design method as stated in the

"Statement of Problem" previously, In program research, experiments, analytical & numerical work are combined as described below.

- Identify existing design methods given in literature review papers, in order to identify the types of connection that have already been tested.

- Scrutinizing the dataset using Microsoft Excel in order to identify the possible design solution.

- Evaluate the response of the panels, screws & existing lateral force design used.
- Using regression analysis to identify the possible solution to develop the accurate design equation.
- Assemble an experimental program using Orient Strand Board & Gypsum panels. Testing it on the Universal Testing Machine (UTM).

The above objectives will be supported by using a Regression analysis (Microsoft Excel) in conjunction to lab experiments in order to extrapolate the experimental results in order for the development of an accurate design solution guideline thereby leading to efficient use of CFS design systems.

## 1.4 Research Methodology

This research takes into account an extensive literature review & experimental program.

The thorough review of CFS structural properties in conjunction with other CFS panels. The extensive study allows to understand the detailed past studies, their pros and cons. Assessing the shear strength of the connection between the panel and frame helps identify inadequacies in CFS structures. In addition, the AISI-S400 design code is reviewed comprehensively. Using the literature review, a new experimental program was developed in order to observe different sheathing connections.

## 1.5 Outline of thesis

This research comprises of seven chapters. Each chapter runs into flow of the next one in order to present the idea of methodology and motive behind the development of a design criteria of the ultimate shear strength in CFS systems.

In **Chapter 1**, the introduction gives an extensive overview of the CFS design systems. It flows by the main objectives of this study and methodology in the research used.

Eventually, the thesis layout is provided.

In **Chapter 2**, a review of literature on the particular authors is studied. This study examines the CFS member's capacity and the sheathing board's effect and screws on Cold Form Steel panels.

In **Chapter 3**, the AISI –S400 guidelines are discussed, which includes experimental and numerical studies, in order to formulate a reliable calculation of the ultimate shear strength. The numerical study is presented in Excel format.

In **Chapter 4**, the numerical study is presented in Microsoft Excel. The tables presented are studied via the literature review. The dataset is created in the form of separate work- sheet for each sheathing panel (OSB, Gypsum, and Plywood). The study further on filters down to configuration of sheathing, spacing of fastener (edge distance), steel thick- ness, and type of fastener (length, diameter & head diameter) type of failure in screw varies in order to understand the connection performance.

Using the relevant factors mentioned above the nom-inal strength shear for screw con- nections is established by (AISI, 1996). Further, the ratios are calculated to determine the accuracy of the equations used by AISI. This is determined as the experimental study.

The analytical study is established through finding the minimum of the nominal shear strength using equations stated in the chapter. Ratios are calculated to assess the analytical equa- tion given.

In **Chapter 5**, regression analysis is performed using Excel. The equation format is given in the form of a dependent variable "Y" which is the "Ultimate shear strength". Multiple Independent variables used are thickness of panel & steel frame, ultimate strength of steel, average of ultimate strength for panel, diameter of fastener & ulti- mate strength of single connection.

In **Chapter 6**, experimental program is explained. All designs are as per the variation given in the excel chart. The steel frame is attached to the panels (OSB & gypsum) using two different fasteners (screws & ring nails). Steel plate are also attached to hold the specimen in place in the UTM. The proposed design criteria are validated through comparison with experimental data and numerical simulations.

Finally, **in Chapter 7**, the conclusion from the current study is shown as well as suggestions for future recommendation.

### 1.6 Description of Cold form steel systems

Cold formed steel systems (CFS) are modern construction applications whose usage has been increasing over time due to the vast benefits in terms of main load bearing structures, with high durability, high strength & the ability of it being corrosion-free. With many advantages on its side, it also has an economic value, due to the simplicity of assembling and erection, short execution time, and few man-hours. In addition, the use of recyclable materials, the flexibility of systems and the possible reuse of elements assure a low environmental impact. CFS allows to resolve “in-built” concerns rising in the construction frames, as well as, seismic activity safety and without making any compromise on the performance requirements of the buildings. Typical CFS products can be used to build both structural and non-structural architec- tural systems [1]. In construction projects where CFS is used, proper fasteners such as bolts, nails & screws are necessary to assure the proper performance connections used in CFS sys- tems. Commonly used fasteners are self- drilling screws (SDS). These allow to screw together metal cladding frames without pre-drilling. To understand the behavior of CFS systems, leading to effectiveness of connection design, we need to understand the fac- tors such as number and type of screws being used and modes of failures.

Fasteners are used between the sheathing panel and CFS frame to develop resistance to avoid lateral buckling. The type of screw being used depends on the type of con- struction. When fastening rigid materials such as plywood, OSB to

CFS studs or joists normally require a fastener head that will lie flush with the sheathing. (Fig. 2)

## 1.7 Description of lateral force existing systems in Cold-

### 1.7 Formed Steel construction

The possibility for construction of walls and floors under vertical as well as horizontal loading. Using two different seismic resistance approaches “Strap braced” & “sheathing- braced design” [1]. Strap bracing does not include sheathing boards or fasteners. The profile, is considered a stand-alone frame. As for the sheathing braced method the load bearing capacity is calculated by sheathing boards connected via screws to the main steel design and driven out through failure mechanisms such as tilting, bear- ing & pull out (notation given as T, B, PT) at sheathing connections. Both techniques are designed to work together to create a strong and stable building. In a strap braced design, the straps are connected to the sheathing, which then transfers the load to the framing and foundation. This combination of sheathing and straps creates a strong and rigid structure that can resist lateral forces. Gypsum wall boards and other types of calcium silicate boards are normally combined with other types of sheathing material to have much improved seismic behavior. CFS systems that have neither braces nor cladding attached, do not possess much shear resistance and as such, only sheathed CFS panels are discussed. [1]

When fastening CFS studs to CFS tracks, e.g., strap-bracing screw choices are partially determined by the CFS member’s thickness. A self-piercing screw works best with thin materials, such as non-structural CFS studs (less than or equal to

0.033” thick steel). When it comes to penetration into thicker, structural CFS studs, self-drilling screws are more suitable.

A single connection controls the response of a shear wall by connecting the wall to the building’s framing in a way that allows the wall to move and deform under lateral loads. The connection must be strong enough to transfer the lateral load from the wall to the framing, but it must also allow the wall to move and deform without causing damage to the wall or the building.

There are several factors that affect the response of a shear wall, including the size and thickness of the wall, the type of connection used, and the spacing and size of the fram- ing members. A single connection that is too weak or too stiff can cause the wall to fail, while a connection that is flexible can cause excessive deformation and damage to the wall and the building.

## 1.8 Description of screws

To ensure that a shear wall responds appropriately to lateral loads, it is important to in- stall the connections between the wall and the building’s framing. This involves using fasteners to ensure that the wall is properly supported. A single connection that is ap- appropriately designed can allow the wall to move and deform under lateral loads without causing damage to the wall or the building.

The grip range of a screw is the thickness of the material that the screw can securely fasten. Specifically, it is the distance between the undersides of the head of the screw to the tip of the screw. When selecting screws, it is vital that the grip range of the screw is long enough to securely fasten the joining material. If the grip range is too short, the screw may not be able to securely hold the material, leading to potential failure or loosening over time.

Screw length is an important consideration when selecting screws for a particular appli- cation. The length of the screw should be chosen based on the thickness of the material being joined and the depth of the pre-drilled hole. If the screw is too short, it may not provide sufficient holding power or may not be able to reach the material being fas- tened. If the screw is too long, it may damage the material or protrude out the other side. The length of a screw refers to the distance between the tip of the screw and the underside of its head. In other words, it is the measurement of the screw from end to end, excluding the head. When selecting screws, it is important to choose the appropri- ate length based on the specific application. In Cold-Formed Steel (CFS) construction, screws used for attaching sheathing panels typically range in diameter from #8 to #12. The most common screw diameter used for CFS sheathing applications is #8, but #10 and #12 screws can also be used for heavier-duty applications. The length of the screws used for CFS sheathing panels will depend on the thickness of the sheathing panel and the thickness of the steel framing members being fastened to.

Multiple experimental studies have been outlined in this research proposal showing the strength of the sheathing connections between cold formed steel systems and panels un- der monotonic and cyclic shear loads. This proposal also has taken into consideration

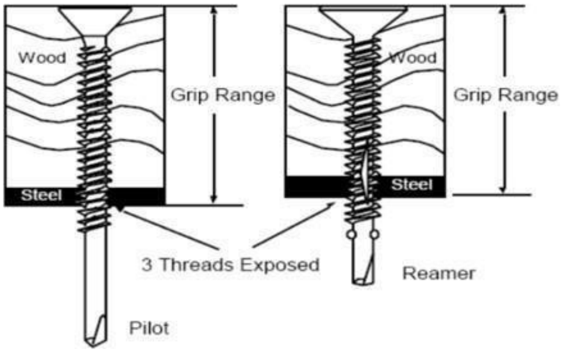


Figure 1.2: Source: CFSEI’s Technical Note on Cold-Formed Steel Construction — Screw Fastener Selection for Cold-Formed Steel Frame Construction, F102-11, Nov. 2011.

the behavior and failure of CFS panel sheathing connections; effect of the edge distance

between the sheathing and the thickness of the boards on behavior and strength;

Figure 1.3: Strap Bracing Design

Figure 1.4: Sheathing Brace Design

### 1.9 Description of panels

CFS sheathing systems with different board panels (OSB, GWB, gypsum, plywood, & calcium silicate or steel sheets) are being used widely in residential and commercial buildings. This is used to bui-ld both struc-tural and non-structural systems. When con- structed it is a sandwiched frame element, in which the internal cavities can be used for pipes, insulated material etc.

Oriented strand board (OSB) is popular in construction because it is strong, cost-effective, and easy to work with. It also has good dimensional stability, opting as less likely to expand or shrink in response to changes in temperature or humidity. OSB is a type of engineered wood product that is commonly used in construction as a substitute for ply- wood or solid wood panels. It is made by shredding and compressing wood strands into a flat, layered panel (0.5–0.7 mm thick). The process of making OSB board begins with harvesting and debarking trees. The resin-coated wood strands are then spread evenly across a conveyor belt, with each layer oriented in a different direction

(transverse or longitudinal) to increase the panel’s strength and stability. The mat is cold pressed to allow air and water to escape and then hot pressed at around 205ºC for approximately 10 minutes to create a flat, solid panel. As discussed in this research there are different sizes of OSB board panels used, also shown below in Fig.4

Once the OSB board has been formed, it is cut into the desired shape and size. The edges of the board are often profiled to allow for easier fitting and joining with other panels. The final product is a versatile construction material that can be used for a va- riety of applications, such as roof sheathing, wall sheathing, and subflooring. Gypsum

Figure 1.5: 11mm/12mm/15mm/18mm OSB Manufacturer Oriented Strand Board Sandwich Panel [1]

(GWB aka gypsum wall board) board, also known as drywall or plasterboard, is a type of building material that is used to create walls and ceilings in homes and commercial buildings. It is made by sandwiching a core of gypsum (a soft sulfate mineral) between two sheets of paper or fiberglass mats. The gypsum is crushed through finding natural deposits in the earth and grounded into a fine powder. This powder is mixed with water to form a paste-like substance, which is then spread onto a continuous sheet of paper or fiberglass mat.

The paper or fiberglass mat with the gypsum paste is then passed through a set of rollers, which compress the mixture and ensure that it is evenly distributed across the surface of the sheet. The edges of the sheet are trimmed to the desired size, and the sheet is left to dry and harden. Once the gypsum board has dried, it is ready to be cut into smaller pieces and used in construction. Gypsum board is a popular construction material be- cause it is easy to work with, fire-resistant, stopping the chances of spreading fire which ensures life safety and has good sound insulation properties. The thermal properties en- sure a good balance of indoor humidity and temperature. A water-resistant gypsum board and water-repellent face paper may be used as a base for wall tile in baths, show- ers, and other areas subject to wetting. Plywood is a type of wood panel made by gluing

Figure 1.6: Types of gypsum boards [2]

together thin layers, of wood veneer. These layers are usually laid perpendicular to one another, which gives the plywood its strength and durability. The process of making plywood involves peeling thin sheets of wood from a log, which are then flattened and cut into uniform thickness. These thin sheets are then layered on top of each other, with each layer's grain running perpendicular to the layer beneath it. The layers are then glued together under high pressure and heat, creating a strong and stable panel. Plywood can be made from a variety of wood species, including hardwoods like oak and maple, and softwoods like pine and fir. It is commonly used in construction in CFS systems, and other woodworking applications due to its strength, durability, and versatility.

Figure 1.7: Plywood 4mm, 6mm, 9mm, 12mm, 15mm, 18 mm [3]

Calcium silicate board is a building material that is commonly used in high-temperature environments, such as in industrial settings, fireproofing applications, and commercial kitchens. It is made from a mixture of calcium silicate and other inorganic materials. The process of making calcium silicate board starts with mixing calcium silicate with water and other additives to form a slurry. The slurry is then poured into molds, and pressed together to remove excess water and form a flat panel. The panel is then dried and cured at high temperatures to harden the calcium silicate mixture. Once cured, the panel is cut to the desired size and shape.

Calcium silicate board is known for its high strength, durability, and resistance to moisture and fire. It is often used in construction applications where other materials, such as gypsum board or plywood, would not be suitable.

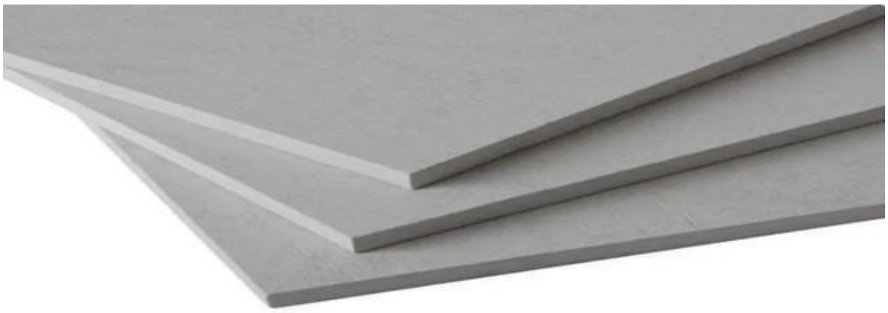


Figure 1.8: Calcium silicate boards for floors and roofs [4]

## 1.10 Description of failure mechanisms

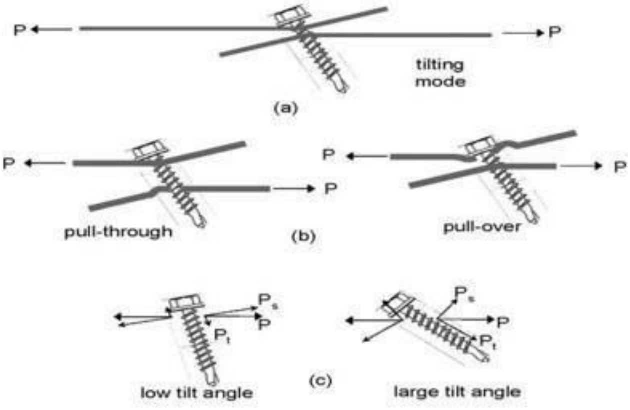


Figure 1.9: Fastener behavior in shear connections [5]

In shear connections, fasteners are used to connect two or more structural elements together by transferring shear forces between them. Fasteners such as bolts, screws, and rivets are commonly used in these types of connections. When a shear force is applied to a connection, the fasteners must be able to resist the force and maintain the structural integrity of the connection. This is achieved through a combination of friction and tension forces. There are several failure mechanisms that can occur in shear connections, including:

**Bearing Failure:** This occurs when the applied load is transferred to the fastener and exceeds the bearing capacity of the material. Bearing failure usually results in the de- formation or crushing of the material, which can reduce the strength and stiffness of the connection.

Figure 1.10: Bearing failure [6]

**Shear Failure:** This occurs when the applied load exceeds the shear strength of the fas- tener. Shear failure can result in the complete rupture or fracture of the fastener, which can compromise the structural integrity of the connection.

Figure 1.11: Bending and shearing off

**Tension Failure:** This occurs when the applied load exceeds the tensile strength of the fastener. Tension failure can result in the fastener stretching or breaking, which can reduce the clamping force and cause the connection to loosen.

Figure 1.12: Tensile crack under screw location

**Titling failure:** This occurs when the connected member tilts or rotates around the bolt or weld axis. It is caused by inadequate shear strength or stiffness of the connected members or by excessive deformation of the connected members. This type of failure can result in a reduction of the shear capacity of the connection.

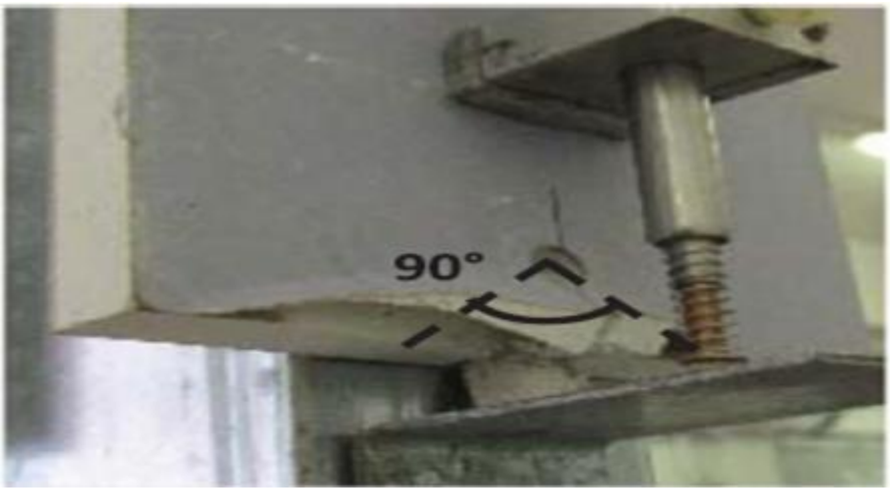


Figure 1.13: Tilting failure at 90° angle

**Pull-out failure:** This occurs when the connected member is pulled out of the con- nection due to insufficient bearing strength or clamping force. Pull-out failure is more common in bolted connections and can be prevented by ensuring proper bolt installation torque and sufficient bearing area.

Figure 1.14: Pull-out failure [6]

**Pull-through failure:** This occurs when the connected member fails in tension due to insufficient tensile strength or thickness. It is more common in welded connections and can be prevented by using thicker or stronger members or by increasing the number of welds.

Figure 1.15: Pull through failure [9]

**Edge tearing:** occurs when the material around the edge of a hole in a connection plate or beam web begins to tear due to the high stresses induced by the applied load. This type of failure is most likely to occur when the connection is loaded in tension and is particularly common in thicker plates or when the hole is located too close to the edge of the plate.

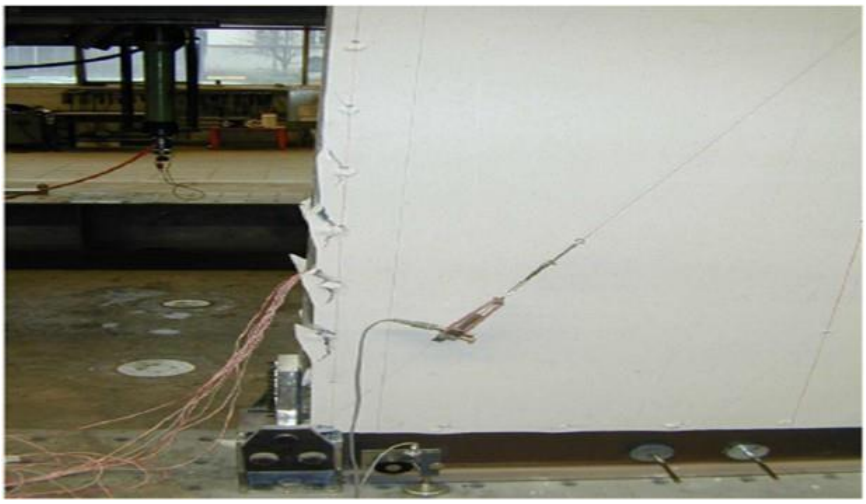


Figure 1.16: Fracture of the edge (tension) [10]

**Local buckling:** occurs when a portion of the connection plate or beam web buckles under compressive stress, reducing the overall strength of the connection. This can happen when the plate or web is too thin or when it is not sufficiently braced against lateral deformation.

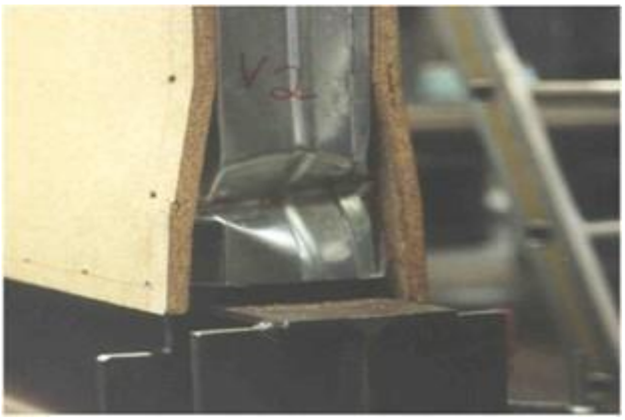
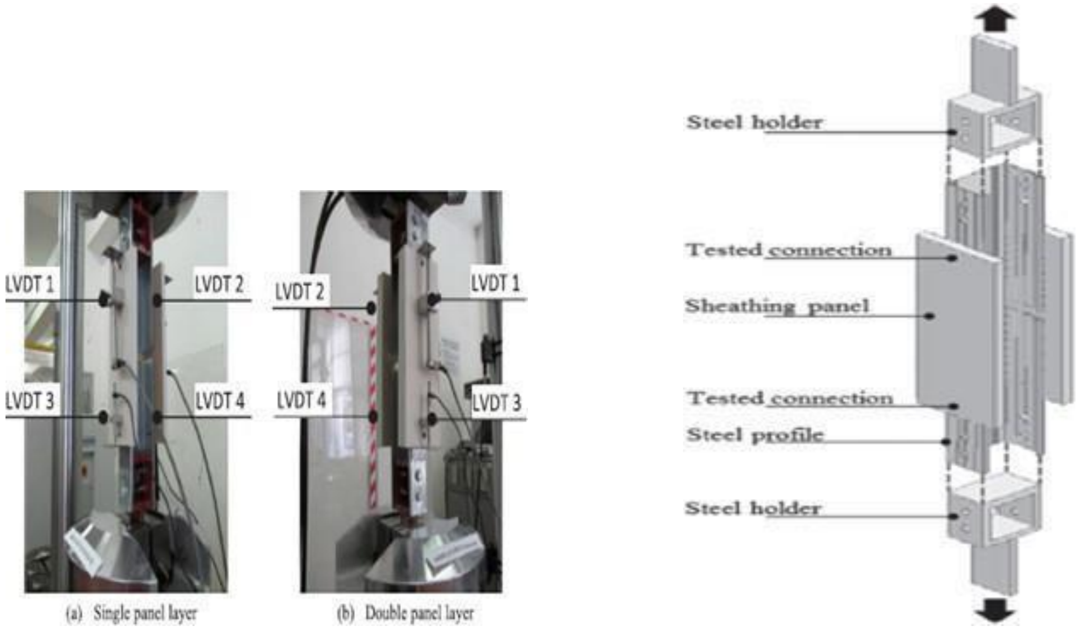


Figure 1.17: Buckling of the cold-formed profile at the bottom of the compression stud. [10]

# Chapter 2: LITERATURE REVIEW

## 2.1 Review of available literature

In the experimental study of Luigi Fiorino, the study aimed to investigate the perfor- mance of various screw connections used to attach gypsum or cement-based panels to cold-formed steel (CFS) framing. Conducting, monotonic and cyclic loading tests to evaluate the connection's resistance to vertical and lateral loads. Both monotonic and cyclic loading modes are employed to assess the connection's performance under different loading scenarios. Monotonic loading helps determine the ultimate strength, while cyclic loading evaluates the connection's behavior under repeated loading. The performance of the connection was affected by factors such as panel type, screw size and spacing, and framing member thickness. Use of larger diameter screws and closer screw spacing improved the connection's resistance to both vertical and lateral loads. Gypsum-based panels had better performance than cement-based panels. The study's findings provide valuable information for designers and engineers involved in the con- struction of CFS structures with gypsum or cement-based panels to make informed decisions about screw size, spacing, and panel type for optimal performance and dura- bility. Also, identified areas for further research, such as the effect of different framing configurations and loading conditions on the connection's performance.



(a) Single panel layer (b) Double panel layer

Figure 2.1: Transducers at the laboratory of the Dept. of Structures for Engg and Arch. of University of Naples Federico II [8]

Tests were performed by using a universal test machine and four linear variable differ- ential transducers (LVDTs) were used to measure the relative displacements between panels and steel profiles, as shown.

A study done by L. A. Fülöp and D. Dubina proposes design criteria for seam and sheeting-to-framing connections of cold-formed steel (CFS) shear panels, based on ex- perimental tests and numerical simulations. The study focuses on two types of connec- tions: side lap seams and sheeting-to-framing connections, which are critical for the overall performance of CFS shear panels. The proposed design criteria take into ac- count factors such as material properties, connection geometry, and loading conditions, and are based on established design methodologies and standards. The paper presents a set of design equations for calculating the strength and stiffness of seam and sheeting- to-framing connections, and provides guidelines for selecting appropriate connection types and details. The proposed design criteria are validated through comparison with experimental data and numerical simulations, and are found to provide accurate predic- tions of connection performance. The study concludes that the proposed design criteria can be used to improve the design and performance of CFS shear panels, leading to more efficient and cost-effective structural solutions.

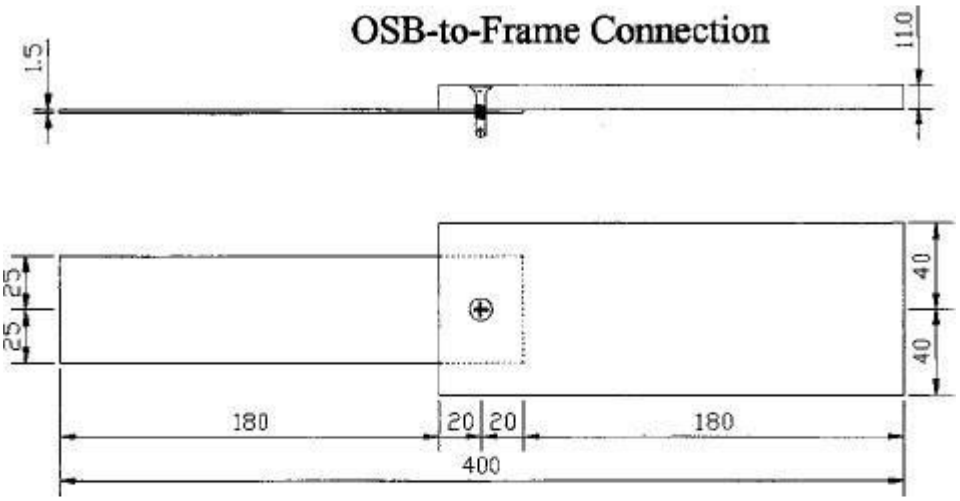


Figure 2.2: Tested OSB-to-steel skeleton connections. [11]

Above as shown connection topology was prepared using the wall panel test connect- ing OSB to steel skeleton. The testing of these specimens yielded very inhomogeneous results depending on the direction and density of fibers in the vicinity of the screw and between the screw and the margin of the OSB panel.

The experimental findings of Ashok Jammi and S. Arul Jayachandran aimed to inves- tigate the performance of screw connections between cold-formed steel (CFS) framing and sandwich sheathing panels under axial, shear, and combined loading conditions. The performance of the connections was evaluated using both monotonic and cyclic loading tests. The results showed that the connection strength and stiffness were in- fluenced by factors such as screw type, screw spacing, and panel thickness. The study found that increasing the screw spacing reduced the connection strength and stiffness, while using longer screws increased the strength and stiffness. The cyclic loading tests revealed that the connections exhibited significant strength degradation under repeated loading, with the degree of degradation depending on the loading direction and loading history. The study also found that the failure mode of the connections varied depending on the loading direction and the type of sheathing panel used. The findings of the study can be used to inform the design of screw connections between CFS framing and sand- wich sheathing panels, and can help to improve the overall performance and durability of these systems.

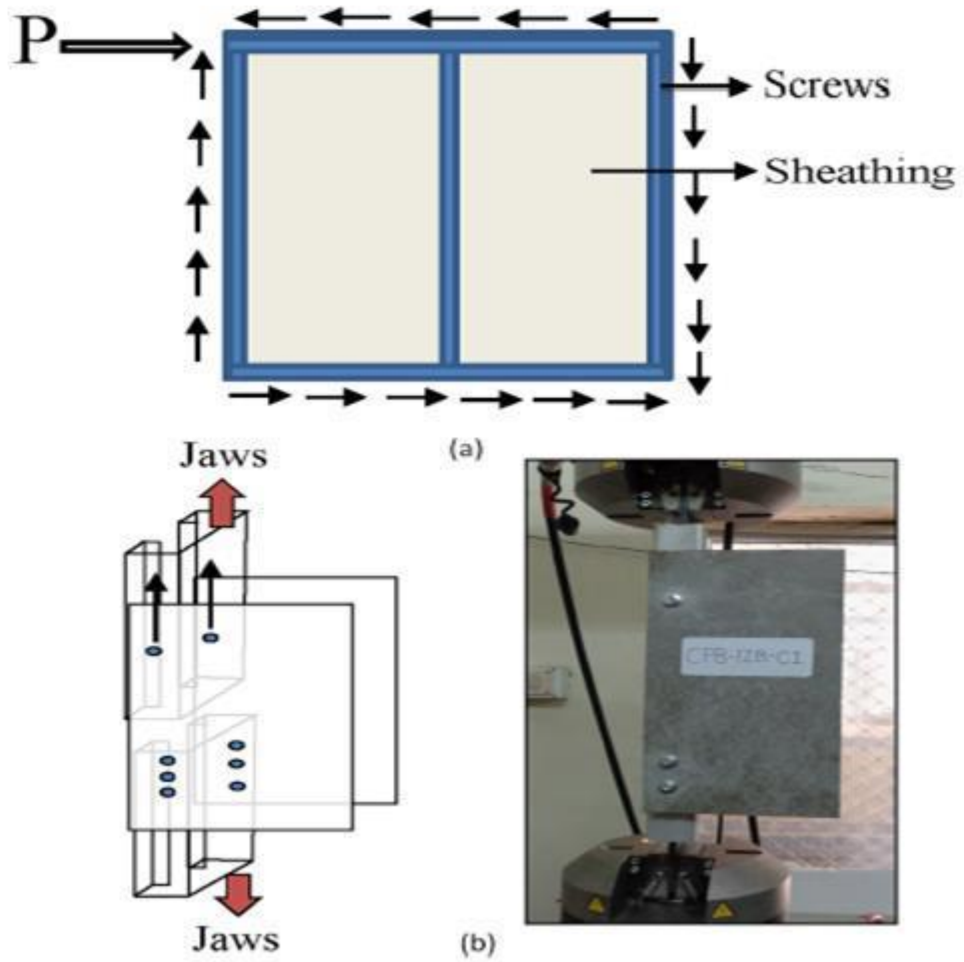


Figure 2.3: (a) Shear experiencing direction by screws in wall panel test. (b) Experi- mental study setup [6].

The assembly shown in Fig. 20(b) is constructed to examine the fastener to comprehend shear parallel to the free edge of the sheathing. This is to research the problem of screw in case of full-scale wall panel tests Fig. 20 (a). Fig. 20(b) depicts the experimental assembly used. The red arrows displayed (solid color) are where the UTM jaws are fixed. This is established by utilizing two 12 mm thickness mild steel holders, bolted at the UTM jaws on one end and the other to the CFS profiles web.

In a study, proposed by Jo¨rg Lange and Bernd Naujoks the results showed that the shear walls exhibited significant strength and stiffness under both horizontal and verti- cal loads, and were able to resist large deformations before failure. The failure modes of the shear walls varied depending on the type of framing member and sheathing panel used, with some configurations exhibiting brittle failure modes and others exhibiting more ductile failure modes. The numerical simulations were used to validate the ex- perimental results and to investigate the behavior of the shear walls under different loading scenarios. The study provides valuable information for designers and engineers involved in the construction of CFS shear walls, and can help to improve the overall performance and safety of these systems.

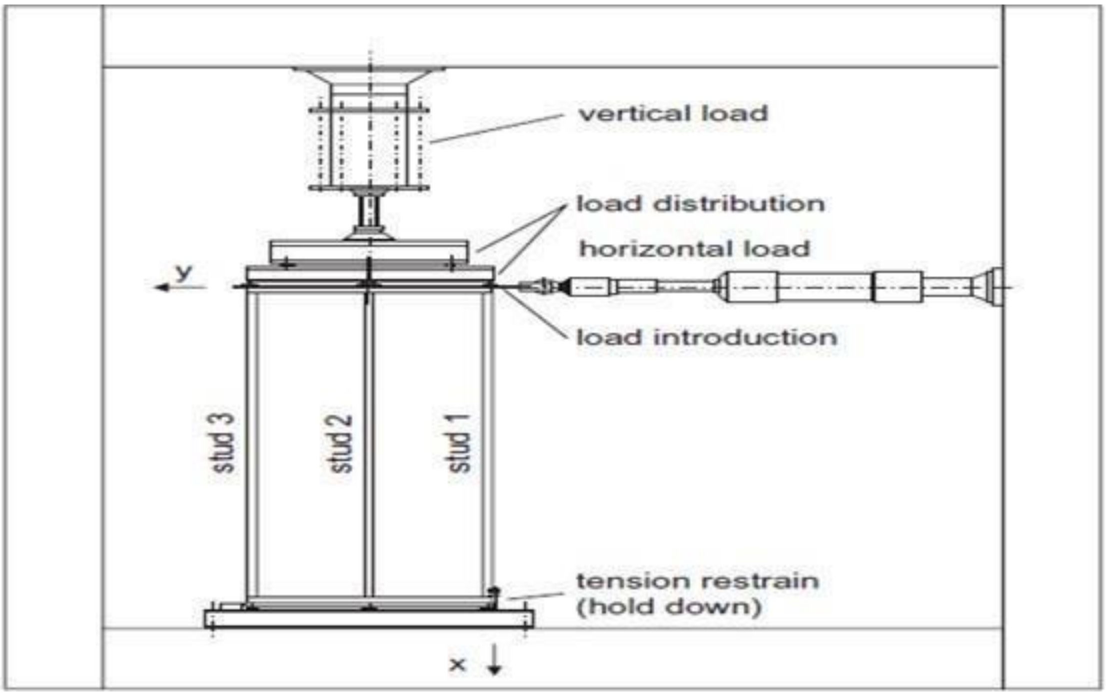


Figure 2.4: Test rig for walls under horizontal and vertical load. [10]

The horizontal loading (Fig. 21) is shown on a load cycle on the standards for testing shear walls in timber frame construction, DIN EN 594/07 96. In a study by M. Nithyadharan and V. Kalyanaraman investigated the behavior of screwed connections in cold-formed steel (CFS) framing and calcium silicate board (CSB) panels under both monotonic and cyclic loading conditions. The CSB panels were at- tached to the CFS framing using self-tapping screws with varying lengths and spacing. The experimental tests showed that the strength and stiffness of the screw connections were influenced by factors such as screw length, screw spacing, and panel thickness. The study found that increasing the screw spacing reduced the strength and stiffness of the connections, while increasing the screw length improved the strength and stiff- ness. The cyclic loading tests revealed that the screw connections exhibited significant strength degradation under repeated loading, with the degree of degradation depend- ing on the loading direction and loading history. The failure modes of the connections varied depending on the loading direction and the type of screw used, with some config- urations exhibiting pullout failure modes and others exhibiting shearing failure modes.

Fig. 22 (a) below shown the setup of the screw connection test frame, to study the shear in screws parallel to the nearest free edge of the sheathing. The initial two linear variable differential transducers (LVDT’s) were conducted with two tests on the two faces of the specimen Fig. 22(b), parallely with UTM transducer, for the relative displacement measurement between the stud & sheathing.

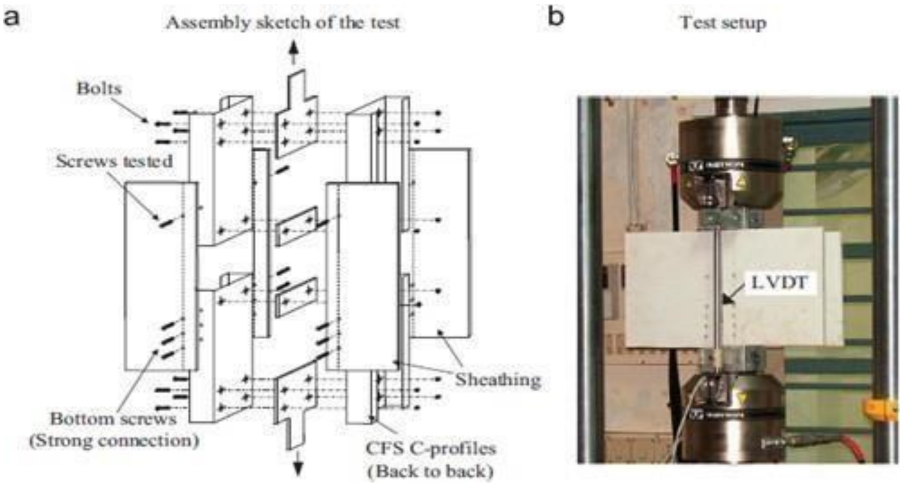


Figure 2.5: Test specimen and setup. [9]

A finding by Reynaud Serrette and David Nolan states that the behavior of wood struc- tural panels (WSP) attached to cold-formed steel (CFS) framing members using pneu- matically driven knurled steel pins (KDSP). The key parameters studied include the pin length, pin spacing, panel thickness, and loading direction. The experimental tests in- cluded both monotonic and cyclic loading tests, and were used to evaluate the strength and stiffness of the WSP-CFS connections under different loading conditions. The study found that increasing the pin length and reducing the pin spacing improved the strength and stiffness of the connections. The failure modes of the connections varied depending on the loading direction, with some configurations exhibiting pullout failure modes and others exhibiting lateral buckling failure modes. The study also found that the WSP-CFS connections exhibited significant strength degradation under repeated loading, with the degree of degradation depending on the loading direction and loading history.

The findings of the study can be used to inform the design of WSP-CFS connections using KDSP, and can help to improve the overall performance and durability of these systems.

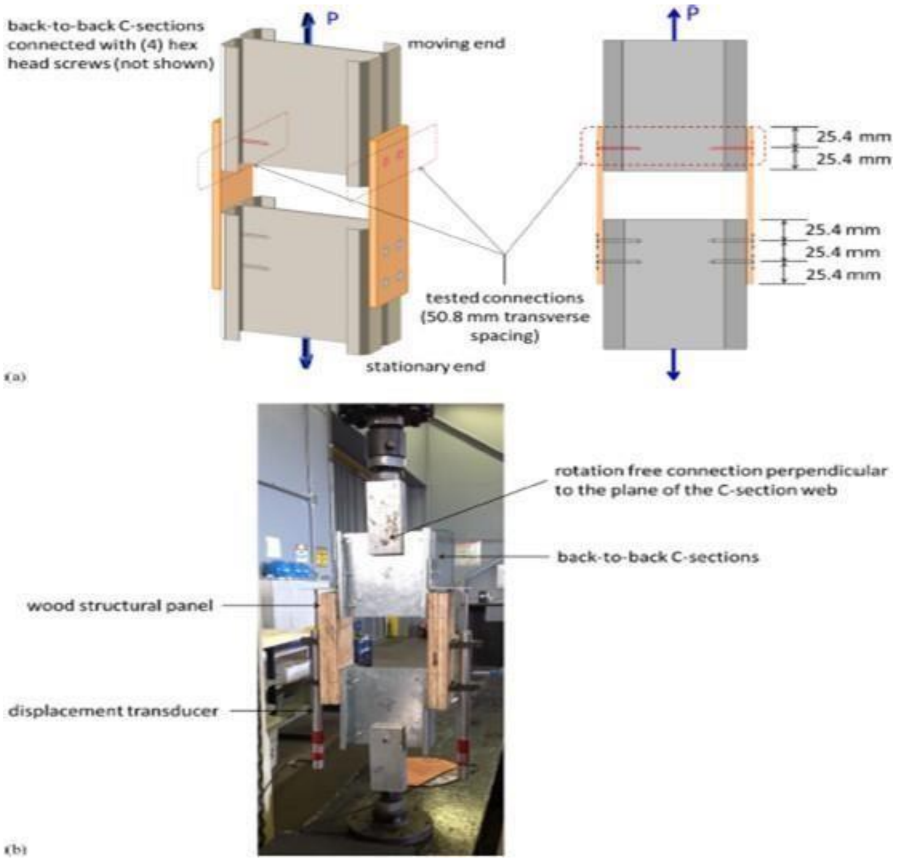


Figure 2.6: Test setup (a) Illustration (b) actual [12]

This test program shows a loading scheme and a symmetric lap-shear test setup, in Fig. 23, Strips of WSP (long side parallel to the longitudinal axis of the full sheet and the applied load) were connected to the flanges of the back of the CFS C- sections. The upper C-sections connected four pins to the WSP strips, the lower C-sections and eight screws are connected to the opposite end of each strip. The average connection deformation at the pins was used to measure through the displacement transducers. The study conducted by K.D. Peterman, N. Nakata, and B.W. Schafer focuses on the hysteretic behavior of cold-formed steel (CFS) stud-to-sheathing connections under cyclic loading conditions. The experimental tests were conducted using a custom- designed testing apparatus, and included both monotonic and cyclic loading tests. The study found that the behavior of the connections was strongly influenced by factors such as fastener type, fastener spacing, and sheathing type. The cyclic loading tests showed that the connections exhibited significant strength degradation and stiffness degradation under repeated loading, with the degree of degradation depending on the loading direction and loading history. The study also found that the connections exhibited pinching and hysteresis loops in their load-displacement response, indicating the presence of en- ergy dissipation mechanisms. The findings of the study can be used to inform the design of CFS stud-to-sheathing connections, and can help to improve the overall performance and durability of these systems. The study highlights the importance of considering the hysteretic behavior of CFS connections in the design of cold-formed steel structures, and provides valuable insights into the behavior of these systems under cyclic loading conditions.

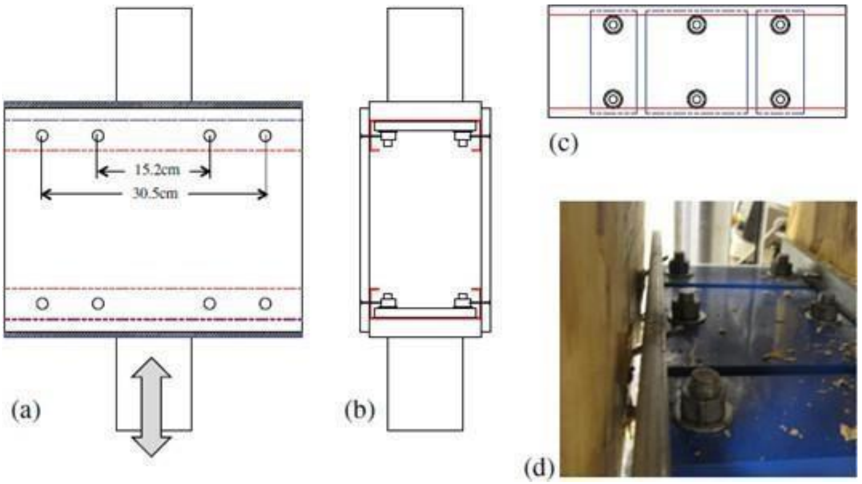


Figure 2.7: (a) Front view of loaded specimen, dashed lines indicate hidden stud, arrow indicates direction and location of loading [13]

The Fig. 24 depicts the response of eight stud-fastener sheathing combinations in shear. Each stud is fastened with two fasteners per flange, resulting in a total of eight fasteners. Fig. 24(a) demonstrates 15.2cm  or 30.5 cm fastener spacing's explored in this testing.

As illustrated in Fig. 24(d), in the test arrangement, the fasteners will tilt, and, with high deformations, the points of the fasteners may bear on the web of the channel. Fig. 24(c) Full fastener movement was allowed at both of the tested fastener spacings, and fastener tip bearing was not seen throughout the experiments.

The key parameters of study done by Tiziano Sartori and Roberto Tomasi is aimed to investigate the behavior of panel- to- framing connections in timber shear walls under monotonic and cyclic loading conditions. The parameters studied include the fastener type, fastener spacing, sheathing thickness, and loading direction. The exper- imental tests were conducted using a custom-designed testing apparatus, and included both monotonic and cyclic loading tests. The study found that the behavior of the con- nections was strongly influenced by the fastener type and spacing, with larger and closer fasteners resulting in increased strength and stiffness. The cyclic loading tests showed that the connections exhibited significant strength degradation and stiffness degradation under repeated loading, with the degree of degradation depending on the loading direc- tion and loading history. The study also found that the connections exhibited pinching and hysteresis loops in their load-displacement response, indicating the presence of en- ergy dissipation mechanisms. The findings of the study can be used to inform the design of wood shear walls and sheathing to framing connections, and can help to improve the overall performance and durability of these systems.

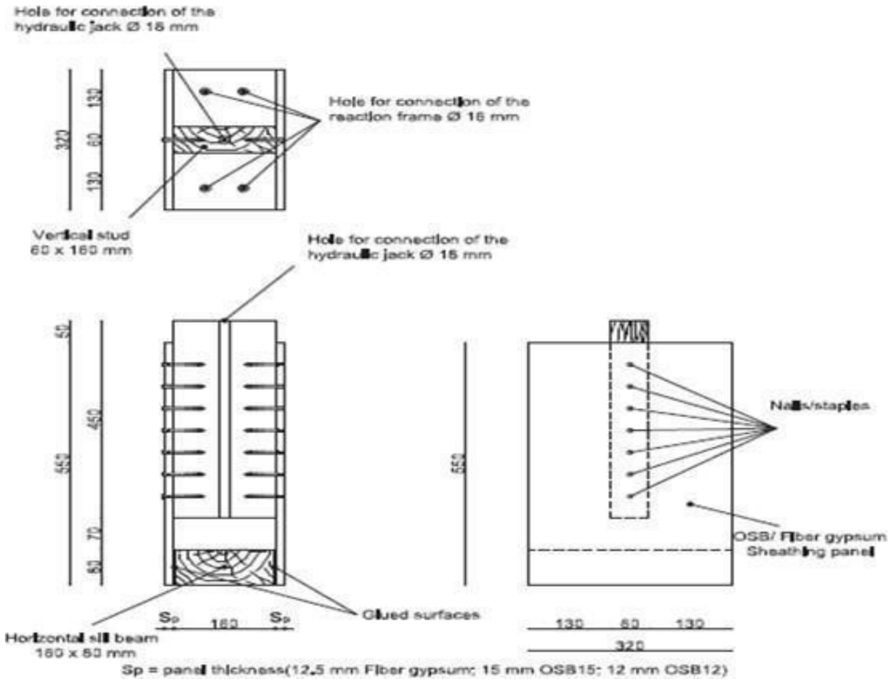


Figure 2.8: Specimen geometry [14]

Since the goal is to evaluate the mechanical properties of the nailed sheathing to frame member connection the specimen shown above in Fig. 25 is generally used in shear tests, based on two side boards (OSB, gypsum) and a solid wood element held together by fasteners.

The lateral boards are glued to a wooden beam rigidly connected to the steel frame, while the wooden beam is connected through a hydraulic jack by a steel threaded bar inserted axially through a central hole. The instrumentation layout is shown in Fig.26.

The four LVDT transducers are used to measure the displacement.

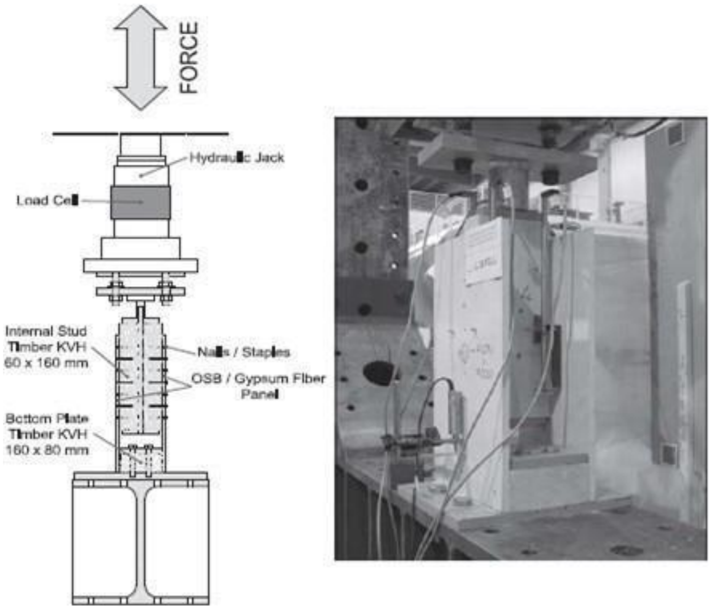


Figure 2.9: Test setup Illustration and actual. [14]

A study by S.G. Buonopane, G. Bian, T.H. Tuna, and B.W. Schafer aims to develop computationally efficient models of cold-formed steel shear walls with wood sheath- ing, which can accurately predict their behavior under monotonic and cyclic loading conditions. The key findings of the study include the development of fastener-based models for the connections between the sheathing and the framing, which can accu- rately capture their behavior. The models were validated using experimental data, and were found to accurately predict the load-displacement response of the shear walls un- der both monotonic and cyclic loading conditions. The study found that the strength and stiffness of the shear walls were strongly influenced by factors such as the fastener type, fastener spacing, and sheathing thickness. The developed models can be used to opti- mize the design of cold-formed steel shear walls with wood sheathing, and can help to improve the overall performance and durability of these systems. The study highlights the importance of considering the behavior of the connections between the sheathing and framing in the design of cold-formed steel shear walls, and provides a computa- tionally efficient method for accurately predicting their behavior. The test setup is a fastener-based modeling approach for the full cyclic behavior of wood-framed shear walls. It has also been incorporated into the nail-pattern analysis module of the SAWS and SAP Wood software, as well as into general purpose finite-element software. Each OSB or gypsum board panel is modeled as a separate rigid body (RigidDiaphragm in OpenSees), with nodes at every fastener location and a master node at the center of the panel.

A study by Luiz C.M. Vieira Jr. and Benjamin W. Schafer aims to investigate the lat- eral stiffness and strength of sheathing braced cold-formed steel stud walls, which are commonly used as lateral load resisting systems in buildings. The study found that the lateral stiffness and strength of the walls were strongly influenced by the sheathing thickness, with thicker sheathing resulting in increased stiffness and strength. The stud depth and spacing were found to have a lesser effect on the stiffness and strength of the walls, while the edge fastener spacing had a prominent effect on the strength of the walls. The study also found that the walls exhibited significant strength degradation and stiffness degradation under repeated loading, with the degree of degra- dation depending on the loading direction and loading history.

The findings of the study are to inform the of sheathing braced design cold-formed steel stud walls, and can help to improve the overall performance and durability of these systems.

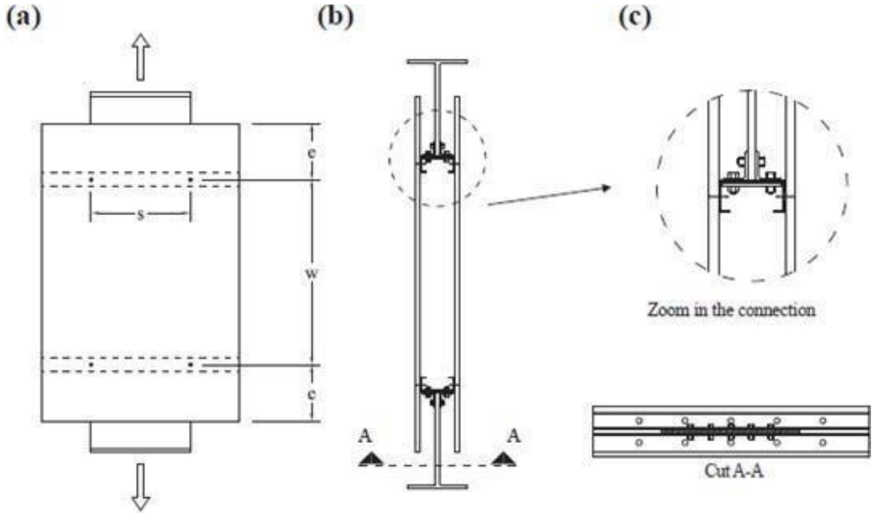


Figure 2.10: Test setup of lateral stiffness test for measurement of local translational stiffness of stud–fastener–sheathing assembly. (a) Front view, (b) side view, and (c) details [15]

Winter’s method for determining the translational stiffness of a stud–sheathing assem- bly employs a simple symmetrical shear test, as illustrated in Fig. 27 Two sections of studs are connected by identical sheathing on both sides and then pulled laterally (per- pendicular to the long direction of the studs).

The key parameters in a study conducted by Sivaganesh Selvaraj, Mahendrakumar Madhavan, and Hieng Ho Lau aimed to develop a design method for sheathed cold- formed steel (CFS) point-symmetric wall frame studs, which can accurately predict the strength and deformation behavior of these systems. The proposed design method is based on the strength of the sheathing-fastener connection, and uses a series of equa- tions to predict the strength and deformation behavior of the wall frame studs under both monotonic and cyclic loading conditions. The study includes experimental tests to validate the proposed design method, and found that it accurately predicts the strength and deformation behavior of the wall frame studs under various loading conditions. The study also found that the strength of the sheathing-fastener connection is the crit- ical factor controlling the overall strength and deformation behavior of the wall frame studs. The proposed design method can be used to optimize the design of sheathed CFS pointsymmetric wall frame studs.

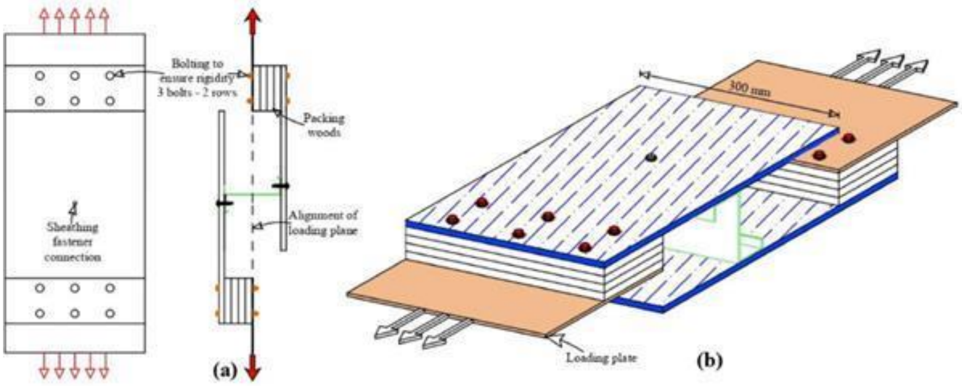


Figure 2.11: New setup to determine the strength of the sheathing fastener connection against the twist of the point symmetric shaped CFS stud. [16]

The flanges of the CFS studs were pulled in opposite direction by the machine grips to simulate a forward cross-sectional twist. It should also be noted that the test setup is de- veloped such that the sheathing fastener connections in both top and bottom flanges of the CFS stud are loaded simultaneously, which is the actual force distribution scenario in the CFS wall frame with sheathing.

The findings in a study by S. Swensen, G.G. Deierlein, and E. Miranda focuses on the behavior of screw and adhesive connections to gypsum wallboard in both wood and cold-formed steel-framed wallettes. The results of the study indicate that the use of adhesives can significantly increase the strength and stiffness of the connections, par- ticularly for steel-framed wallettes.

The study also found that the number and spacing of screws have a significant impact on the behavior of the connections, and that proper installation is critical for achieving optimal performance. The study provides insights into the study of fastener and adhe- sive connections to gypsum wallboard, and can help inform the design and construction of wood and cold-formed steel-framed wall systems.

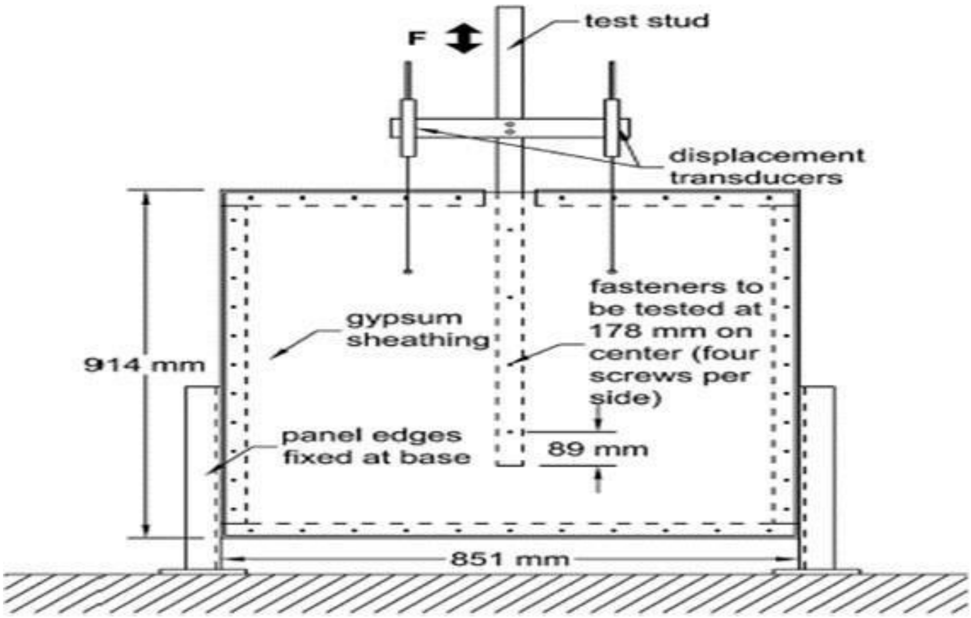


Figure 2.12: Drywall fastener test configuration [17]

All specimens were tested in a 250 kN (55 kip) MTS 322 test frame controlled using MTS **MultiPurpose TestWare**. Wood screws were used to attach the vertical edge studs to steel channels that were bolted to the testing machine base. Using large wood screws, the center stud was connected to a steel loading yoke, which was attached to the loading piston.

The key parameters in Jihong Ye, Xingxing Wang, and Mengyuan Zhao study focuses on the shear behavior of screw connections in cold-formed steel (CFS) sheathing. The results of the study indicate that the load-carrying capacity and stiffness of the connec- tions are influenced by the screw diameter, screw spacing, and sheathing thickness. The study also found that the failure modes of the connections include screw bending, screw pullout, and sheathing splitting.

The study provides insights into the fastener connections in Cold Form Steel sheathing, and can help inform the design and construction of CFS sheathed walls and roofs.

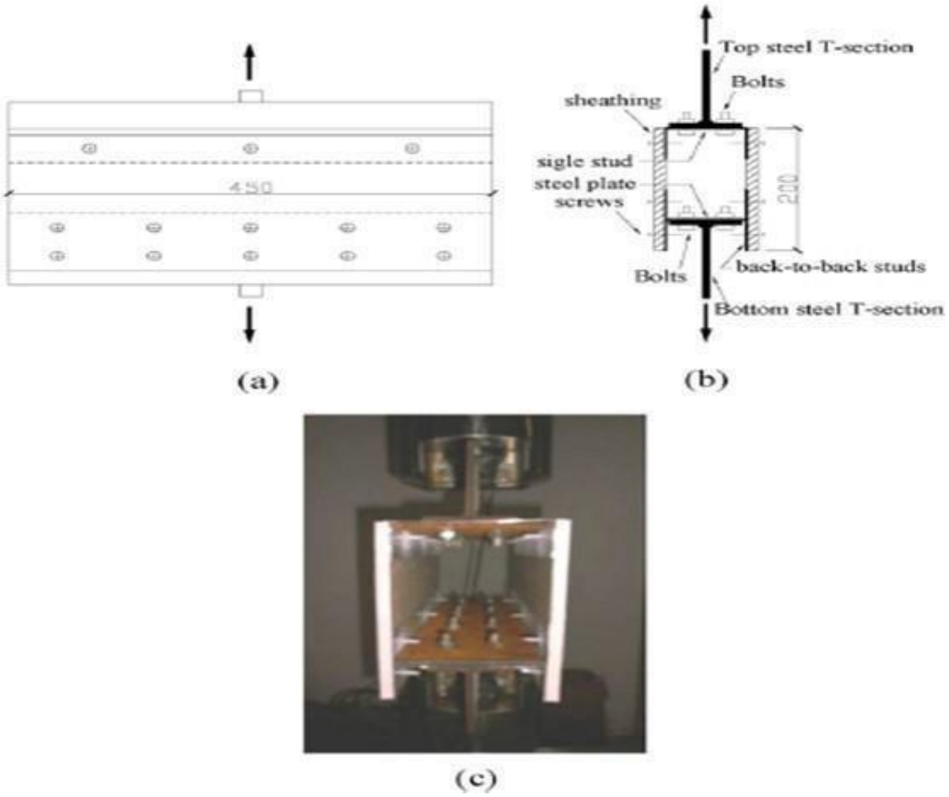


Figure 2.13: Screw connection test setup (a) Front (b) Profile (c) Test setup [18]

In Fig. 20, The entire test setup consisted of a couple of steel T-sections and two steel plates. CFS studs were fastened to the steel T-section pair first, followed by the steel plates on the inner side of each stud. Two steel plate sizes, 450 mm × 80 mm and 450mm× 136mm, were selected to align with the specimens featuring studs measuring 0.9 mm and 1.2 mm thick, respectively. Sheathings were fastened with six screws (including the opposite flanges) for the top frame and two rows of five fastener for the bottom frame to prevent failure in the fastener at the top end connection.

In this chapter past research/background/theory/standards/formula on ultimate strength capacities are provided first. This depends on various factors including the screw parameters of the sections of CFS panels such as thickness and the type and strength of the cold formed steel material. Methods of calculating these capacities are discussed. Followed by explanation on the available behavioral dataset collected by the past research done.

# Chapter 3: AISI S400 GUIDELINES

## 3.1 Explanation of Section J4 of AISI S100

Section J4 of AISI S100 provides detailed design requirements. These provisions en- sure that the member is designed to resist the applied loads and that it is not susceptible to failure due to lateral-torsional buckling or shear. The nominal shear strength for screw connections is determined by the following (AISI, 1996)

- $P_{ns} = 4.2 \sqrt{t_2^3 d_p}\, F_{u2}$ Equation 1 ref AISI-S400 J.4.3.1
- $P_{ns} = 2.7\, t_1\, d_p\, F_{u1}$ Equation 2 ref AISI-S400 J4.3.1-2
- $P_{ns} = 2.7\, t_2\, d_p\, F_{u2}$ Equation 3 ref AISI-S400 J4.3.1-3
- $P_{not} = 0.85\, t_c\, d\, F_{u2}$ Equation 4
- $P_{nov} = 1.5\, t_1\, d_w\, F_{u1}$ Equation 5

Where $F_{u2}$ is the tensile strength of the steel frame, correspondingly $t_2$ is the thickness of the steel frame. The tensile strength of the steel frame is found from corresponding author’s papers as mentioned in the excel sheet.

Accordingly, $d_p$, is the screw diameter mentioned in the research paper. $t_1$ is the thick- ness of the sheathing panel found from various research papers mentioned in the excel sheet.

To find $F_{u1}$ the tensile strength was studied according to the panel direction. The ten- sile strength was analyzed through transverse and longitudinal axis. Both the strengths were then taken and an average tensile strength Avg Fu was calculated. This allowed to calculate the AISI nominal strength calculation as shown above. This allowed to calculate the minimum $P_{NS}$. Correspondingly, allowing to calculate the ratio of the nominal shear strength to the ultimate shear strength of the screws. Section J.4.3.1. Of AISI S100 provides the equations and procedures for calculating the nominal flexural strength subjected to lateral loads. The section provides detailed information on how to determine the moment amplification factor and effective length factor based on the length and end conditions of the member. The section also provides limitations on the section properties.

# Chapter 4: ANALYSIS OF EXPERIMENTAL DATA

## 4.1 Orient Strand Board (OSB)

Table 4.1: OSB Dataset

| **Author** | **Edge distance (mm)** | **Thickness of frame** | **Thickness of Panel** | **Shear Strength (kN)** | **Type of fastener** | **Diameter of fastener (mm)** | **Length of fastener (mm)** | **Ultimate Strength for single connection (N)** | **Type of loading** |
|---|---|---|---|---|---|---|---|---|---|
| K.D. Peterman | 38 | 0.84 | 11.1 | 0.885 | SDS | 4.2 | 4.16 | 1.77 | M/C |
| B.W. Schafer | 20 | 1.81 | 11.9 | 1.014 | SDS | 4.2 | 4.17 | 1.2 | M/C |
| Luiz Vieira, Schafer | 15.2 | 1.81 | 12 | 2.01 | Simpson #8 | 6.35 | 83 | 0.489 | M |
| Jo¨rg Lange | 20 | 1.5 | 12 | 1.96 | SDS | 4.2 | 4.16 | 1.363 | M |
| Fu¨lo¨p and Dubina | 30 | 0.42 | 12.7 | 1.95 | SDS | 4.8 | 22 | 1.8 | M/C |
| R. Serrette, D. Nolan | 25 | 0.84 | 12.7 | 2.05 | Knurled steel pins | 2.5 | 38 | 0.86 | M |
| Ye et al. | 15 | 0.9 | 18 | 2.01 | SDS | 4.8 | 22 | 1.55 | M/C |
| Sartori, Tomasi |  | 2.8 | 15 | 1.72 | smooth nails | 2.5 | 60 | 1.14 | M/C |

A total of 13 experiments were conducted to investigate screw connections between steel studs and 2 different types of sheathing materials: gypsum board, and oriented strand board (OSB). The objective was to analyze how factors like stud thickness, screw diameter, and edge distance affect the performance of these connections. Shown above, is the dataset collected for the OSB panel only. The author column identifies all the research papers studied. The edge distance is the distance between the edge of the ma- terial being fastened (in this case, OSB) and the point where the fastener (screw) is inserted into the framing member (OSB – to – steel). The edge distance needs to be adequate for the size of the fastener being used, in order to ensure a strong and secure connection.

The thickness of the frame column in CFS sheathing panels typically refers to the thick- ness of the steel studs (typically in C shape or track shape) used to create the frame of the panel.

The thickness of the panel (OSB) column in CFS (cold-formed steel) sheathing panels refers to the thickness of the sheathing material that is attached to the steel frame of the panel.

The shear strength column above is referred to as shear strength in CFS (cold-formed steel) sheathing panels to resist forces that cause the panel to slide or shear along its plane.

The type of fasteners studied in the research above self-drilling screws (SDS), knurled pins, smooth nails, steel pins, ring nails & screw nails.

Self-drilling screws: - are screws that have a drill bit at the tip. This eliminates the need for pre-drilling and makes the installation faster and easier. SDSs are commonly used in CFS sheathing panels as they can easily penetrate through the thin-gauge steel studs and attach the sheathing material securely to the frame.

Knurled pins: These are pins with a textured surface that provides a better grip when inserted into the material being fastened. Knurled pins are often used in conjunction with adhesive to attach sheathing material to CFS framing. The knurled surface of the pin helps to anchor it in the material, while the adhesive provides additional bonding strength.

Smooth nails: These are nails with a smooth surface, without any texture. Smooth nails are commonly used in sheathing applications as they can be easily driven into the sheathing material and the CFS framing. However, because they rely solely on friction to hold the sheathing material to the framing, they may be less reliable.

Steel pins: These are straight metal pins. They can be easily driven into the CFS fram- ing to hold the sheathing material in place until permanent fasteners, such as SDSs or knurled pins, can be installed. Steel pins are typically removed after the permanent fas- teners are installed.

Ring shank nails: are likely a type of nail that has a ringed texture or pattern on its shaft. The ringed texture provides enhanced grip and holding power compared to smooth- shank nails. These nails are commonly used to fasten steel studs, tracks, and other components in a CFS system.

Screw nail: are fasteners that combine features of both screws and nails. These are often used in steel framing applications where a strong and secure connection is needed. The diameter of the fastener refers to the thickness or width of the fastener shaft or thread. The diameter of the fastener is an important factor in determining its load- carrying capacity and its ability to resist pullout forces.

Length of fastener: This refers to the distance between the head of the fastener and the end of the shaft or thread. The length of the fastener is important in ensuring that it can penetrate both the sheathing panel and the steel frame to achieve a secure attachment. OSB (Oriented Strand Board) panel span rating refers to the maximum distance, in inches, that the panels can span between supporting members (such as joists or studs) while still maintaining their structural integrity. The rating is typically given as two numbers separated by a slash, such as 24/16, 32/16, or 48/24. Here's a detailed expla- nation of what these numbers mean and how they affect panel shear strength:  First Number (e.g., 24, 32, 48): indicates the maximum recommended spacing between the supporting members. For example, with a rating of 24, the panels should be in- stalled with no more than 24 inches between joists or studs.

Second Number (e.g., 16): represents the maximum allowable span when the panels are used for roof sheathing. In this example, with a rating of 24/16, the panels can span up to 16 inches when used on a roof.

Effect on Panel Shear Strength: The span rating directly affects the panel's shear strength and load-bearing capacity.

The ultimate strength of a single connection in CFS (cold-formed steel) sheathing pan- els refers to the maximum load or force that a single fastener or connection can sustain before it fails.

Monotonic loading refers to the application of a steadily increasing load or force to the panel until it reaches its maximum load-carrying capacity or fails. Cyclic loading, on the other hand, refers to the repeated application of loads or forces to the panel over a period of time. This type of loading is often used to simulate the effects of seismic or wind-induced forces on the panel.

| Author | Layers of Panels | Loading protocol | Loading rate (mm/sec) | Height of flange/ Web (whichever nail is connected to) | Type of failure in screw | $f_u$ (MPa) | $f_y$ (MPa) |
|---|---|---|---|---|---|---|---|
| K.D. Peterman | S | CUREE | Constant load throughout the test at one full cycle every 16sec | F | PT | 448 | 345 |
| B.W. Schafer | S | AISI S213-07 | | | PT | 448 | 345 |
| Luiz Vieira, Schafer | D | | | F | | 448 | 345 |
| Jörg Lange | S | ECCS 1985 | | F | LB | 460 | 320 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Fu¨lo¨p and Dubina | S | ECCS | 1983 | T,PT | 440.6 | | |
| R. Serrette, D. Nolan | S | AISI 2012 | 0.042 | F | T | 374 | 311 |
| Ye et al. | S | | 0.03 | F | T,B,E | 448 | 345 |
| Sartori, Tomasi | D | CUREE | 0.05 | | E,T,B | 600 | |

Referring to the loading protocol column, it refers to the specific sequence and magni- tude of loads or forces that are applied to the connection during testing. Examples of loading protocols that are commonly used in CFS connections are the CUREE, ECCS, AISI protocol.

The CUREE (Consortium of Universities for Research in Earthquake Engineering) pro- tocol is a loading protocol that is commonly used in seismic testing of CFS sheathing connections. The protocol involves applying a series of simulated seismic forces to the connection in different directions and frequencies, while monitoring the response of the connection. The ECCS (European Convention for Constructional Steelwork) protocol is a loading protocol that is commonly used in testing the capacity of CFS sheathing connections to resist wind loads. The protocol involves applying a series of gradually increasing wind pressures to the connection in a specific pattern, while monitoring the deflection and stress response of the connection. The AISI (American Iron and Steel Institute) standards for the loading protocol in CFS connections, may involve apply- ing simulated seismic loads to the connection in a specific pattern, while measuring the deflection and stress response of the connection. The AISI S213 standard for CFS framing, for example, specifies loading protocols for testing the lateral resistance of CFS stud-to-track connections under wind loads.

Type of failure column is discussed in the failure mechanism section. The $f_u/f_y$ column refers to the ultimate strength and yield strength of the connection.

# Chapter 5: EXPERIMENTAL PROGRAM

## 5.1 Experimental Setup

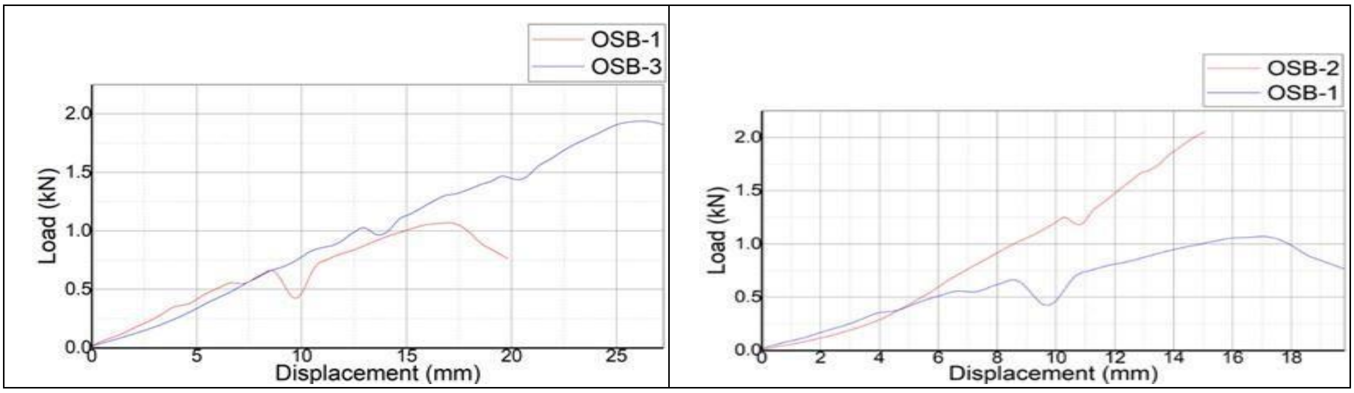
The experimental program involved different panel-to-CFS connections. The connec- tions under investigation were made of different panels (Orient strand board & gypsum) which are fastened to steel grade CFS profiles with ring shank nails and screw nails. Since the usage of ring shank nails and screw nails is being used with an edge distance of 15mm (In accordance with ASTM C1513-19: "Standard Specification for Steel Tapping Screws for Cold-Formed Steel Framing Connections") and at a monotonic loading rate of 0.05mm/s. OSB sheets were purchased from ACE hardware store in, Dubai of thickness ranging from (9mm, 11mm, 15mm, 18mm.) Gypsum sheets were purchased locally (12mm). “C” shaped steel frame was procured from Rawat industrial area. In accordance to AISI standards the specimen was subjected to conventional tension tests in the UTM at the NICE testing lab. Two small steel sheets of 152mm x 71mm with 6.35mm thickness on each side of the steel frame were procured to hold into place on the UTM clamp plates in accordance with ASTM ASTM E564-19. In particular, the steel frame is distinguished in four series obtained by four steel grade coils with nominal thicknesses of 1.14mm, 2.45mm, 1.2mm and 0.90 mm. The rational for choosing the specimen assembly is that different delayed failure modes may be observed, thereby sustaining higher shear force.

Table 5.1: Test Matrix

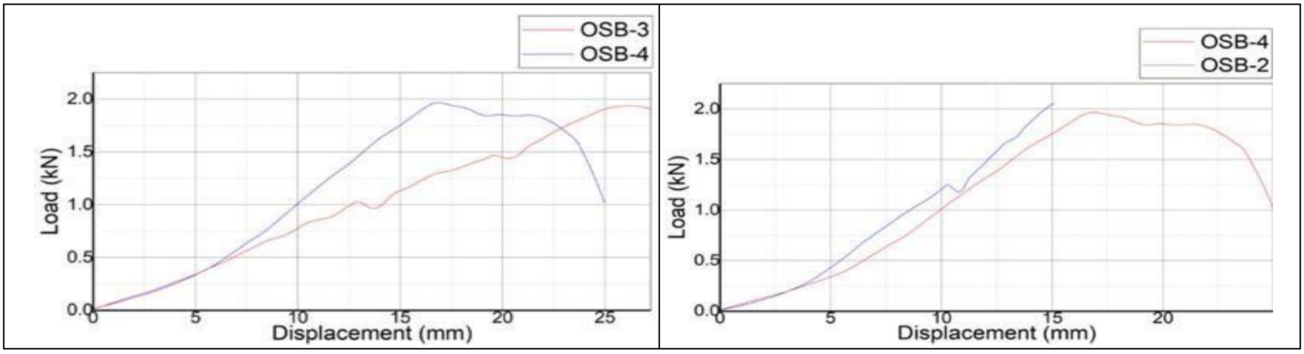
| **Panel type** | **Thickness of panel (mm)** | **Height of panel (mm)** | **Width of panel (mm)** | **Diameter of fastener (mm)** | **Thickness of frame (mm)** | **Height of frame (mm)** | **Width of frame (mm)** | **Comments** |
|---|---|---|---|---|---|---|---|---|
| OSB -1 | 9 | 305 | 127 | 2.5(ring nail) | 1.14 | 254 | 76 | OSB -1 can be compared with OSB -3 to see the effect of the panel thickness on strength of OSB sheathed connection. |
| OSB - 2 | 11 | 305 | 127 | 2.5(ring nail) | 1.14 | 254 | 76 | OSB -2 can be compared with OSB -1 to see the effect of the panel thickness on strength of OSB sheathed connection. |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| OSB - 3 | 15 | 305 | 127 | 3.8(ring nail) | 1.14 | 254 | 76 | OSB -3 can be compared with OSB -4 to see the effect of the panel thickness on strength of OSB sheathed connection. |
| OSB - 4 | 18 | 305 | 127 | 3.8(ring nail) | 1.14 | 254 | 76 | OSB -4 can be compared with OSB -2 to see the effect of the panel thickness on strength of OSB sheathed connection. |
| OSB - 5 | 9 | 305 | 127 | 2.5(ring nail) | 2.45 | 254 | 76 | OSB -5 can be compared with OSB -1 to see the effect of the thickness of frame on strength of OSB sheathed connection. |
| OSB - 6 | 11 | 305 | 127 | 2.5(ring nail) | 2.45 | 254 | 76 | OSB -6 can be compared with OSB -2 to see the effect of the thickness of frame on strength of OSB sheathed connection. |
| OSB - 7 | 15 | 305 | 127 | 3.8(ring nail) | 2.45 | 254 | 76 | OSB -7 can be compared with OSB -8 to see the effect of the thickness of frame on strength of OSB sheathed connection. |
| OSB - 8 | 18 | 305 | 127 | 3.8(ring nail) | 2.45 | 254 | 76 | OSB -8 can be compared with OSB - 7 to see the effect of the thickness of frame on strength of OSB sheathed connection. |
| Gypsum -1 | 12 | 305 | 127 | 3.8 (ring nail) | 1.2 | 254 | 76 | Gypsum -1 can be compared with Gypsum -2 to see the effect of the diameter of the fastener on strength of sheathed connection. |
| Gypsum -2 | 12 | 305 | 127 | 3.5 (screw nail) | 1.2 | 254 | 76 | Gypsum -2 can be compared with Gypsum -1 to see the effect of the diameter of the fastener on strength of sheathed connection. |
| | | | | | | | | |

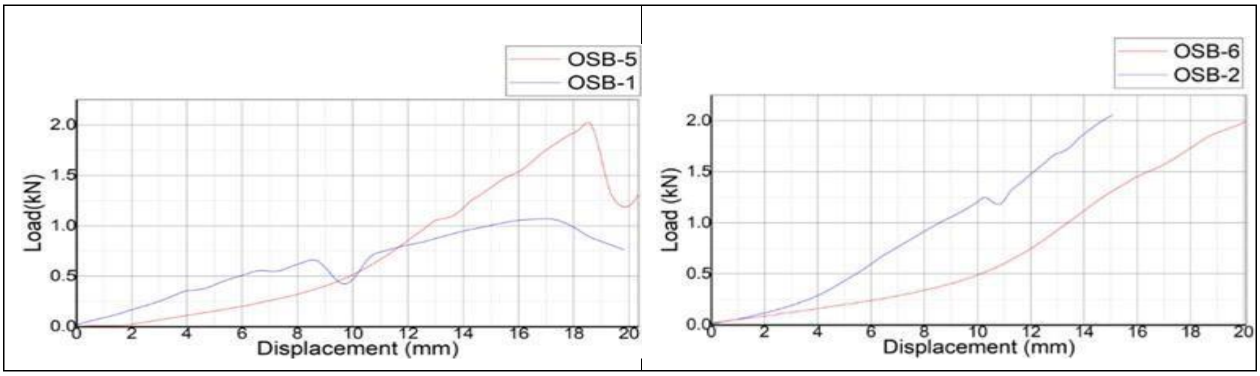
| Panel type | Thickness of panel (mm) | Height of panel (mm) | Width of panel (mm) | Diameter of fastener (mm) | Thickness of frame (mm) | Height of frame (mm) | Width of frame (mm) | Comments |
|---|---|---|---|---|---|---|---|---|
| Gypsum -3 | 12 | 305 | 127 | 4.5(screw nail) | 0.9 | 254 | 76 | Gypsum -3 can be compared with Gypsum -4 to see the effect of the diameter of the fastener on strength of sheathed connection. |
| Gypsum -4 | 12 | 305 | 127 | 4.8(screw nail) | 0.9 | 254 | 76 | Gypsum -4 can be compared with Gypsum - 5 to see the effect of the diameter of the fastener type on strength of sheathed connection. |
| Gypsum -5 | 12 | 305 | 127 | 5.1(ring nail) | 0.9 | 254 | 76 | Gypsum -5 can be compared with Gypsum - 4 to see the effect of the diameter of the fastener type on strength of sheathed connection. |



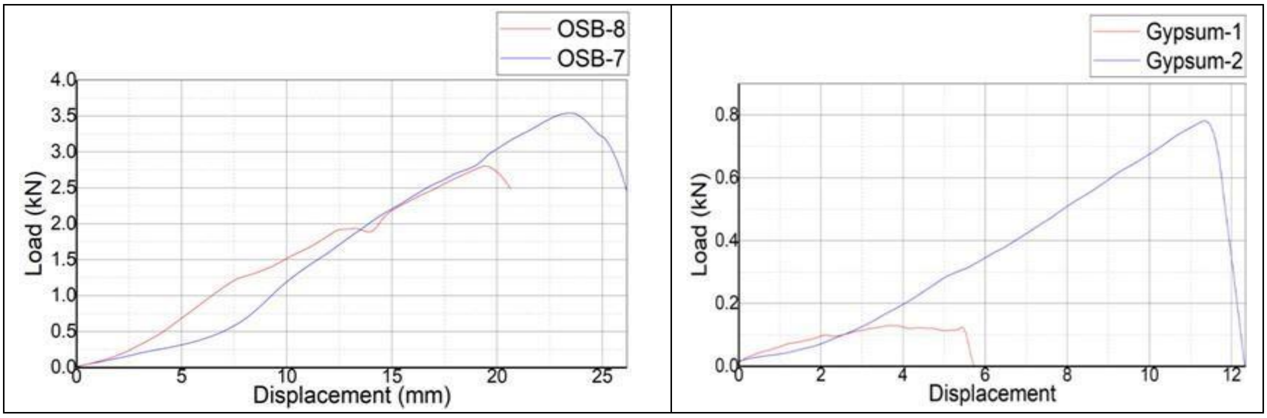
(a) OSB-3 vs. OSB-1 (b) OSB-2 vs. OSB-1



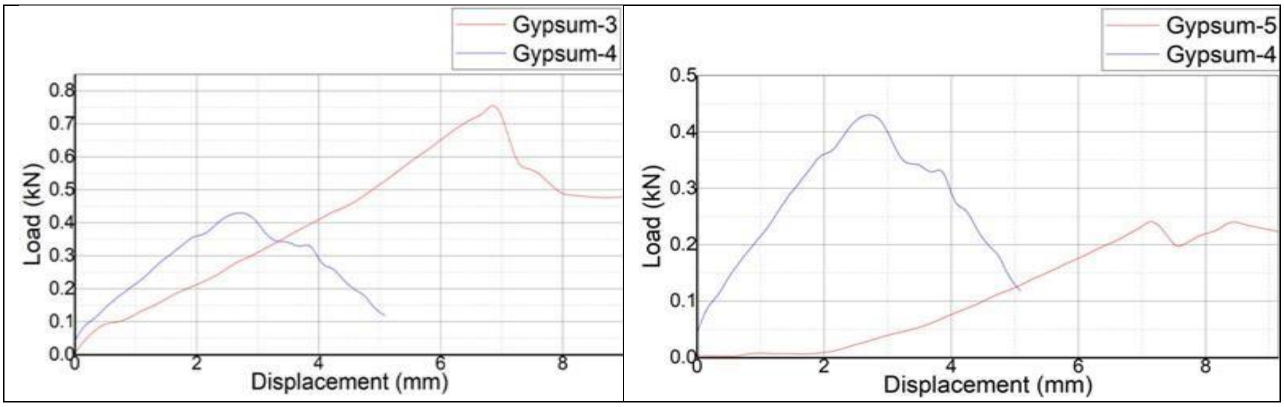
(c) OSB-3 vs. OSB-4 (d) OSB-4 vs. OSB-2



(e) OSB-5 vs. OSB-1 (f) OSB-6 vs. OSB-2



(g) OSB-7 vs. OSB-8 (h) Gyp-1 vs. Gyp-2



(i) Gyp-3 vs. Gyp-4 (j) Gyp-4 vs. Gyp-5

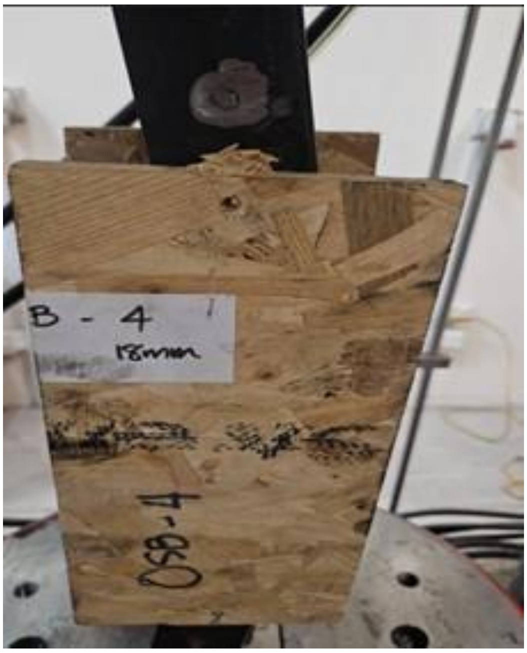
Table 5.2: Experimental results for specimens characterized for one panel layer

| **Label** | **$\sigma_y$ [MPa]** | **$\sigma_{max}$[MPa]** | **Yield Strength ($f_y$) (kN)** | **Ultimate Strength ($f_u$) (kN)** | **Stiffness (N/mm)** | **Ultimate displacement ($\Delta u$) (mm)** |
|---|---|---|---|---|---|---|
| OSB -1 | 1.9 | 3.49 | 0.93 | 3.49 | 164.16 | 21.26 |
| OSB _ 2 | 3.54 | 5.21 | 1.74 | 2.56 | 169.98 | 15.06 |
| OSB - 3 | 2.6 | 5.21 | 1.28 | 2.56 | 90.748 | 28.21 |

| | | | | | | |
|---|---|---|---|---|---|---|
| OSB - 4 | 1.89 | 5.36 | 0.93 | 2.63 | 105.28 | 24.98 |
| OSB - 5 | 1.78 | 5.1 | 0.88 | 2.5 | 122.85 | 20.35 |
| OSB - 6 | *1.73* | 5.26 | 0.85 | 2.58 | 128.42 | 20.09 |
| OSB - 7 | 3.64 | 8.47 | 1.79 | 4.16 | 158.89 | 26.18 |
| OSB - 8 | 5.43 | 7.27 | 2.67 | 3.57 | 173.04 | 20.63 |
| Gypsum -1 | 1.59 | 1.86 | 0.78 | 0.92 | 137.31 | 6.7 |
| Gypsum -2 | 1.16 | 2.74 | 0.57 | 1.34 | 108.58 | 12.34 |
| Gypsum -3 | 1.74 | 2.84 | 0.85 | 1.39 | 154.27 | 9.01 |
| Gypsum-4 | 1.55 | 2.25 | 0.76 | 1.1 | 171.6 | 6.41 |
| Gypsum-5 | 1.21 | 1.72 | 0.6 | 0.84 | 91.8 | 9.15 |

**(a) OSB-1 pull-through failure (PT)** **(b) OSB-2 bearing failure (B)**

(c) OSB-3 back-edge tearing combined with bearing(B+E)

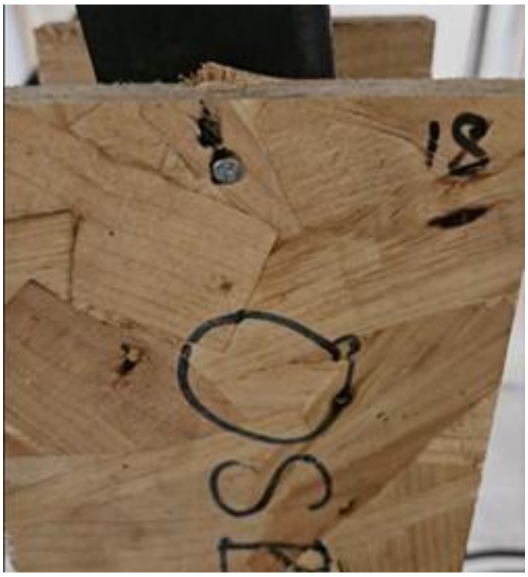


(d) OSB-4 Pull-through with edge tearing(PT+E)

(e) OSB-5 bearing failure/edge tearing (B+E)

(f) OSB-6 tilting & bearing (T+B)

(g) OSB-7 bearing failure/ edge tearing (B+E)



(h) OSB-8 Pull-through (PT)

(i) GYP-1 bearing failure/ edge-tearing (B+E)

(j) GYP-2 bearing failure/ edge-tearing (B+E)

(k) GYP-3 bearing failure/ edge-tearing (B+E)

(l) GYP-4 bearing failure/ edge-tearing (B+E)

(m) GYP-5 bearing failure/ edge-tearing (B+E)

For all tested connections the failure mechanism was affected by the edge loading distance, which was equal to 15 mm. In fact, for this value of the edge loading distance, the most common observed failure mechanism for all tests was the breaking of the panel edge. In particular, for connections with standard gypsum board, the panel portion affected by the rupture can be obtained by considering a diffusion of about 90° starting from the screw.

Whereas for the solutions with Orient Strand board, the rupture involved a lower panel portion.

**Effect of the profile thickness**

- OSB -3 is compared with OSB -1 to see the effect of the panel thickness on strength of OSB sheathed connection.

  The variation in strength between OSB-3 (15mm panel thickness) 1.922 kN and OSB-1 (9mm panel thickness) 1.06 kN is significant, with OSB-3 having an ultimate strength that is 81.32% greater than OSB-1. This substantial increase is due to the increased panel thickness, which enhanced the load-bearing capacity, and overall resistance to failure.

- OSB -2 compared with OSB -1 to see the effect of the panel thickness on strength of OSB sheathed connection.

  The ultimate strength of OSB-2 (11mm panel thickness) is 92.45% higher than OSB-1 (9mm panel thickness) due to increased material thickness, which results in improved load distribution, higher flexural strength, enhanced shear and tensile strength. The increase in thickness, resulting in a much higher ultimate strength.

- OSB -3 can be compared with OSB -4 to see the effect of the panel thickness on strength of OSB sheathed connection

  The ultimate strength 1.96kN of OSB-4 (18mm panel thickness) is 1.98% higher than ultimate strength 1.922 kN OSB-3 (15mm panel thickness). The slight increase in strength demonstrates that while thicker panels generally improve structural performance, the rate of improvement decreases with higher thicknesses. The 3mm increase in thickness enhances load-bearing capacity, flexural strength, shear and tensile strength, but the relative improvement is less pronounced compared to initial increases in thickness. • OSB -4 can be compared with OSB -2 to see the effect of the panel thickness on strength of OSB sheathed connection.

  The ultimate strength 1.96kN of OSB-4 (18mm panel thickness) is 3.92% lower than OSB-2 (11mm panel thickness) with a strength of 2.04kN. This counterintuitive result indicates that factors other than panel thickness, such as fastener performance, and load distribution efficiency, play significant roles in determining the strength of sheathed connections. Thinner panels may exhibit better performance in specific applications due to optimal material properties, better stress distribution, and superior fastener holding power. •

OSB -5 compared with OSB -1 to see the effect of the thickness of frame on strength of OSB sheathed connection. OSB-5 has a 2.45 mm frame thickness whereas OSB-1 has a 1.14mm frame thickness

The increase in frame thickness from OSB-1 (1.14mm) to OSB-5 (2.45mm) results in an 87.74% increase in ultimate strength. The thicker frame provides greater stiffness, improved load distribution, enhanced connection strength, reduced shear stresses. This increase seems to be associated with a change in failure mode from pull-through failure to bearing failure.

- OSB -6 can be compared with OSB -2 to see the effect of the thickness of frame on strength of OSB sheathed connection. OSB-6 has a 2.45 mm frame thickness whereas OSB-2 has a 1.14mm frame thickness

  Connection shear strength increases with increases in framing thickness. This increase seems to be associated with a change in failure mode from tilting & bearing combined failure to only bearing failure.

- OSB -7 can be compared with OSB -8 to see the effect of the thickness of panel on strength of OSB sheathed connection.

  So, the variation in strength between OSB-7 (15mm panel thickness) 3.55kN and OSB-8 (18mm panel thickness) 2.78kN, considering the thickness of the panel, is approximately 27.7%

**Effect of the screw diameter: -**

- Gypsum -1 with a 3.8mm ring shank nail compared with Gypsum -2 to see the effect of the fastener on strength of sheathed connection. Adopting a 3.5mm angular threaded screw nails for Gyp-2, higher strength values were recorded, about 1.34 kN.

  This is primarily due to the load distribution provided by the threaded design of the screws, despite their slightly smaller diameter. Smaller-diameter screw nails are more susceptible to edge tearing. Which was the common failure mode observed.

- Gyp-3 diameter 4.5 angular screw nail. Gyp-4 diameter 4.8 angular screw nail adopting a 4.5mm angular threaded screw nails for Gyp-3, higher strength values were recorded, about 1.39 kN.

  The insertion process of a larger diameter screw (4.8mm) can cause more disruption to the gypsum material, potentially leading to weakening around the screw hole. The higher strength values recorded for Gyp-3 (4.5mm angular threaded screw nails) compared to Gyp-4 (4.8mm angular threaded screw nails) can be attributed to a combination of better thread engagement, more effective stress distribution, material compatibility, and reduced disruption of the gypsum during insertion.

- Gypsum -4 can be compared with Gypsum -5 to see the effect of the fastener type on strength of sheathed connection. Adopting a 4.8mm angular threaded screw nails for Gyp-4, rather than ring shank nails show higher strength values were recorded, about 1.1 kN.

  Threaded screws generally have higher withdrawal resistance compared to ring shank nails. This is because the threads bite into the material more effectively, providing greater resistance to forces that try to pull the fastener out. The 4.8mm angular threaded screw nails provide a better load distribution, and higher withdrawal resistance compared to ring shank nails.

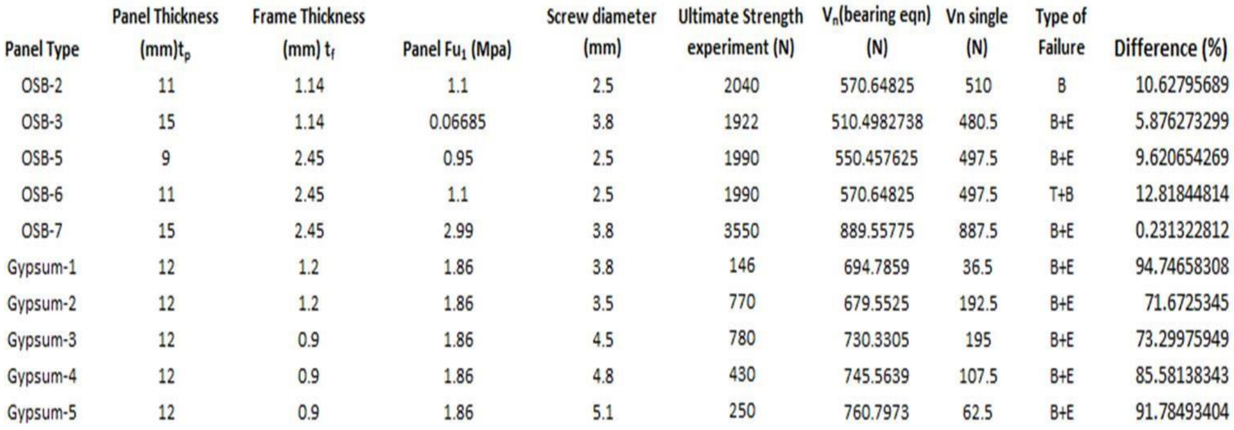
**Validation of Design Criteria: -**

Explanation below of how the proposed design equation is validated.

Bearing

$$Vn= 501.8295+ (2.275t_pFu_1d)$$

The bearing equation shows validity for OSB panel. For Gypsum the regression model was unfavorable. The data set is limited which is why the regression model doesn’t fit the design equation.  This needs to be further explored as mentioned in the future recommendation section.  The ultimate strength is the peak strength obtained from the UTM. The Vn bearing value is using the designed equation of the single screw connection. The Vn single is the ultimate strength value divided by 4 (as there were 4 fasteners in the screw connection). This gives the value of the single screw connection. The type of failure is observed through the experimental assembly through the UTM. The difference showcases the validation of the design values between Vn bearing equation and Vn single (experimental) of the single screw connection proving it to be optimal for OSB.

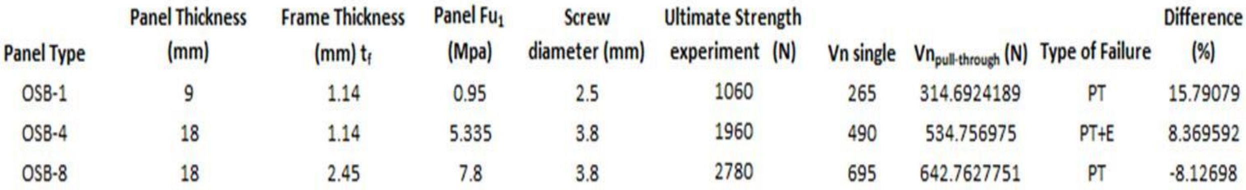


| Panel Type | Panel Thickness (mm)$t_p$ | Frame Thickness (mm) $t_f$ | Panel $Fu_1$ (Mpa) | Screw diameter (mm) | Ultimate Strength experiment (N) | $V_n$(bearing eqn) (N) | Vn single (N) | Type of Failure | Difference (%) |
|---|---|---|---|---|---|---|---|---|---|
| OSB-2 | 11 | 1.14 | 1.1 | 2.5 | 2040 | 570.64825 | 510 | B | 10.62795689 |
| OSB-3 | 15 | 1.14 | 0.06685 | 3.8 | 1922 | 510.4982738 | 480.5 | B+E | 5.876273299 |
| OSB-5 | 9 | 2.45 | 0.95 | 2.5 | 1990 | 550.457625 | 497.5 | B+E | 9.620654269 |
| OSB-6 | 11 | 2.45 | 1.1 | 2.5 | 1990 | 570.64825 | 497.5 | T+B | 12.81844814 |
| OSB-7 | 15 | 2.45 | 2.99 | 3.8 | 3550 | 889.55775 | 887.5 | B+E | 0.231322812 |
| Gypsum-1 | 12 | 1.2 | 1.86 | 3.8 | 146 | 694.7859 | 36.5 | B+E | 94.74658308 |
| Gypsum-2 | 12 | 1.2 | 1.86 | 3.5 | 770 | 679.5525 | 192.5 | B+E | 71.6725345 |
| Gypsum-3 | 12 | 0.9 | 1.86 | 4.5 | 780 | 730.3305 | 195 | B+E | 73.29975949 |
| Gypsum-4 | 12 | 0.9 | 1.86 | 4.8 | 430 | 745.5639 | 107.5 | B+E | 85.58138343 |
| Gypsum-5 | 12 | 0.9 | 1.86 | 5.1 | 250 | 760.7973 | 62.5 | B+E | 91.78493404 |

Pull-through

$$Vn= 877.2687+ (0.640581t_pFu_1d)$$

The Pull-through equation shows validity for OSB panel.



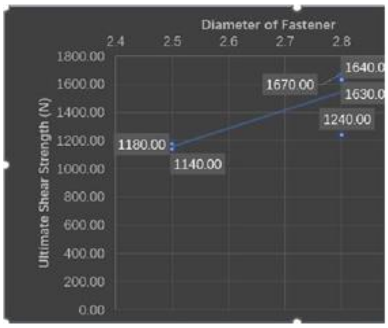
| Panel Type | Panel Thickness (mm) | Frame Thickness (mm) $t_f$ | Panel $Fu_1$ (Mpa) | Screw diameter (mm) | Ultimate Strength experiment (N) | Vn single | $Vn_{pull\text{-}through}$ (N) | Type of Failure | Difference (%) |
|---|---|---|---|---|---|---|---|---|---|
| OSB-1 | 9 | 1.14 | 0.95 | 2.5 | 1060 | 265 | 314.6924189 | PT | 15.79079 |
| OSB-4 | 18 | 1.14 | 5.335 | 3.8 | 1960 | 490 | 534.756975 | PT+E | 8.369592 |
| OSB-8 | 18 | 2.45 | 7.8 | 3.8 | 2780 | 695 | 642.7627751 | PT | -8.12698 |

Pull-out

$$Vn= 1409.894+ (1.106654t_fFu_2d_h)$$

No results were obtained with the pull-out failure. In this experimental program the connected member did not pull out of the connection due to insufficient bearing strength.

## 5.2 Detailed Analysis for Orient Strand Board

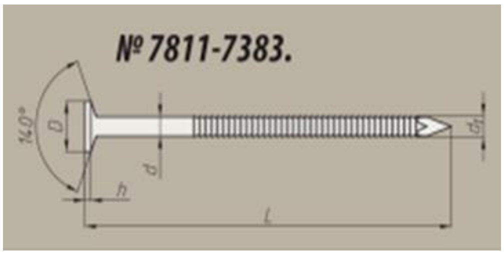


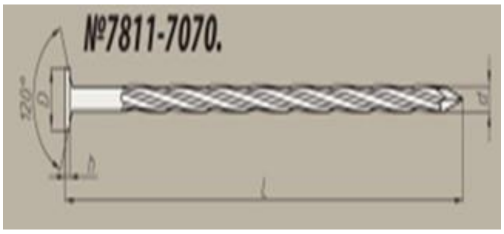
| Constants for this plot | |
|---|---|
| Thickness panel | 15 mm |
| $F_u$ Steel | 600 MPa |
| Thickness frame | 1.6 mm |

Figure 5.3: Representation of affect of diameter on strength of connection

### *5.2.1 Discussion*

- Plots shown here in which only diameter of fastener variable is changing against strength.
- Ring Nails: provides added grip and holding power when driven into wood, making them less likely to pull out or loosen over time. Well-suited for load-bearing and structural applications. Ring nail shows a 1.5 times better resistance compared to the screw nail.
- Screw nail: features threads along its shaft, providing additional grip and holding power. Designed to be driven into materials with a twisting or screwing motion. They are commonly used in applications where a strong and secure connection is needed.





Reference: https://www.indiamart.com/proddetail/ring-shank-wire-nails-2849563233455.html

Reference: https://marshallindustrial.co.uk/drive-screw-nails-galvanised-2-5kg-pack/

### *5.2.2 Interpretation*

- As the diameter of the fastener increases, the ultimate shear strength of the connection also tends to increase.
- The spacing between fasteners (50-100mm) is crucial for the overall performance of the connection.

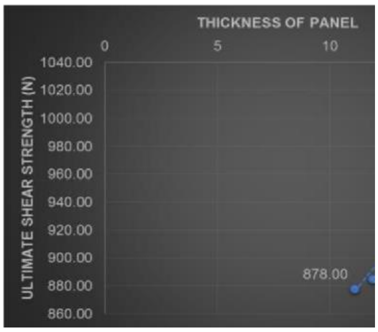
- Screws (2.5mm) engage with the material through threading, which provides more resistance against pull-out forces. Smaller-diameter screw nails might be more susceptible to edge tearing.

### *5.2.3 Possible Explanation*

- A larger diameter fastener provides more contact area between the screw and the materials being connected (such as the panel and the frame). This increased contact area distributes the shear load over a larger surface, which can lead to higher shear strength, reduce stress concentrations at the edges, and lower the risk of crushing the connected materials.
- The ring nail (2.8mm) shows a 1.5 times better resistance compared to the screw nail (2.5mm).
- By changing diameter of screw the strength is not changing drastically. However incase, of nails the change in strength is due to the diameter.

### *5.2.4 Discussion*

- Plots shown here in which only thickness of panel variable is changing against strength.



| Constants for this plot | |
|---|---|
| Diameter | 2.5mm |
| $F_u$ Steel | 310MPa |
| Thickness frame | 0.84mm |

Figure 5.5: Representation of affect of thickness of panel on strength of connection

### *5.2.5 Effect of OSB Panel Thickness on Shear Strength*

- As the thickness of the OSB panel increases from 11.1mm to 18.3mm, there is a general trend of an increase in ultimate shear strength on a single screw connec- tion. This is evident from the data points (878.00 N to 1014.00 N). Suggesting that thicker OSB panels result in stronger screw connections, resulting in a greater bearing area and, consequently, higher resistance to shear forces.

- If panels are increased 1.2 x the thickness the strength will also increase 1.2 x which distributes the load more effectively reducing the risk of bearing and pull- through failures.

### *5.2.6 OSB Panel Ratings*

- The information about the OSB panel ratings (e.g., 24/16, 32/16, 40/20, 48/24) adds further context to the data the OSB panel span rating consists of two num- bers, with the first number representing the maximum spacing between support- ing members and the second number indicating the maximum allowable span for roof sheathing. These ratings are critical for ensuring the panels' shear strength and load-bearing capacity.

### *5.2.7 Effect on Panel Shear Strength*

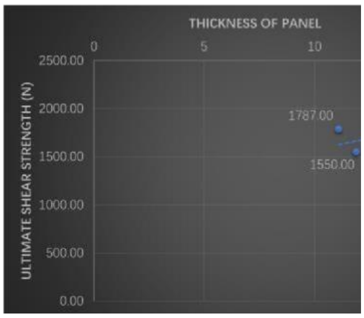
- A lower first number (e.g., 24) means the panels are intended for use where the supporting members are placed closer together. This results in a stiffer and stronger floor or wall assembly.

- A higher second number (e.g., 24) for roof sheathing indicates that the panels are designed to handle less load and allow for wider spacing of rafters or trusses.

### *5.2.8 Failure Modes*

- The increase in ultimate shear strength with thicker OSB panels is an indication that the connection is becoming more resistant to tilting, bearing, and pull- through failures,

- In conclusion, the data analysis demonstrates a positive correlation between the thickness of the OSB panel and the ultimate shear strength of a single screw connection in a CFS shear wall. The information on OSB panel ratings further supports this trend.



| Constants for this plot | |
|---|---|
| Diameter | 2.5 mm |
| $F_u$ Steel | 448 MPa |
| Thickness frame | 1.37 mm |

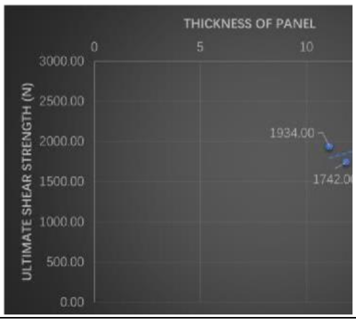
Figure 5.6: Representation of affect of thickness of panel on strength of connection

### *5.2.9 Explanation*

- The increase in OSB panel thickness provides a larger volume of material for the screw to engage with, resulting in a greater bearing area and, consequently, higher resistance to shear forces.
- Thicker OSB panels distribute the load more effectively across a larger area, re- ducing the risk of bearing and pull-through failures. Additionally, the increased thickness helps prevent tilting by providing more material for the screw to grip.

- In conclusion, the data analysis demonstrates a positive correlation between the thickness of the OSB panel and the ultimate shear strength of a single screw connection in a CFS shear wall. The information on OSB panel ratings further supports this trend.

- The steel of the frame is the governing failure as it could be the weak part since due to changes in the thickness of the panel there’s not much difference in the shear strength.



| Constants for this plot | |
|---|---|
| Diameter | 2.5 mm |
| $F_u$ Steel | 345 MPa |
| Thickness frame | 2.45 mm |

Figure 5.7: Representation of affect of thickness of panel on strength of connection

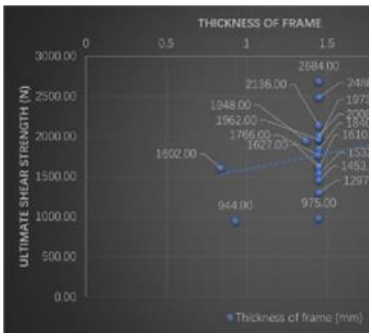
### *5.2.10 Explanation*

- The increase in OSB panel thickness provides a larger volume of material for the screw to engage with, resulting in a greater bearing area and, consequently, higher resistance to shear forces.

- Thicker OSB panels distribute the load more effectively across a larger area, re- ducing the risk of bearing and pull-through failures. Additionally, the increased thickness helps prevent tilting by providing more material for the screw to grip.
- In conclusion, the data analysis demonstrates a positive correlation between the thickness of the OSB panel and the ultimate shear strength of a single screw connection in a CFS shear wall. Thicker panels provide more material for the screw to engage with, resulting in increased strength and improved resistance to various failure modes. The information on OSB panel rating further supports this trend.

- The steel of the frame is the governing failure as there’s not much change in the connection strength.

**As calculated via Ratio the formulas are underestimated in error by 48% in OSB sheathing.**



| Constants for this plot | |
|---|---|
| Diameter | 4.2 mm |
| $F_u$ Steel | 448 MPa |
| Thickness of panel | 11.1 mm |

Figure 5.8: Representation of affect of thickness of frame on strength of connection

- Plots shown here in which only thickness of frame variable is changing against strength.

*5.2.11 Effect of Steel Stud Thickness on Shear Strength*

- There is a noticeable relationship between steel stud thickness and shear strength, which appears to be influenced by several factors.

*5.2.12 Explanation*

- The ultimate shear strength depends on multiple variables, including steel stud thickness. Thicker steel studs generally provide increased shear strength due to higher material volume and tensile strength. This aligns with the overall trend of increasing shear strength as steel thickness increases.

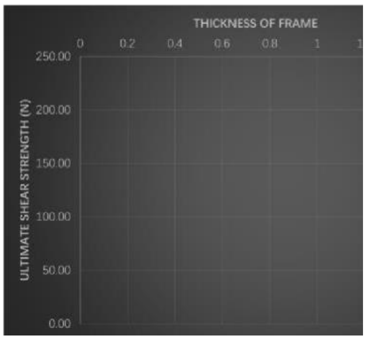
*5.2.13 Screw Type and Installation*

- The choice of Simpson Strong Tie QuikDrive 8 screws and their installation pa- rameters, such as edge distances and spacing, significantly impacts shear strength. Proper installation is crucial to avoid common failure modes like tilting, bearing, and pull-through. The spacing of screws along the perimeter and field studs en- sures proper load distribution and attachment strength.

- In conclusion, the analysis reveals a clear relationship between steel stud thick- ness and the ultimate shear strength of single screw connections in CFS shear walls. Thicker steel studs generally lead to increased shear strength.

### *5.2.14 Interplay with Steel Stud Thickness*

The combination of fastener specifications and spacing is likely optimized for the range of steel thicknesses tested (1.45mm to 0.93mm).



| Constants for this plot | |
|---|---|
| Diameter | 4.17 mm |
| $F_u$ Steel | 448 MPa |
| Thickness of panel | 11.1 mm |

Figure 5.9: Representation of affect of thickness of frame on strength of connection

- Plots shown here in which only thickness of frame variable is changing against strength.

### *5.2.15 Effect of Steel Stud Thickness*

- The dataset provides shear strength data for a fixed steel stud thickness of 1.81 mm showing variations in ultimate shear strength under different conditions, which can be attributed to the effects of thickness of frame, moisture & instal- lation practices.

### *5.2.16 Environmental Conditions*

- The use of three environmental conditions (normal, over-driven, humid) indicates a thorough investigation into the impact of moisture and installation quality on connection strength. Sheathing materials like OSB can be sensitive to moisture, and their properties may change under different environmental conditions. This can affect the performance of the connections.

### *5.2.17 Stud Type*

- The choice of a specific steel stud type (362S162-68) is consistent throughout the testing. This ensures that the variations in shear strength are primarily attributed to the thickness of frame being studied, such as installation practices.

### *5.2.18 Fasteners*

- The use of Number 8 fasteners with specified dimensions ensures consistency in fastening methods across the tests. This allows for a direct comparison of the effects of other variables, such as moisture and installation quality.

- In conclusion, this dataset provides valuable insights into the behavior of sheath- ing connections under different conditions while keeping the steel stud thickness constant. The variations in shear strength observed can be attributed to the en- vironmental conditions and installation practices of the steel frame. This analy- sis contributes to a better understanding of the factors that influence connection strength in CFS shear walls and underscores the importance of considering these variables in design and construction practices.

## 5.3 Gypsum

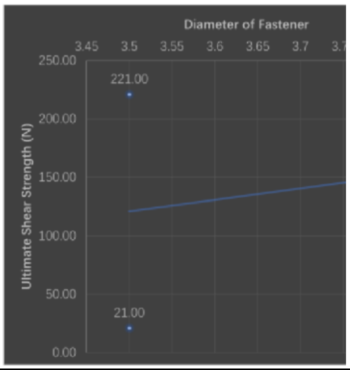
Table 5.3: Gypsum dataset

| **Author** | **Type of panel** | **Edge distance (mm)** | **Thickness of frame** | **Thickness of Panel** | **Shear Strength (kN)** | **Type of fastener** | **Diameter of fastener (mm)** | **Length of fastener (mm)** | **Ultimate Strength for single connection (N)** |
|---|---|---|---|---|---|---|---|---|---|
| K.D. | | 38 | 0.84 | 12.7 | 1.92 | SDS | 3.5 | 3.51 | 0.43 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Peterman | | | | | | | | | |
| Ashok Jammi | | 20 | 0.84 | 12.5 | 0.966 | SDS | 4.2 | 13 | 0.219 |
| Luiz Vieira, Schafer | | 152.4 | 1.3 | 11.1 | 7<br>1.5 | Simpson #6 | | 83 | 0.249 |
| B.W. Schafer | | 20 | | 11.9 | 0.966 | SDS | 4.2 | 4.17 | 0.43 |
| Luigi Fiorino | GWB - S- 35-6 | 15 | 0.6 | 12.5 | 0.22 | SDS | 3.5 | 3.51 | 0.27 |
| Luigi Fiorino | RGWB-S- 39-6 | 15 | 0.8 | 12.5 | 0.33 | | 3.9 | | 0.42 |
| Luigi Fiorino | GWB-D-35-6 | 15 | | 12.5 | | | 3.5 | 3.51 | 0.47 |
| Ye et al. | GWB | 15 | 0.9 | 18 | 1.95 | SDS | 4.2 | 4.16 | 0.56 |
| Jörg Lange | GFB | 20 | 1.81 | 12 | 1.97 | SDS | 4.2 | 4.16 | 1.541 |
| Luigi Fiorino | GFB- S-39-6 | | 1.5 | 12.5 | 0.1 | SDS | 3.9 | | 0.71 |
| Luigi Fiorino | GFB-D-39-6 | | 1.5 | 12.5 | 0.1 | SDS | 3.5 | 3.51 | 0.89 |
| Selvaraj, Madhavan | 12.5 | 2.5 | 4.2 | 270 | 3.26 | 1.65 | | | |
| Sartori, Tomasi | | | 1.5 | 12.5 | 1.45 | Staples | 1.6 | 55 | 1.38 |

| Author | Type of loading | Layers of Panels | Loading protocol | Loading rate (mm/sec) | Height of flange/ Web (whichever nail is connected to) | Type of failure in screw | $f_u$ (Mpa) | $f_y$ (MPa) |
|---|---|---|---|---|---|---|---|---|
| K.D. Peterman | M/C | S | CUREE | Load rate was constant throughout the test at one full cycle every 16 s. | Flange | B | 448 | 345 |
| Ashok Jammi | M/C | S | ASTM E2126 19 | 0.05 | Web | PO | 407 | 350 |
| Luiz Vieira, Schafer | M | D | | | Flange | | 448 | 345 |
| B.W. Schafer | M/C | S | | | | B | 448 | 345 |
| Luigi Fiorino | M | S | | 1 | Flange | T&BE | 270 | 140 |
| Luigi Fiorino | M | S | | 1 | Flange | T&BE | 270 | 140 |
| Luigi Fiorino | M | D | | 1 | Flange | T&BE | 270 | 140 |
| Ye et al. | M/C | S | | 0.03 | Flange | T&BE | | 345 |
| Jörg Lange | M | D | | | Flange | E | | 320 |
| Luigi Fiorino | M | S | | 1 | | T &BE | 270 | 140 |
| Luigi Fiorino | M | D | | 1 | | T &BE | 270 | 140 |
| Selvaraj, Madhavan | | | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Sartori, Tomasi | M/C | D | CUREE | | | E,T,B | 617 | |



| Constants for this plot | |
|---|---|
| Thickness of frame | 0.6 mm |
| $F_u$ Steel | 270 MPa |
| Thickness of panel | 12.5mm |

Figure 5.10: Representation of affect of diameter of fastener on strength of connection

- Plots shown here in which only diameter of fastener variable is changing against strength.

### 5.3.1 *Effect of Fastener Diameter*

- The dataset provides insight into how the diameter of the fastener influences the ultimate shear strength of single screw connections.

- Variations in ultimate shear strength are observed with changes in fastener diam- eter. The higher ultimate shear strength is shown in 3.5mm fastener due to the double Gypsum fibre sheathing panel used.

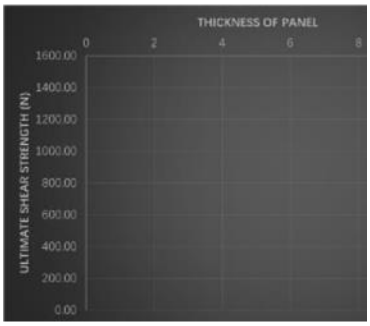
### 5.3.2 *Panel Specifications*

- The study involved different types of panels (gypsum plasterboard, gypsum-fibre board, impact-resistant special gypsum board, and cement-based board).

### 5.3.3 *Interaction with Steel Profiles*

- These panels were fastened to DX51D+Z steel grade CFS (cold-formed steel) profiles with self-tapping or self-drilling screws which also play a role in deter- mining the overall shear strength.

- The CFS profiles’ dimensions and material properties may affect the loadcarrying capacity of the connection.



| Constants for this plot | |
|---|---|
| Thickness of frame | 0.9 mm |
| $F_u$ Steel | 448 MPa |
| Diameter | 4.8 mm |

Figure 5.11: Representation of affect of thickness of panel on strength of connection

- Plots shown here in which only thickness of panel variable is changing against strength.

- Thicker studs could improve the shear capacity of the GWB-to-stud connections.

- An increase in screw diameter is noted to result in a 9.8 % decline in shear capac- ities for specimens with GWB sheathings.

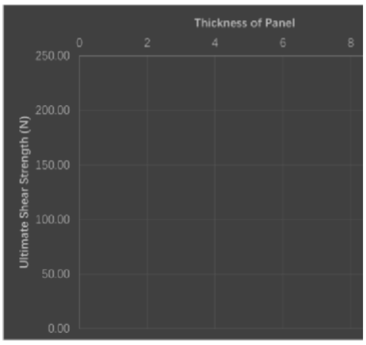
- Sheathing orientation is mentioned, highlighting a minor effect on shear capaci- ties for GWB sheathings with large edge distances.

- Self-drilling bugle head screws with a diameter of 4.8 mm are used for fastening.

  The screw type and diameter affect the connection’s performance.

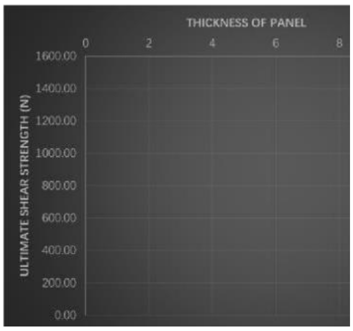
- Plots shown here in which only thickness of panel variable is changing against strength.

- Plots shown here in which only thickness of panel variable is changing against strength.



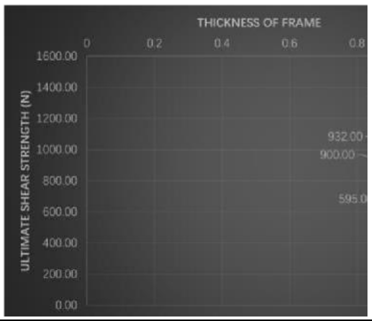
| Constants for this plot | |
|---|---|
| Thickness of frame | 0.6 mm |
| $F_u$ Steel | 270 MPa |
| Diameter | 3.5 mm |

Figure 5.12: Representation of affect of thickness of panel on strength of connection



| Constants for this plot | |
|---|---|
| Thickness of frame | 1.2 mm |
| $F_u$ Steel | 448 MPa |
| Diameter | 4.8 mm |

Figure 5.13: Representation of affect of thickness of panel on strength of connection



| Constants for this plot | |
|---|---|
| Thickness of panel | 12 mm |
| $F_u$ Steel | 448 MPa |
| Diameter | 4.8 mm |

Figure 5.14: Representation of affect of thickness of frame on strength of connection

- Plots shown here in which only thickness of frame variable is changing against strength.

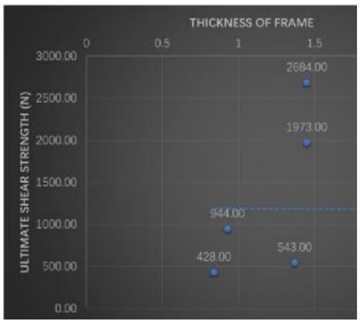
- The effect of edge distance on the shear capacity of the connection, indicating that edge distance has a more significant impact than screw diameter or stud thickness.
- As the edge distance increased, shear capacity improved under tension, but the improvement under compression was not significant.

### *5.3.4 Effect of Frame Thickness on Strength*

- The graph illustrates variations in the ultimate shear strength (y-axis) while chang- ing the thickness of the frame (x-axis) from 0.9 mm to 1.2 mm.

- Based on the data, there is a trend suggesting that as the frame thickness increases, the ultimate shear strength of the connection also tends to increase.

- The edge distance is a critical factor influencing shear capacity, but it doesn’t negate the influence of frame thickness as shown in the graph.



| Constants for this plot | |
|---|---|
| Thickness of panel | 12.7 mm |
| $F_u$ Steel | 448 MPa |
| Diameter | 3.5 mm |

Figure 5.15: Representation of affect of thickness of frame on strength of connection

- Plots shown here in which only thickness of frame variable is changing against strength.

## 5.4 Plywood

Table 5.4: Plywood dataset

| **Author** | **Type of Panel** | **Edge distance (mm)** | **Thickness of frame** | **Thickness of Panel** | **Shear Strength (kN)** | **Type of fastener** | **Diameter of fastener** | **Length of fastener** | **Ultimate Strength for single connection** |
|---|---|---|---|---|---|---|---|---|---|
| Jörg Lange | | 20 | 1.5 | 13 | 1.97 | SDS | 4.2 | 4.16 | 1.504 |
| Selvaraj, Madhavan | | 2.5 | 6 | 1.32 | | | | 0.66 | |

| **Author** | **Type of loading** | **Layers of Panels** | **Loading protocol** | **Loading rate (mm/sec)** | **Height of flange/ Web (whichever nail is connected to)** | **Type of failure in screw** | $f_u$ **(MPa)** | $f_y$ **(MPa)** |
|---|---|---|---|---|---|---|---|---|
| Jörg Lange | M | S | | | Flange | LB | | 320 |
| Selvaraj, Madhavan | | | | | | PT,<br>BREAKAGE | | |

Selvaraj, Madhavan literature is a Pull-through literature. This research will disregard the paper.

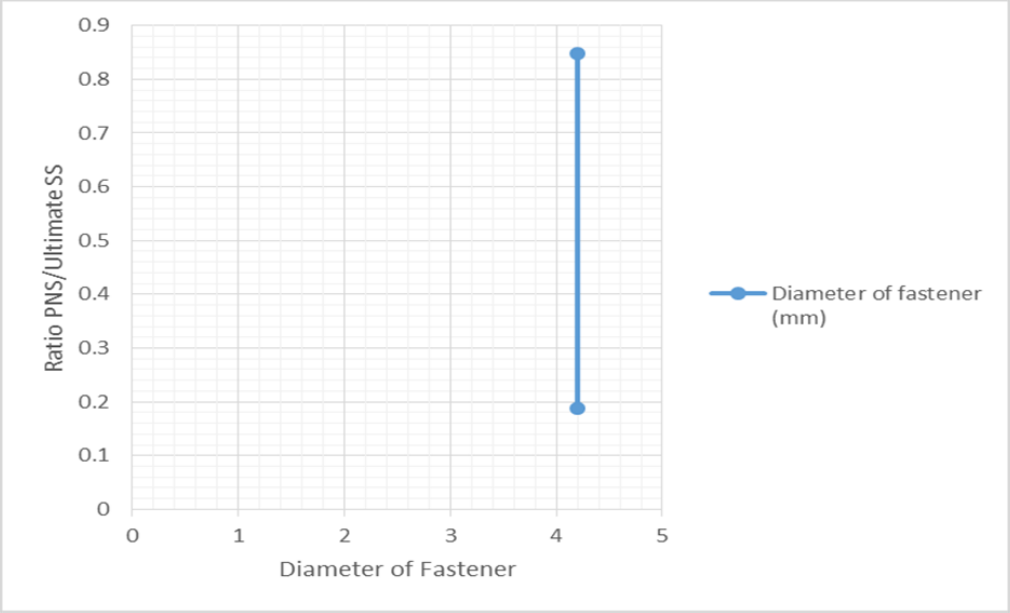


Figure 5.16: Ratio vs. Dia. of fastener

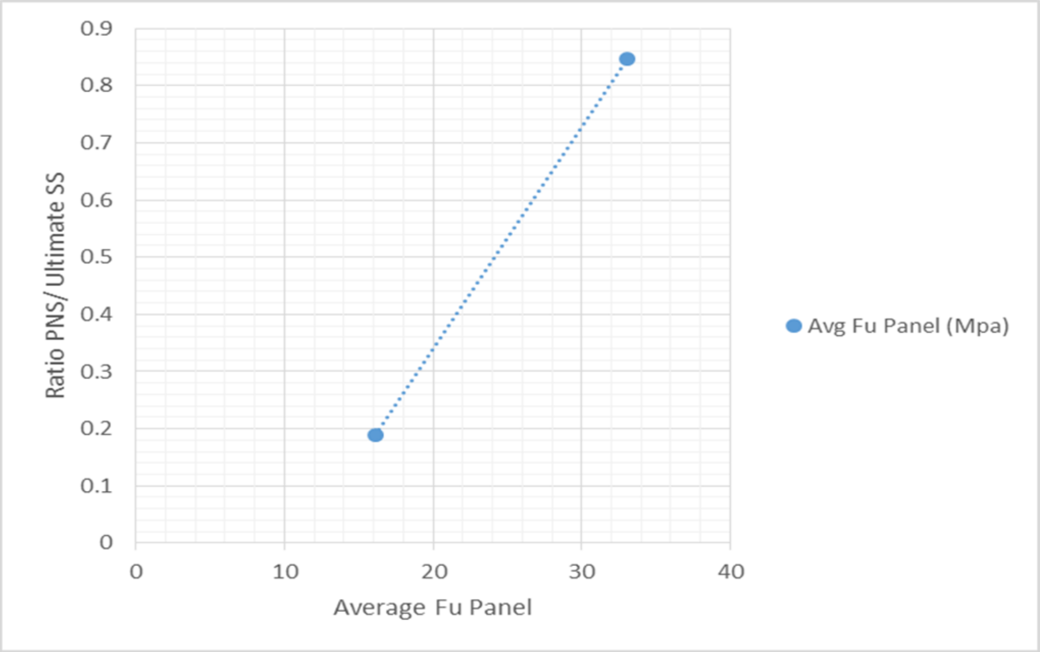


Figure 5.17: Ratio vs. Avg. $F_u$ panel

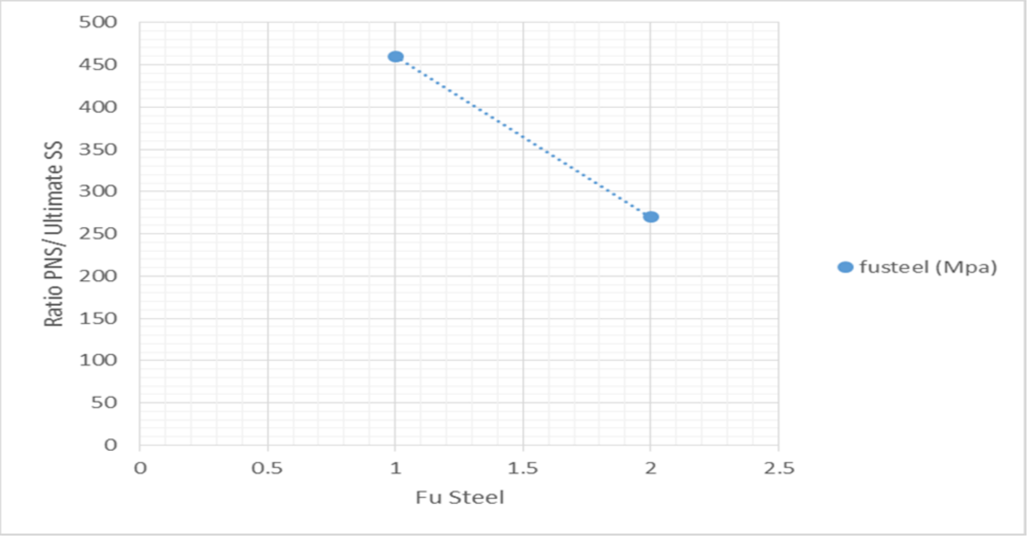


Figure 5.18: Ratio vs. $F_u$ Steel

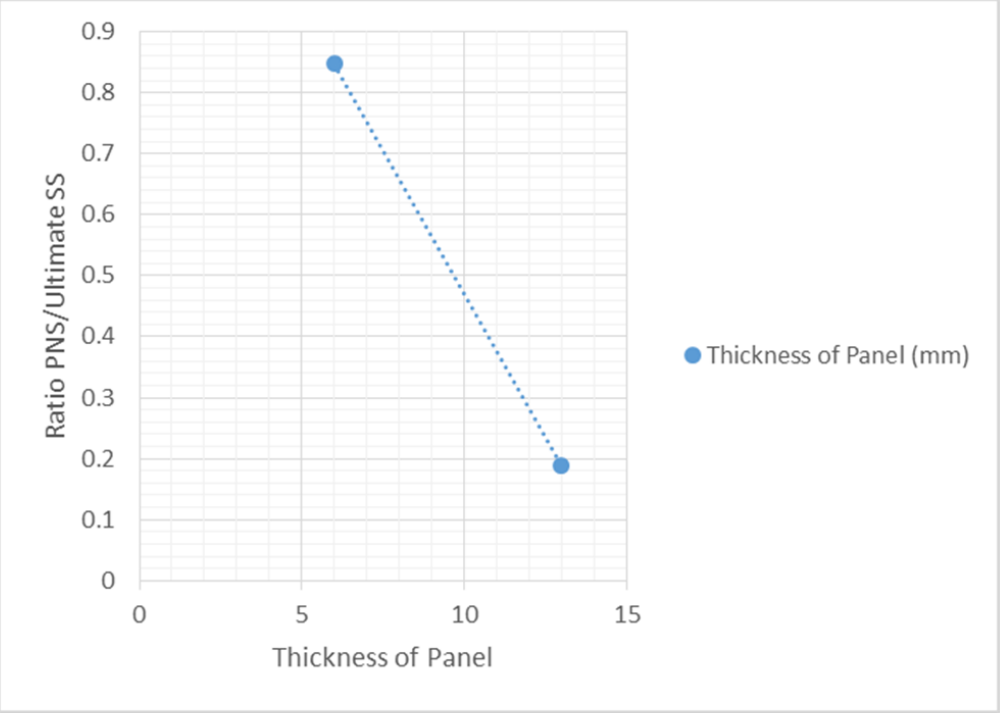


Figure 5.19: Ratio vs. Thickness of Panel

| | |
|---|---|
| Average | 0.517577 |
| Standard Deviation | 0.465981 |

# Chapter 6: REGRESSION ANALYSIS

## 6.1 Orient Strand Board (OSB)

All excel worksheets are organized by each panel. $fu_{longitudinal}$ , $fu_{transverse}$ found using literature. APA follows standardized testing procedures (e.g. tension test), such as those outlined in ASTM (American Society for Testing and Materials) standards. The results obtained from these tests, along with statistical analysis and consideration of safety factors, are used to establish the design values for $F_u$ and $F_y$ in both the transverse and longitudinal directions for OSB, plywood & gypsum panels.

| Author | Thickness of Panel (mm) | Thickness of frame (mm) | fupanel Longitudinal (Mpa) | fupanel Transverse (Mpa) | fuSteel (Mpa) | Avg Fu (Mpa) | Diameter of fastener (mm) | Pns (1) (N) | Pns (2) (N) |
|---|---|---|---|---|---|---|---|---|---|
| K.D. Peterman | 11.1 | 0.84 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 2968.73121 | 1847.20095 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| K.D. Peterman | 11.1 | 1.37 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 6183.47772 | 1847.20095 |
| K.D. Peterman | 11.1 | 2.46 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 14878.3445 | 1847.20095 |
| B.W. Schafer | 11.9 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 9390.09123 | 1980.33255 |
| B.W. Schafer | 11 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 9390.09123 | 1830.5595 |
| B.W. Schafer | 11.1 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 9390.09123 | 1847.20095 |
| Luiz Vieira, Schafer | 11.1 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 6.35 | 11546.0127 | 2792.79191 |
| Jo¨rg Lange | 12 | 1.5 | 16.84 | 12.51 | 460 | 14.675 | 4.2 | 7273.92227 | 1996.974 |
| Jo¨rg Lange | 3 | 1.5 | 16.84 | 12.51 | 460 | 14.675 | 4.2 | 7273.92227 | 499.2435 |
| Fu¨lo¨p and Dubina | 12.7 | 0.42 | 6.22 | 6.22 | 440.6 | 6.22 | 4.8 | 1103.54066 | 1023.76224 |
| R. Serrette, D. Nolan | 11.1 | 0.84 | 6.22 | 6.22 | 310 | 6.22 | 2.5 | 1584.89289 | 466.0335 |
| R. Serrette, D. Nolan | 11.9 | 0.84 | 6.22 | 6.22 | 310 | 6.22 | 2.5 | 1584.89289 | 499.6215 |
| R. Serrette, D. Nolan | 15.1 | 0.84 | 16.84 | 12.51 | 310 | 14.675 | 2.5 | 1584.89289 | 1495.74938 |
| R. Serrette, D. Nolan | 18.3 | 0.84 | 11.15 | 4.58 | 310 | 7.865 | 2.5 | 1584.89289 | 971.524125 |
| R. Serrette, D. Nolan | 11.1 | 1.37 | 6.22 | 6.22 | 448 | 6.22 | 2.5 | 4770.65663 | 466.0335 |
| R. Serrette, D. Nolan | 11.9 | 1.37 | 6.22 | 6.22 | 448 | 6.22 | 2.5 | 4770.65663 | 499.6215 |
| R. Serrette, D. Nolan | 15.1 | 1.37 | 16.84 | 12.51 | 448 | 14.675 | 2.5 | 4770.65663 | 1495.74938 |
| R. Serrette, D. Nolan | 18.3 | 1.37 | 11.15 | 4.58 | 448 | 7.865 | 2.5 | 4770.65663 | 971.524125 |
| R. Serrette, D. Nolan | 11.1 | 2.45 | 6.22 | 6.22 | 345 | 6.22 | 2.5 | 8785.9255 | 466.0335 |
| R. Serrette, D. Nolan | 11.9 | 2.45 | 6.22 | 6.22 | 345 | 6.22 | 2.5 | 8785.9255 | 499.6215 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| R. Serrette, D. Nolan | 15.1 | 2.45 | 16.84 | 12.51 | 345 | 14.675 | 2.5 | 8785.9255 | 1495.74938 |
| R. Serrette, D. Nolan | 18.3 | 2.45 | 11.15 | 4.58 | 345 | 7.865 | 2.5 | 8785.9255 | 971.524125 |
| Ye et al. | 11 | 0.9 | 11.15 | 6.22 | 448 | 8.685 | 4.8 | 3519.74894 | 1238.1336 |
| Ye et al. | 12 | 0.9 | 11.15 | 6.22 | 448 | 8.685 | 4.8 | 3519.74894 | 1350.6912 |
| Ye et al. | 18 | 0.9 | 11.15 | 4.58 | 448 | 7.865 | 4.8 | 3519.74894 | 1834.7472 |
| Sartori, Tomasi | 15 | 1.6 | 16.84 | 12.51 | 600 | 14.675 | 2.5 | 8064 | 1485.84375 |
| Sartori, Tomasi | 12 | 1.6 | 16.84 | 12.51 | 600 | 14.675 | 2.5 | 8064 | 1188.675 |

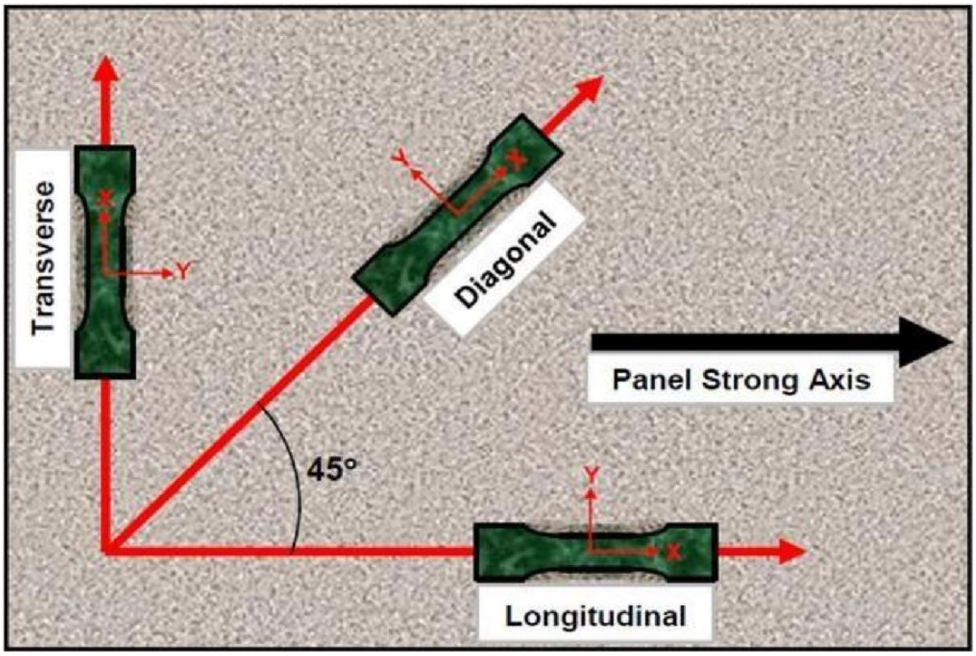


Figure 6.1: Direction of OSB test specimen [19]

Tensile strength test: This test measures the resistance of OSB to forces applied parallel to the direction of the strands. Specimens are prepared, typically in the form of strips, and subjected to tensile forces until failure occurs. This test helps determine properties such as ultimate tensile strength, yield strength, and elongation. $Avg_{fu}$ was

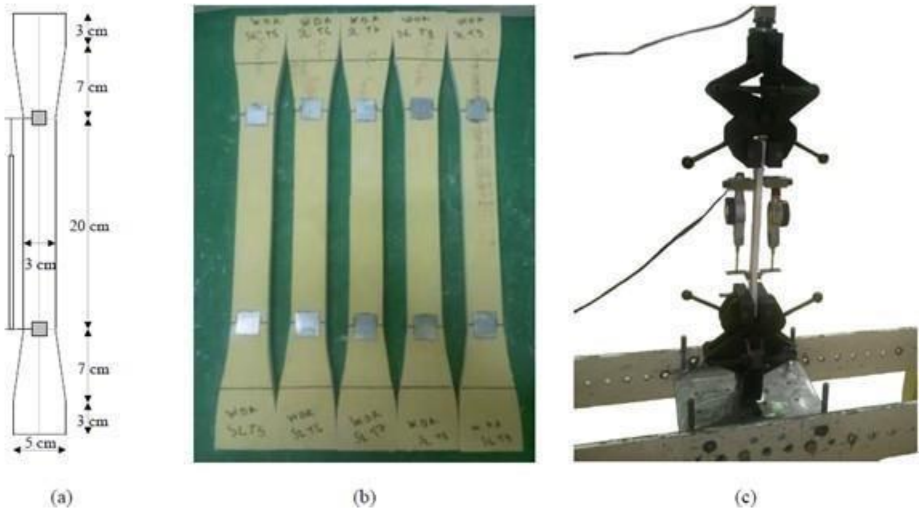


Figure 6.2: Specimen for tension test (a) geometry (b) photo (c) test setup [20]

found using $fu_{longitudinal}$, $fu_{transverse}$. $P_{NS1}$, $P_{NS2}$, $P_{NS3}$ equation shown as above mentioned equations in Chapter #3. PNSmin calculated via the minimum value of the nominal screw strength, using PNS1, PNS2, and PNS3. The ratio calculated is the PNS- min divided by the Ultimate shear strength of the single connection which was found through the literature papers. The ratio calculated represents the accuracy of the equa- tion. When the value is closest to 1 the ratio confirms the accuracy. This value is a comparison of the analytical strength to the experimental strength of the connection.

The analytical failure was calculated using the nominal shear strength.

| **Author** | **Thickness of Panel (mm)** | **Thickness of frame (mm)** | $fu_{panel}$ **(MPa) Longitudinal** | $fu_{panel}$ **(MPa) Transverse** | $fu_{Steel}$ **(Mpa)** | *Avg Fu* **(Mpa)** | **Diameter of fastener (mm)** | $P_{ns}$ **(1) (N)** | $P_{ns}$ **(2) (N)** |
|---|---|---|---|---|---|---|---|---|---|
| K.D. Peterman | 11.1 | 0.84 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 2968.73121 | 1847.20095 |
| K.D. Peterman | 11.1 | 1.37 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 6183.47772 | 1847.20095 |
| K.D. Peterman | 11.1 | 2.46 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 14878.3445 | 1847.20095 |
| B.W. Schafer | 11.9 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 9390.09123 | 1980.33255 |
| B.W. Schafer | 11 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 9390.09123 | 1830.5595 |
| B.W. Schafer | 11.1 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 4.2 | 9390.09123 | 1847.20095 |
| Luiz Vieira, Schafer | 11.1 | 1.81 | 16.84 | 12.51 | 448 | 14.675 | 6.35 | 11546.0127 | 2792.79191 |
| Jörg Lange | 12 | 1.5 | 16.84 | 12.51 | 460 | 14.675 | 4.2 | 7273.92227 | 1996.974 |
| Jörg Lange | 3 | 1.5 | 16.84 | 12.51 | 460 | 14.675 | 4.2 | 7273.92227 | 499.2435 |
| Fülöp and Dubina | 12.7 | 0.42 | 6.22 | 6.22 | 440.6 | 6.22 | 4.8 | 1103.54066 | 1023.76224 |
| R. Serrette, D. Nolan | 11.1 | 0.84 | 6.22 | 6.22 | 310 | 6.22 | 2.5 | 1584.89289 | 466.0335 |
| R. Serrette, D. Nolan | 11.9 | 0.84 | 6.22 | 6.22 | 310 | 6.22 | 2.5 | 1584.89289 | 499.6215 |
| R. Serrette, D. Nolan | 15.1 | 0.84 | 16.84 | 12.51 | 310 | 14.675 | 2.5 | 1584.89289 | 1495.74938 |
| R. Serrette, D. Nolan | 18.3 | 0.84 | 11.15 | 4.58 | 310 | 7.865 | 2.5 | 1584.89289 | 971.524125 |
| R. Serrette, D. Nolan | 11.1 | 1.37 | 6.22 | 6.22 | 448 | 6.22 | 2.5 | 4770.65663 | 466.0335 |
| R. Serrette, D. Nolan | 11.9 | 1.37 | 6.22 | 6.22 | 448 | 6.22 | 2.5 | 4770.65663 | 499.6215 |
| R. Serrette, D. Nolan | 15.1 | 1.37 | 16.84 | 12.51 | 448 | 14.675 | 2.5 | 4770.65663 | 1495.74938 |
| R. Serrette, D. Nolan | 18.3 | 1.37 | 11.15 | 4.58 | 448 | 7.865 | 2.5 | 4770.65663 | 971.524125 |
| R. Serrette, D. Nolan | 11.1 | 2.45 | 6.22 | 6.22 | 345 | 6.22 | 2.5 | 8785.9255 | 466.0335 |
| R. Serrette, D. Nolan | 11.9 | 2.45 | 6.22 | 6.22 | 345 | 6.22 | 2.5 | 8785.9255 | 499.6215 |
| R. Serrette, D. Nolan | 15.1 | 2.45 | 16.84 | 12.51 | 345 | 14.675 | 2.5 | 8785.9255 | 1495.74938 |
| R. Serrette, D. Nolan | 18.3 | 2.45 | 11.15 | 4.58 | 345 | 7.865 | 2.5 | 8785.9255 | 971.524125 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Ye et al. | 11 | 0.9 | 11.15 | 6.22 | 448 | 8.685 | 4.8 | 3519.74894 | 1238.1336 |
| Ye et al. | 12 | 0.9 | 11.15 | 6.22 | 448 | 8.685 | 4.8 | 3519.74894 | 1350.6912 |
| Ye et al. | 18 | 0.9 | 11.15 | 4.58 | 448 | 7.865 | 4.8 | 3519.74894 | 1834.7472 |
| Sartori, Tomasi | 15 | 1.6 | 16.84 | 12.51 | 600 | 14.675 | 2.5 | 8064 | 1485.84375 |
| Sartori, Tomasi | 12 | 1.6 | 16.84 | 12.51 | 600 | 14.675 | 2.5 | 8064 | 1188.675 |

The ratios calculated above ($P_{NS}$/Ultimate SS connection) give a relative compar- ison between the design and experimental shear strengths and provide the accuracy of the design calculations. Ratios close to 1 indicate good agreement between the design and experimental values, while ratios significantly higher or lower than 1 suggest po- tential discrepancies or uncertainties in the design approach. It is important to note that the nominal shear strength is typically higher than the peak experimental shear strength because it incorporates additional factors of safety and conservative assumptions to en- sure the structural reliability. The ratio calculated is the result of dividing the value of Pnov (Pull- over) by the value of $P_{not}$ (Pull-out). This ratio helps compare the two different approaches used in AISI S100. A ratio close to 1 indicates that the two formulas yield similar results and the design approach is consistent. These ratios can be used to assess the accuracy and reli- ability of the different formulas.

Another ratio calculation, is for the multi- variable regression analysis. The analysis is performed in this research for the shear strength in screw connections. To compare the peak experimental shear strength values given in the literature. “Y” being the dependent variable used to assess the shear strength in the regression analysis. Values closer to 1 govern the accuracy of the regression analysis done.

The summary output report generated through Microsoft excel shows the multi-variable regression analysis. The equation format that it has generated is in the form of:

$$Y = \beta_o + \beta 1(X_1 \times X_2 \times X_3) + \beta_2(X_4 * X_5 * X_6) + \epsilon \tag{6.1}$$

Where:-

- $\beta_{xi=0,1,2}$ = Coefficients for each independent variable
- $\epsilon$ = *Errorterm*
- Y = Dependent variable
- X = Independent variable

| **Author** | **Type of failure in screw - Updated (analytical)** | **Ratio $P_{NS}$/Ultimate SS connection - Updated** | **Ratio $P_{nov}/P_{not}$** | **Ratio Y/Ultimate SS connection - Experimental MRA 1** |
|---|---|---|---|---|
| K.D. Peterman | PO | 0.18994237 | 1.273107 | 0.55591091 |
| K.D. Peterman | PO | 0.22692886 | 0.892104 | 0.47270032 |
| K.D. Peterman | PO | 0.3170072 | 0.558925 | 0.63062662 |
| B.W. Schafer | B | 0.96601588 | 0.788151 | 1.92494164 |
| B.W. Schafer | B | 0.89295585 | 0.728543 | 1.83261879 |
| B.W. Schafer | B | 0.90107363 | 0.735166 | 1.84287688 |
| Luiz Vieira, Schafer | PT | 0.82934576 | 0.487368 | 1.87622154 |
| Jörg Lange | B | 0.55806338 | 0.450384 | 2.01710958 |
| Jörg Lange | B | 0.25112852 | 0.241277 | 1.06508824 |
| Fülöp and Dubina | PO | 0.4194512 | 1.490922 | 1.63880535 |
| R. Serrette, D. Nolan | B | 0.53078986 | 1.197801 | 2.57983785 |
| R. Serrette, D. Nolan | B | 0.27958674 | 1.284129 | 1.29129343 |
| R. Serrette, D. Nolan | B | 0.28611686 | 3.844381 | 1.28092879 |
| R. Serrette, D. Nolan | B | 0.54571006 | 2.497015 | 2.61054359 |
| R. Serrette, D. Nolan | B | 0.30066677 | 0.508191 | 1.41324531 |
| R. Serrette, D. Nolan | B | 0.28680913 | 0.544818 | 1.28184498 |
| R. Serrette, D. Nolan | B | 1.35014493 | 1.631056 | 2.48731767 |

| | | | | |
|---|---|---|---|---|
| R. Serrette, D. Nolan | B | 0.58070779 | 1.059409 | 1.53767112 |
| R. Serrette, D. Nolan | B | 0.23128213 | 0.369012 | 1.06286864 |
| R. Serrette, D. Nolan | B | 0.49272337 | 0.395607 | 2.15396847 |
| R. Serrette, D. Nolan | B | 0.73465097 | 1.184356 | 1.15613897 |
| R. Serrette, D. Nolan | B | 0.40751851 | 0.769267 | 1.05858803 |
| Ye et al. | B | 0.703485 | 0.827555 | 1.77974743 |
| Ye et al. | B | 0.76743818 | 0.902787 | 1.86056235 |
| Ye et al. | PO | 0.93469091 | 1.226325 | 2.34545187 |
| Sartori, Tomasi | PT | 0.20466167 | 1.132996 | 0.46916752 |
| Sartori, Tomasi | PT | 0.68314655 | 0.906397 | 1.38938085 |

The independent multi- variables are thickness of Panel, thickness of frame, $fu_{Steel}$, Avg

$Fu_{P\ anel}$, diameter of fastener. The two products of the independent variable include:-

- Thickness of panel ($X_1$) × Avg $Fu_{panel}$ ($X_2$) × diameter ($X_3$)
- Thickness of frame ($X_4$) × $fu_{Steel}$ ($X_5$) × diameter ($X_6$)

Multiplying both the products with their respective $\beta$ coefficients. The dependent vari- able is the experimental ultimate shear strength of the connection studied through the literature mentioned in chapter # 2.

- Y represents the dependent variable we want to predict.

- $X_1, X_2, \cdots, X_n$ are the independent variables that we believe have an impact on

  Y.

- $\beta_0, \beta_1, \beta_2, \cdots, \beta_n$ are the coefficients representing the strength and direction of the relationship between each independent variable and the dependent variable.

- $\epsilon$ represents the residual or error term, accounting for unexplained variability in the data.

Table 6.1: Summary output multiple regression analysis

| Summary Output | Regression Statistics |
|---|---|
| Multiple R | 0.446704199 |
| R Square | 0.199544641 |
| Adjusted R Square | 0.132840028 |
| Standard Error | 1653.496478 |
| Observations | 27 |

Multiple R is the correlation coefficient between the dependent variable and all the in- dependent variables in the regression model. In this case, the coefficient is 0.446, which indicates a moderate positive correlation between the ultimate shear strength and the de- pendent variables as they are influenced by the various factors. One of the reasoning's being that the larger the diameter the greater stability it provides to withhold the panel & frame.

R Square, this statistic represents the proportion of the total variation in the dependent variable that can be explained by the independent variables in the model. In this case, 19.9% of the variation in the dependent variable (the ultimate shear strength) can be partially accounted for or predicted by changes in the independent variables. A value of 1.0 for R-squared indicates that the model perfectly explains all of the variability in the dependent variable, while a value of 0 indicates that the model explains none of the variability. The remaining 80.1% of the variability in the dependent variable is not accounted for by the independent variables in the model. However, as shown there is a statistically significant relationship between the ultimate shear strength and the inde- pendent variables (as determined by the significance F-test).

Adjusted R Square, is a modified version of R Square that takes into account the number of independent variables in the model. The adjusted R Square value of

0.1328 indicates that the independent variables are not very effective at explaining the variation in the dependent variable.

Standard Error, is a measure of the accuracy of the estimates of the coefficients in a regression model which are the differences between the actual values of the dependent variable and the predicted values from the regression model. In this case, the standard error is 1653.49.

Observations refers to the number of data points used to perform the regression anal- ysis. In this case, there are 27 observations. Shown above, is the analysis of variance

Table 6.2: ANOVA multiple regression analysis

| ANOVA | | | | | |
|---|---|---|---|---|---|
| | df | SS | MS | F | Significance F |
| Regression | 2 | 16357643.64 | 8178821.821 | 2.99146688 | 0.069190329 |
| Residual | 24 | 65617214.43 | 2734050.601 | | |
| Total | 26 | 81974858.07 | | | |

(ANOVA) table for the multiple regression model.

Df, this stands for degrees of freedom and represents the number of independent pieces of information in the analysis. In this case, there are 2 degrees of freedom for the re- gression and 24 degrees of freedom for the residual.

SS: This stands for sum of squares and represents the sum of the squared differences between the predicted values and the actual values for each data point. In this case, the sum of squares for the regression is 16357643.64 and the sum of squares for the residual is 65617214.43.

MS: This stands for mean square and is calculated by dividing the sum of squares by the degrees of freedom. In this case, the mean square for the regression is 8178821.821 and the mean square for the residual is 2734050.601.

F: This is the F-statistic, which is calculated by dividing the mean square for the regres- sion by the mean square for the residual. It is used to test the overall significance of the regression model. In this case, the F-statistic is 2.9914, which indicates that the model is not statistically significant.

Significance F: This is the p-value associated with the F-statistic, which is used to deter- mine whether the regression model is statistically significant. In this case, the p-value is 0.222, which is greater than the commonly used significance level of 0.05. This indicates that the model is not statistically significant and we cannot reject the null hy- pothesis that the regression coefficients are zero.

Table 6.3: Coefficients & std. error for multiple regression analysis

| | Coefficients | Standard Error |
|---|---|---|
| Intercept, $\beta_o$ | 1013.585336 | 715.344169 |
| $\beta_1$ panel variable | 3.411875347 | 1.812563692 |
| $\beta_2$ frame variable | -0.084409563 | 0.399446758 |
| SUM of $\epsilon$ | | 717.5561794 |

The intercept term ($\beta_o$) represents the estimated value of the dependent variable when all the independent variables are zero. In this case, it could indicate the baseline value of the dependent variable when the panel and frame thickness, average $F_u$ of the panel, diameter of the screw, and $F_u$ of the steel are all zero.

The coefficient $\beta_1$ represents the estimated change in the dependent variable for a one- unit increase in the panel thickness, holding all other variables constant. A positive value (3.411875347) suggests that an increase in panel thickness is associated with an increase in the ultimate shear strength, assuming other factors remain the same. The coefficient $\beta_2$ represents the estimated change in the dependent variable for a one- unit increase in the frame thickness, holding all other variables constant. A negative value (-0.084409563) indicates that an increase in frame thickness is associated with a decrease in the ultimate shear strength, assuming other factors remain the same. The standard errors associated with each coefficient provide an estimate of the uncer- tainty or variability in the coefficient's estimate. Lower standard errors indicate more precise estimates.

The sum of $\epsilon$ represents the sum of the residuals in the regression model. Residuals are the differences between the observed values and the predicted values of the dependent variable. The sum of $\epsilon$ helps to evaluate the overall model fit and the accuracy of the predictions.

## 6.2 Gypsum

Table 6.4: Summary output multiple regression analysis

| SUMMARY OUTPUT | Regression Statistics |
|---|---|
| Multiple R | 0.478626897 |
| R Square | 0.229083706 |
| Adjusted R Square | 0.229083706 |
| R Square | 0.088917108 |
| Standard Error | 731.3007843 |
| Observations | 14 |

Table 6.5: ANOVA multiple regression analysis

| ANOVA | | | | | |
|---|---|---|---|---|---|
| | df | SS | MS | F | Significance F |
| Regression | 2 | 1748122.006 | 874061 | 1.634367 | 0.239078067 |
| Residual | 11 | 5882809.208 | 534800.8 | | |
| Total | 13 | 7630931.214 | | | |

Table 6.6: Coefficients & std. error for multiple regression analysis

| | Coefficients | Standard Error |
|---|---|---|
| Intercept, $\beta_o$ | 1479.685323 | 756.0687371 |
| $\beta_1$ panel variable | -9.212833343 | 5.652449365 |
| $\beta_2$ frame variable | 0.225353397 | .201291653 |
| SUM of $\epsilon$ | | 761.9224781 |

## 6.3 Failure Mechanism Equations

### *6.3.1 Bearing*

$$V_n = 501.8295 + (2.275\, t_p\, F_{u1}\, d) \qquad (6.2)$$

✓ Equation accuracy calculated at 99%.

### 6.3.2 Pull-through

$$V_n = 877.2687 + (0.640581\ t_p\ F_{u1}\ d) \tag{6.3}$$

✓ Equation accuracy calculated at 99%.

### 6.3.3 Pull-out

$$V_n = 1409.894 + (1.106654\ t_f\ F_{u2}\ d_h) \tag{6.4}$$

✓ Equation accuracy calculated at 99%.

### 6.3.4 All-in-one (OSB, Gypsum, Plywood)

$$V_n = 628.1173 + (1.725 t_p\ F_{u1}\ d) + (0.0375\ t_f\ F_{u2}\ d_h) \tag{6.5}$$

✓ Equation accuracy calculated at 99%.

# Chapter 7: CONCLUSION

## 7.1 Conclusion

Cold formed steel systems (CFS) are modern construction applications whose usage has been increasing over time due to the vast benefits in terms of main load bearing structures, with high durability, high strength & the ability of it being corrosion-free. With many advantages on its side, it also has an economic value, due to the simplicity of assembling and erection, short execution time, and few man-hours. In addition, the use of recyclable materials, the flexibility of systems and the possible reuse of elements assure a low environmental impact.

The objective of this research proposal is to suggest the design criteria to evaluate the ul- timate shear strength of the connection between the panel and steel frame utilizing the available experimental studies on them. The development of accurate design criteria will enable better design guidelines for structural engineers thereby leading to efficient use of CFS building systems.

The multiple experimental studies examined in this research include self-drilling screws (SDS), screw diameter, the frame thickness, as well as also assessing the strength of the sheathing connections between cold formed steel systems and panels under monotonic and cyclic shear loads. This research proposal also has taken into consideration the behavior and failure of CFS panel sheathing connections; effect of the edge distance between the sheathing and the thickness of the boards on behavior and strength. For the past 20 years, testing has been done majorly with monotonic & cyclic testing. The test database comprises of 8 experiments in OSB. 3 monotonic tests were performed in OSB, 5 tests were performed with monotonic and cyclic testing. 9 experiments took place in gypsum, 6 monotonic & cyclic tests were performed, and 3 monotonic tests were performed. 2 experiments were performed in plywood, 1 was monotonic. In OSB 6 single shear panels & 2 double shear panel experiments were done. In gyp- sum, 5 single shear panels & 4 double shear panel experiments were performed. In plywood, 1 single shear panel experiment was performed. Finally, the test results are compared with the available theoretical predictions and the experimental data from lit- erature.

The current design method in section J4 of AISI-100 lacks in accurately predicting the design shear strength of (gypsum, OSB, plywood) CFS sheathing. This research used the formulas to predict the strength of the sheathing connection but they were inaccu- rate. As calculated the formulas are underestimated in error by 48% in OSB sheathing, 71% in Gypsum sheathing & 49% in plywood sheathing. The failure mechanisms are also not good as calculated in the ratio column. Values less than 1 indicate weaker analysis in analytical form. Values closer to 1 indicate a stronger analysis in analytical form.

In order to predict a better way of analytical strength, regression analysis was performed. Using, two independent variables of the frame and panel resulted in predicting a better relationship between the shear strength of the screw connection and the two independent variables. The experimental setup performed shows that for OSB sheathing, optimizing panel and frame thickness can substantially improve connection strength. The regression model is accurate and supports the validation through the design equations.

For gypsum sheathing, the choice of fastener type is crucial, with threaded screw nails providing better structural performance compared to ring shank nails. The regression model wasn't validated due to the limited dataset that needs to be explored further. Design shear strength is always less than peak shear strength. This formulated equation governs design strength.

## 7.2 Future recommendation

In the future, this formula from regression analysis can govern the experimental evalu- ation using various sheathing connection assemblies. Future research should explore the interplay between panel and frame thickness in gypsum screw connections in CFS shear wall systems and investigate the long-term performance of gypsum fastener types under different loading conditions. These insights can guide the development of more resilient and efficient building practices.

## References

[1] Lee, S. H., & Kim, H. J. (2019). Shear resistance of light-gauge steel-framed shear walls with wood-based sheathing. Engineering Structures, 197, 109425.

[2] Uddin, N., & Rogers, C. A. (2015). Experimental study of sheathing-to- frame connections in cold-formed steel-framed shear walls. Journal of Structural Engineering, 141(10), 04015036.

[3] Zhang, Y., Wang, Y., & Lin, Y. (2021). Shear behavior of cold-formed steel- framed shear walls with gypsum sheathing. Journal of Constructional Steel Re- search, 181, 106428.

[4] American Iron and Steel Institute (AISI). (2016). AISI S100-16: North Amer- ican specification for the design of cold-formed steel structural members. AISI.

[5] M. Santhosh Kumar and M. S. Sridhar, ”Design Criteria for Evaluating Sheath- ing Connection Strength in Cold-Formed Steel Shear Walls,” International Jour- nal of Civil Engineering and Technology, vol. 8, no. 4, pp. 430–443, 2017.

[6] L. Fiorino and A. Mandara, ”Experimental Investigation of Screw Connec- tions for Gypsum and Cement-Based Panels to Cold-Formed Steel Framing,” Journal of Structural Engineering, vol. 143, no. 9, pp. 04017138, 2017. DOI: 10.1061/(ASCE)ST.1943-541X.0001833.

[7] L. A. Fu¨ lo¨ p and D. Dubina, ”Design Criteria for Seam and Sheeting-to- Framing Connections of Cold-Formed Steel Shear

Panels,” Journal of Structural Engineering, vol. 144, no. 6, pp. 04018058, 2018. DOI: 10.1061/(ASCE)ST.1943-541X.0002097.

[8] A. Jammi and S. A. Jayachandran, ”Performance of Screw Connections be- tween Cold-Formed Steel Framing and Sandwich Sheathing Panels,” Journal of Structural Engineering, vol. 145, no. 2, pp. 04018245, 2019. DOI: 10.1061/(ASCE)ST.1943- 541X.0002284.

[9] J. Lange and B. Naujoks, ”Behavior of cold-formed steel shear walls under monotonic and cyclic loading,” Journal of Structural Engineering, vol. 139, no. 7, pp. 1133-1143, Jul. 2013.

[10] M. Nithyadharan and V. Kalyanaraman, ”Behavior of screw connections be- tween cold-formed steel framing and calcium silicate board panels under mono- tonic and cyclic loading,” Journal of Constructional Steel Research, vol. 103, pp. 1-12, Mar. 2014.

[11] R. Serrette and D. Nolan, ”Behavior of wood structural panels attached to cold-formed steel framing using pneumatically driven knurled steel pins,” Journal of Structural Engineering, vol. 143, no. 8, Aug. 2017.

[12] K. D. Peterman, N. Nakata, and B. W. Schafer, ”Hysteretic behavior of cold-formed steel stud-to-sheathing connections under cyclic loading,” Journal of Structural Engineering, vol. 142, no. 11, Nov. 2016.

[13] T. Sartori and R. Tomasi, ”Behavior of sheathing to framing connections in wood shear walls under monotonic and cyclic loading conditions,” Journal of Structural Engineering, vol. 143,

no. 10, Oct. 2017, article no. 04017147. DOI: 10.1061/(ASCE)ST.1943-541X.0001865.

[14] S.G. Buonopane, G. Bian, T.H. Tuna, and B.W. Schafer, ”Efficient models of cold-formed steel shear walls with wood sheathing under monotonic and cyclic loading conditions,” Journal of Structural Engineering, vol. 144, no. 3, Mar. 2018, article no. 04018006. DOI: 10.1061/(ASCE)ST.1943-541X.0002065.

[15] L.C.M. Vieira Jr. and B.W. Schafer, ”Lateral stiffness and strength of sheath- ing braced cold-formed steel stud walls,” Journal of Constructional Steel Re- search, vol. 168, May 2020, article no. 106074. DOI: 10.1016/j.jcsr.2020.106074.

[16] S. Selvaraj, M. Madhavan, and H. H. Lau, ”Design method for sheathed cold-formed steel point-symmetric wall frame studs,” Journal of Structural Engi- neering, vol. 147, no. 10, Oct. 2021, Art. no. 04021127.

[17] S. Swensen, G. G. Deierlein, and E. Miranda, ”Behavior of screw and ad- hesive connections to gypsum wallboard in wood and cold-formed steelframed wallettes,” Journal of Structural Engineering, vol. 146, no. 10, Oct. 2020, Art. no. 04020238.

[18] J. Ye, X. Wang, and M. Zhao, ”Shear behavior of screw connections in cold- formed steel sheathing,” Journal of Structural Engineering, vol. 147, no. 3, Mar. 2021, Art. no. 04020248.

[19] S. Sartori, ”Experimental investigation on sheathing-to-framing connections in wood shear walls,” Journal of Structural Engineering, vol. 146, no. 5, May 2020, Art. no. 04020051.

[20] Selvaraj, S. (2019). Sheathing-Fastener Connection Strength Based De- sign Method for Sheathed CFS Point-Symmetric Wall-Frame Studs. Journal of Structural Engineering, 145(8), 04019075. doi: 10.1061/(ASCE)ST.1943- 541X.0002323.

[21] American Iron and Steel Institute, ”Specification for the Design of

Cold- Formed Steel Structural Members,” AISI S100, Washington, D.C., 1996.