INVESTIGATION OF THE SLIP MODULUS BETWEEN COLD-FORMED STEEL AND PLYWOOD SHEATHING

by

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Abstract

Cold-formed steel members quickly are becoming a popular material for both commercial and residential construction around the world. Their high strength to weight ratio makes them a viable alternative to timber framing. In most cases cold-formed steel is used as a repetitive member in floor, wall, or roof assemblies. Structural sheathing is used in conjunction with the framing members in order to transfer loads between individual members. This sheathing is connected mechanically to the cold-formed steel through a variety of methods. The most common method uses screws spaced at close intervals, usually between 6 to 12 inches on center. When such assemblies are constructed, load is transferred from the sheathing through the connectors into the cold-formed steel, forming a composite assembly in which load is transferred and shared between two materials, providing a higher strength and stiffness over individual members themselves. The amount of load that can be transferred is dependent on the amount of slip that occurs when the assembly is loaded. This slip value describes the amount of composite action that takes place in the assembly. The amount of slip can be described by a value called the slip modulus. The composite, or effective, bending stiffness can be calculated using the slip modulus. In current design of cold-formed steel composite assemblies this composite action is not being taken into account due to a lack of research and understanding of the composite stiffness present in these assemblies. Taking composite action into account can lead to decreased member sizes or increased spacing of members, thereby economizing design. Furthermore, improved understanding of the effective stiffness can lead to more accurate design for vibrations in floor systems. This thesis tests cold-formed steel plywood composite members in an effort to verify previously established slip modulus values for varying steel thicknesses and establishes new values for varying fastener spacings. The slip modulus values obtained are used to calculate effective bending stiffness values in an effort to prove that composite action should be utilized in design of cold-formed steel composite assemblies.

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Dedication

This is dedicated to my parents, Pat and Kent Martin, the rest of my family, and all of my friends here at school and back at home. I would not be here today without their continued support and encouragement.

Chapter 1 - Introduction

Cold-formed steel is a popular building material in both residential and commercial construction. While cold-formed steel has been around since the 1800s, its use in buildings increased in the 1940s after the first United States specification was published in 1946 (Macdonald, 2008). It can be used to construct floors, walls, and roofs that are both lightweight and stiff. Several advantages over use of timber members for the same applications can be cited. Compared to timber members, cold-formed steel is impervious to termite, fungus and moisture degradation, has a much higher strength-to-weight ratio, and is 100% recyclable.

Cold-formed steel is most commonly used as a repetitive member in assemblies containing other materials. Plywood, gypsum board, or oriented strand board (OSB) may be attached to a series of cold-formed steel members as a load-distributing element referred to as sheathing. This sheathing works as a load transfer mechanism to carry the loads between the closely spaced steel members. The sheathing is attached to the steel by means of mechanical fasteners, often screws, spaced at close intervals ranging anywhere from 6 to 18 inches. Load is first seen by the sheathing and then transferred through the fasteners to the steel members, which in turn take the load out to their supporting elements. The load is transferred from the sheathing to the framing member through a variety of means. Composite action, load sharing, and residual capacity all account for additional strength of the assembly to varying degrees.

Composite action describes the amount of slip between two connected materials when load is induced. The slip between the two connected materials will decrease as the amount of connection increases. Furthermore, the load carrying capacity of the assembly will be increased. In the case of cold-formed steel plywood composite assemblies, this composite action is not being taken into account in design. A floor, wall or roof system designed with cold-formed steel is based on the capacity of a single member. This is a conservative approach and does not accurately reflect the actual stiffness of the floor system. By taking composite action into account, member sizes may be decreased or member spacing may be increased. This will lead to less overall material used in construction and will thus save on construction material and labor costs.

Vibration in floor systems is a large concern with light-framed construction. Vibration is dependent on the overall effective stiffness of the system being considered. Since vibration is a serviceability limit state, building codes specify a deflection limit criteria based on the span of the members, and do not take into account the stiffness of the system. This may not accurately reflect how the system will behave given its fundamental natural frequency. The fundamental natural frequency of the floor system describes the lowest frequency at which a structure will vibrate after a sudden impact load. Once the amount of composite action present in cold-formed steel timber composite structures is established and a design method has been developed that takes this composite behavior into account, floor vibration predictions can be calculated more accurately.

Previous studies by Matsen Ford Design Associates (Chan et al, 2009) and Northcutt (2012) were conducted in order to establish an approximate slip modulus value for various fastener and steel thickness conditions. In this study, tests were conducted in order to further establish slip modulus values for various cold-formed steel thicknesses, and to establish new values for varying fastener spacing. Effective stiffness values were calculated based on these slip values to show that composite action is present in cold-formed steel timber composite assemblies, and should be accounted for in design.

Chapter 2 - Literature Review

The overall goal of this research is to verify slip modulus values found previously by Northcutt and Matsen Ford Design Associates and to find slip modulus values for different fastener spacing. Many factors such as material thicknesses, fastener spacing, and ultimate load all play a role in the determination of the slip modulus and its impact on overall building construction. Composite action, repetitive member assemblies, and floor vibrations were addressed prior to research in order to fully understand the impact of the slip modulus on behavior and design.

Composite Action

A full understanding of composite action is required in order to better present the research of a slip modulus determined for cold-formed steel timber composite assemblies. Composite action has been researched greatly using other materials. Timber-timber, concrete-timber, and concrete-steel are just a few examples of other types of composite systems. Studying their behavior can help in understanding the mechanics of different materials combined in a composite system.

Two different materials are often mechanically connected in building construction. These two materials can behave differently under the same load due to both material and geometric properties. The two materials will help transfer loads between one another depending on how much mechanical connection is present, giving additional strength to the assembly. According to Salmon, "Composite action is developed when two load-carrying structural members…are integrally connected and deflect as a single unit (Salmon, 2009)." As load is applied to the two members in flexure, the bottom fiber of the top member will elongate under tension stress as the top fiber of the bottom member will try to shorten under compression. This difference in stresses creates horizontal shear, which causes slip to occur between the two members. The amount of slip that occurs between the two members describes the amount of composite action. Higher slip values indicate less force transfer and therefore less composite action. A single linear strain relationship from the top fiber of the compression member to the bottom fiber of the tension member describes fully composite action. This linear strain results in a single neutral axis for the two members. This means no slip will occur between the two materials. Figure 2-1 shows a graphic representation of this relationship.

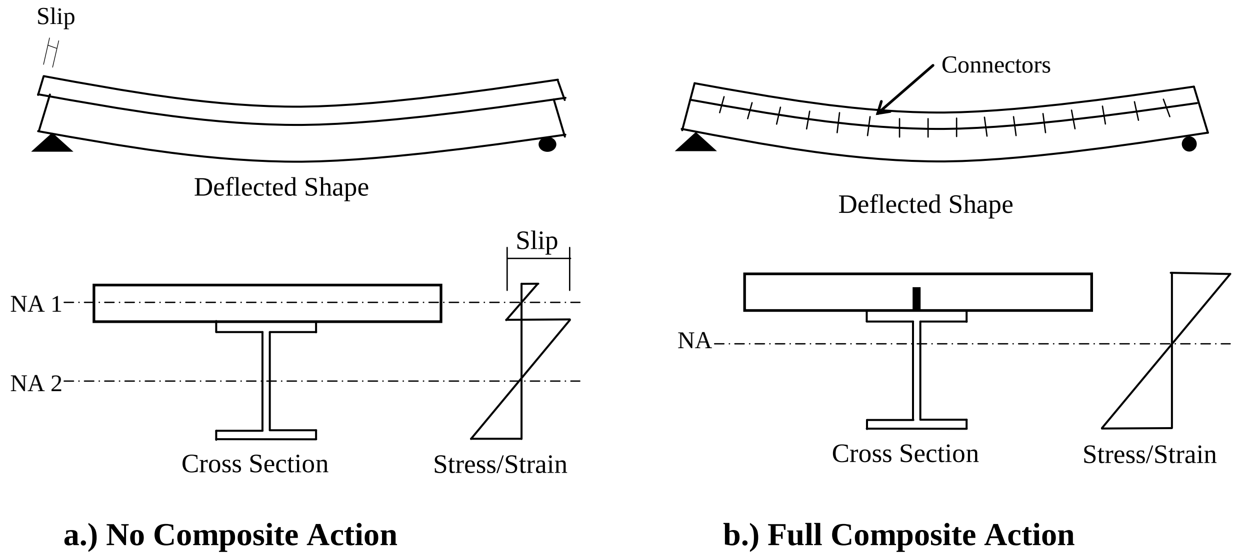

Figure 2-1 Effect of Composite Action on Beam Loaded in Flexure

Steel-Concrete Composite

Often in construction concrete is placed as a floor slab on top of a steel framing system, sometimes by itself and other times placed on top of metal deck. Steel shear studs are welded to the top flange of the steel beam and the concrete is placed around them. As vertical load is applied, the two members will deflect separately. Shear stresses will develop between the slab and beam as the surfaces try to elongate and compress, respectively. The shear studs help to transfer this force between the slab and beam. The strength and stiffness of the floor system are both increased when composite action is utilized. This increased strength and stiffness gained by the concrete slab leads to a reduction in both weight and depth of the beam required for a given span. According to Salmon, a savings in steel weight between 20-30% can be achieved by taking advantage of composite action.

The stiffness of a floor system utilizing composite construction is greater than that of the stiffness of a floor system considering the concrete and steel separately. The moment of inertia is greatly increased by considering the strength of the concrete in both the parallel and perpendicular directions with respect to the steel beams. This stiffness is affected by the width of the concrete considered to be effective in resisting the loads, known as the effective flange width. Effective flange width is theoretically based on elastic plate bending theory, but in design is simplified by the American Institute of Steel Construction (AISC) to be the smaller of the beam spacing or half the beam span for an interior beam.

Cold-Formed Steel Timber Composites

Another form of composite construction is cold-formed steel members connected by wood structural panels. These cold-formed steel members are framed parallel to one another at close intervals much like traditional wood framing, commonly spaced at 16” or 24” on center. Wood panels are mechanically attached to the framing members. Wood structural panels can be plywood, laminated veneer lumber (LVL), and oriented strand board (OSB), as well as a variety of other types. The American Forest and Paper Association (AF&PA) National Design Specification (NDS) describes wood structural panels as “wood-based products that have been rated for use in structural applications (NDS 2005).” In current design of cold-formed steel timber composite systems, composite action is not being accounted for (Northcutt, 2012). The amount of composite action present in an assembly will affect the stiffness and vibration properties of the system. If the additional stiffness is not being accounted for, the floors may not behave in the manner that is expected. In an effort to determine the amount of composite action present in various cold-formed steel timber assemblies, several studies have been conducted, two in regards to finding the slip modulus between the two materials. The slip modulus is a value that represents the stiffness of a connection between two different materials. Higher slip modulus values increase the rigidity of the assembly and therefore the effective moment of inertia. Increasing the moment of inertia will increase the assembly stiffness, leading to higher strength capacities. Higher available strength can lead to smaller member sizes, greater member spacings, or even greater spans.

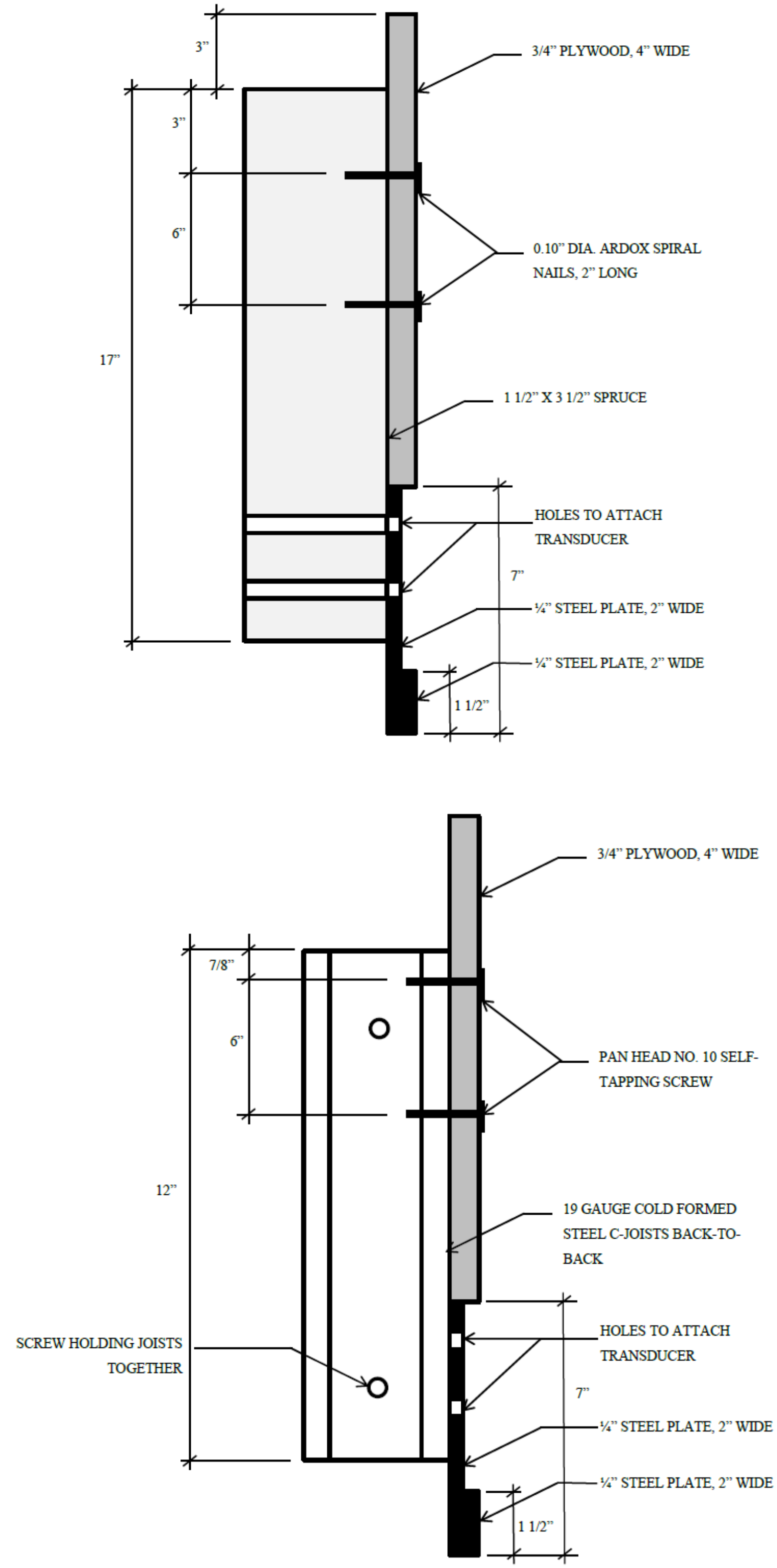

Figure 2-2 Timber-Timber and Cold-Formed Steel-Timber Test Setups (Northcutt, 2012)

Matsen Ford Design Associates conducted a study in 2009 to determine the slip modulus for cold-formed steel timber composite floor assemblies. The goal of the study was to determine the slip modulus in an effort to enhance design for vibration of cold-formed floor assemblies. Two test series were conducted. The first series tested timber-timber composites to establish a baseline for the test procedure. The second series tested cold-formed steel timber composites. Each series tested three different connection types. Nails or screws, glue, and nails/screws together with glue were each tested for their effects on the slip modulus. The test specimen setup for the timber-timber and the cold-formed steel timber test series are shown in Figure 2-2. The timber-timber specimens were constructed using 2x4 joists attached to ¾" plywood. Nails were 0.1 in (2.5 mm) diameter with ardox-spiral threads hammered into the wood. For tests involving glue, a ¼" bead was applied and cured for three days. In the nail and glue specimens nails were applied after the glue dried. The cold-formed steel-timber composite specimens used back to back 4" C-shaped steel studs and ¾" plywood. Screws were #10 self-drilling screws with square socket heads. For specimens utilizing glue, the glue was applied in the same manner as for the timber-timber composite specimens.

Tests were carried out in a tension manner (pull tests) to better simulate behavior in flexure. The loading rate used was based on International Organization for Standardization (ISO) standard 6891 (1983). The tests were conducted in the following manner.

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 slip of 0.591 in (15 mm)

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

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

ii) Maintain load for 30 seconds

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

iv) Maintain load for 30 seconds

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

The slip modulus as defined by ISO 6891 is calculated in Equation 2-1. In the Matsen Ford Design Associates study, the slip modulus was normalized to reflect the effect of the number and spacing of fasteners on the slip modulus. The normalized slip modulus is given in Equation 2-2.

Equation 2-1 Slip Modulus

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

Equation 2-2 Normalized Slip Modulus

$$K_N = \frac{K}{ns}$$

K = Slip Modulus from Equation 2-1 (lb/in)

$0.4P_u$ = 40% of ultimate load (lb)

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

n = number of fasteners

s = fastener spacing

The values obtained by Matsen Ford Design Associates were shown to be significantly affected by fastener spacing in both the timber-timber and cold formed steel-timber composite tests. For the cold-formed steel timber composite, the slip modulus decreased from 1135 to 693 lb/in/in when the fastener spacing increased from 6” to 12”. For connections involving screws and glue, the converse was observed. As the screw spacing increased, the slip modulus increased as well. An increase from 1121 to 1370 lb/in/in was observed when spacing went from 6” to 12”. The authors attributed this to the fact that while the fasteners were spaced further more glue could be applied, which provides more composite action than fasteners alone. The values taken as the slip modulus for each connection type were lower bounds, and therefore conservative. For screw connections, K_N=650lb/in/in and for glue and screw connections K_N=1100lb/in/in. These values are for the cold-formed steel timber composite assemblies. Table 2-1 summarizes these results.

Table 2-1 Chan Recommended Normalized Slip Modulus Values

Connection		Slip Modulus K_N (lb/in/in)	Recommended K_N (lb/in/in)
Type	Spacing (in)		
Screw	12	693	650
	6	1135	
Screw & Glue	12	1370	1100
	6	1121	

In the study titled "Slip Modulus of Cold-Formed Steel Members Sheathed With Wood Structural Panels" by Northcutt in 2012 the slip modulus for cold-formed steel timber composites was studied further in an effort to verify the results of the Matsen Ford Design Associates study and to establish further values for various steel thicknesses. The fasteners used were #10 TEKS self-drilling screws, spaced at a constant 12" for every test series. Specimens were constructed using two-foot sections of 6" deep C-shaped cold-formed steel members with a 24"x6"x1/2" thick plywood member attached to each flange of the cold-formed steel with two screws spaced 12" apart. One exception to the above description was the variation in the plywood thickness for tests involving 97 mil (12ga) steel. For these tests, plywood thickness was increased to 23/32" thick in an effort to replicate standard construction practice. A typical test specimen is shown in Figure 2-3. Cold-formed steel thicknesses tested were 33mil, 54mil, 68mil, and 93mil (20ga, 16ga, 14ga, and 12ga, respectively). At the time of the study 43mil (18ga) steel was not available.

Tests were conducted in a manner similar to those conducted by Matsen Ford Design Associates with the exception of the direction of loading. Northcutt used a push test rather than a pull test. The loading procedure and rate were in accordance with ISO 6891 in which an estimated failure load was established by testing the first specimen to failure. Three specimens were tested for each test series and the slip deformation curves were recorded. Failure modes observed during testing were primarily tilting of the screws which was caused by bearing failure of the plywood and was therefore a slower failure. However, in the 12ga test series screw shear was observed which was sudden. The author attributed this to the significant increase in both steel and plywood thicknesses, which caused the screws to fail in shear prior to any plywood bearing failure.

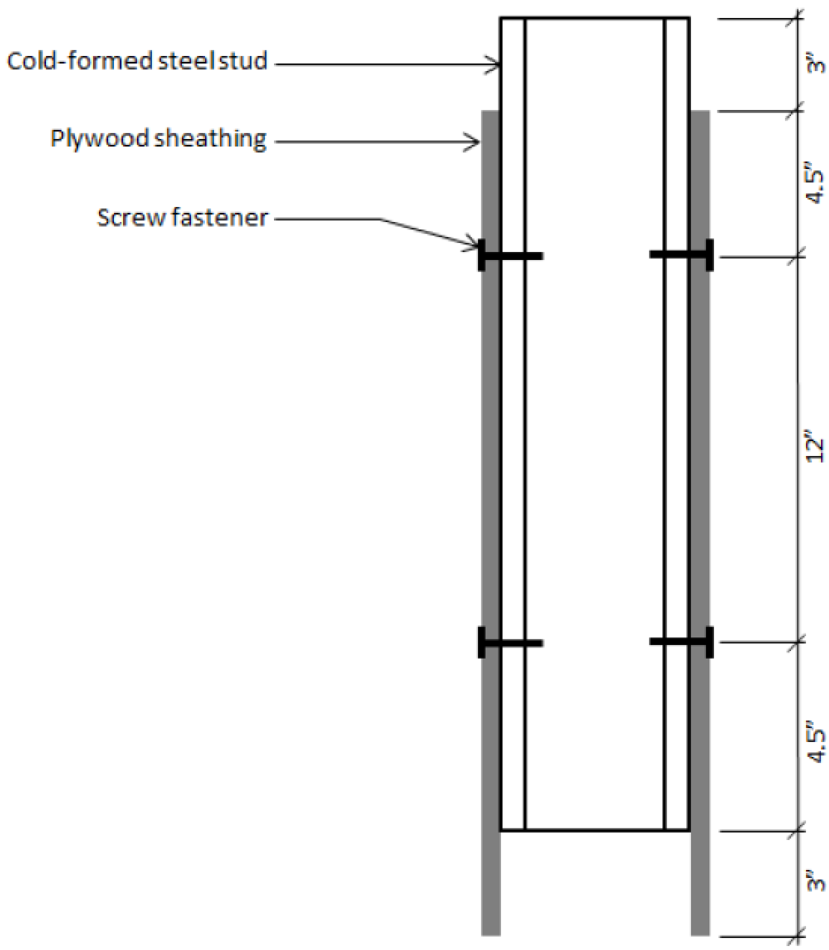

Figure 2-3 Typical Test Specimen Setup (Northcutt, 2012)

Results of the study appeared consistent with the hypothesis that as the thickness of the cold-formed steel member increases, the slip modulus will also increase. The normalized slip modulus was calculated based on recorded slip values using Equations 2-1 and 2-2. After the normalized slip was calculated for each specimen, results were statistically averaged and recommended values were developed. Values from Test Series 3, 68mil (14ga), were disregarded due to excess deviation in the three samples. Recommended values for the remaining three series are given in Table 2-1.

Table 2-2 Northcutt Recommended Normalized Slip Modulus Values

Test Series	Steel Thickness	Plywood Thickness	Recommended K_N (lb/in/in)
T1	33mil (20ga)	1/2"	140
T2	54mil (16ga)	1/2"	560
T4	97mil (12ga)	23/32"	640

Using these recommended values, the shear bond coefficient was calculated in order to find the effective stiffness of the composite material. The shear bond coefficient is found using Equation 2-3 and the effective stiffness is found using Equation 2-4.

Equation 2-3 Shear Bond Coefficient

$$\gamma = \frac{1}{1 + \frac{\pi^2 s E_s A_s}{K L^2}}$$

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)

Equation 2-4 Effective Bending Stiffness

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

$(EI)_{eff}$ = 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 composite 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 composite centroid (in)

The effective bending stiffness of the three steel thicknesses was calculated based on a floor system consisting of joists spaced at 16 inches on center and spanning ten feet. Every test series showed a large percentage increase in the effective stiffness when composite action was considered. Results are summarized in Table 2-2.

Table 2-3 Increase in Effective Bending Stiffness Values

Test Series	Steel Thickness	$E_J I_J$ (lb*in^2)	$(EI)_{EFF}$ (lb*in^2)	Percent Increase
T1	33mil (20ga)	5.19E+07	8.45E+07	63%
T2	54mil (16ga)	8.29E+07	1.16E+08	40%
T4	97mil (12ga)	1.39E+08	2.21E+08	59%

Results of the Northcutt study show that when composite action between the cold-formed steel and plywood was taken into account, the effective bending stiffness of the floor system showed an average 54% increase. Northcutt recommends further investigation to confirm results as well as varying parameters such as fastener spacing, size, or type, sheathing thickness, and loading rate in order to determine a method for determining the slip modulus. Once slip modulus values and a calculation method have been established, a composite factor for use in design needs to be developed and standardized. Current design of cold-formed floor systems only takes the flexural stiffness of the joist itself into account. Member spacing may be increased or depth decreased with the increase in stiffness from consideration of composite action. Furthermore, vibration calculation results may vary with the different stiffness values for the same system and will need to be investigated further.

Repetitive Member Assemblies

It is important to discuss how load is shared and transferred between members when they are combined in series. A member can be considered a part of a repetitive member system by the 2005 AF&PA National Design Specifications (NDS) as "…members used as joists, truss chords, rafters, studs, planks, decking, or similar members which are in contact or spaced not more than 24" on center, are not less than three in number and are joined by floor, roof or other load distributing elements adequate to support the design load." The NDS provides a factor to account for strength gained by members in a repetitive assembly. Such repetitive member factors were studied by Clayton in "Repetitive Member Factor Study for Cold-Formed Steel Framing Systems (2010)". Clayton first investigated the repetitive member effects on wood assemblies since they have been established and are utilized in design and then expanded to determine a factor for cold-formed steel, which is currently not being utilized in design. Load sharing, residual capacity, and composite action all help provide extra strength to repetitive member assemblies to different degrees.

Load Sharing

Load sharing in wood assemblies is dependent on the high variability of strength and stiffness properties between individual pieces of lumber. Consider a floor assembly of three sawn lumber members connected by sheathing. If one of these members is weaker in bending, it will deflect more than the other more stiff members. In turn, more load will be transferred to the stiffer members, until they deflect the same amount as the weaker member. This describes the concept of load sharing. The overall strength of the assembly increases as compared to the individual strength of the weaker member since the two stronger members help carry load and add capacity. Assembly size plays an important role in how much load transfer occurs between stronger and weaker members. As the number of members in an assembly increases, the probability of weaker members also increases, leading to a higher probability of faster first member failure. This decreases the overall strength of the assembly according to Clayton. The main component of load sharing is a concept known as mutual restraint. The stiffness of the load distributing element, or sheathing, correlates to the ability of the individual members in the assembly to deflect together. Two theoretical models are discussed by Clayton. The first model contains an infinitely rigid load distributing element. The members will deflect together due to the rigidity of the sheathing, and the weakest member will fail first. This will cause the other members to take on the load from the weaker member, thereby increasing the strength of the system. The second model uses an infinitely flexible sheathing element. Here, the members will deflect individually, and therefore the strength of the system will equal the strength of the weakest member. Actual real-world assemblies lie somewhere in between these two systems, meaning that mutual restraint will provide some amount of strength increase.

Repetitive Member Factor

After investigating repetitive member factors for timber construction, Clayton determined a factor for cold-formed steel assemblies sheathed with oriented strand board (OSB). A composite section of a 6 inch deep cold-formed steel stud connected to a 1/2 inch thick piece of OSB with a flange width of 16 inches was used to calculate the ratio of allowable composite moment to allowable moment of the stud alone. Using the transformed area method, the composite section properties were calculated and an allowable bending moment was calculated. The ratio of the composite moment to the stud moment was determined to be 1.27. These calculations assumed full composite action and no slip between the two materials. After investigating the composite action effects, load sharing effects were investigated. Steel is known to have much less variance in properties from member to member. However, enough material variance was found to exist that a load sharing factor of 1.027 was found. An overall recommended preliminary repetitive member factor of between 1.14 and 1.29 was established, with limitations given by the author based on several factors. First, the calculations performed did not take connection slip or sheathing gaps into account, and are therefore upper bounds. Second, the sheathing is assumed to be in compression. Connections must be screws, and sheathing must be wood-based (OSB or plywood, not gypsum board).

Vibrations in Cold-Formed Steel Floor Systems

An important parameter in determining vibration characteristics of floor systems is the relative stiffness of the system. Vibration is a serviceability limit state and is largely subjective, with different people having different opinions on acceptable levels of vibration. Because of this, vibration is very hard to design for. Different building codes have limits in place for deflections due to various loading conditions, but vibration itself is not covered. The National Association of Home Builders (NAHB) limits the deflection of a span of length L to L/480 under uniform live loads to control vibration (Xu, 2007). This limit was based on timber floor construction. As floor systems become lighter, vibration concerns increase.

In a study by Kraus, in conjunction with NAHB, titled “Floor Vibration Design Criterion For Cold-Formed C-Shaped Supported Residential Floor Systems (1997)”, four different design criteria were compared with regards to provisions controlling deflection and vibration response of floor systems. The study focused on criteria for wood framed floor systems. The following four criteria were compared: the Swedish Building Technology Design Guide equations developed by Sven Ohlsson, the Australian Standard Domestic Framing Code, United States Proposed Timber Floor Vibration Criterion developed by Johnson, and the Canadian Timber Floor Criterion developed by Donald Onysko. After the vibration criteria were established, twelve full-sized floors were constructed to determine vibration acceptability and a method for predicting the fundamental frequencies of cold-formed steel floor systems. Testing involved impact loading, concentrated loading, and walking both parallel and perpendicular to the framing direction.

Joist Size (inxinxmil)	10 psf Dead + 30 psf Live Spacing (in)			10 psf Dead + 40 psf Live Spacing (in)		
	12	16	24	12	16	24
2x6x33	11'-6"	10'-5"	8'-8"	10'-5"	9'-6"	7'-9"
2x6x43	12'-6"	11'-4"	9'-11"	11'-4"	10'-4"	9'-0"
2x6x54	13'-5"	12'-2"	10'-8"	12'-2"	11'-1"	9'-8"
2x6x68	14'-4"	13'-1"	11'-5"	13'-1"	11'-10"	10'-4"
2x6x97	15'-11"	14'-5"	12'-7"	14'-5"	13'-1"	11'-5"
2x8x33	14'-9"	12'-9"	8'-10"	13'-2"	10'-7"	7'-1"
2x8x43	16'-11"	15'-4"	13'-3"	15'-4"	13'-11"	11'-10"
2x8x54	18'-2"	16'-6"	14'-5"	16'-6"	15'-0"	13'-1"
2x8x68	19'-6"	17'-8"	15'-5"	17'-8"	16'-1"	14'-0"
2x8x97	21'-7"	19'-7"	17'-2"	19'-7"	17'-10"	15'-7"
2x10x43	20'-4"	17'-10"	14'-7"	18'-5"	15'-11"	13'-0"
2x10x54	21'-10"	19'-10"	17'-4"	19'-10"	18'-0"	15'-9"
2x10x68	23'-5"	21'-4"	18'-7"	21'-4"	19'-4"	16'-11"
2x10x97	26'-1"	23'-8"	20'-8"	13'-8"	21'-6"	18'-9"
2x12x54	25'-6"	23'-2"	19'-0"	13'-2"	20'-9"	17'-0"
2x12x68	27'-4"	24'-10"	21'-9"	24'-10"	22'-7"	19'-9"
2x12x97	30'-5"	27'-8"	24'-2"	27'-8"	25'-1"	21'-11"

Table 2-4 NAHB Prescriptive CFS Joist Span Table (Adapted from Kraus, 1997)

During design, an accurate prediction of mid-span deflection is needed. Several methods and equations were discussed and compared with respect to predicted versus measured deflection. The Steel Joist Institute (SJI) American Institute of Steel Construction (AISC) both developed equations to determine the number of joists or beams effective in resisting concentrated loads. The number of effective joists depends on the distance from the joist having the applied load to the joist with zero deflection. The SJI equation was developed for beam spacing less than 30 inches, while the AISC equation was for spacing greater than 30 inches. The NAHB developed prescriptive tables for the design of cold-formed steel floor systems with regards to allowable deflection for various member sizes, spans, and loading. Table 2-3 shows the allowable spans for various joist sizes and has been reproduced from Kraus' thesis. The allowable spans are based on deflection criteria of L/480 for live loads and L/240 for dead and live loads (total).

After all vibration and deflection criteria and design methods were compared with laboratory testing results, Kraus developed a proposed design method that took the most suitable parts from each method. The proposed design procedure is as follows.

1. **Equation 2-5** Calculate critical central floor deflection based on Onysko criteria (Canadian criteria)

   $$y_{crit} = \frac{37.32}{L^{1.3}}$$

   y_{crit} = critical central floor deflection (inches)

   L = floor span (inches)

2. **Equation 2-6** Calculate predicted deflection of one joist due to a concentrated midspan load of 225 pounds.

   $$A_{ot} = \frac{225L^3}{48EI}$$

   A_{ot} = predicted single joist deflection from 225 lb concentrated load at midspan (in)

   E = joist modulus of elasticity (psi)

   I = joist moment of inertia (in^4)

3. **Equation 2-7** Calculate the number of effective joists from SJI Equations

   $$N_{eff} = 1 + 2\Sigma\left(\cos\frac{x\pi}{2x_o}\right)$$

   N_{eff} = number of effective joists

   X = distance from center joist to joist under consideration (in)

   x_o = distance from center joist to the edge of effective floor = 1.06εL (in)

   L = joist span (in)

$\varepsilon = (D_x/D_y)^{0.25}$

D_x = flexural stiffness perpendicular to joists = $E_c t^3/12$ (lb*in)

D_y = flexural stiffness parallel to joists =EI_t/S (lb*in)

E_c = modulus of elasticity of sheathing (psi)

E = modulus of elasticity of joist (psi)

T = sheathing thickness (in)

I_t = moment of inertia of joist alone (in^4)

S = joist spacing (in)

4. **Equation 2-8** Calculate predicted central floor deflection.

$$A_o = \frac{A_{ot}}{N_{eff}}$$

A_o = predicted central floor deflection (in)

A_{ot} = predicted single joist deflection from 225 lb midspan concentrated load

N_{eff} = number of effective joists

5. **Equation 2-9** Compare predicted deflection to critical deflection

If $A_o < y_{crit}$;	Acceptable
If $A_o > y_{crit}$;	Unacceptable
If $y_{crit} < A_o \leq 1.1\, y_{crit}$;	Marginal

It can be seen from the equations above that composite action is not taken into account. Kraus tested five specimens comprised of two cold-formed steel C-shaped joists spaced 24 inches apart with OSB sheathing attached to the top flanges. Loads were applied to the joists alone prior to sheathing attachment, and then applied to the system with sheathing. Measured deflections were used to calculate the composite moment of inertia, which was then used to calculate the effective width of the OSB sheathing. This effective width ranged from 1 to 4 inches. Kraus concluded that "…the OSB cannot be relied upon to provide a significant amount of composite action for design purposes. Thus, the moment of inertia that should be used in the calculation of deflection and frequency is simply the bare joist moment of inertia."

Chapter 3 - Test Setup and Procedure

Test Plan

In order to further study the slip modulus of cold-formed steel timber composites, both CFS thickness and connector spacing were varied and tested. Two separate test series were conducted. The first series kept the fastener spacing and plywood sheathing thickness constant while the steel thickness varied. The second series examined the effect of fastener spacing on the slip modulus values. Each test group contained a total of three specimens and one preliminary specimen, making a total of four for each group. The preliminary test specimen was used to determine the estimated ultimate load for that group. Tables 3-1 and 3-2 show configurations for both test series.

Table 3-1 Test Series 1-Steel Thickness

	Steel		Sheathing	Screw	Screw
Series	Size	Gage	Thickness	Size	Spacing
T43	43 mil	18	1/2"	#10	12
T54	54 mil	16			
T97	97 mil	12			

Table 3-2 Test Series 2-Fastener Spacing

	Steel		Sheathing	Screw	Screw
Series	Size	Gage	Thickness	Size	Spacing
TF-6	43 mil	18	1/2"	#10	6
TF-8					8
TF-10					10
TF-12					12

Test Specimens

Specimens comprised of two pieces of plywood, measuring six inches wide by 24 inches long, attached to one 24 inch metal stud section. The geometry of the test specimens aims to limit eccentricities during loading. The loading conditions are explained below. The plywood was attached using #10 TEKS self-drilling screws. The number of screws depended on the fastener spacing being tested. Two small metal angles were attached to the plywood on each side of a specimen using wood screws. A strain pod was attached to each metal angle. A small hole was drilled into the top of each stud and a screw threaded into the hole, one on each side. The strain pod string was hooked around the small screw at the top using nylon fishing wire. This allowed for the slip between the steel and plywood to be accurately measured. Two strain pods were used in order to provide more accurate measurement. Figure 3-1 is an example of a specimen prior to testing, showing the configuration of the strain pod attachment. Figure 3-2 shows the measurements of specimens with the four different fastener spacings.

Figure 3-1 Strain Pod Attachment

Plywood used was 1/2” birch, and was higher quality than would typically be used in floor construction. At the time of material acquisition, this was the only option in the thickness desired. The increased strength was not reflected in stiffness calculations and should be a consideration in future testing. Cold-formed steel used was 600S162, meaning 6” deep structural grade metal studs with 1.625” flange depth. As explained earlier, three different thicknesses were used depending on the test series, but all studs were 600S162.

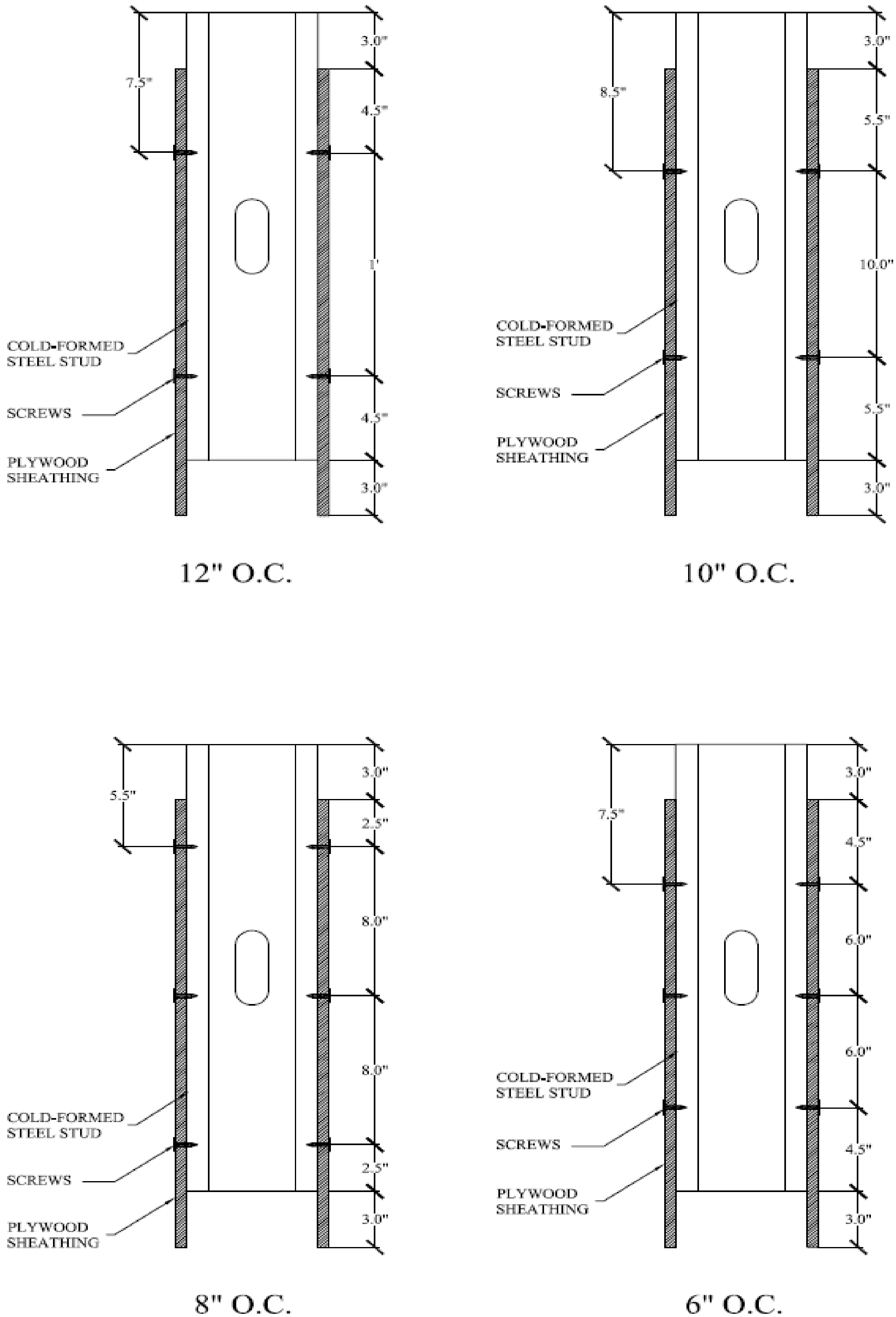

Figure 3-2 Test Specimen Setup for Varying Fastener Spacing

Test Procedure

Push tests were conducted in a manner similar to Northcutt's testing in order to produce comparable results. The test procedure complied with ISO 6891. The push test was carried out by a Shimadzu AG-IC, shown in Figure 3-3. The machine applied load to the steel section through a loading plate at the top of the specimen, shown in Figure 3-4. Each test series had one trial specimen in order to determine an estimated failure load, P_{est}. Load was applied to the trial specimen at a rate of 0.0394 inches/minute (1 mm/min) until specimen failure. A value of 0.591 in (15 mm) of slip was used as a maximum for failure in the event the specimen did not fail prior to 0.591 in of slip. This value was used by both Northcutt and Chan et al and is defined in the ISO 6891 standard. Further testing was conducted in a stroke-controlled manner, using a rate of 0.0394in/min similar to the trial specimen. Tests were carried out using the following procedure.

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 slip
2.) Estimate the load at which failure will occur in the future specimens, P_{est}, based on 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 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 is considered when the screw slips more than 15 mm. If the difference between P_u and P_{est} is more than 20% of P_{est}, a new ultimate load must be estimated and used for the next specimen. If the difference is less than 20%, continue to step 5.
5.) Plot the load and displacement curve
6.) Determine the slip modulus
7.) Compare results to previous study

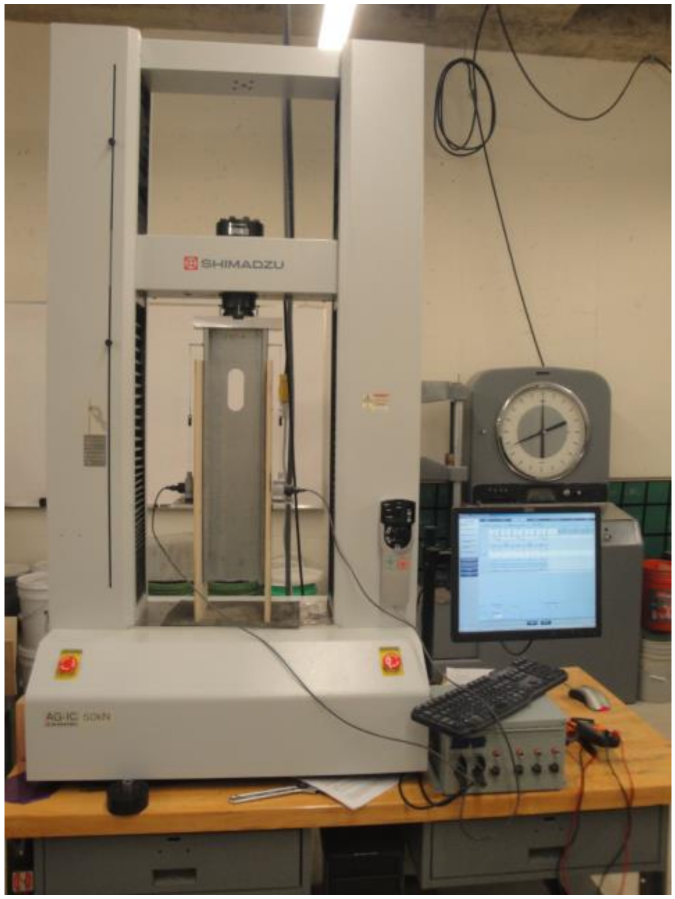

Figure 3-3 Shimadzu Testing Machine

Figure 3-4 Loading Plate

Data Measurement and Recording

Each strain pod measured the slip between the cold-formed steel and plywood on the side it was attached to. The strain pods were connected to the Shimadzu machine through a computer interface. The loading pattern was put into an integrated computer program called Trapezium which controlled the machine and recorded measurements taken by the strain pods. Measurements were taken every two pounds. The data was extracted after completion of each test and analyzed to ensure the ultimate load was within 20% of the estimated load. Two strain pods were used in order to provide more accurate results, as one side of a specimen may slip more than the other at a given time. The values were then averaged for each specimen and plotted against force. Results are discussed in the following chapter.

Chapter 4 - Test Results

Test results are summarized in Table 4-1 and show the critical values for calculation of the normalized slip modulus along with the failure mechanism observed.

Table 4-1 Summary of Test Results

Test Series	Steel Gage (mils)	Plywood Thickness	Fastener Spacing	Fasteners Per Specimen	Specimen	Max Force (lb)	Max Slip (in)	40% P_{est} (lb)	Slip @ 40% P_{est} (in)	Failure Mode
T43	43	1/2"	12	4	T43 A	3226	0.646	1280	0.045	Screw Tilting
	43	1/2"	12	4	T43 B	3267	0.619	1280	0.082	Screw Tilting
	43	1/2"	12	4	T43 C	3421	0.541	1280	0.055	Screw Tilting
T54	54	1/2"	12	4	T54 B	3194	0.696	1320	0.082	Screw Tilting
	54	1/2"	12	4	T54 C	3390	0.508	1320	0.091	Screw Tilting
	54	1/2"	12	4	T54 D	3454	0.641	1320	0.059	Screw Tilting
T97	97	1/2"	12	4	T97 B	3377	0.686	1460	0.054	Screw Tilting
	97	1/2"	12	4	T97 C	2776	0.265	1460	0.042	Screw Tilting
	97	1/2"	12	4	T97 D	3278	0.517	1460	0.041	Screw Shear
TF6	43	1/2"	6	6	TF6 B	3320	0.227	1600	0.064	Plywood
	43	1/2"	6	6	TF6 C	3659	0.247	1600	0.042	Plywood
	43	1/2"	6	6	TF6 D	4627	0.523	1600	0.073	Plywood
TF8	43	1/2"	8	6	TF8 B	4609	0.396	1940	0.061	Plywood
	43	1/2"	8	6	TF8 C	4654	0.650	1940	0.081	Plywood
	43	1/2"	8	6	TF8 D	5013	0.660	1940	0.088	Plywood
TF10	43	1/2"	10	4	TF10 B	3614	0.796	1320	0.082	Screw Tilting
	43	1/2"	10	4	TF10 C	3173	0.723	1320	0.065	Screw Tilting
	43	1/2"	10	4	TF10 D	3107	0.718	1320	0.113	Screw Tilting
TF12	43	1/2"	12	4	T43 A	3226	0.646	1280	0.045	Screw Tilting
	43	1/2"	12	4	T43 B	3267	0.619	1280	0.082	Screw Tilting
	43	1/2"	12	4	T43 C	3421	0.541	1280	0.055	Screw Tilting

The following figures show the displacement vs. load curves for each specimen, with each graph representing one test series. The graphs for Test Series T43 and Test Series T12 are combined because they share the same specimen configuration. For variation of the steel thickness, screws were spaced at 12”, and for variation of fastener spacing, 43 mil steel was used.

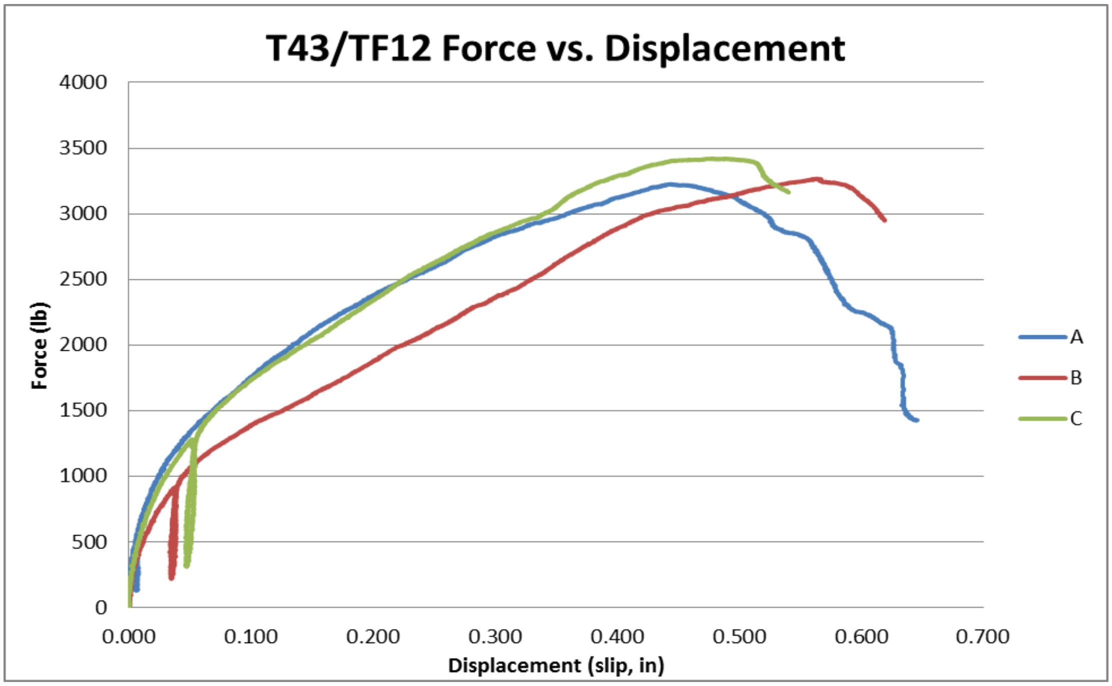

Figure 4-1 Test Series T43/TF12 Force vs. Displacement

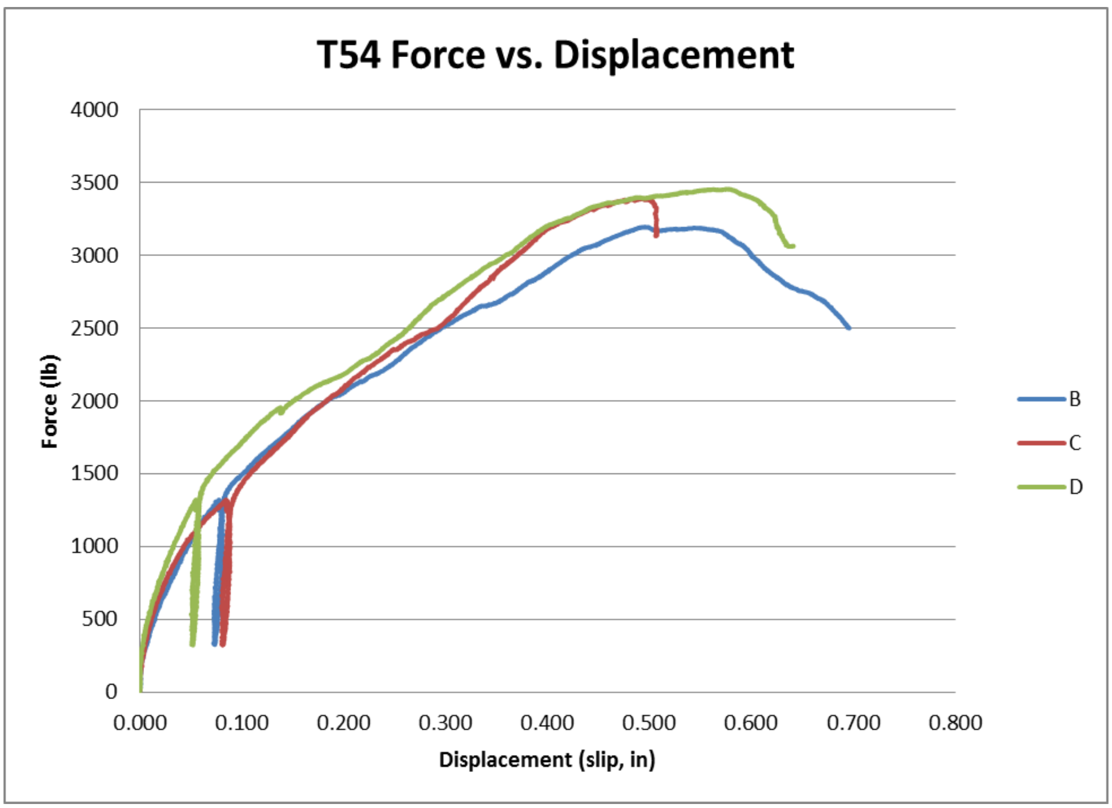

Figure 4-2 Test Series T54 Force vs. Displacement

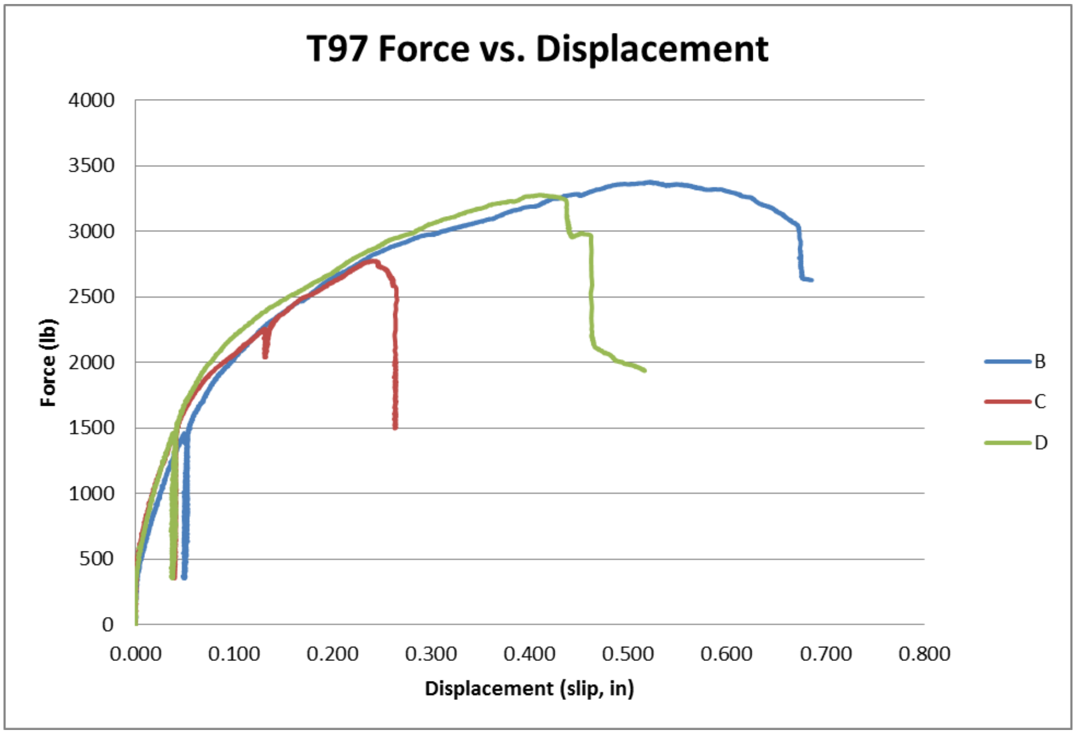

Figure 4-3 Test Series T97 Force vs. Displacement

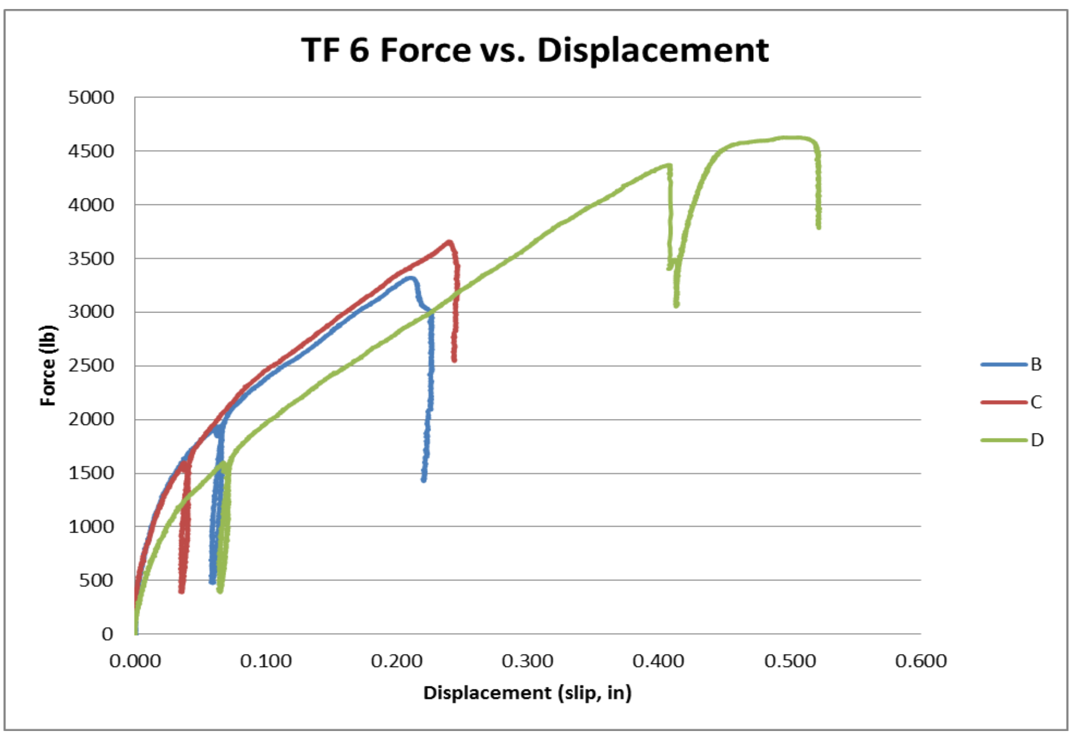

Figure 4-4 Test Series TF6 Force vs. Displacement

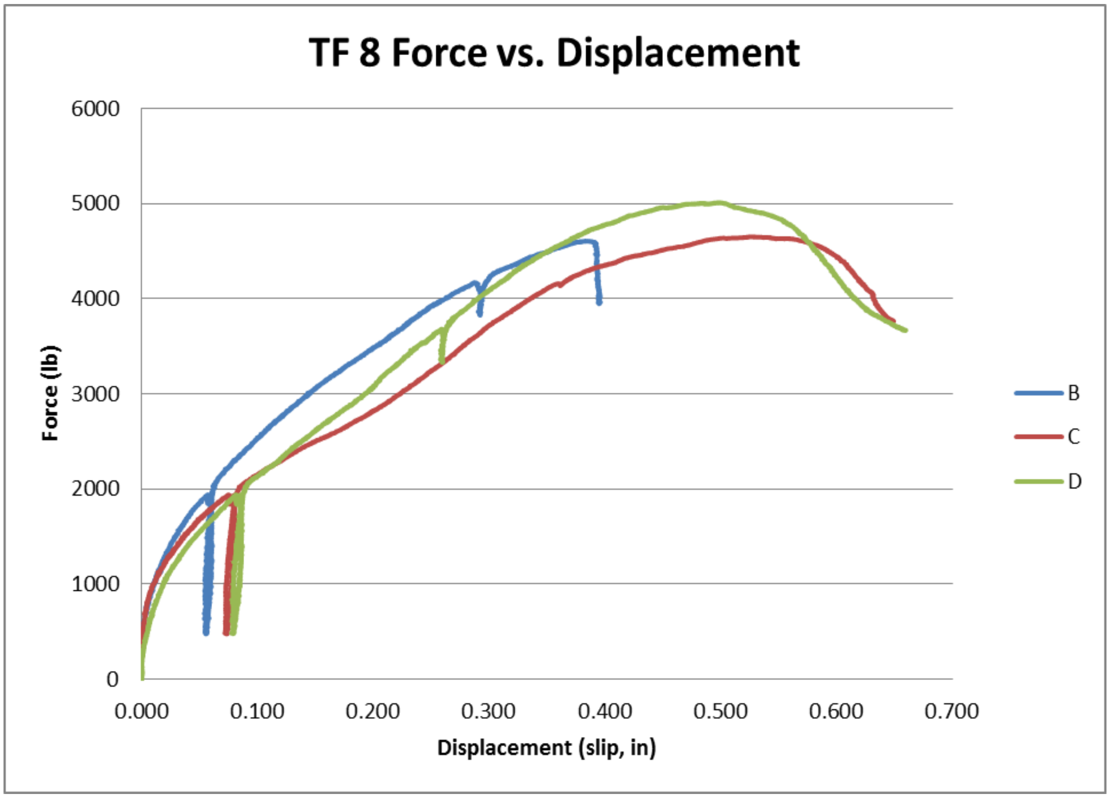

Figure 4-5 Test Series TF8 Force vs. Displacement

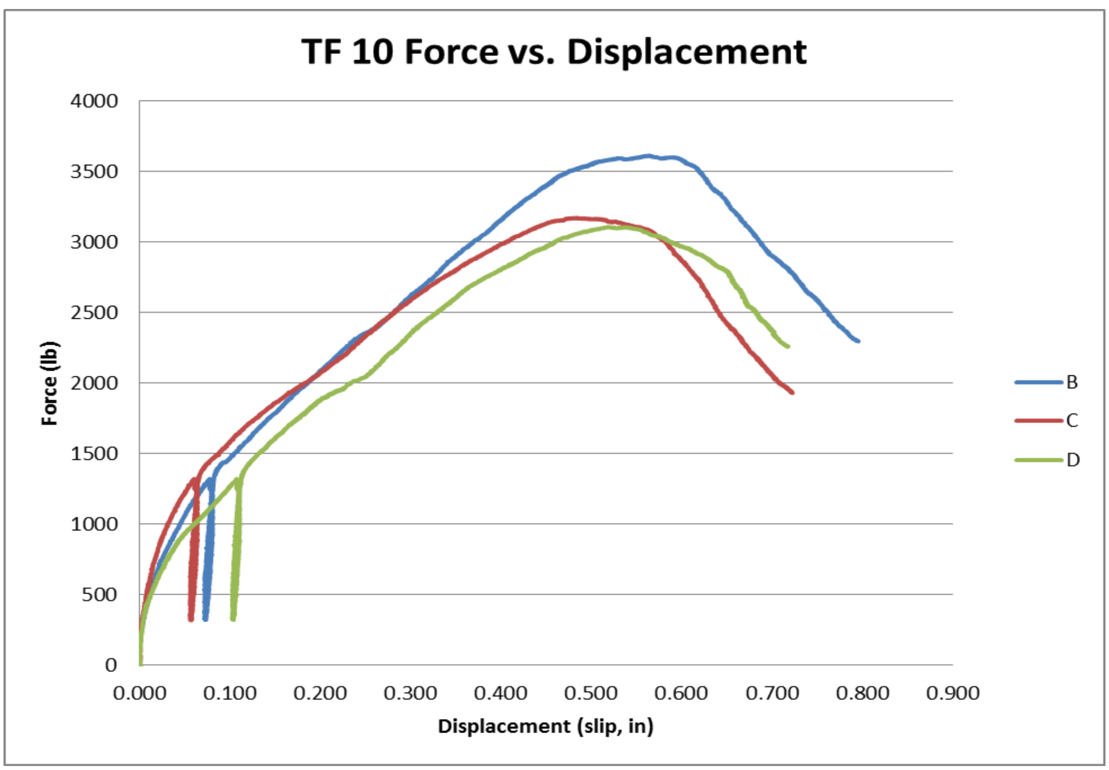

Figure 4-6 Test Series TF10 Force vs. Displacement

Table 4-1 shows a summary of the slip modulus values for each test specimen organized by test series, as well as statistical data for each series. The confidence interval was calculated based on a 95% two tailed normal distribution. The slip modulus K is calculated using Equation 2-1 and then normalized to find K_N using equation 2-2, repeated here for convenience. Values were calculated based on data in Table 4-1. Results and comparisons are discussed in the following chapter.

Table 4-2 Slip Modulus Values and Statistical Data

Test Series	Slip Modulus K (lb/in)	Normalized Slip Modulus K_N (lb/in/in)	Standard Deviation σ	Median	Mean	COV	Confidence Interval Lower Bound
T43	28444 15610 23273	593 325 485	134.53	485	468	0.288	315
T54	16098 14505 22373	335 302 466	86.66	335	368	0.236	270
T97	27037 34762 35610	563 724 742	98.41	724	676	0.145	565
TF6	25000 38095 21918	694 1058 609	238.60	694	787	0.303	517
TF8	31803 23951 22045	663 499 459	107.75	499	540	0.199	418
TF10	16098 20308 11681	402 508 292	107.84	402	401	0.269	279
TF12	28444 15610 23273	593 325 485	134.53	485	468	0.288	315

Equation 2-1 Slip Modulus

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

Equation 2-2 Normalized Slip Modulus

$$K_N = \frac{K}{ns}$$

K = Slip Modulus from Equation 2-1 (lb/in)

K_N = Normalized Slip Modulus (lb/in/in)

$0.4P_u$ = 40% of expected ultimate load (lb)

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

n = number of fasteners

s = fastener spacing (in)

Chapter 5 - Conclusion

Discussion of Results

Failure Modes

Failures observed consisted mostly of screw tilting, which corresponds to crushing of the plywood under the bearing stress of the screws. As load was applied to the specimen, the screws began tilting due to plywood bearing failure and slip occurred between the steel and plywood. In some cases, the screws would be tilted so much the head would become totally withdrawn into the plywood after failure. Figure 5-1 shows a typical screw tilting failure and screw head withdrawal into the plywood. One specimen experienced screw shear. This was in the thickest steel tested, 97 mil (12 ga), and can be seen in Figure 5-2. This failure mode was expected given the increase in thickness of steel. Prior to testing, screw shear was expected to occur in this test series based on results of Northcutt's study. All specimens that utilized 97 mil steel experienced screw shear in her study. It should be noted that Northcutt increased the plywood thickness from 1/2" to 23/32" in order to simulate common construction practices. This increase in plywood thickness likely lead to greater strength against crushing, causing screw failure to control over plywood failure. In this study, the plywood remained at 1/2", and was weaker than that observed in the Northcutt study and thus failed prior to screw shear in most cases.

Figure 5-1 Screw Tilting and Withdrawal Failure

Figure 5-2 Screw Shear Failure

Figure 5-3 Plywood Crushing Failure in TF8 Specimen

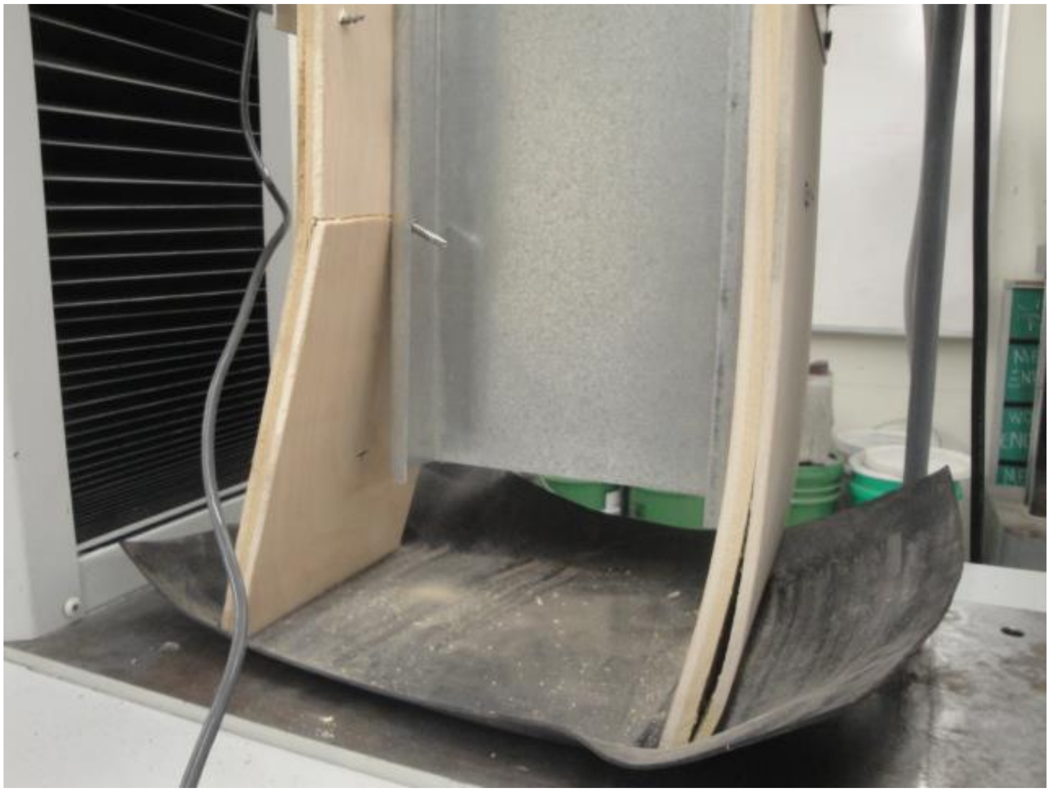

Figure 5-4 Plywood Bending/Cracking Failure in TF6 Specimen

Figure 5-5 Lesser Screw Tilt/Withdrawal in TF6 Specimen

Comparison of Results

In Test Series F6 and F8, which had three screws spaced at 6" and 8" respectively, plywood crushing and bending at the bottom of the specimen was observed. From Figures 5-3, 5-4, and 5-5 it can be seen that less screw tilt occurred because the bottom pieces of plywood crushed and bent. This suggests that for decreased fastener spacing, design will be controlled by the strength of the plywood. Full loading was transferred from the steel through the screws and down into the plywood, which failed prior to high values of screw slippage. Table 4-1 shows maximum load and maximum slip values for both TF6 and TF8 test series. Test Series TF6 showed lower ultimate loads when compared with TF8, but also showed much lower maximum slip values. TF6 had an average maximum load of 3784 lb, while TF8 had an average ultimate load of 4759 lb. Average maximum slip values were 0.332 in and 0.569 in for TF6 and TF8 respectively. This comparison lends to the hypothesis that for closer screw spacing slip will be reduced, thus increasing composite behavior. While TF8 specimens had higher ultimate loads, the higher slip values show that less load was taken by the connections causing the plywood around the screws to fail, rather than the whole specimen deflecting as one composite unit. The failures in series TF6 show that the specimens deflected more as a unit until specimen failure occurred, rather than connection failure. This is reflected in the calculation of the slip modulus for each series. Series TF6 had a greater normalized slip than series TF8, supporting the hypothesis that as fastener spacing decreases, slip modulus values decrease, thereby increasing the shear bond coefficient and the effective bending stiffness.

Test series TF10 showed lower slip modulus values than TF8, but also lower than series TF12. TF10 had lower ultimate loads than both TF12 and TF8, and also higher maximum slip values. Poor specimen construction is one possible explanation for the inconsistent results obtained. Further testing with fasteners spaced at 10 inches should be conducted in order to determine a more accurate slip modulus.

Table 5-1 shows slip modulus values obtained in previous studies by Chan et al and Northcutt compared to values obtained in this study, organized by specimen comparison. The Matsen Ford Design Associates (Chan et al) study utilized 19 gage cold-formed steel studs, which is fairly close to the 18 ga/43 mil steel used in this study. Plywood used by Chan et al was 3/4" thick while plywood in this study was 1/2" thick. For fasteners spaced at 12" apart, Chan et al obtained a normalized slip modulus value of 693 lb/in/in while this study determined a value of 315 lb/in/in. The slightly thicker cold-formed steel and much thicker plywood used by Chan are potential reasons for the difference between the two values. At the time of Northcutt's study, 43 mil steel was not available. However, she did test 33 mil (20 ga) and 54 mil (16 ga). Slip modulus values for these two thicknesses were 140 lb/in/in and 560 lb/in/in respectively. The value of 315 lb/in/in obtained in this study for 43 mil (18 ga) lies roughly in the middle of Northcutt's values. This suggests a relatively linear relationship between steel thickness and the slip modulus.

Table 5-1 Comparison of Results to Previous Studies

Study	Steel Thickness	Fastener	Spacing	Plywood Thickness	K_N (lb/in/in)
Northcutt	16 ga.	#10 TEKS	12	1/2"	560
Martin	16 ga.	#10 TEKS	12	1/2"	270
Northcutt	12 ga.	#10 TEKS	12	23/32"	640
Martin	12 ga.	#10 TEKS	12	1/2"	565
Chan	19 ga	#10 TEKS	12	3/4"	693
Martin	18 ga	#10 TEKS	12	1/2"	315
Chan	19 ga	#10 TEKS	6	3/4"	1135
Martin	18 ga	#10 TEKS	6	1/2"	517

This linear relationship did not hold true in this study for the 54 mil test series. A decrease in the slip modulus was observed when the thickness increased from 43 mils to 54 mils. Poor specimen construction could be one possible explanation for this outcome. The centers of the pieces did not always align perfectly with the centers of the cold-formed steel, causing skewed or uneven bases. Values obtained in this study are about half of those obtained by Northcutt. A recommended value of 560 lb/in/in was given for 54 mil cold-formed steel, while the lower bound obtained in this study was 270 lb/in/in. Due to these lower-than-expected results, no recommendation is given for the slip modulus for 54 mil (16 ga) cold-formed steel timber composite assemblies.

Values for Test Series T97 compared well with those obtained in Northcutt's study for 97 mill steel. The lower bound obtained in this study for the normalized slip modulus was 565 lb/in/in. Northcutt recommended a value of 640 lb/in/in. In Northcutt's study, the plywood thickness was increased for the 97 mil tests from 1/2” to 23/32”. This study kept the plywood thickness constant at 1/2”. The thicker plywood in Northcutt’s study could be one reason for the increased slip modulus value, as it was not a drastic increase. If the previously mentioned linear relationship were to hold true, the slip modulus for 97 mil steel and 1/2” plywood would be roughly 980 lb/in/in. This amount of increase was not observed in either Northcutt’s or this study, however, which suggests a nonlinear relationship. As the steel thickness increases, the slip modulus is controlled by the strength and thickness of the plywood rather than the steel. This is supported by the lower slip modulus value found in this study corresponding to the less thick plywood used.

Effective Stiffness

Based on the slip modulus values obtained, the effective bending stiffness of a cold-formed steel timber composite assembly can be calculated. The effective bending stiffness represents the amount of composite action present in a composite assembly when load is induced on the specimen in flexure, which will be greater than the bending stiffness of a cold-formed steel joist alone. The effective stiffness takes into account both the axial stiffness and the flexural stiffness of both the sheathing and the cold-formed joist, and calculates stiffness with respect to the distance between the centroid of each component and the composite centroid. The effective stiffness will be calculated based on an assembly configuration of cold-formed steel joists spaced at 16” on center spanning 10 ft, which could represent a typical floor, roof or wall assembly configuration. First, the shear bond coefficient must be calculated based on Equation 5-1. The effective bending stiffness can then be calculated using Equation 5-2. Plywood axial stiffness and flexural stiffness values (EA_S and EI_S) were obtained from the AF&PA National Design Specifications (NDS) 2005 Table M9.2-2 and were adjusted to reflect an effective width of 16 inches. Cold-formed steel joist axial stiffness and flexural stiffness values were obtained from the American Iron and Steel Institute (AISI) Manual for Cold-Formed Steel Design 2008 edition.

Equation 5-1 Shear Bond Coefficient

$$\gamma = \frac{1}{1 + \frac{\pi^2 s E_s A_s}{KL^2}}$$

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)

Equation 5-2 Effective Bending Stiffness

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

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

EI_S = bending stiffness of sheathing (lb*in^2)

γ = shear bond coefficient

EA_S = axial stiffness of sheathing (lb)

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

EI_J = bending stiffness of joist (lb*in^2)

EA_J = axial stiffness of joist (lb)

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

Table 5-2 Shear Bond Coefficient

Series	K_N (lb/in/in)	s (in)	EA_S (lb)	L (in)	γ
T43/TF12	315	12	5533333	120	0.007
T54	270	12	5533333	120	0.006
T97	565	12	5533333	120	0.012
TF6	517	6	5533333	120	0.022
TF8	418	8	5533333	120	0.014
TF10	279	10	5533333	120	0.007

Table 5-3 Effective Bending Stiffness

Series	γ	Sheathing			Joist			$(EI)_{eff}$ ($lb*in^2$)	% Increase
		EA_S (lb)	EI_S ($lb*in^2$)	a_1 (in)	EI_J ($lb*in^2$)	EA_J(lb)	a_2 (lb)		
T43/TF12	0.007	5.53E+06	1.67E+05	2.278	6.73E+07	1.38E+07	0.972	8.07E+07	19.98%
T54	0.006	5.53E+06	1.67E+05	2.420	8.29E+07	1.61E+07	0.830	9.44E+07	13.84%
T97	0.012	5.53E+06	1.67E+05	2.714	1.39E+08	2.80E+07	0.536	1.48E+08	6.26%
TF6	0.022	5.53E+06	1.67E+05	2.278	6.73E+07	1.38E+07	0.972	8.12E+07	20.63%
TF8	0.014	5.53E+06	1.67E+05	2.278	6.73E+07	1.38E+07	0.972	8.09E+07	20.26%
TF10	0.007	5.53E+06	1.67E+05	2.278	6.73E+07	1.38E+07	0.972	8.07E+07	19.99%
								Avg=	**16.83%**

Tables 5-2 and 5-3 summarize the shear bond coefficient and effective bending stiffness calculations for each test series. As can be seen in Table 5-3, the effective bending stiffness is shown to be increased by an average of 16.8% over the stiffness of a cold-formed steel joist alone. This is a markedly smaller percentage increase when compared to Northcutt's study. Although this study found a smaller increase, an increase was still observed. This supports the hypothesis that composite behavior is present in cold-formed steel timber composite assemblies, and that this stiffness can be taken into account in design of floor, roof, and wall assemblies constructed from cold-formed steel members sheathed with plywood.

Conclusions

Cold-formed steel timber composite assemblies do exhibit composite action. All test series in both the steel thickness and fastener spacing parameters showed an increase in the effective stiffness. Further research should be conducted so that this composite action can be taken into account in design of floor, wall, and roof assemblies of cold-formed steel sheathed with plywood or other wood structural panels.

Increases in cold-formed steel thickness lead to an increase in the slip modulus and thus the effective stiffness of cold-formed steel timber composite assemblies. This relationship does not appear to be linear. As the thickness increases, the rate of increase in the slip modulus decreases.

Fastener spacing seems to have a large effect on the slip modulus of cold-formed steel timber composites. Decreasing the spacing of screws increases the slip modulus and thus the effective stiffness of these assemblies. For fasteners spaced at 6 inches, plywood strength seems to control failure. Fasteners spaced at 8 inches may or may not be controlled by plywood strength and should be tested further to determine the controlling design parameter.

Specimen construction quality played a role in accuracy and consistency of results. Real-world construction may result in flaws. This research could be a good representation of a lower bound for the amount of composite action in flawed construction. Future research should consider real-world construction practices.

Recommendations for Further Research

Future research on this subject should test with larger sample sizes per test series. As only three specimens per series were tested in both this study and Northcutt's study, one specimen could throw off the statistical data quite a bit, leading to inaccurate averages and higher coefficients of variation. In order to truly develop an idea of the slip modulus value for various steel thicknesses and fastener spacings, at least 15 specimens per series should be tested to provide more accurate results.

Specimen construction quality should be more controlled in future testing. Some accidental eccentricities were present in this study that lead to premature failure or uneven loading, which produced more varied results. A different specimen configuration could also be beneficial for tests in which plywood failed first, as in series TF6 and TF8.

Further testing should consider the effects of the plywood on the failure of cold-formed steel timber composite assemblies. Plywood species, span rating, and thickness should be varied and tested to see if a relationship exists between these parameters and the slip modulus.

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