SLIP MODULUS OF COLD-FORMED STEEL MEMBERS SHEATHED WITH WOOD STRUCTURAL PANELS

by

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Abstract

Cold-formed steel framing sheathed with wood structural panels is a common method of construction for wall, roof and floor systems in cold-formed steel structures. Since wood structural panels are attached with screws at relatively close spacing, a certain amount of composite behavior will be present. However, the benefit of composite behavior of this system is currently not being taken advantage of in the design of these structural systems. While composite effects are present, they are not yet being accounted for in design due to a lack of statistical data. To determine the amount of composite action taking place in these systems, the slip modulus between steel and wood is required. The slip modulus reflects the amount of shear force able to be transferred through the screw connection, to either member of the composite system. This thesis presents the results of a study conducted to determine values of the slip modulus for varying thicknesses of cold-formed steel and plywood sheathing. Push tests were conducted and the slip moduli were determined based on ISO 6891 and ASTM D1761. Compared with data from a previous preliminary study performed by others, the values determined from these tests for the slip modulus were deemed reasonable. The determination of the slip modulus will lead to the ability to calculate a composite factor. Determination of a composite factor will allow cold-formed steel wood structural panel construction to become more economical due to the available increase in bending strength.

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Dedication

This is dedicated to my grandmother, Sue Northcutt. Without her this would not have been possible.

Chapter 1 - **Introduction**

Cold-formed steel wood structural panel construction (CFSWSPC) is the use of cold-formed steel members sheathed with wood structural panels and attached using screws. Cold-formed steel members have been used in building construction since the 1850s, however it has only been widely used in buildings since the 1940s (Yu, 2000). Cold-formed steel and wood structural panels are already being used together widely in building construction, however CFSWSPC is not being used as a composite material. Further study must be completed in order to more accurately understand the composite behavior of the material and produce design guidelines allowing engineers to design for the composite behavior.

The use of cold-formed steel in building construction has many advantages over similarly designed timber framing systems:

- High strength and stiffness
- Low transportation costs due to the ability of cold-formed steel to interlock and save freight space
- Uniform material properties allow for more economical design and fewer issues at the time of construction due to poor material quality
- Environmentally conscious: while plywood comes from renewable sources, steel is continually recyclable with no degradation in performance, from product to product (AISI, 2012)

Cold-formed steel construction is most typically used for repetitive member systems such as floor joists, roof joists, roof rafters and wall studs. In each of these structural systems sheathing is normally attached to the cold-formed steel members. In many cases the sheathing is wood structural panels. Because this sheathing is already being attached, the cold-formed steel members and the sheathing may act in a composite manner to resist bending. If this is the case, more strength is actually available in the structural system than what is currently being designed for.

When the wood structural panel and cold-formed steel joist are connected, the screw connection will resist the horizontal shear force between the members when bending is present. When the entire horizontal shear load is not able to be transferred through the connection, due to gaps between the materials or connection spacing, this is called slip. The extent to which this slip occurs can be related with a value called the slip modulus. The slip modulus can be used to calculate the shear flow coefficient, the effective composite bending stiffness, and ultimately the composite factor for CFSWSPC. Obtaining the slip modulus for CFSWSPC will allow the design of the system to be more economical. Accounting for composite action will allow for the possibility of smaller member sizes or greater strength for the system.

Chapter 2 - **Background**

As previously mentioned, CFSWSPC is typically used in a repetitive member assembly such as a floor, roof or wall application. The *Standard Guide for Evaluating System Effects in Repetitive-Member Wood Assemblies* (ASTM, 2003) defines a timber-timber composite structure (TTCS) repetitive member assembly as a system in which a transverse load-distributing element connects three or more members. This definition can also be applied to CFSWSPC. In the case of CFSWSPC the transverse load-distributing element is the wood structural panel, which is one of the most commonly used load distributing elements for most low-rise buildings in North America according to Rosowsky, et al. (2004) in *Partial Factor Approach to Repetitive-Member System Factors.* The member used in this case is a cold-formed steel member.

According to ASTM D6555-03, *Standard Guide for Evaluating System Effects in Repetitive Member Wood Assemblies,* "The apparent stiffness and strength of repetitive member wood assemblies is generally greater than the stiffness and strength of the members in the assembly acting alone. The enhanced performance is a result of load sharing, partial composite action and residual capacity obtained through the joining of members with sheathing or cladding, or by connections directly." (ASTM, 2003)

ASTM D6555-03 (2003) defines "composite action" of TTCS as "interaction of two or more connected wood members that increases the effective section properties over that determined for the individual members." To simplify, as stated previously, the addition of the sheathing as a member increases the section properties because the system is then able to be designed as a t-beam, and not a simple joist. Figure 2-1 shows an illustration of cold-formed steel members in a repetitive member system attached to wood structural panel (plywood, in this case) and the effective t-beam created by the two members. Effective t-beams with partial composite action can be modeled by numerous structural analysis formulations which include the finite difference method, the finite element method, the direct stiffness method and the exact analytical model. The direct stiffness method is used in this thesis.

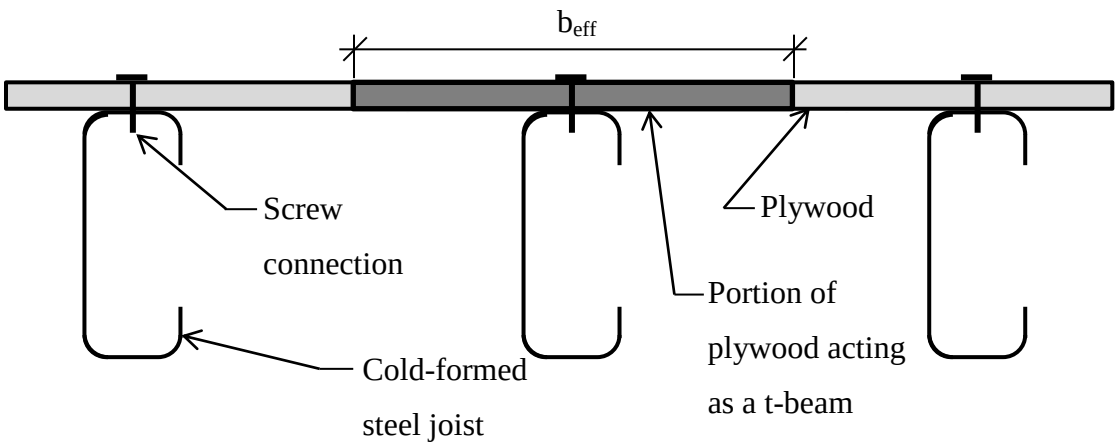

Figure 2-1 CFSWSPC Effective T-Beam

A joist that is sheathed with plywood does not act simply as a beam carrying the loads. The plywood sheathing acts along with the joist and forms a composite t-beam (see Figure 2-1). The t-beam is comprised of the cold-formed steel member as the web and the plywood sheathing as the flange. To resist bending in the composite member, the plywood acts as the compression flange and the bottom of the cold-formed steel member acts as the tension flange. However, due to the non-rigid connection between the dissimilar elements of the CFSWSPC, full composite action may not exist, and thus should not be assumed. Full composite action occurs when the two elements being attached act as if they are one solid element, even if they have different material properties. In order to have full composite action, the connection between the two elements must be completely rigid, thus for CFSWSPC only partial composite action takes place. Partial composite action is a condition in which full composite action is not able to be used or developed. The slip modulus is the measure of the shear flow that is able to be transferred through the connection. The connection and possibility of gaps between the joist and sheathing creates the non-rigid connection in which slip must be accounted for. This non-rigid connection can be accounted for using a slip modulus.

The slip modulus is a value reflective of the stiffness of a connection between two materials. When members are in bending, the composite action results in increased flexural rigidity by increasing the effective moment of inertia of the cross section of the composite

members. Composite action decreases as the rigidity of the connection between the cold formed steel and the sheathing decrease.

Similar to CFSWSPC, timber-concrete composite structures use timber members as the joist and concrete as the transverse load distribution element. Many aspects of timber-concrete composite structures are similar to that of CFSWSPC. Stresses that develop at the interface of the two composite materials can be expected to result in slip in both composite constructions. The slip modulus is a value that allows the slip to be taken into account in design while accepting that some composite action does occur in a composite assembly such as timber-concrete composite structures or CFSWSPC. *Connectors for Timber-Lightweight Concrete Composite Structures* by Steinberg, et. al. (2003), defined the slip modulus as the initial stiffness of the composite material. The initial stiffness is the quotient of the load at 40% of the estimated ultimate load and the accompanying slip at that moment, in accordance to the International Organization for Standardization (ISO) 6891.

In order to determine the magnitude of the partial composite action taking place, the slip modulus must be determined. For CFSWSPC the additional stiffness and strength of the composite structure is not currently being accounted for in design. In the same way as the TTCS described above, CFSWSPC can gain stiffness and strength in a repetitive member system by partial composite action obtained through the joining of the members with sheathing.

This thesis determines a lower bound for the slip moduli for CFSWSPC in order allow designers to account for the aforementioned increase in stiffness and strength through composite action between the cold formed steel and wood structural panel sheathing.

Chapter 3 - **Literature Review**

In order to fully understand the behavior of CFSWSPC, it is necessary to look at other types of composite materials and their behaviors. While the materials may be different, many of the mechanisms and behaviors are very similar. These help to verify data found from the testing of CFSWSPC. Additionally, it is important to fully understand the completed studies with this same composite material. This understanding enables further progress in the study of CFSWSPC.

Timber-Lightweight Concrete Composite Structures

Timber-concrete composite is another relatively new composite construction method being used. Those designing timber-concrete composite have also had to address the issue of slip modulus and composite action. In the study *Connectors for Timber-Lightweight Concrete Composite Structures* by Steinberg, et al. (2003) timber members are connected to a concrete slab such that when the composite is subject to bending forces due to gravity loads the timber member is in tension and the concrete is in compression. The Steinberg study attempts to determine the best possible connector for the composite structure.

Steinberg's study is comprised of 4x10 timber joists, spaced at 24" on center, overlaid with ¾" timber plywood which is used as the formwork for the 2" thick lightweight concrete slab which is installed on top and connected with screws at a 45 degree angles. The nature of timber-concrete composite structures differs from CFSWSPC in that the screw connecting the elements is primarily loaded in tension rather than shear. The tensile load makes the 45 degree connection more efficient and results in about twice the stiffness and load capacity as screws installed at a normal direction to the face of the concrete.

The materials were set up for testing such that the concrete layer was sheathed on either side by timber plywood as well as the timber joist. The materials were then connected by two screws, varying in size and type, on either side, as shown in Figure 3-1 and Figure 3-2. The specimens were tested using a push-out test; load was placed on the specimen at the top of the concrete portion of the specimen and the specimen was supported only by the timber joists.

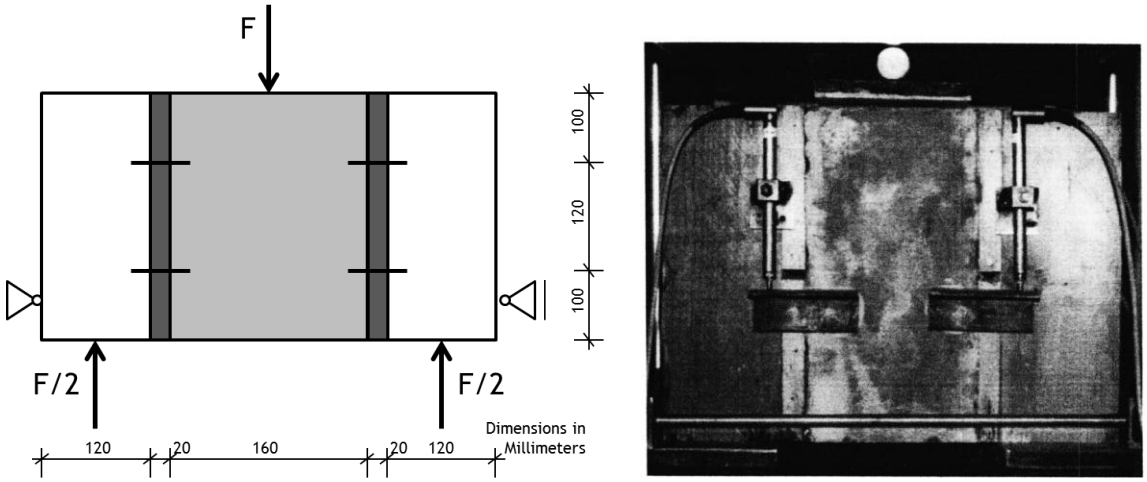

Figure 3-1 Timber-Lightweight Concrete Composite Test Set-up, with permission of ASCE

In accordance to the ISO 6891, the load was applied at a rate of 1mm per minute and first increased to 40% of the ultimate load, followed by a relief of the load to 10% of the ultimate load, and finished by loading to failure, as shown in Figure 3-2. The ultimate load was defined as the load at which the materials had slipped 15mm or the load at which the member failed by another mechanism prior to 15mm of slip occurring. The determination of failure at 15mm of slip is also in accordance with ISO 6891. Each series of tests consisted of four specimens. The first specimen was tested in order to determine the ultimate load by which the test procedure for the final three specimens was able to be designed.

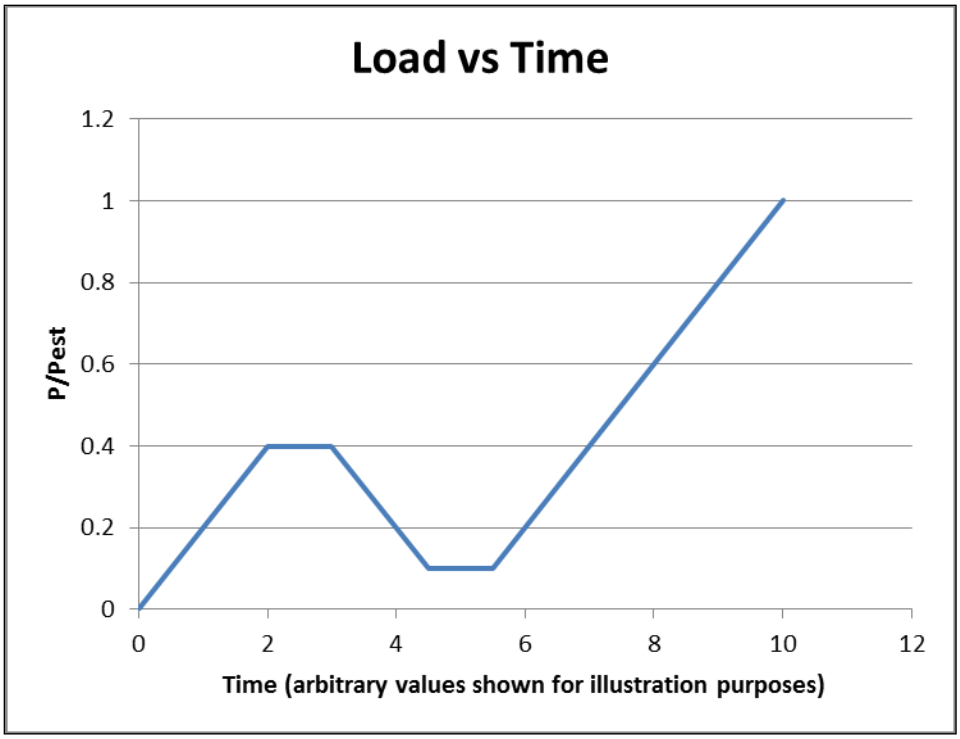

Figure 3-2 Loading Curve

Steinberg, et al. determines the slip modulus using the quotient of the load at 40% of the estimated load and the accompanying measurement of slip, as shown in Equation 3-1 and in accordance with ISO 6891.

Equation 3-1 Slip Modulus

$$K = \frac{0.4P_u}{v_{0.4}}$$

Where:

K = Slip Modulus (lb/in)

P_u = Ultimate Load (lb)

$v_{0.4}$ = measured slip at 40% P_u (in)

This study is a good example of how to determine the slip modulus of a composite material. While different conclusions are being derived from the results of the experiments performed, the ultimate goal is the same: to determine the slip modulus of the composite

material. In examining the methods that the slip modulus is determined for other composite materials, this may validate the methods for which the slip modulus of CFSWSPC is determined.

From the slip modulus value calculated, the shear bond coefficient is then determined. The shear bond coefficient represents the amount of shear force able to be transferred from one part of the composite member through the connection to the other part of the composite member. The calculation for the shear modulus is shown in Equation 3-2.

Equation 3-2 Shear Modulus

$$\gamma = \frac{1}{1 + \frac{\pi^2 s E_S A_S}{K L^2}}$$

Where:

γ = shear bond coefficient

s = spacing of connectors (in)

E_S = Modulus of Elasticity of sheathing (psi)

A_S = Area of sheathing (in^2)

K = slip modulus (lb/in)

L = length of member (in)

The use of the shear bond coefficient allows for the effective bending stiffness of the overall composite to be calculated. This takes into account the original stiffness of each individual material, the slip that occurs at the interface of the composite materials and the composite action that takes place due to the connection(s). The effective stiffness calculation is shown in Equation 3-3.

Equation 3-3 Effective Stiffness

$$(EI)_{eff} = E_S I_S + \gamma E_S A_S a_1^2 + E_J I_J + E_J A_J a_2^2$$

Where:

$(EI)_{eff}$ = effective stiffness of composite ($lbin^2$)

$E_S I_S$ = bending stiffness of sheathing (lbin^2)

γ = shear bond coefficient

$E_S A_S$ = axial stiffness of sheathing (lb)

a_1 = distance between sheathing centroid and CFSWSPC centroid (in)

$E_J I_J$ = bending stiffness of joist (lbin^2)

$E_J A_J$ = axial stiffness of joist (lb)

a_2 = distance between joist centroid and CFSWSPC centroid (in)

The effective stiffness is then used to calculate the total bending stress in the timber member of the composite system.

Timber-Timber Composite Structures

Timber-timber composite structures have been highly investigated and designed. Many of the methods of construction and design of TTCS are similar to CFSWSPC, therefore the mechanisms by which TTCS is designed can be applied to CFSWSPC. It is for this reason of transference that a thorough study of TTCS is necessary in order to understand some of the possible behavior and design considerations necessary for CFSWSPC.

In the more traditional TTCS the behavior of repetitive member systems has been investigated much more extensively. Many tests have been conducted to determine the stiffness added to timber joists by attaching timber sheathing. In *Light-Frame Wall and Floor Systems,* (1989) Sherwood attempts to understand the composite behavior of TTCS using various methods of subflooring as well as various methods of connections. Sherwood cites a 13 percent increase in stiffness was noted when plywood subfloor was nailed to timber joists.

Sherwood compared different thicknesses and types of sheathing, sizes and grades of joists, spans and spacing, with the deflection and two-way action of repetitive member systems. Sherwood determined from his studies that it is the axial stiffness of the sheathing that adds the strength and stiffness to the repetitive member system. When the sheathing is rigidly fastened to the joists with no gaps, full composite action takes place. When no connection between the joists and the sheathing occurs the two materials act completely independently. In typical construction

the connections are not able to be completely rigid, for constructability there must be spacing between connections and thus the actual case is somewhere between the two extremes.

According to Sherwood, when using nail or screw fasteners, the degree to which the fastener resists slip is dependent on its ability to resist pullout/slip and spacing of the fasteners. The actual relationship between the lateral load and the slip is nonlinear, however for the ranges of design loads the relationship between the lateral load and the slip has been simplified to a linear relationship, as shown in Equation 3-4. This is allowed because the joists, which carry the majority of the load, do behave linearly, and the overall composite structure behaves in a nearly linear manner. When the relationship between lateral load and slip is simplified to linear, the interlayer stiffness of the mechanical fasteners can be calculated as follows:

Equation 3-4 Mechanical Fastener Interlayer Stiffness

$$S = \frac{k}{s}$$

Where:

S = interlayer stiffness (lb/in/in)

k= nail stiffness (lb/in), also known as the load/slip ratio, or how much load the member is able to withstand per inch of slip

s = spacing between fasteners (in)

In "First-Order Reliability Analysis of Wood Structural Systems," by W. M. Bulleit and W. F. Liu (1995), timber-timber composite structures are analyzed. Timber sheathing of 5/8" thickness attached with 8d common nails at 8" on center to timber joists are studied as floor and roof systems. The layout and setup of the repetitive member, composite structures are shown in Figure 3-3. Two-way action, partial composite action, and other irregularity effects dealing specifically with the non-uniform nature of timber material were taken into account. Computer simulation, BSAF, and an approximate method, the beam-spring method, were used to analyze the lifetime behavior of said wood floor and roof systems, from initial load to failure. The

computer analysis itself is less applicable to the study of CFSWSPC than the method of accounting for partial composite action of the repetitive member system.

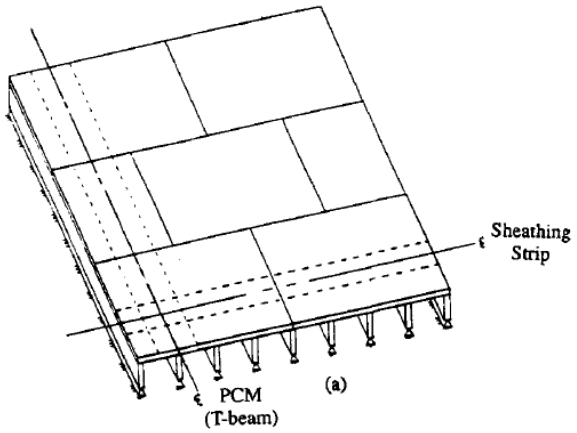

Figure 3-3 Typical Wood System Layout, with permission of ASCE

The Bulleit study determines system factors, Ψ, for load and resistance factor design (LRFD) and analyzes a number of system factors and how they affect the system factor, Ψ. The main system factor that applies is the effect of the sheathing and connectors. The values of the system factor vary so slightly for different thicknesses of sheathing, the sheathing thickness is found to not be a contributing factor to the variation of the system factor. The stiffness of the connectors does affect the system factor slightly, however this amount is so small compared to the stiffness itself that it is found to be insignificant.

The consequences of these determinations are important to the study of CFSWSPC because only one thickness of sheathing per thickness of cold-formed steel and one type of connector used in both the Bulleit study and the current study of CFSWSPC. Based on this the slip modulus determined shall be valid no matter the thickness of sheathing used or the rigidity of the connector.

The article, *Partial Factor Approach to Repetitive-Member System Factors* by David Rosowsky and G. Yu (2004), outlines the current procedures outlined in the National Design Specification (NDS) for designing a repetitive-member system. According to Rosowsky, within the NDS the repetitive use factor of 1.15 accounts for load-sharing as well as partial composite T or I-beam action within a repetitive-member system.

While it is appropriate for the repetitive member factor to be applied to increase the allowable bending stress of dimensional lumber, the current repetitive member factor is based only on simple statistical model. The statistical model is based on the increase of load-carrying capacity and stiffness when multiple joists are attached together by a transverse load distributing element, such as plywood sheathing. Further study and testing needs to be completed to more fully comprehend the partial composite action of the joist and sheathing system. Part of the analysis of the system requires the calculation of a Partial Composite Action Factor (PCA Factor). The PCA factor as defined by Rosowsky is merely a ratio of the maximum stress in the timber joist by itself to the maximum stress in the partial composite section of the effective t-beam created by the joist and sheathing. The PCA Factor is defined as shown in Equation 3-5.

Equation 3-5 Partial Composite Factor

$$K_{PCA} = \frac{6(EI)h}{6(EI)h + h_s[EI - (EI_S + EI_C)]}$$

Where:

K_{PCA} = partial composite factor

E = modulus of elasticity (psi)

I = moment of inertia (in^4)

h = distance from the centroid of the member to the sheathing (in)

h_s = height of the stud (in)

Cold-Formed Steel Wood Structural Panel Composite

In *Repetitive Member Factor Study for Cold-Formed Steel Framing Systems,* Scott Clayton (2010) outlines the characteristics of a Repetitive Member Assembly of Cold form steel framing system. While Clayton's report focuses on the repetitive member factor and its use with cold-formed steel, he also outlines the composite behavior present in CFSWSPC systems and compares CFSWSPC to TTCS.

In order for Clayton to determine the repetitive member factor, the composite factor must be determined. It was assumed that the screw connection between the cold-formed steel stud and the sheathing provided full composite action. This assumption allows the transformed area method to be used to calculate the member strength, much like when calculating the strength of a reinforced concrete beam.

In the study by Clayton, it was also assumed that the full flange width of the effective t-beam was able to be utilized and that the steel stud was solid, with no holes punched in the web. The full flange width was taken conservatively at 16 inches in order to limit the flange width, and thus its strength. An illustration of the effective t-beam is shown in Figure 3-4. Conversely, the assumption that the web of the cold-formed steel member is solid is not conservative, however the effect of holes punched in the web is negligible in bending.

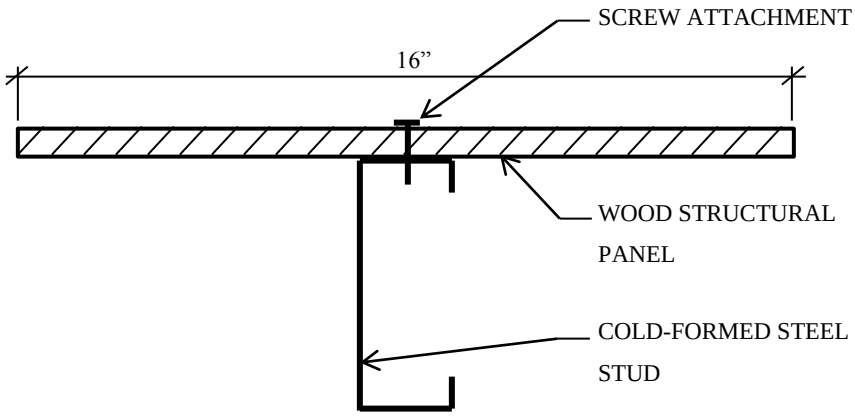

Figure 3-4 Composite Section of Clayton CFS Stud and Wood Structural Panel

The transformed area method was used to determine the composite action effects by Clayton. The area of the sheathing is transformed to an equivalent area of cold-formed steel using a ratio of the moduli of elasticity. The transformed area method is used so that the composite t-beam can be analyzed as if it is one material. The neutral axis is calculated, and then used to determine the maximum moment of the composite section. The composite factor is the ratio of the maximum moment of the composite section to that of the non-composite member.

The composite factor is determined by Clayton to be 1.27 when using 6" deep, 1.625" wide, 33mil cold-formed steel members attached to ½" OSB sheathing with a 24/0 span rating with #8 steel screws. This means that 27% of additional strength is due to the composite action in the member. While calculations were done to ensure that the screws were able to transfer the maximum shear force in the member, it has been shown in TTCS that full composite action is not plausible in reality. It is reasonable to assume that this is also the case for CFSWSPC. While the screws may be able to transfer the full shear force, they may also slip. In order to adjust the composite factor for the reality that full composite action does not take place, the slip modulus of CFSWSPC must be determined.

Matsen Ford Design Associates conducted a preliminary test to find the slip modulus of CFSWSPC, *The Study of Slip Modulus for Cold Form Steel-Timber Composite Floor Structures* (Chan, et al., 2009). The purpose of the investigation was to determine the slip modulus so that design for floor vibration when using CFSWSPC could be more accurate.

The Matsen Ford Design Associates study investigated a variety of different connection types and connection spacing. The two connection types examined were mechanical connectors and mechanical connectors with glue. The three different connector spacings evaluated were a single connector, two connectors with 6" spacing, and two connectors with 12" spacing. Materials for each test, including joist, plywood, and connector information are given in Table 3-1.

Table 3-1 Tables of Materials for TTCS and CFSWSPC

Materials for Tests (2x4 Joist/Plywood)

Materials	**Description**
Joist	1 1/2" x 3 1/2"
Plywood	3/4" thick
Nails	25 mm dia. ardox spiral
Glue	LePage PL400 subfloor & deck adhesive

Materials for Tests (Cold Formed C-Joist/Plywood)

Materials	**Description**
Plywood	3/4" thick
Cold Formed C-Joist	Back-to-back cold-formed c-joist
Screws	Self-drilling screw, square socket Wafer head, TEKS style 3 #10 – 24 x 1 1/4", 0.470" head 23/32" thread length
Glue	LePage PL400 subfloor & deck adhesive

Two series of tests were conducted in "The Slip Modulus of Cold Formed Steel-Timber Composite Floor Structures." The first series was conducted with timber-timber composite structures. Using timber joist members and plywood sheathing, this test series was conducted in order to validate the lab procedure and ensure the values produced were similar to those already being used. The second test was using CFSWSPC. Using cold-formed steel joists and plywood sheathing, this test series was conducted to determine a slip modulus to be used for the design of floor vibration.

Matsen Ford Design Associates used the pull out test was used to determine the slip modulus. The materials were set up such that the slip condition of the joist and plywood would be simulated at the interface of the two composite materials. The joist material was connected to the plywood using the given number of fasteners and glue, depending upon the test set. The set-up of the TTCS is shown in Figure 3-5 and the set-up of the CFSWSPC is shown in Figure 3-6.

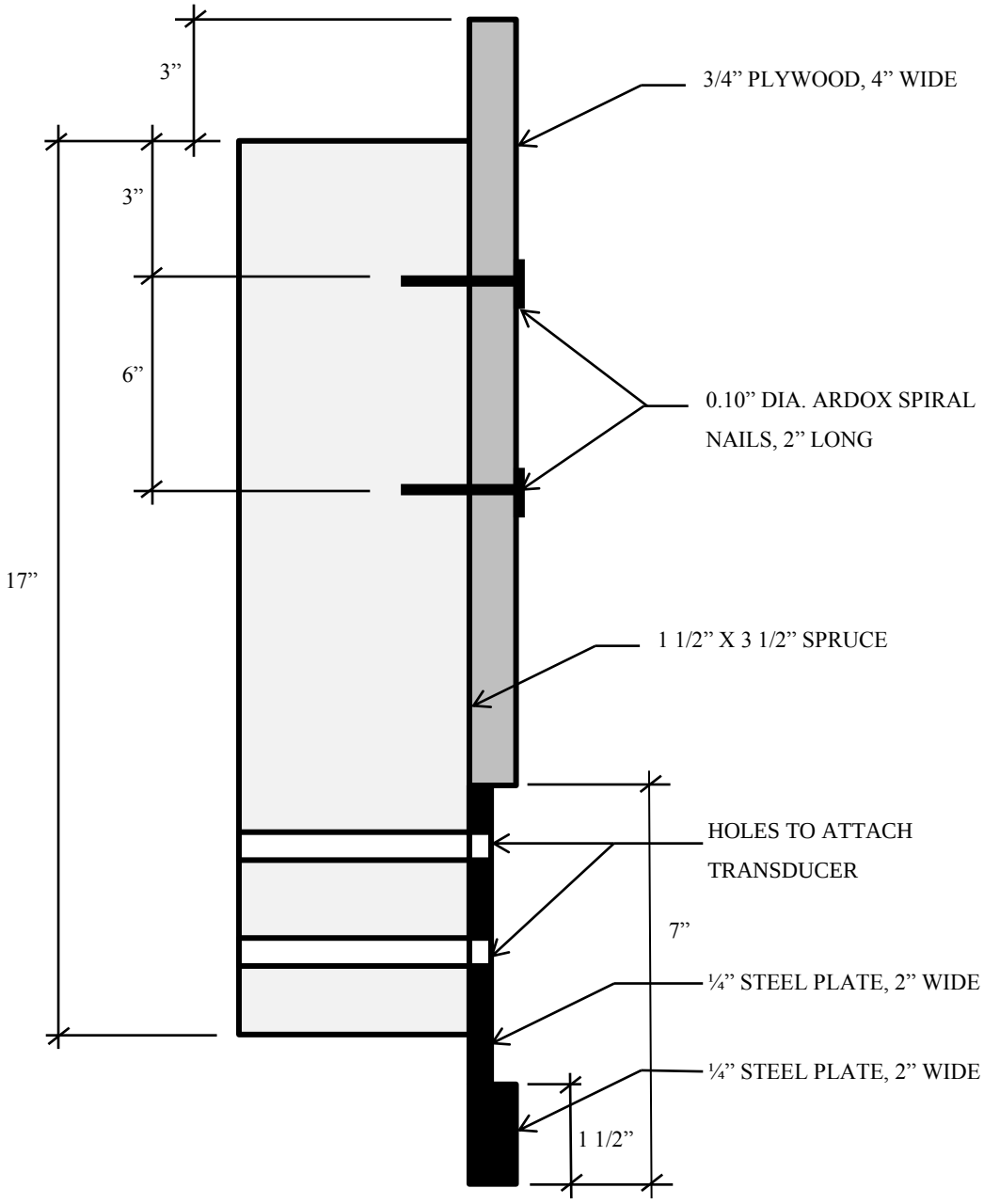

Figure 3-5 TTCS Test Set-up

In the Matsen Ford Design Associates study the TTCS specimen was fabricated by nailing 3/4” plywood with at 2x4 joist. For the nailed type connection, 25mm diameter nails were simply hammered in. For the nailed and glue type connection, a 1/4” bead of glue was applied to the surface of the plywood and joist and held in place and allowed to dry for three days prior to testing.

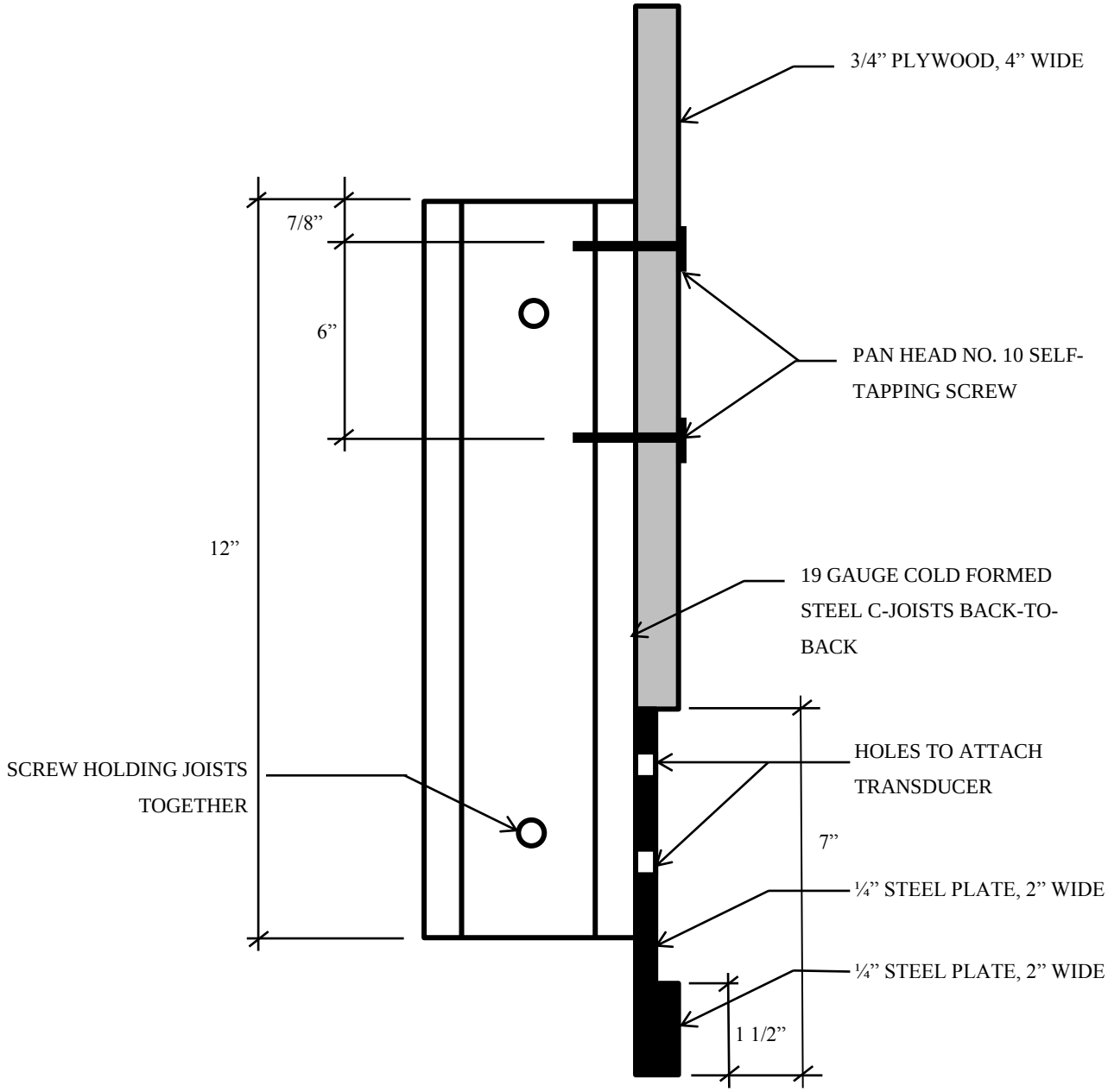

Figure 3-6 CFSWSPC Test Set-up

The CFSWSPC specimen in the Matsen Ford Design Associates study was fabricated by drilling screws through the 3/4” plywood into cold-formed c-joists. Self-tapping screws, #10 size, with square sockets and a 0.47” diameter were screwed in using an electric drill. For the screwed type connection, only the screws were applied. For the screw and glue type connection, a 1/4” bead of glue was applied to the surface of the 3/4” plywood and 5/8” cold-formed steel

joist (MC460) and allowed to dry for three days before testing. The materials for each test series are shown in Table 3-1.

In the Matsen Ford Design Associates study the load was applied in accordance with ISO 6891, as shown in Figure 3-7 (from concrete-timber composite section). The load was applied at a constant rate of motion of 1.0 mm/min (0.0394 in/min). The test procedure was also based on ISO 6891 as well as ASTM D 1761 and was conducted as follows:

Step 1) Conduct a preliminary test to determine $*P_{max}$

$*P_{max}$ is defined as the load corresponding to the failure of the specimen or a 15 mm slip

Step 2) Estimate $*P_{est}$ based on P_{max}

$*P_{est}$ is the estimated failure load based on P_{max} obtained from the preliminary test

Step 3) Apply load according to ISO 6891 until failure (load application curve shown in Figure 3-7)

i) Apply load until it reaches $0.4xP_{est}$

ii) Maintain load for 30 s

iii) Relieve load from $0.4xP_{est}$ to $0.1xP_{est}$

iv) Maintain load for 30 s

v) Increase load to 70% P_{est}

vi) Increase load until failure

Step 4) Compare P_{max} and P_{est}

If the difference between P_{max} and P_{est} is less than 20%, go to step 6. Otherwise, Step 5

Step 5) Re-estimate P_{est} and redo Step 3 to 4

Step 6) Plot the load and deformation curve

Step 7) Determine the slip modulus

Step 8) Compare the slip modulus with the value established in the ATC Vibration Design Guide to validate the experimental setup and approach

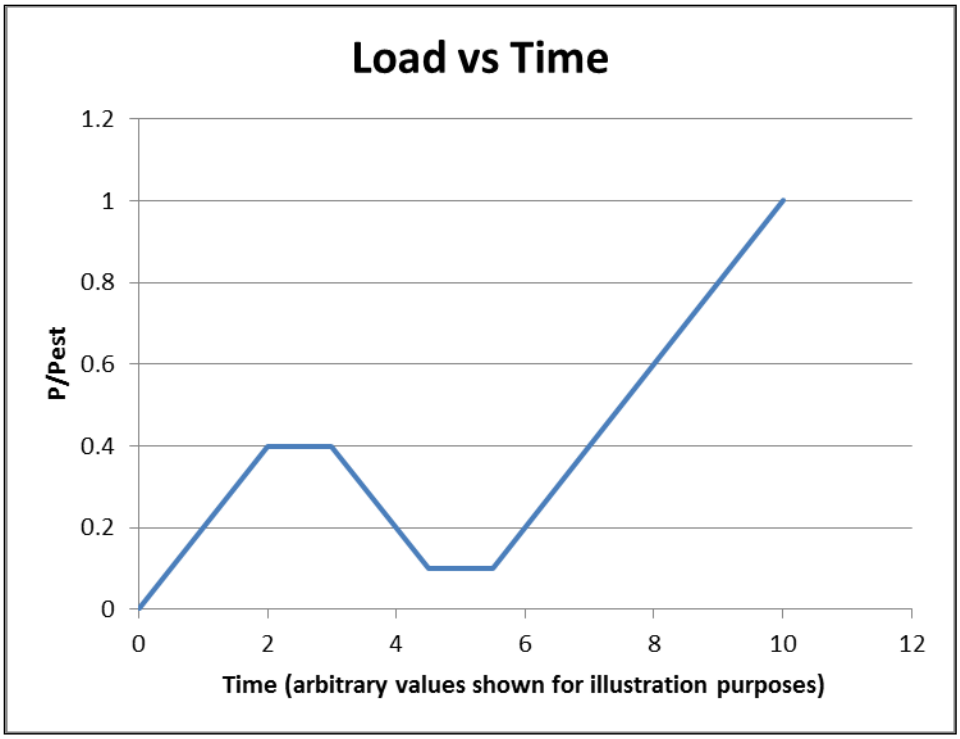

Figure 3-7 Load Application Curve

The slip was measured using a transducer during testing by Matsen Ford Design Associates. The transducer was attached to the side of the test specimen such that the measurement did not account for deformation of the individual materials themselves. As the test progressed, the slip was continuously measured.

The slip modulus was calculated by Matsen Ford Design Associates as the quotient of the load at 40% of the estimated ultimate load and the accompanying slip, as given in Equation 3-6. This value is then normalized in order to determine a value able to be compared to the ATC Design guide. The value is normalized by dividing the previous value by the total length of the specimen and the number of screws installed, as shown in Equation 3-7.

Equation 3-6 Slip Modulus

$$K = \frac{0.4P_U}{v_{0.4}}$$

Equation 3-7 Normalized Slip Modulus

$$K_N = \frac{(\frac{0.4P_u}{v_{0.4}})}{ns}$$

Where:

K = slip modulus (lb/in)

K_N = normalized slip modulus (lb/in/in)

P_u = ultimate load (lb)

$v_{0.4}$ = measured slip at 40% of P_u (in)

n = number of screws

s = spacing of screws (in)

The results of the test by Matsen Ford Design Associates produced two design values for CFSWSPC, using the lower bounds of each of the connection types. The design value of the slip modulus using only screw connection is 650 lb/in/in. The design value of the slip modulus using screw and glue connection is 1100 lb/in/in.

The sample size of the Matsen Ford Design Associates study was three samples per connection type per test series. Due to the small sample size, irregularities from the materials or the fabrication may significantly alter the data and conclusions of the study. Further investigation is needed in order to validate the values calculated for the slip modulus of CFSWSPC.

Chapter 4 - **Test Plan and Procedure**

Test Plan

The study of the slip modulus of CFSWSPC was comprised of four test series. All series were using cold-formed steel joists with plywood sheathing and two screws, spaced at 12", on each side of the member. Table 4-1 shows the different combinations of materials for each CFSWSPC test series. 43 mil cold-formed steel studs were not available at the time of this study.

Table 4-1 Test Series

Series	Test Combinations		
	Steel Thickness	Plywood Thickness	Screw Size
T1	33 mil (20ga)	1/2"	#10
T2	54 mil (16ga)	1/2"	#10
T3	68 mil (14ga)	1/2"	#10
T4	97 mil (12ga)	23/32"	#10

The CFSWSPC was tested using variation in the cold-formed steel thickness and plywood thickness; however the connection type and spacing remained constant. Materials for the tests can be found in Table 4-2. The connection method and spacing chosen to test is based upon "The Study of Slip Modulus for Cold Form Steel – Timber Composite Floor Structures (Chan, 2009)." 12 inch screw spacing is the normal spacing used for roof, floor and wall sheathing for members not located at a sheathing panel joint. Thus, to imitate most typical construction methods this test is limited to the use of two connectors with 12" spacing.

Table 4-2 Materials for Tests

Materials	Description
Cold Formed C-Joist	33 mil, 43 mil, 54 mil, 97 mil
Plywood	23/32", 1/2" thick
Screws	#10 self-drilling, self-tapping TEKS 5, 1 7/16", Phillips Flat Head

The plywood thickness was changed for test T4 four to imitate typical construction practices. While it is still possible that a design for a roof rafter or load bearing wall that calls for 1/2" plywood to be attached to 97 mil cold-formed steel, it is more likely that the plywood thickness would be increased. Each test series was run a minimum of three times to obtain significant data. Materials for Experiment are given in Table 4-2.

Apparatus

Pull out tests were used to determine the slip modulus of CFSWSPC. The apparatus used for the test are shown in Table 4-3 and Figures 4-1 through 4-4.

Table 4-3 Test Apparatus

Apparatus	Description
MTS Machine	Machine can apply loads up to 55 kips. It operates at either a constant stroke or constant force and has an accuracy of +/- 1% when calibrated. Last calibrated 3-21-11. See Figure 4-1
Loading plate	1.25" steel plate to distribute the load evenly to the cross section of the specimen from the MTS Machine. See Figure 4-2
Screws with washers and angle plate	Fabricated to aid in the measurement of slip such that they did not affect the material performance. See Figure 4-3
Transducer	Schaevitz DC-EC 2000 LVDT. The transducer measures the slip between the cold formed steel and plywood during the test. It has a sensitivity of 0.001". See Figure 4-4

Figure 4-1 MTS Testing Machine

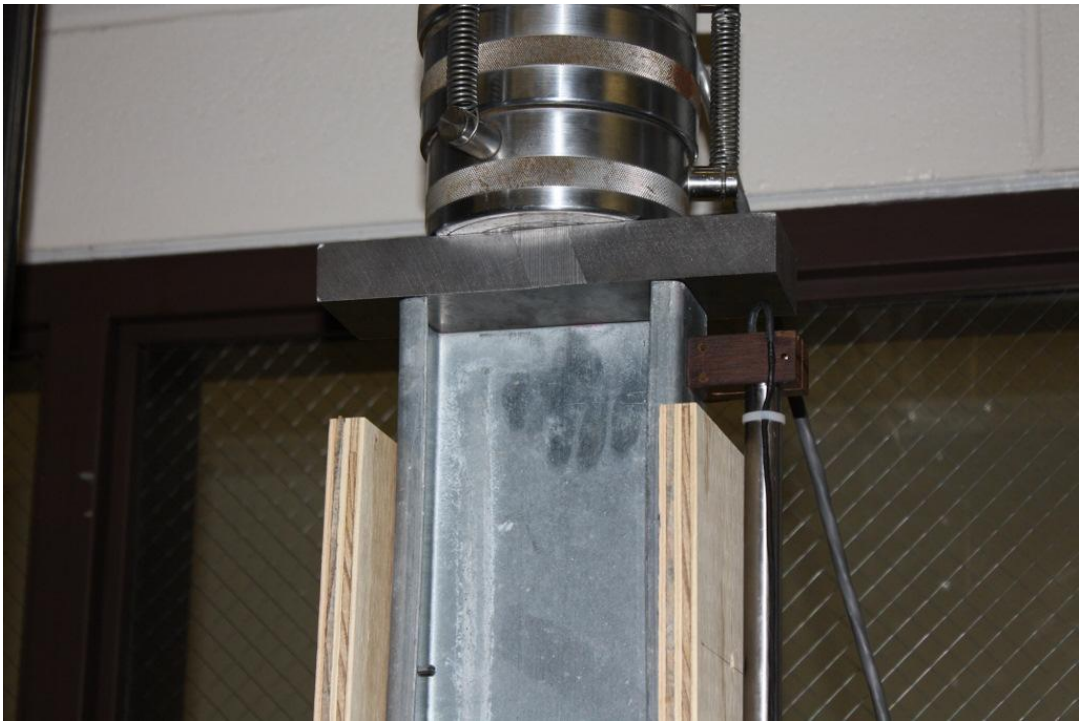

Figure 4-2 Loading Plate

Figure 4-3 Screws with Washers and Angle Plate

Figure 4-4 Transducer

Experimental Procedure

The plywood is typically produced in four foot by eight foot sheets, thus the first step of the procedure was to cut the plywood into six inch by twenty four inch pieces. The steel also was cut into two foot sections. The components are measured and marked for assembly. Specimens are assembled using two pieces of plywood and one steel section. Two self-screwing, self-tapping screws are used to attach each piece of plywood to each side of the steel member, as shown in Figure 4-5. The transducer must be attached to the steel and the plywood, so a small hole is drilled in the steel for the insertion of a bolt and a small angle iron is attached using a small screw to the plywood. All screws are installed using a hand held drill.

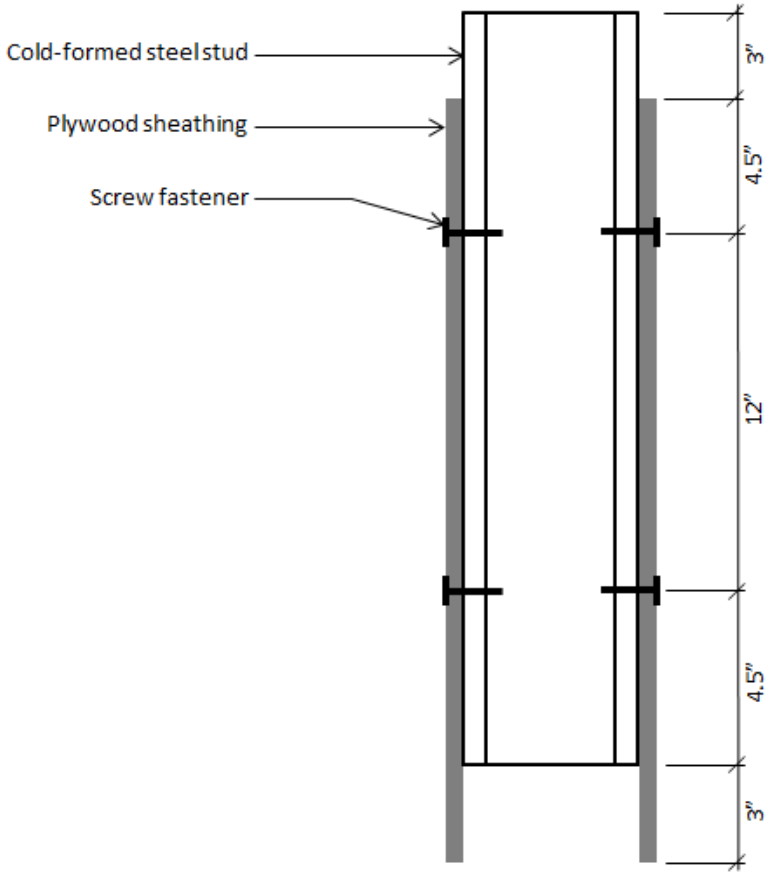

Figure 4-5 Test Specimen Set-up

Before beginning a test the transducer must be set to zero and the MTS machine must be adjusted such that the specimen is secured but not loaded on the machine. This ensures that the

slip reading has a reference value of zero rather than an alternate number and allows for easy measurements during the loading phase. The specimen is then loaded using a constant displacement of 0.0394 in/min (1 mm/min).

For each series one initial specimen is loaded to failure to determine the ultimate load for the rest of the test series. This test procedure was based on ISO 6891 and ASTM D1761. The procedure is as follows:

1) Conduct a preliminary test to determine the ultimate load in order to set up the proceeding tests. The ultimate load, P_u, is defined as the load corresponding to specimen failure or 15mm of slip.
2) Estimate the load at which failure will occur in the future specimens, P_{est}, based upon the ultimate load, P_u
3) Apply load according to ISO 6891 as follows:
    i. Apply load until it reaches $0.4*P_{est}$
    ii. Maintain load for 30 seconds
    iii. Relieve load from $0.4*P_{est}$ to $0.1*P_{est}$
    iv. Maintain load for 30 seconds
    v. Increase load to $0.7*P_{est}$
    vi. Increase load until failure
4) Compare the ultimate load, P_u, to the estimated load, P_{est}. The ultimate load is the load at which failure occurs. Failure may occur by a number of different mechanisms, however screw shear and screw tilting were the only two observed in this study. Screw tilting failure is determined to be at a tilting or slip value of 15mm or 0.591in. If the difference between P_u and P_{est} is more than 20%, the test must be thrown out and a new specimen must be tested. If the difference is less than 20% continue to step 5.
5) Plot the load and displacement curve
6) Determine the slip modulus

The use of 15mm as a benchmark for tilting failure of the specimen was used previously by Chan, et al (2009) in the preliminary study to determine the slip modulus of CFSWSPC and governed by ISO 6891. This value seems reasonable, for if even half an inch of slip occurs in a composite member, the deflection will cause much greater stresses on nearby elements within the structure.

The load curve as described in step 3 above is shown in Figure 4-6. In Figure 4-6: step 3i is shown from time 0 to 2, step 3ii is shown from time 2 to 3, step 3iii is shown from time 3 to 4.5, step 3iv is shown from time 4.5 to 5.5, step 3v and 3vi are continuous and shown from time 5.5 to 10.

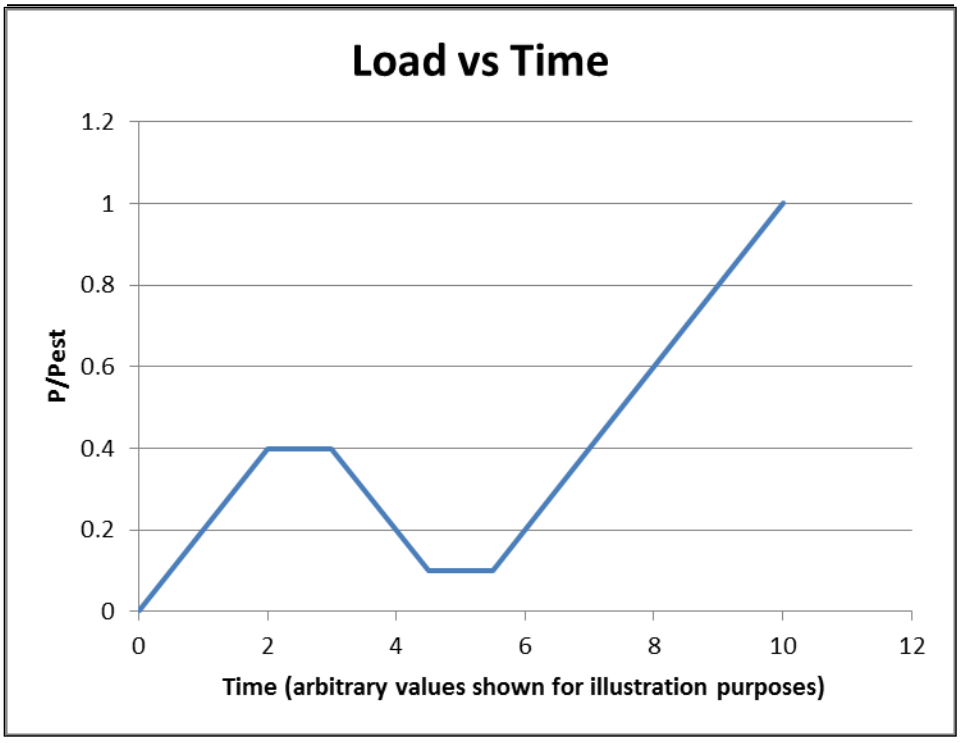

Figure 4-6 Loading Curve

Slip Measurement

A transducer (see Table 4-3) was attached to the side of each specimen in order to measure the slip between the two materials. Slip was measured through the entirety of the test, from when the initial load was applied through failure. Mechanisms of failure were either screw

shear or screw tilting. The failure mechanism for each specimen is shown in Table 5-1. Data was recorded every 0.001 inches of slip.

Chapter 5 - **Test Results**

Test Data

The data collected has been compiled for use in Table 5-1. Figures 5-1 through 5-4 show the load vs. displacement curve of each specimen, grouped by test series.

Table 5-1 Test Results

Test Series	Steel Gauge	Plywood Thickness	Specimen Code	Maximum Force	Maximum Slip	40% P_u	Slip at 40% P_u	Failure Mechanism
	ga (mil)	in		lb	in	lb	in	
1	20 (33)	1/2"	20A	1986	0.4625	845	0.1290	Screw Tilting
	20 (33)	1/2"	20B	2052	0.6006	845	0.0816	Screw Tilting
	20 (33)	1/2"	20C	1708	0.6055	845	0.1010	Screw Tilting
2	16 (54)	1/2"	16A	2488	0.6019	1084	0.0352	Screw Tilting
	16 (54)	1/2"	16B	2657	0.5099	1084	0.0383	Screw Tilting
	16 (54)	1/2"	16C	2570	0.6006	1084	0.0259	Screw Tilting
3	14 (68)	1/2"	14A	2483	0.6051	1127	0.0808	Screw Tilting
	14 (68)	1/2"	14B	2865	0.6039	1127	0.0464	Screw Tilting
	14 (68)	1/2"	14C	2971	0.6046	1127	0.0361	Screw Tilting
4	12 (97)	23/32"	12A	3393	0.2110	1354	0.0322	Screw Shear
	12 (97)	23/32"	12B	3936	0.3979	1354	0.0324	Screw Shear
	12 (97)	23/32"	12C	3294	0.8922	1354	0.0463	Screw Shear

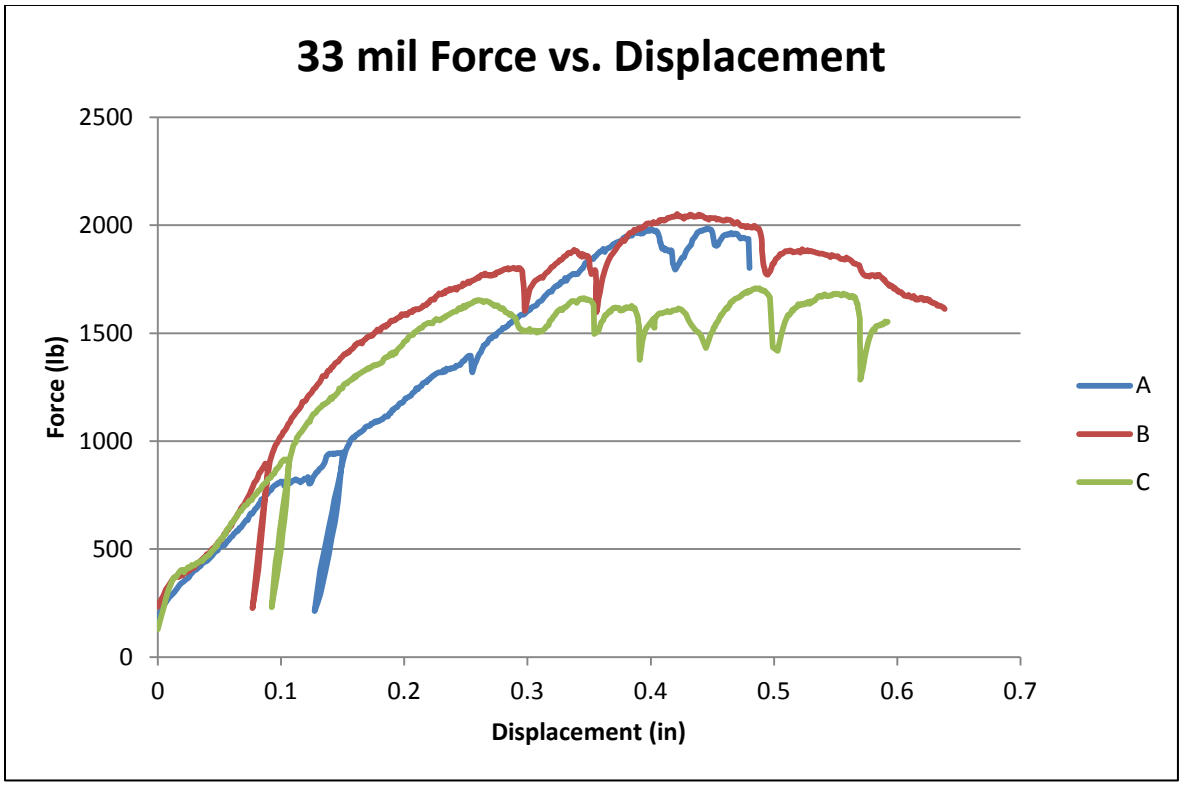

Figure 5-1 Test Series 1 Force vs. Displacement

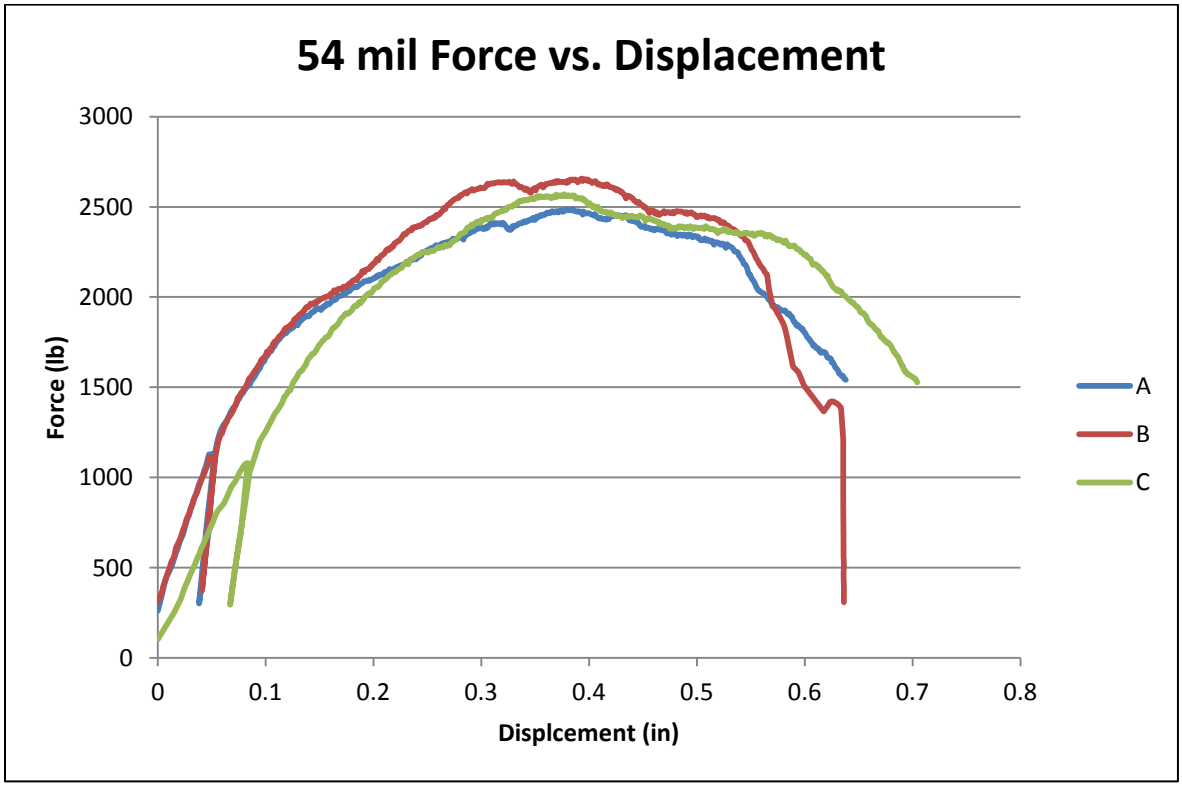

Figure 5-2 Test Series 2 Force vs. Displacement

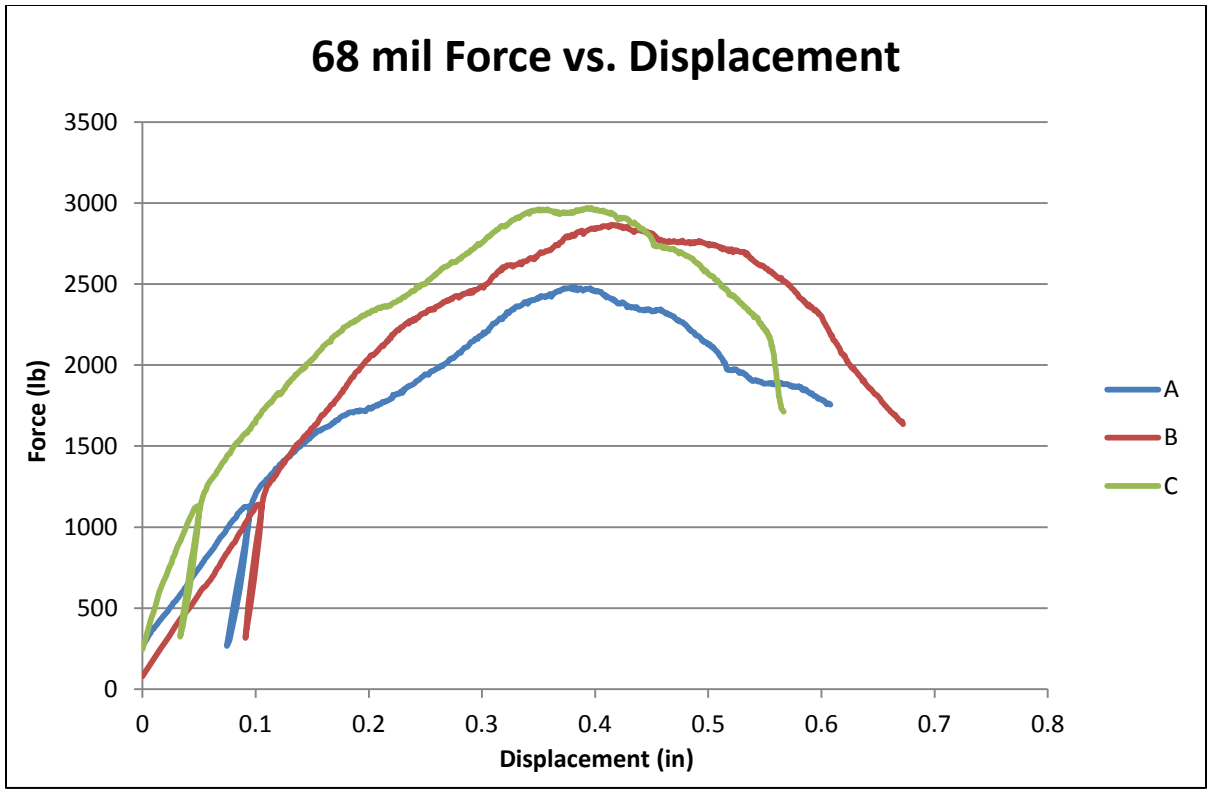

Figure 5-3 Test Series 3 Force vs. Displacement

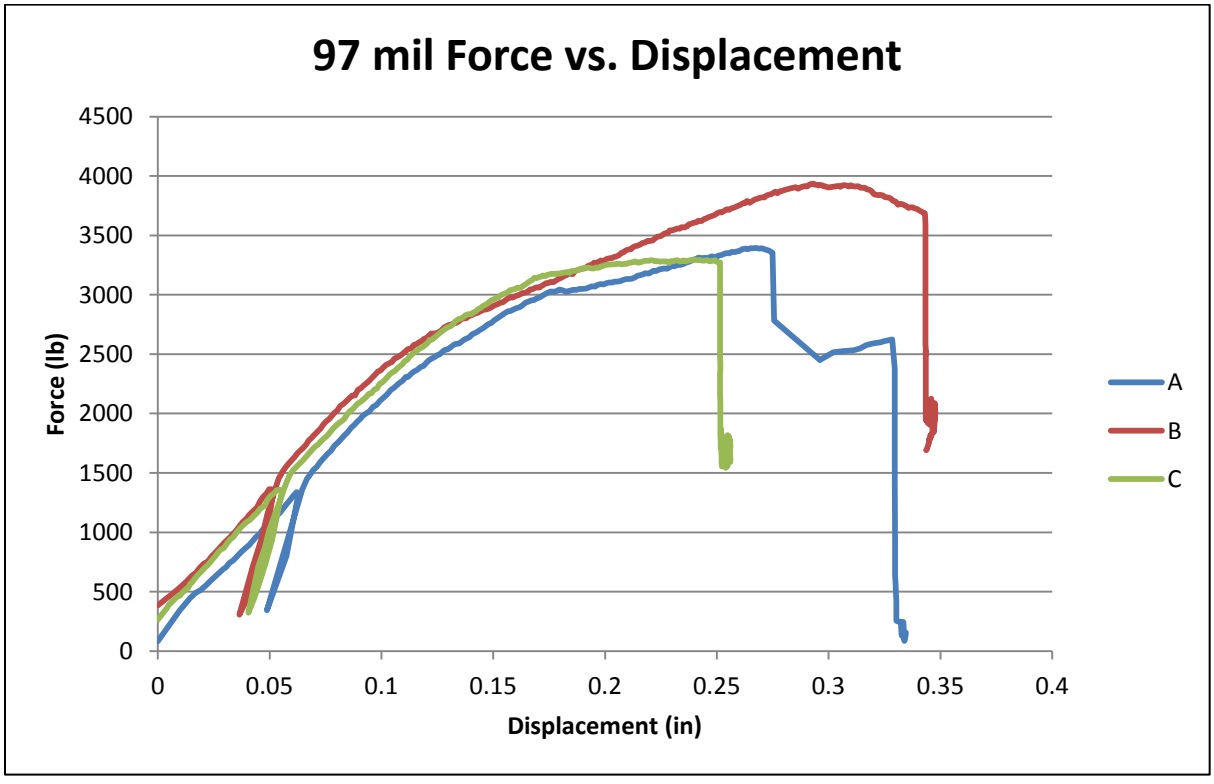

Figure 5-4 Test Series 4 Force vs. Displacement

Chapter 6 - **Conclusion**

Discussion of Results

The maximum load for each specimen generally increased as the thickness of the cold-formed steel joists increased. The maximum slip also generally increased with exceptions in the 97 mil steel tests (test series 4). This is likely due to the brittle and sudden nature of the screw shear failure that took place for test series 4. Test series 1, 2 and 3 had screw tilting failure mechanisms, and thus were slower, more predictable failures.

The sharp decline at the end of each of the force vs. displacement curves of the 97 mil specimens in Figure 5-4 is due to the failure mechanism of screw shear. The failure was so rapid that the testing apparatus was still taking data during the failure and after the composite materials had likely collapsed. The remaining tests shown in Figures 5-1 through 5-3 all had much less sudden curves, thus indicating the slow, predictable failure as mentioned previously.

The most common mode of failure was screw tilting. This occurred either when the screws were no longer effective in attaching the plywood and the steel or when the slip between the two materials was measured by the transducer as 15mm. Figures 6-1 and 6-2 show screw tilting failures. Figure 6-1 shows that all four screws connecting the plywood to the cold-formed steel are rotated due to the load; in this case the specimen was considered failed because the slip had reached 15mm. Figure 6-2 gives an up close look at the angle of the screw. Note that the screws were originally installed at a 90 degree angle to the face of the plywood.

Figure 6-1 Screw Tilting Failures

Figure 6-2 Screw Tilting Failure, Close Up

The mode of failure for each specimen in test series T4 was screw shear. The change in the mechanism of failure can likely be attributed to the greater thickness of plywood and cold-formed steel. The materials were able to prevent the screws from tilting as they did in the series T1 through T3 tests, and thus the screw ultimately failed in shear. Figure 6-3 shows the inside of the plywood, where a cold formed steel joist was attached, prior to screw shear failure. This shows the screw bearing deformation of the plywood, and while some deformation was present, the amount of tilting was extremely small compared with that of the tilting failures shown in Figures 6-1 and 6-2. Figure 6-4 shows the opposite side of the cold-formed steel stud. The stud shown was previously attached to the plywood shown in Figure 6-3 and shows the screw still in the cold-formed steel stud on the opposite half of the screw shear failure.

Figure 6-3 Screw Shear Failure

Figure 6-4 Screw Shear Failure

The results found in this study appear reasonable, in comparison with the previous preliminary study by Matsen Ford Design Associates. This study expanded the scope of research to multiple thicknesses of cold-formed steel studs, and thus some variation was expected in comparison to the preliminary study's results.

Implications for Practice

In order for the slip modulus to be useful, it must be shown that effective bending stiffness of the CFSWSPC is greater than the bending stiffness of the cold-formed steel joist alone. In order to calculate the effective bending stiffness of the composite material, the normalized slip modulus and the shear bond coefficient must be calculated for each test series. Calculations of these values are to follow.

The slip modulus was calculated for each test specimen. The slip modulus is equal to the quotient of forty percent of the ultimate load and the corresponding amount of slip at that load, as shown in Equation 6-1. The slip modulus then must be normalized. A normalized slip modulus will reflect the composite action for one screw connection, per inch of sheathing, as shown in Equation 6-2. In order to normalize the slip modulus the original value must be divided by both

the number of connections per specimen and the vertical spacing between the screws, in this case 12 inches. Table 6-1 shows the values for the slip modulus and normalized slip modulus for each specimen.

Equation 6-1 Slip Modulus

$$K = \frac{0.4P_U}{v_{0.4}}$$

Equation 6-2 Normalized Slip Modulus

$$K_N = \frac{(\frac{0.4P_U}{v_{0.4}})}{ns}$$

Where:

K= slip modulus (lb/in)

K_N = normalized slip modulus (lb/in/in)

P_u = ultimate load (lb)

$v_{0.4}$ = measured slip at 40% of P_u (in)

n = number of screws

s = spacing of screws (in)

Table 6-1 Slip Modulus Calculations

Test Series	Specimen Code	Slip Modulus	Normalized Slip Modulus
		lb/in	lb/in/in
1	20A	6549	136.4
	20B	10353	215.7
	20C	8364	174.3
2	16A	30809	641.9
	16B	28315	589.9
	16C	41871	872.3
3	14A	13944	290.5
	14B	24283	505.9
	14C	31211	650.2
4	12A	42046	876.0
	12B	41787	870.6
	12C	29242	609.2

When examining the slip modulus values in Table 6-1, Test 14A appears inconsistent with the other values within test series 3. Test 14A also appears inconsistent with trend that as the thickness of the cold-formed steel increases, the slip modulus increases. While the value of the maximum load was still within the required 20% of the estimated ultimate load, this test will be thrown out because of the high variation. During testing the specimen seemed to perform similarly to the other two within the series; however more slip occurred early within the test than in tests 14B and 14C. The large amount of slip that was present when 40% of the ultimate load was applied is the cause for the smaller normalized slip modulus value.

The value of the slip modulus affects the extent to which full composite action is designed for. A higher slip modulus results in a larger composite factor, and thus allows for more composite action to be designed for.

Recommendations

The statistics for the normalized slip modulus are shown in Table 6-2, using a 95%, two tailed probability.

Table 6-2 Normalized Slip Modulus Statistical Data

Test Series	Standard Deviation, σ	Mean	Median	Confidence Interval	Coefficient of Variation
1	32.38	175.4	174.3	137 – 214	0.185
2	122.7	701.4	641.9	557 – 846	0.175
3	147.8	482.2	505.9	307 – 657	0.307
4	124.5	785.3	870.6	638 – 932	.159

From these tests with #10 screws spaced at 12” on center, the following nominal slip modulus values are recommended:

- 140 lb/in/in for 33 mil cold-formed steel with 1/2" plywood sheathing
- 560 lb/in/in for 54 mil cold-formed steel with 1/2” plywood sheathing
- 640 lb/in/in for 97 mil cold-formed steel with 23/32” plywood sheathing

These recommended values are based on the lower bound of the 95%, two tailed probability confidence intervals. The confidence interval is the range of values with which 95% of tests will fit into. The values recommended are nominal values and will need a safety factor applied for design applications.

640 lb/in/in is the recommended slip modulus for the 97 mil test series and 560 lb/in/in is the recommended slip modulus for the 54 mil test series, rather than 650 lb/in/in as previously recommended in the Matsen Ford Design Associates study due to slightly lower results. The results for the 33 mil test series were significantly lower than those recommended in “The Study of Slip Modulus for Cold Formed Steel-Timber Composite Floor Structures (Chan et al., 2009).”

In order to provide accurate recommendations for test series 3, further study is recommended. Using a 95% two-tailed probability the confidence interval is 307 to 657 and the coefficient of variation is twice as large as the other tests; this appears to be unreliable and no recommendation will be given. Due to the small number of samples and high variation in the test

data, a high standard deviation was recorded in the calculation of the slip modulus, and thus a lack of reliability of the values present.

Based on these recommended values for the slip modulus, some calculations must be completed in order to prove that the use of the slip modulus is warranted in design. If the effective bending stiffness of the CFSWSPC is significantly greater than that of the cold-formed steel joist alone, such that it will make a difference in design, it will be warranted to use the slip modulus to increase the available strength in design.

The shear bond coefficient is used to determine the effective bending stiffness of the composite material. The effective bending stiffness will show the relative amount of stiffness increase from the composite action of the CFSWSPC compared to the stiffness of the cold-formed steel joist alone. The value of the shear bond coefficient relates the amount of shear force able to be transferred through the connection. The shear bond coefficient is dependent upon the slip modulus, determined from individual tests, the length of the member, the modulus of elasticity of the sheathing, the area of the sheathing and the spacing of the connectors, as shown in Equation 6-3. Values of the shear bond coefficient are shown in Tables 6-3, 6-4 and 6-5.

Equation 6-3 Shear Bond Coefficient

$$\gamma = \frac{1}{1 + \frac{\pi^2 s E_S A_S}{K L^2}}$$

Where:

γ = shear bond coefficient

s = spacing of connectors (in)

E_S = Modulus of Elasticity of sheathing (psi)

A_S = Area of sheathing (in^2)

K = slip modulus (lb/in)

L = length of member (in)

Table 6-3 Test Series 1 Shear Bond Coefficient

Test Series 1	
Shear Bond Coefficient	Reference
K= 140 lb/in/in	
s= 12 in	
E_sA_s= 5533333 lb	(NDS 2005, Table M9.2-2)
L= 120 in	
γ= 0.003	

Table 6-4 Test Series 2 Shear Bond Coefficient

Test Series 2	
Shear Bond Coefficient	Reference
K= 560 lb/in/in	
s= 12 in	
E_sA_s= 5533333 lb	(NDS 2005, Table M9.2-2)
L= 120 in	
γ= 0.012	

Table 6-5 Test Series 4 Shear Bond Coefficient

Test Series 4	
Shear Bond Coefficient	Reference
K= 640 lb/in/in	
s= 12 in	
E_sA_s= 7800000 lb	(NDS 2005, Table M9.2-2)
L= 120 in	
γ= 0.010	

The shear bond coefficient is used to determine the effective stiffness of the composite. This takes into account both the axial stiffness and bending stiffness of each of the materials that the composite is composed of, the shear bond coefficient, and the size of each of the composite components through the distance between the centroid of each individual member and the overall

composite centroid. If the shear bond coefficient increases the effective stiffness of the composite also increases.

The effective stiffness is calculated using Equation 6-4. Ultimately, the effective stiffness is the measure of whether there is a benefit to considering the composite action of CFSWSPC. Tables 6-6, 6-7 and 6-8 show the calculations for effective stiffness for each test series.

Equation 6-4 Effective Stiffness

$$(EI)_{eff} = E_S I_S + \gamma E_S A_S a_1^2 + E_J I_J + E_J A_J a_2^2$$

Where:

$(EI)_{eff}$ = effective stiffness of composite (lb*in^2)

$E_S I_S$ = bending stiffness of sheathing (lb*in^2)

γ = shear bond coefficient

$E_S A_S$ = axial stiffness of sheathing (lb)

a_1 = distance between sheathing centroid and CFSWSPC centroid (in)

$E_J I_J$ = bending stiffness of joist (lb*in^2)

$E_J A_J$ = axial stiffness of joist (lb)

a_2 = distance between joist centroid and CFSWSPC centroid (in)

Table 6-6 Test Series 1 Effective Stiffness

Test Series 1	
Effective Stiffness	Reference
$E_s I_s$= 166666.7 lb*in^2	(NDS 2005, Table M9.2-2)
γ= 0.003067	
$E_s A_s$= 5533333 lb	(NDS 2005, Table M9.2-2)
a_1= 1.44802 in	
$E_J I_J$= 51910000 lb*in^2	(AISI CFS Design Manual 2008)
$E_J A_J$= 9976000 lb	(AISI CFS Design Manual 2008)
a_2= 1.80198 in	
$(EI)_{eff}$ 8.45E+07 lb*in^2	
*Assuming 16" spacing of joists	

Table 6-7 Test Series 2 Effective Stiffness

Test Series 2	
Effective Stiffness	Reference
E_sI_s= 166666.7 lb*in^2	(NDS 2005, Table M9.2-2)
γ= 0.012155	
E_sA_s= 5533333 lb	(NDS 2005, Table M9.2-2)
a_1= 1.835774 in	
E_JI_J= 82940000 lb*in^2	(AISI CFS Design Manual 2008)
E_JA_J= 16124000 lb	(AISI CFS Design Manual 2008)
a_2= 1.414226 in	
$(EI)_{eff}$ 1.16E+08 lb*in^2	
*Assuming 16" spacing of joists	

Table 6-8 Test Series 4 Effective Stiffness

Test Series 4	
Effective Stiffness	Reference
E_sI_s= 166666.7 lb*in^2	(NDS 2005, Table M9.2-2)
γ= 0.009878	
E_sA_s= 5533333 lb	(NDS 2005, Table M9.2-2)
a_1= 1.542179 in	
E_JI_J= 1.39E+08 lb*in^2	(AISI CFS Design Manual 2008)
E_JA_J= 28014000 lb	(AISI CFS Design Manual 2008)
a_2= 1.707821 in	
$(EI)_{eff}$ 2.21E+08 lb*in^2	
*Assuming 16" spacing of joists	

The effective bending stiffness of the composite member compared to the bending stiffness of the cold-formed steel member alone (E_JI_J) is notably larger. For example, for test series 4, the effective stiffness of the cold-formed steel member alone is only $1.39\text{x}10^8$ lb*in^2, while the composite CFSWSPC is much higher at $2.21\text{x}10^8$ lb*in^2. The bending stiffness of the CFSWSPC is 1.59 times greater than the individual cold-formed steel member alone. The

bending stiffness of the cold-formed steel member is currently all that is used in design. It can be seen then that CFSWSPC systems are much more economical than a design based on the CFS member alone.

Conclusion

Recommended values for the slip modulus are 140 lb/in/in for 33 mil cold-formed steel, 560 lb/in/in for 54 mil cold-formed steel and 640 lb/in/in for 97 mil cold-formed steel.

The bending stiffness is increased by an average factor of 1.54 when comparing the composite member to the cold-formed steel member alone. This is a significant increase and warrants the values of the slip modulus to be used to increase the strength of the system in bending. While full composite action is not present, the values of the slip moduli indicates that shear forces are transferred through the connection and thus the partial composite action is significant. Through these tests and in conjunction with the previous study conducted by Matsen Ford Design Associates, it is clear that it is appropriate to use the composite action to improve the accuracy of the design of CFSWSPC.

Limitations

The limitations of this study are as follows.

- Each of the materials was only supplied from one source. While all the materials are standardized, it is possible that a difference of storage conditions at each of the sources could affect the results.
- This study's intent was only to check strength parameters. The effects of vibrations have not been taken into account. The effects of vibrations could be critical in floor systems, especially if they are made lighter due to the increase in strength available for the CFSWSPC.
- This thesis only used one loading rate to determine the slip modulus. Different rates of loading may yield different results.
- Only one screw spacing (12") was tested. Further tests using smaller spacing may yield higher slip modulus values.

- The number of tests per test series was small. In order to narrow the confidence interval, more tests must be performed. A more narrow confidence interval would likely increase the lower bound and allow for a larger slip modulus value to be recommended.

Recommendations for Further Research

Further study should be conducted to support the values that have been recommended for the slip modulus. Additionally, expanding the variables of similar experiments to determine their effects on the slip modulus is suggested. Other possible variables to study further include, but are not limited to, sheathing thickness, cold-formed steel thickness, loading rate, screw type, screw size, screw spacing. Ultimately, further study would confirm a method for determining the slip modulus of a given CFSWSPC construction type.

Further study should also include developing a method for determining the effective flange width of the t-beam that is assumed in CFSWSPC. Currently, the entire flange width is used, but this assumption needs to be verified.

Based upon slip modulus values, a composite factor must be developed for design. The composite factor will allow for standardization in design and an increase of available strength in cold-formed steel wood structural panel systems. Increasing this strength of the system will allow the weight of the system to decrease, which may cause vibration issues for floor system design. Analysis of the vibration of the lighter weight floor system should be conducted.

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