TESTS TO DETERMINE THE JOINT STIFFNESS AND RESISTING MOMENT CAPACITY OF JOINTS IN TIMBER-FRAMED HOUSES

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Abstract Wind and earthquake disasters in the world often cause billions of dollars of damage to buildings, most of which is to non-engineered buildings-especially homes. Wood-frame homes use connectors which exhibit partial rotational rigidity at the member ends, making analytical solutions difficult. This is due to the lack of information about the complex behavior of the connections used in this type of buildings, and the lack of a methodology to analyze light-timber frame buildings with flexible joints. In order to address the problem, tests were carried out to determine the moment rotation characteristics of wood joints made of conventional and mechanical fasteners. During testing, it was discovered that rotational stiffness of timber connections is a function of the axial load as well as the shear load. Comparison of ordinary timber connections with metal connectors were made. Resisting moment capacity of joints with different connectors were also determined. This paper presents a new method of testing for joint stiffness and the experimental results obtained during testing for stud-to-bottom-plate, rafter-to-ridge-board, and top-plate-to-rafter connections using different connectors.

Key Words Timber, Joints, Connections, Moment-Resisting Capacity Tests, Rotational Stiffness.

چکیده هر ساله زلزله و طوفانهای شدید، میلیاردها دلار خسارت به ساختمانها، بخصوص مسکونی غیر مهندسی، وارد می سازند. این سازه ها غالباً از اتصالاتی استفاده می کند که دارای اتصالات نیمه گیردار بوده و رفتار آنها بستگی کامل به گیرداری آنها دارد. این امر، بخصوص در ساختمانهای چوبی که از اعضای چوبی و اتصالات ساده بهره می برند، بیشتر نمایانگر می باشد. با توجه به پیچیدگی رفتار این گونه سازه ها، آزمایشاتی برای تعیین درجه گیرداری این نوع اتصالات در سازه های چوبی طراحی گردید که این مقاله نتایج حاصله و همچنین روش آزمایشات را بطور کامل توضیح می دهد. در این آزمایشات سه نوع اتصال مورد بررسی قرار گرفت که هر اتصال برای اتصالات تشکیل یافته از میخ و همچنین اتصالات مکانیکی، آزمایش گردید. با عنایت به نتایج آزمایشگاهی، گیرداری انواع اتصالات چوبی معین و در قالب یک فرمول ریاضی برآورد شده است و در این مقاله ارائه می گردد.

INTRODUCTION

Each year, wind disasters in the world cause billions of dollars of damage to buildings, most of which is to non-engineered buildings--especially homes. Numerous post-disaster investigations have identified the weak links in these buildings [1]. However, rigorous research is needed not only to identify the weak joints but also to find effective solutions [2].

It is often claimed that non-engineered buildings such as homes made of timber are more difficult to analyze than engineered buildings. Perhaps, this is due to the lack of information about the complex behavior of the connections used in non-engineered timber buildings, and the lack of a methodology to analyze light-timber frame buildings with flexible joints.

In conventional analysis and design of engineered frame-works, it is customary to represent joint behavior by idealized joint models. Two of the most commonly used models are rigid-joint and pinned-joint models. For the rigid-joint model, it is assumed that the joints are infinitely rigid. This means the angles between the adjacent members remain unchanged as the frame deforms. For the pin-joint (hinged-Joint) model, it is assumed that no moments are transmitted at the joint. Although these two idealized models are simple to use and easy to implement in analysis and design, their validity for wood-frame houses are questionable. Wood-frame houses use connectors such as nails, straps, and toothed plates which offer partial resistance to moments and forces generated at joints subjected to lateral forces. Also, multiconnector timber joints are likely to transfer moments due to the very nature of their construction. Such structures exhibit partial rotational resistant at the member ends.

The purpose of this study was to test and determine the real behavior and the resisting moment capacity of joints used in timber-framed houses. The research includes laboratory tests of flexible joints of typical wood-frame houses so that the joint characteristics can be understood and used for predicting the response of such structures in analytical models [3].

LITERATURE REVIEW

Little work has been reported to date on the rigidity characteristics and ways to analyze light-frame timber structures with non-rigid joints. In 1972, Suddarth [4] developed a computer program, the Purdue Plan Structural Analyzer (PPSA), which performs rigid analysis of framed structures. Partial rigidity was handled by inserting fictitious, small elements of low stiffness in joints. The difficulty in this approach is selecting the properties of those fictitious elements.

Maraghechi and Itani [5], in 1982, reported on the influence of truss plate connectors on the analysis of light-framed structures. Structures were modeled using two types of elements, a beam element and a joint element-essentially the same as in the Purdue method. Joint properties were obtained experimentally. This method was applied to trusses and plane frames with toothed metal plate connectors. However, no attempt was made to model and analyze joints with more than two members, nail joints, and connectors other than toothed plates. The experimental procedures developed could only measure joint properties under pure tension, shear or moment loads without considering the combined effect of these loads on joint properties. Experimental tests pertain applicable only to toothed-plate connectors and not to other connectors.

In 1987, Gupta and Kuo [6] presented a model to predict the behavior of houses. A finite-element technique was used to model structures with nine degrees of freedom representing displacements of various connections. The model assumed nail properties and joint stiffnesses. Emphasis was placed on predicting the behavior of shear walls under uplift. The model was used to predict the behavior of an experimental building tested by Tuomi and McCutcheon [7] in 1978. Even though the final model results were in good agreements with the test data, the authors admitted the complexity involved and doubted the capability of the finite-element method. The model ignores the bending and axial deformations in the studs and the rotational stiffness of the stud joints. The same assumptions were made in modeling the roof system. This method seems complicated, and it adjusts joint behavior arbitrarily to fit experimental data.

Conner, et al. [8] reported the results of the test conducted on rafter-to-top-plate connections. These connections include 16d box nails, strapping, clip angles, extra nailing with pole barn nails and lag screws. In these tests average load capacities were determined while no attempts were made to establish the joint characteristics.

Nateghi and Liu [3,9,10] presented a flexible joint model to analyze the behavior of wood-frame houses under wind action, and some ways to determine the joint characteristic curves to be used in the model.

Canfield et al. [11] studied the uplift resistance of various rafter-wall connections. They tested several types of connectors and obtained the average uplift capacity of each connection ending with a design example which considered the joint as a pin connection. In general, very few studies have considered wood joints as semi-rigid connections and very little data are available on the rigidity of these connections.

STRUCTURAL IDEALIZATION--JOINT-STIFFNESS

An important consideration in predicting structural response is the behavior of joints. Tests conducted on steel structures have demonstrated that the moment transmitted by a flexible (semi-rigid) joint is a function of the relative rotation of the members as shown in Figure 1(a). A flexible joint can be presented by a spring of rotational stiffness--rigidity (K) shown in Figure 1(b) such that $M = K\O$, where M is the moment and $\O$ is the angle of the rotation of the spring. In this study, joints are idealized as rotational springs and the rigidity of these rotational springs are determined from the moment-rotation curve obtained from testing the joints. A linear behavior is assumed for determination of the joint stiffness.

EXPERIMENTAL PROGRAM

The objectives of the test program are: **a)** to provide information on the joint-stiffness of wood joints used in conventional construction of wood-frame houses, such as stud-to-bottom-plate, rafter-to-top-plate, and rafter-to-ridge board joints as shown in Figure 2, **b)** to collect data on the rotational stiffness of wood joints so that they can be used in structural analysis utilizing flexible joint model, and **c)** to compare the effectiveness of nail joints to joints using mechanical fasteners.

All joints were tested under increasing load until failure by using a servo-controlled MTS loading machine. The general schematic of the test setup is shown in Figure 3.

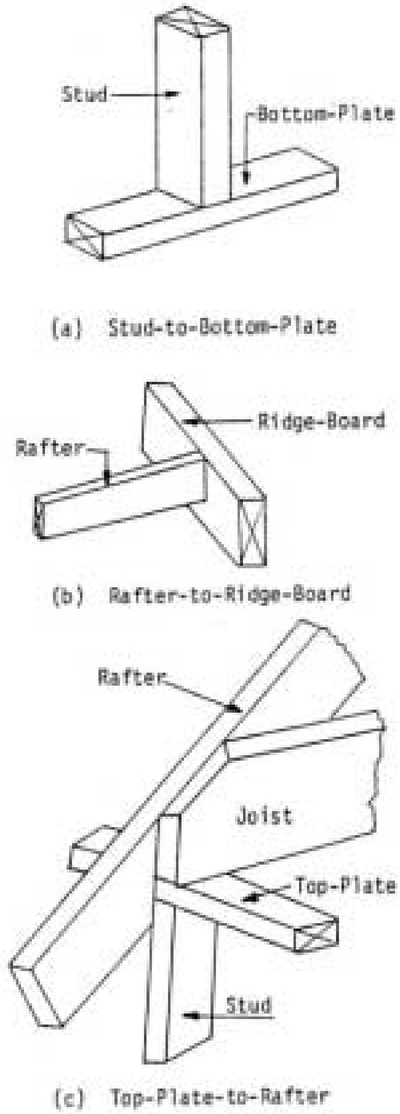


Figure 2. Test Joints.

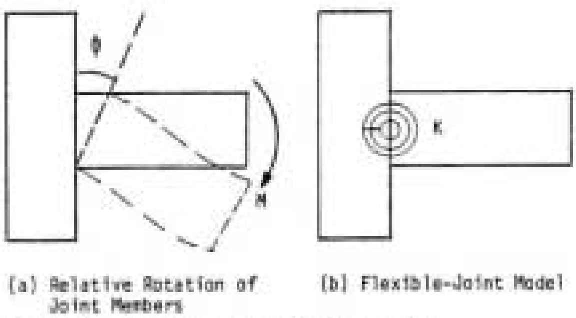


Figure 1. Joint-Rotation and Model Used.

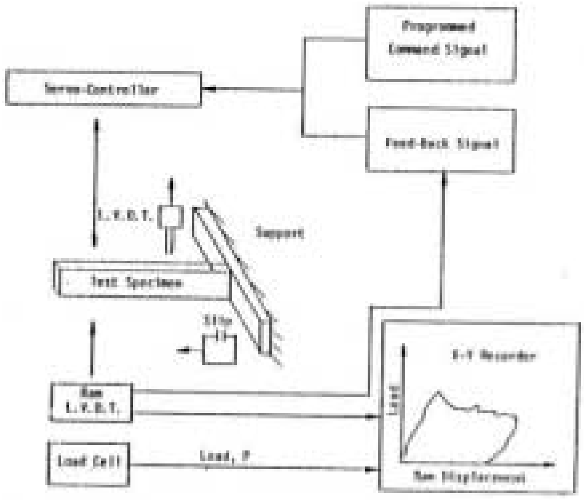


Figure 3. Schematic of Test Set up.

SPECIMEN DETAILS

The tests were conducted by using dissected joints. First, a control group of joints were constructed in accordance with the conventional practice sanctioned by the building codes. Construction details for conventional naillings are shown in Figure 4. These joints were tested under increasing load until failure occurred. Then, a new set of joints were constructed using the same lumber but with improved connectors. Construction details for mechanical fasteners are given in [12].

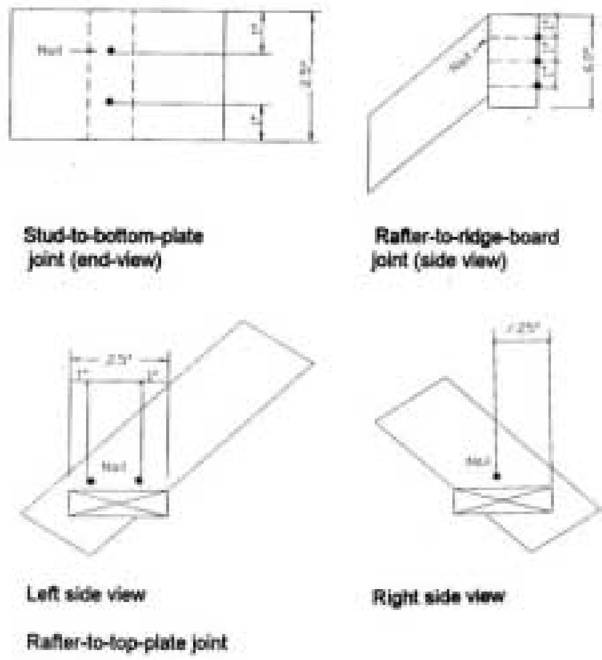


Figure 4. Construction Details of Nailed Joints.

Table 1 provides more information about the test joints. The lumber used for construction of test specimen was grade-2 Fir with an average specific gravity of 0.38 and water content of about 10 percent. The lumber was purchased from local lumber yard and cut to the desired sizes. Attempts were made to

Table 1. Joint Material and Connectors

Joint	Lumber Size		Connector
Stud-to-Bottom-Plate	Stud Bottom-Plate Angle Between	2"×4"×16" 2"×4"×16" 90°	2-16d Common Nail Simpson A35 Simpson A34
Rafter-to-Ridge-Board	Rafter Ridge-Board Angle Between	2"×4"× 16" 2"×4"×20" 72°	3-10d common Nail Simpson H3R Simpson H5R
Rafter-to-Top-Plate	Rafter Top-Plate Angle Between	2"×4"×20" 2"×4"×16" 18°	3-8d Common Nail Simpson H1R Simpson H4R
Joist-to-Top-Plate	Joist Top-Plate Angle Between	2"×4"×20" 2"×4"×16" 90°	3-8d Common Nail Simpson RR

use the parts of the lumber that had no knots or cracks during assembly. A typical dissected rafter-to-top-plate joint made of different connectors is shown in Figure 5. After each test, a piece of timber was cut and tested for specific gravity and water content. Metal clips and other connectors used in tests were made and provided by the Simpson Strong Tie Company. Various connectors which were used in the experimental program are shown in Figure 6.

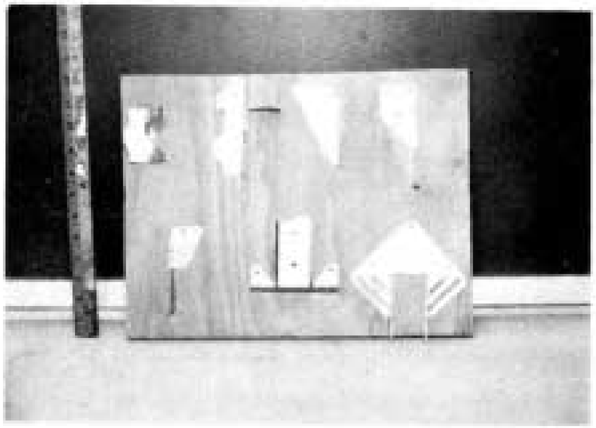


Figure 6. Various Connectors Used in Testing, from Left to Right and Top to Bottom, A34, A35, H4, H5R, H3R, RR and H1.

INSTRUMENTATION

Clip-on-gage

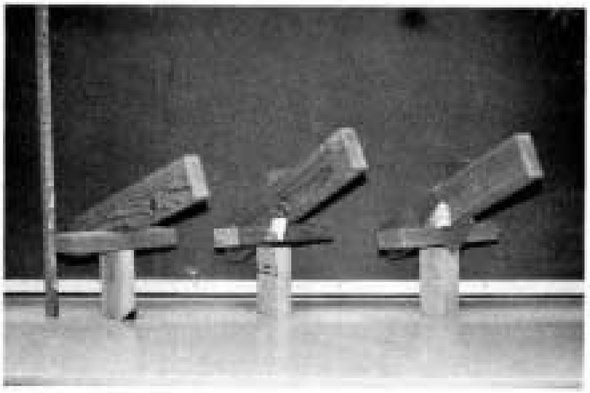
A clip-on-gage (displacement Transducer) was used to measure the slip at the interface of the members of the test specimen. The clip gage had a maximum displacement limit of 0.03 inch. The gage had a calibration factor of 0.001 in/v (inch per volt). The clip was mounted directly on the specimen using a special seating device nailed to the specimen prior to the testing. Figure 7 shows a typical clip-on-gage mounted on a test specimen.

Load Cell

The full bridge tension-compression load cell had a 20,000 pound range and it was used with a 2000 pound cartridge and could measure small loads of 0.02 pound accurately. The load was fastened in series with the hydraulic actuator. The loading head was made of steel and was 5.0 inches long and 1.0 inch in diameter.

L.V.D.T

To monitor the rotation of the test member, an L.V.D.T (Linear Voltage Differential Transducer) was used. It had a calibration factor of 0.0402 in/v. The excitation voltage for the L.V.D.T. was 24 volts D.C. This L.V.D.T. was connected to a power supply and a switch board. This is shown in Figure 8. The switch board was used to monitor the output of the



Figur 5. A Typical Test Specimen with Different Connectors from Left to Right: Nail, H1 and H4 Connectors.

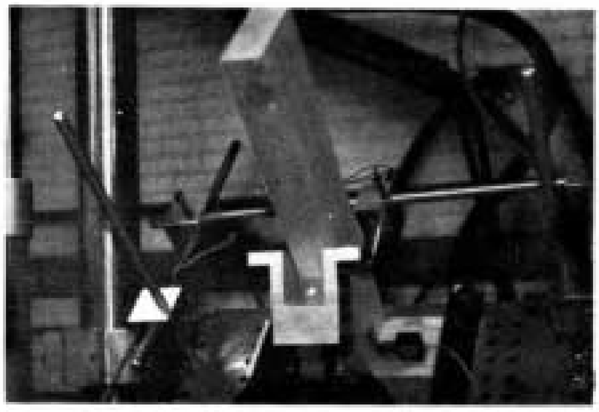


Figure 7. The Set Up of the Clip-on-Gage Mounting.

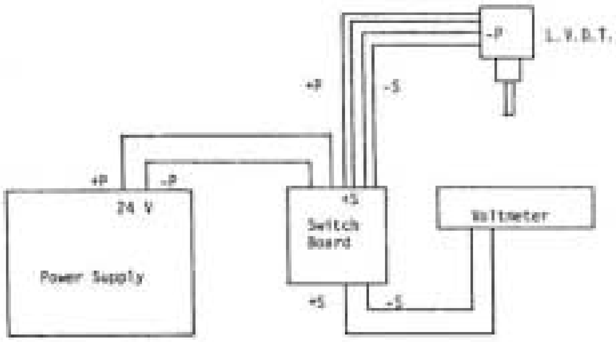


Figure 8. Schematic representation of Recording System.

L.V.D.T. and excitation voltage of the power supply unit. The output from the ram L.V.D.T. was monitored for controlled displacements of the loading device. The L.V.D.T. had a calibration factor of 0.5 in/v.

Recording Device

The MTS controllers were used to monitor and record the output of the ram L.V.D.T., load cell, and the clip-on-gage. The output of the L.V.D.T. mounted on the specimen was recorded using a voltmeter. The schematic of the set up is shown in Figure 8. The load versus ram displacement also was plotted directly on an X-Y recorder (Houston Instruments).

Gripping Device

A set of frictional devices were designed and fabricated to grip an aluminum plate on which the test joint was mounted. These gripping devices were mounted on the fixed columns of the MTS machine. A set of adjustable screws were provided on the gripping devices so that the angle of the aluminum plate could be changed, resulting in different loading angles.

TESTING DETAILS AND PROCEDURE

Tests were performed at the University of Missouri-Columbia Civil Engineering Materials Laboratory. After constructing the test specimen, a clip-on-gage was mounted on the specimen. Using the gripping devices, the specimen then was securely mounted and aligned with the MTS crosshead and columns. After positioning the specimen the spring loaded LVDT was placed on the set up perpendicular to the member and as close as possible to the joint in order to avoid displacement resulted from member bending instead of joint rotation. Then each instrument (MTS and voltmeter) was individually zeroed. Finally, loading was programed on the MTS controller under a displacement-controlled condition. After the completion of each test, a piece of timber from the specimen was cut and tested for water content and specific gravity in accordance with the ASTM standard D1716-77.

After positioning the specimen in the loading frame, the hydraulic ram was moved up under manual control until the loading head made contact with the specimen. The MTS machine was then switched to the displacement control. The rate of the displacement was set at 0.03 inch/min. The ram displacement and the load applied were monitored continuously on an X-Y recorder. The slip of the connection and LVDT outputs for both the hydraulic ram and the specimen were also intermittently monitored and recorded. Figure 9 shows a typical specimen under testing.

FAILURE CRITERIA

Two failure criteria were used for the testing: a)

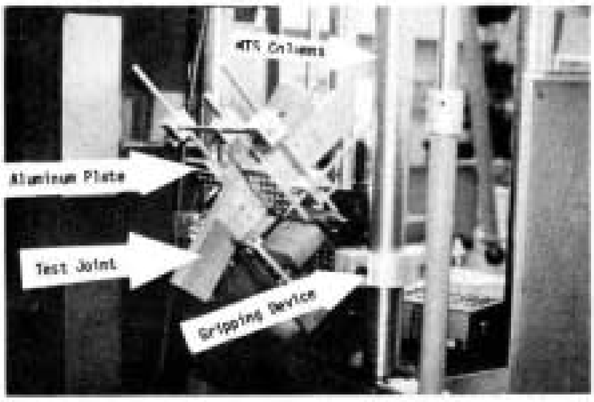


Figure 9. Test Specimen under Loading.

0.015" of slip at the nail level, and b) decrease in load before reaching 0.015' of slip. Loading was increased until one of the above failure conditions was observed. Resistance of joints, against forces and moments beyond this critical slip value, was considered to be caused by the bearing of wood against wood (wedging effect) rather than by the resistance of the connectors. This is specially true for nail joints.

TEST RESULTS AND DISCUSSIONS

Test results

In general, joints resisted loads up to a critical value beyond which joints failed due to a finite slip of the nail bond. A sudden load drop was observed after reaching the critical load, causing a sudden increase in slip.

As loading was continued, joints recovered some

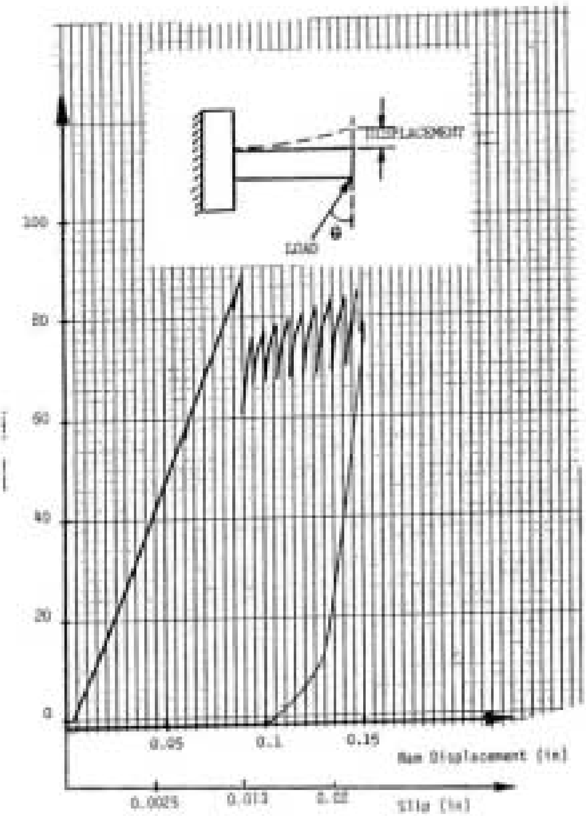


Figure 10. Typical Load-Ram Displacement Curve.

stiffness, but they never reached their initial strength. The critical load usually was observed at the slip of less than 0.015"--the level recommended by ASTM to define failure of nail joints under shear load.

A typical load-displacement relationship recorded by the X-Y plotter is shown in Figure 10. For joints made of connectors, usually loading did not drop in a sudden fashion as it did in the case of nail joints. Loads continued to increase as the ram displacement increased. Testing was stopped when the critical slip of 0.015" was reached.

Two distinct modes of failure were observed from the tests. Some joints failed when nails experienced extensive bending. These joints are usually presented by the highest curve in the moment-rotation diagram as shown in Figure 11. Another group of joints failed when nails experienced extensive slip and very little bending. These joints resulted in the lowest curve in the M-Ø diagram as shown in Figure 11. Variations of wood properties and nailing consistency are believed to be the principal cause of these differences.

DISCUSSIONS

Stud-to-bottom-plate

Three different connectors were used for this particular

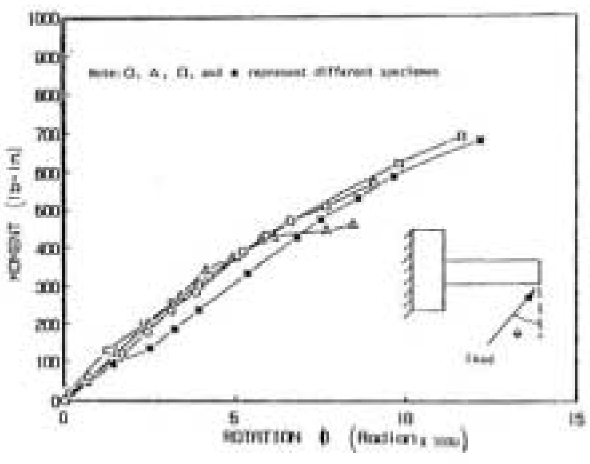


Figure 11. M-Ø for a Typical Joint-Stud-to-Bottom-Plate Joint with 2-16 d Nails Ø=0°.

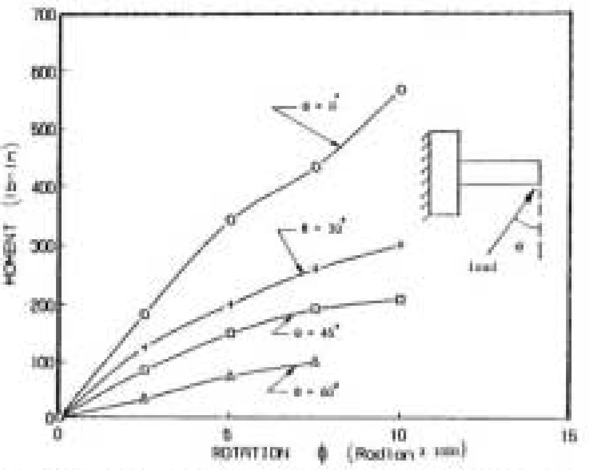


Figure 12. Moment-Rotation Behavior for Stud-to-Bottom Plate Joint (Comparison).

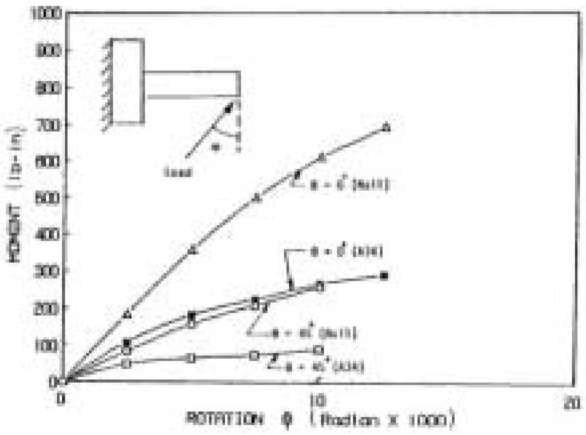


Figure 14. Moment-Rotation Behavior for stud-to-Bottom-Plate Joint (Nail vs A34 Connector).

joint: 2-16d common nails, one Simpson Tie A35, and one Simpson Tie A34. Four different loading angles, 0,30,45, and 60 were used in testing nail joints while two loading angles of 0 and 45 were used for joints made of connectors. Figure 12 gives the plot of the average moment-rotation behavior of nail joints as a function of loading angle. Comparison plots are also given in Figures 13, 14 and 15. A summary of joint stiffness, resisting moments and their relative values are given in Tables 2 and 3.

Figures 12,13,14 and 15 indicate that as the loading angle increased from shear only ($\O = 0°$) to shear and axial tension ($\O = 60°$), both the strength of the joint and the joint stiffness decreased. From the test results for the three connectors, it is clear that Simpson A35 had the highest stiffness. Nail joints performed better than Simpson A34. The surprisingly poor behavior of joints made of Simpson A34 was found to be due to the location of the holes on the connector. From an inspection of the joints, it was found that two of the nails on the A34 connector coincided with the two 16d end nails already in the specimen. This caused internal cracks as shown by the X-rays of the joint in Figure 16. The problem can be avoided by placing the end nails in slightly different locations or by using a different pattern for the holes. As mentioned before,

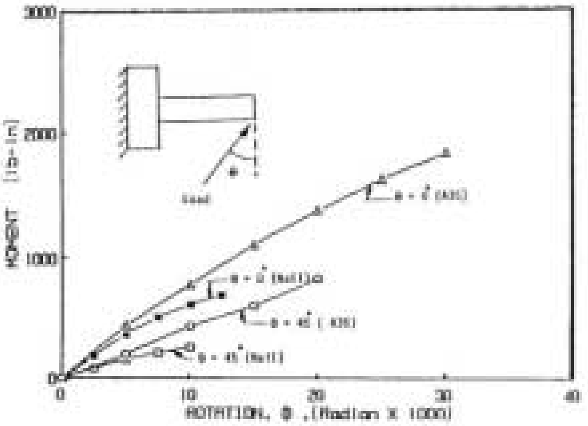


Figure 13. Moment-Rotation Behavior for stud-to-Bottom-Plate Joint (Nail vs A35 Connector).

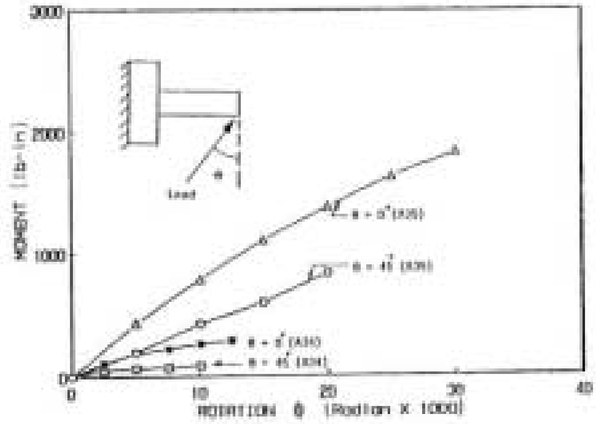


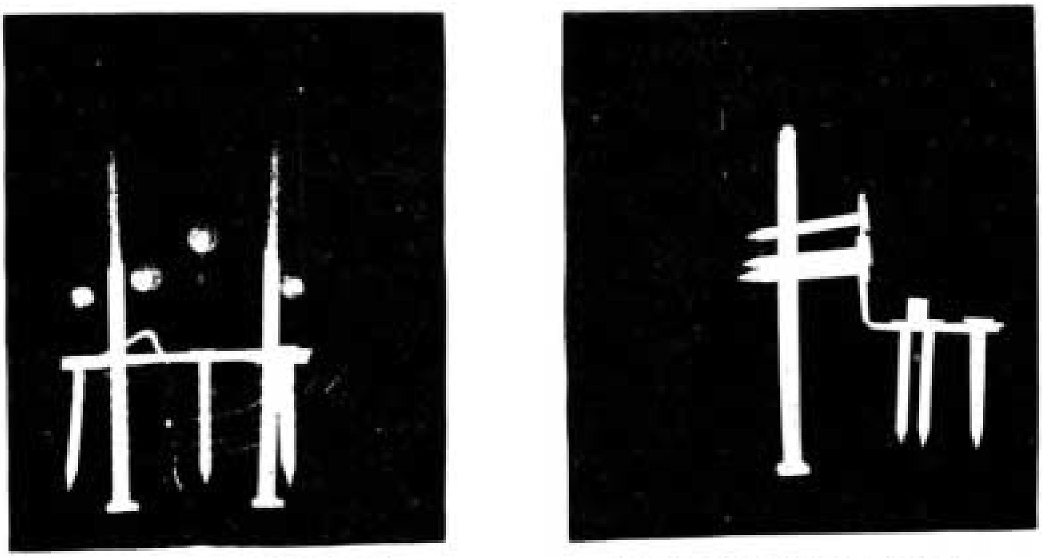
Figure 15. Moment-Rotation Behavior for stud-to-Bottom-Plate Joint (A34 Connector vs A35 Connector).

Table2. Joint stiffness of Stud-to-Bottom-Plate Connection (values are based on average of 3 or more tests)

Angle (degree)	Joint Stiffness, K ($\frac{\text{lb-in}}{\text{Radian}}$)			Relative Stiffness		
	Nail	A35	A34	$\frac{A35}{Nail}$	$\frac{A34}{Nail}$	$\frac{A35}{A34}$
0.0	7×10^4	8×10^4	4.8×10^4	1.24	0.67	1.83
30	5×10^4	N/A	N/A	N/A	N/A	N/A
45	3.5×10^4	4.6×10^4	2.2×10^4	1.34	0.63	1.90
60	1.2×10^4	N/A	N/A	N/A	N/A	N/A

Table 3. Resisting Moment (Stud-to-Bottom-Plate).

Angle (degree)	Resisting Moment (lb-in)			Relative Moment		
	Nail	A35	A34	$\frac{A35}{Nail}$	$\frac{A34}{Nail}$	$\frac{A35}{A34}$
0.0	685	2100	383	3.0	0.56	5.5
30	310	N/A	N/A	N/A	N/A	N/A
45	205	880	100	3.4	0.38	8.8
60	90	N/A	N/A	N/A	N/A	N/A



(Stud-to-Bottom-Plate Joint with 2-16d Common Nails Plus A34 Connector)

a) End View

(Stud-to-Bottom-Plate Joint with 2-16d Common Nails Plus A34 Connector)

b) Side View

Figure 16. X-ray of Test Joint.

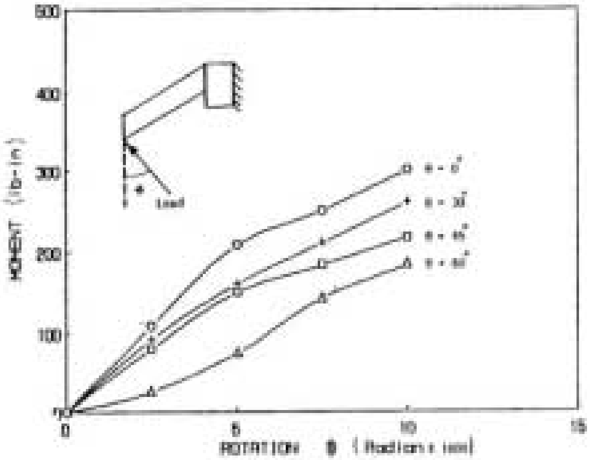


Figure 17. Moment-Rotation Behavior for Rafter-to-Ridge-Board Joint with 3-10d Common Nails (Comparison).

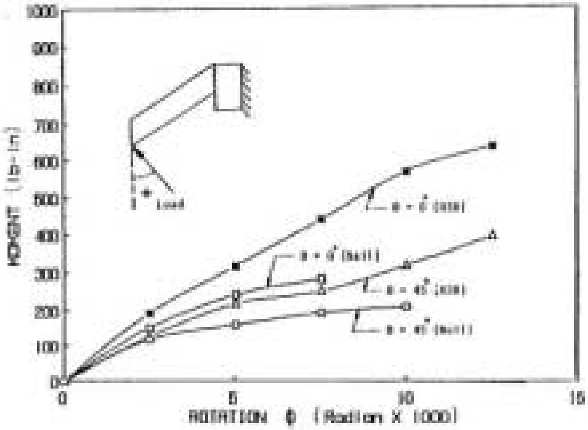


Figure 19. Moment-Rotation Behavior for Rafter-to-Ridge-Board Joint (Nails vs H3R Connector).

A35 out-performed the two other methods of joint construction. From the values of relative moments, it is evident that connector A35 is less sensitive to the axial loads than A34.

Rafter-to-ridge-board

Three different connectors were used for this type of joint, namely, three 10d common nails, one Simpson H3R and one Simpson H5R connector. The moment rotation behavior and their comparative behavior are shown in Figures 17,18,19 and 20. The numerical results are summarized in Tables 4 and 5. Both under shear and under the combined effect of shear and axial tension load, joint stiffness was the highest for H3R, lower for H5R, and the lowest for the nailed joints. The stiffness of nailed joints was most sensitive to the presence of axial tension load. Based on these test results, H3R is the best of three for shear load. However, in the presence of axial tension, Simpson H5R performed the best.

Joist-to-top-plate

Two different connectors were used for the joist-to-top-plate joint. The first was three toenails using 8d common nails, and the second was a Simpson RR connector. Figures 21 and 22 give the comparative

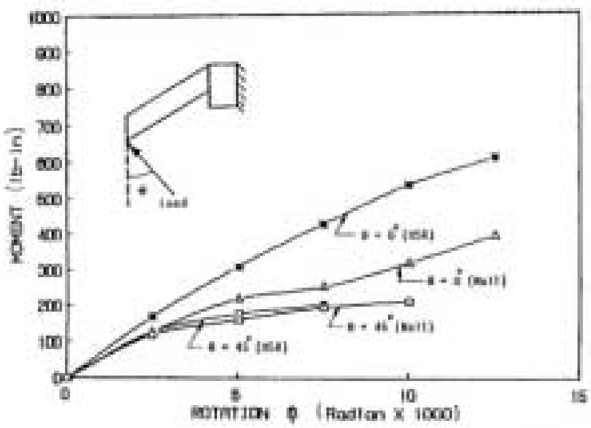


Figure 18. Moment-Rotation Behavior for Rafter-to-Ridge-Board Joint (Nails vs H5R Connector).

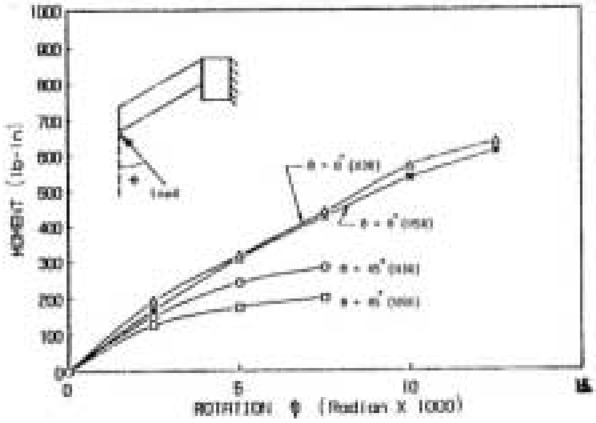


Figure 20. Moment-Rotation Behavior for Rafter-to-Ridge-Board Joint (H5R Connector vs H3R Connector).

Table 4. Joint stiffness (Rafter-to-Ridge-Board).

Angle (degree)	Joint Stiffness, K ($\frac{\text{lb-in}}{\text{Radian}}$)			Relative Stiffness		
	Nail	H3R	H5R	$\frac{\text{H3R}}{\text{Nail}}$	$\frac{\text{H5R}}{\text{Nail}}$	$\frac{\text{H3R}}{\text{H5R}}$
0.0	4.9×10^4	8.4×10^4	6.9×10^4	1.7	1.4	1.22
30	4.4×10^4	N/A	N/A	N/A	N/A	N/A
45	4.2×10^4	6.0×10^4	5.3×10^4	1.43	1.27	1.13
60	1.6×10^4	N/A	N/A	N/A	N/A	N/A

Table 5. Resisting Moment (Rafter-to-Ridge-Board).

Angle (degree)	Resisting Moment (lb-in)			Relative Moment		
	Nail	H3R	H5R	$\frac{\text{H3R}}{\text{Nail}}$	$\frac{\text{H5R}}{\text{Nail}}$	$\frac{\text{H3R}}{\text{H5R}}$
0.0	453	669	700	1.9	1.9	0.69
30	350	N/A	N/A	N/A	N/A	N/A
45	228	277	204	1.2	0.9	1.35
60	200	N/A	N/A	N/A	N/A	N/A

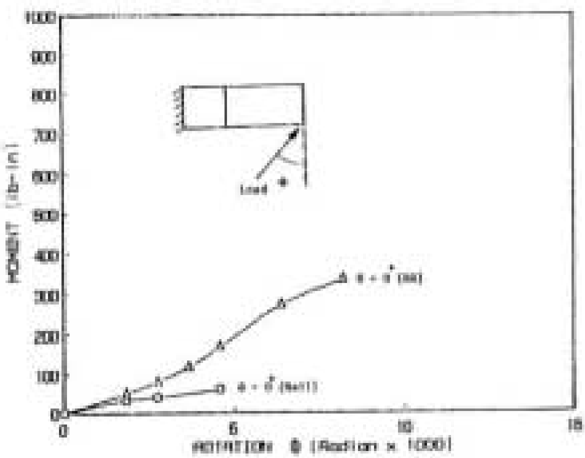


Figure 21. Moment-Rotation Behavior for Joist-to-Top-Plate Joint (Nalis vs RR Connector).

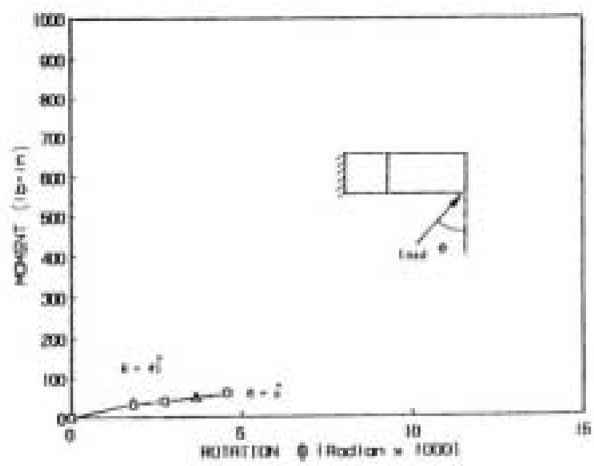


Figure 22. Moment-Rotation Behavior for Joist-to-Top-Plate Joint with 3-8d Common Toenails (Comparison).

moment-rotation behavior of these joints. According to these plots, nailed joints performed poorly. Only two loading angles, 0° and 45° were used in this testing because toenails provided almost no resistance to loading both for pure shear loading and when axial tension load was present. This means that toenailed joints can be considered as pin joints. Connector RR performed much better than the nailed joints, its stiffness was about 200 times and the moment resistance was about 750 times better than the nailed joint. Summary of results in the form of numerical values are given in Table 6.

Rafter-to-Top-plate

Three different connectors, 3-8d common nails, one Simpson H1 and a Simpson H4 were used for testing. Only one angle of 0° was used. This is justified because toenailing has already been proven to perform poorly in presence of shear and axial load. A comparison of moment-rotation behavior for all three joints is shown in Figure 23. Tables 7 and 8 indicate the numerical values for the set of tests. It can be seen

Table 6. Joint Stiffness and Resisting Moment (Joist-to-Top-Plate).

Angle (degree)	Joint Stiffness, K ($\frac{\text{lb-in}}{\text{Radian}}$)		Relative Stiffness	Resisting Moment (lb-in)		Relative Moment
	Nail	RR	$\frac{RR}{Nail}$	Nail	RR	$\frac{RR}{Nail}$
0.0	1.7×10^3	3.2×10^4	1.9	80	600	7.5
45	1.7×10^3	N/A	N/A	60	N/A	N/A

Table 7. Joint Stiffness (Rafter-to-Top-Plate).

Angle (degree)	Joint Stiffness, K ($\frac{\text{lb-in}}{\text{Radian}}$)			Relative Stiffness		
	Nail	H1	H4	$\frac{H1}{Nail}$	$\frac{H4}{Nail}$	$\frac{H1}{H4}$
0.0	2.4×10^4	18.1×10^4	8.9×10^4	7.55	3.7	2.0

Table 8. Resisting Moment (Rafter-to-Top-Plate).

Angle (degree)	Resisting Moment (lb-in)			Relative Moment		
	Nail	H1	H4	$\frac{H1}{Nail}$	$\frac{H4}{Nail}$	$\frac{H1}{H4}$
0.0	200	520	260	2.6	1.3	2.0

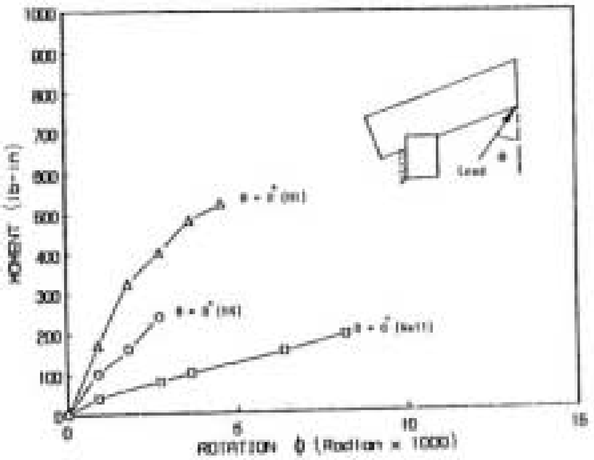


Figure 23. Comparison of Moment-Rotation Behavior for Rafter-to-Top-Plate Joint with Nail, H1, and H4 Connectors.

that connectors behave extremely well in comparison to nails. In general, toenails performed very poorly, approaching the behavior of a pin joint. Empirical Equations of moment-rotation behavior of all tested joints are given in Tables 9,10,11, and 12.

CONCLUSIONS

A new testing was designed and a test method developed to enable the application of axial loads, lateral loads and moments to joints made of nails and mechanical fasteners. Three types of wood joints namely, stud-to-top-plate, rafter-to-top-plate and

Table 9. Empirical Equations for Stud-to-Bottom-Plate Joint.

Connector	Angle (degree)	Joint Behavior M in lb-in Ø in rad/1000
Nail	0	$M = -3.5 + 83\text{Ø} - 2.2\text{Ø}^2$
	30	$M = 3.7 + 50.4\text{Ø} - 2.1\text{Ø}^2$
	45	$M = 0.23 + 36.2\text{Ø} - 1.\text{Ø}^2$
	60	$M = -1.2 + 16.2\text{Ø} - .38\text{Ø}^2$
A35 Simpson	0	$M = 7.2 + 85.2\text{Ø} - .82\text{Ø}^2$
	45	$M = .02 + 41\text{Ø} - .002\text{Ø}^2$
A34 Simpson	0	$M = 5.25 + 42.2\text{Ø} - 1.6\text{Ø}^2$
	45	$M = 4.23 + 17\text{Ø} - .87\text{Ø}^2$

Table 10. Empirical Equations for Rafter-to-Ridge-Board Joint.

Connector	Angle (degree)	Joint Behavior M in lb-in Ø in rad/1000
Nail	0	$M = 12.5 + 42\text{Ø} - \text{Ø}^2$
	30	$M = -4.6 + 13.7\text{Ø} + .56\text{Ø}^2$
	45	$M = 6.9 + 43.5\text{Ø} - 2.4\text{Ø}^2$
	60	$M = 2.3 + 37.2\text{Ø} - 1.16\text{Ø}^2$
H3R Simpson	0	$M = 6.17 + 72.3\text{Ø} - 1.75\text{Ø}^2$
	45	$M = .4 + 70.2\text{Ø} - 4.4\text{Ø}^2$
H5R Simpson	0	$M = .91 + 4.66\text{Ø} - 1.7\text{Ø}^2$
	45	$M = 2.5 + 56.\text{Ø} - 4\text{Ø}^2$

Table 11. Empirical Equations for Joist-to-Top-Plate Joint.

Connector	Angle (degree)	Joint Behavior M in lb-in Ø in rad/1000
Nail	0	$M = .38 + 18.47\text{Ø} - 1.2\text{Ø}^2$
	45	$M = .36 + 19.87\text{Ø} - 1.72\text{Ø}^2$
RR Simpson	0	$M = -6.84 + 30.5\text{Ø} + 1.55\text{Ø}^2$

Table 12. Empirical Equations for Rafter-to-Top-Plate Joint.

Connector	Angle (degree)	Joint Behavior M in lb-in Ø in rad/1000
Nail	0	$M = 6.47 + 28.67\text{Ø} - .78\text{Ø}^2$
H4 Simpson	0	$M = 3 + 102.2\text{Ø} - 6.04\text{Ø}^2$
Hl Simpson	0	$M = 1.14 + 204.6\text{Ø} - 20\text{Ø}^2$

rafter-to-ridge plate using conventional nailing schedule sanctioned by the building codes and special connectors were tested using this procedure. Moment-rotation curves and resisting moment capacities were also established for all tests. Effect of axial load on the joint stiffness was also investigated using different loading angles. Joint stiffness of wood connections decreased as the axial tension in the joint increased. Empirical equations describing the moment rotation behavior of all tests as a function of axial load were obtained. Comparisons were made between the nailed connections and connections made of special connectors. All nailed joints except toenailed joints resulted in some joint stiffness whereas toenails practically behaved as pin connections. In general, it can be concluded that stud-to-bottom-plate and rafter-to-ridge-board nailed joints behave as semi-rigid connections whereas toenailed rafter-to-top-plate acts as pin connection. All connections using connectors behave as semi-rigid connections. Connections made of connectors generally outperformed the nailed connections.

The testing and evaluation of wood joints under combined axial and lateral loadings permit the assessment of the significance of the interaction. Further, by expressing joint stiffnesses in an analytical model in timber structures, the possibility exists for designers to identify the real behavior of the structures considering the real behavior of the connections. And finally, it enables scholars to identify the weak links in a timber-framed structures while subjected to winds and earthquakes.

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