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Deflections of wooden roof trusses based on simple joint tests

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DEFLECTIONS OF WOODEN ROOF TRUSSES BASED ON SIMPLE JOINT TESTS

by A. T. Hansen

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Presented at
the International Symposium on Joints in Timber Structures organized by Timber Research and Development Association and CIB Commission W-18 held in London, England, 30 - 31 March 1965

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DEFLECTIONS OF WOODEN ROOF TRUSSES BASED ON SIMPLE JOINT TESTS

by

A. T. Hansen*

It is common practice among residential wood roof truss fabricators in Canada to determine truss deflections by load testing each new truss design. This can be expensive, particularly if a series of tests is undertaken to develop a family of truss designs. There is some advantage in being able to predict deflection without resorting to full-size tests, not only in developing new designs, but also in checking the results of a truss test to ensure that the test was truly indicative of an average truss and not an atypical one. This paper presents a method of computing the deflections of simple "W" trusses and Howe trusses, the two most common configurations used in house construction. A comparison between the deflections obtained from actual tests on a number of these trusses with those obtained by calculation is also included to compare the accuracy of the proposed method over a range of roof slopes, spans, and joint connections.

CAUSE OF DEFLECTION

When a roof truss is loaded, the resulting deflection at any panel point where members intersect to form a joint is due both to the

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axial lengthening or shortening of the members and to the slippage at the joints. Where glued joints are used, this slippage is negligible and can be ignored (1). In "W" trusses the deflection at the centre of the truss is somewhat greater than at the adjacent panel points due to bending in the lower chord and must also be considered.

DEFLECTION DUE TO AXIAL STRAIN

The deflection at any panel point due to axial strain in the members may be calculated using the formula

$$\Delta e = \sum \frac{SU\ell}{AE} \qquad \ldots\ldots (1)$$

where

Δe = panel point deflection due to axial strain

S = the force on each member due to the roof and ceiling loads

U = the force on each member caused by applying a unit load at the panel point for which the deflection is to be calculated and in the direction the deflection is to be measured

ℓ = length of each member

A = cross-sectional area of each member

E = modulus of elasticity in the axial direction.

The value $SU\ell/AE$ must be calculated separately for each panel member and Δe found by adding these together. The values for S, U, and ℓ have been calculated by the author for simple Howe and "W" trusses and are listed in Tables I and II. For Howe trusses the U stresses are for a unit load applied at mid-span of the lower chord; for "W" trusses, the U values are for a unit load applied at the third point of the lower chord.

The values shown in Tables I and II assume simple pin-connected joints. This may be questioned because truss joints vary in rigidity from glued joints to split-ring joints; rigidity of nailed and toothed connector plate joints is somewhere between these extremes. In calculating deflections due to axial strain in the members, however, the rigidity of the joints is relatively unimportant, and for practical purposes the assumption of pin-connected joints is considered sufficiently accurate (2).

Tables I and II show that for a given truss geometry and where the load and slope are constant, the values of ℓ and S each vary directly in proportion to the span of the truss. For the Howe truss (Table II) the axial load S varies directly as the sum of the ceiling and roof load for most of the members. For "W" trusses (Table I), although the axial load S is not exactly proportional to the sum of the ceiling and roof load, it is almost so, and with the usual ranges of ceiling and roof load one can also consider the value of S as being in direct proportion to the sum of the total load without being in significant error.

When a similar species of lumber is used for all truss members, which is usually the case, the panel point deflection due to axial strain of trusses with similar geometry, slope, and member cross-sections can therefore be expressed as

$$\Delta E = \frac{C_1 W L^2}{E} . \qquad \ldots\ldots (2)$$

If this expression is to be applied to trusses that have different cross-sections than those for which C_1 is determined, a second constant must be introduced to take this into account. The deflection may then be expressed as

$$\Delta E = C_1 C_2 \frac{W L^2}{E} \qquad \ldots\ldots (3)$$

where

ΔE = panel point deflection due to axial strain, in.*

C_1 = constant for a given slope and geometry

C_2 = constant for given member size and geometry

W = total load (roof plus ceiling) per lineal ft of truss, lb

L = span of truss, ft

E = modulus of elasticity, lb per sq in.

The values of C_1 have been determined by the author for both "W" trusses and simple Howe trusses (Tables I and II) by plotting

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* (Footnote - see page 5.)

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*This paper gives British Units throughout. Loads are in pounds, spans are in feet and deflections are in inches. The values of coefficients C_1 and C_2 are dependent on the measuring system used. When the metric system is used therefore, the constants will be different from those in Table III.

Timber sizes are given as nominal inch dimensions. The actual sizes are listed below:

Nominal Size, in.	Canadian Lumber Standard Dressed Size, in.	Metric Equivalent, cm
2 x 4	1 5/8 x 3 5/8	4.1 x 9.2
2 x 6	1 5/8 x 5 1/2	4.1 x 14.0
1 x 8	3/4 x 7 1/2	1.9 x 19.1

Conversion factors to the metric system are as follows:

roof load	- lb/sq ft	x 4.88	=	Kg/sq m
	- lb/lineal ft	x 1.49	=	Kg/m
stress, or modulus of elasticity	- lb/sq in.	x 0.070	=	Kg/sq cm

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the values of $\sum \frac{SUL}{AE}$ for 2/12, 3/12, 4/12, and 5/12 slope trusses, assuming nominal 2- by 4-in. members throughout. The values of $\sum \frac{SUL}{AE}$ were also calculated assuming 2- by 6-in. top and 2- by 4-in. bottom chords, 2- by 4-in. top and 2- by 6-in. bottom chords and 2- by 6-in. top and bottom chords. As the values of C_1 were determined assuming 2- by 4-in. members throughout, the value of C_2 is unity for this condition. The values of C_2 for other member sizes were calculated by dividing the deflections for trusses with other member sizes by that of trusses with 2- by 4-in. members throughout. The values of C_1 and C_2 thus calculated are shown in Table III.

DEFLECTION DUE TO JOINT SLIP

Movement of the wooden members relative to the gusset plates causes deflection at a panel point in much the same manner as axial strain in the members.

The contribution of each member toward the total deflection will be the product of the force U for that member due to the unit load, multiplied by the net change in effective length caused by the joint slippage at both ends of that member due to the total load W. If the displacement of a connector is known for a given load the contribution to deflection at a panel point caused by joint slip for each member can be calculated.

The deflection due to joint slippage, therefore, may be expressed as

$$\Delta P = \Sigma U \Delta \ell \qquad \ldots\ldots (4)$$

where

$\Delta \ell$ = the change in member length due to joint slip.

In an idealized truss design, the load per unit of connector will be the same at every joint. In an idealized connector the displacement, d, of the connector relative to the truss members will be approximately the same at a given load regardless of the orientation of the connector to the direction of loading.

At any given truss loading W, the change in member length, $\Delta\ell$, will be proportional to d. Therefore, for a given slope and geometry of truss, the deflection due to connector slippage can be expressed as

$$\Delta P = C_3 d \qquad \ldots\ldots (5)$$

where

C_3 is a constant for a particular slope and truss geometry

d is displacement of a connector at a load of P, lb per unit of connector, corresponding to a total load, W, lb per lineal ft of truss.

C_3 therefore may be expressed as being equal to $\sum \frac{U \Delta \ell}{d}$.

The following assumptions were made in calculating C_3. In a typical "W" truss (Figure 1a) with metal or plywood plates at all joints, the change in length of L_0L_1 and L_0U_1 will be equal to d. Member L_1L_2, however, will have a change in length equal to 2d since the slip will be equal to d on both ends of the splice plate of the lower chord. The long diagonal L_1U_2 will have a change in length equal to 4d since the slip occurs between the top chord and gusset plate and between the gusset plate and diagonal member at the top end, and between the gusset plate and diagonal member and gusset plate and lower chord at the bottom end. The slippage in U_1U_2 will be zero if the members are in good bearing contact at the peak. Members U_1L_1, L_2U_2 and L_2U_3 can be ignored because their U stress is zero.

Similarly, in a typical Howe truss (Table II), the change in length of L_0L_1, L_1L_2, L_2L_3, L_3L_4, L_0U_1, and L_4U_3 will be equal to d in each case. The movement in U_1U_2 and U_2U_3 will be zero for members in good bearing contact. In the vertical member, U_2L_2, movement occurs between the gusset plate and the upper chords and between the gusset plate and the upright at the top end, and between the gusset plate and upright and between the gusset plate and lower chords at the bottom end. The latter movement, however, should be considerably less than d since the connector will be relatively lightly loaded in the vertical direction. The over-all change in length of L_2U_2, therefore, was assumed to be 3d. The remaining members, L_1U_1, L_2U_1, L_3U_3, and L_2U_3 can be neglected as their U stress will be equal to zero.

Values of C_3 have been calculated by the author for these idealized truss and connector conditions for both "W" and Howe trusses for various slopes (Table III).

The value d can be determined on the basis of a loading test on a simple joint to determine the slip characteristics. The assumptions used in developing the values of C_3 in Table III were that the connectors were designed so that there would be the same displacement between member and plate at each joint. Although this condition is approached for nailed connectors, it may not be the case where certain plate connectors are used which would involve consideration of the orientation of the teeth to the direction of loading. Some judgment may have to be exercised, therefore, in using the values of d determined from a load test on a simple joint. If the displacement of the wood or metal plate d at a given load per connector unit (nail or tooth) is known, it then becomes possible to predict the deflection at any panel point in the full-size truss.

DETERMINATION OF JOINT SLIP, d

Simple loading tests were carried out on three types of connectors: nailed plywood; light-gauge, short-tooth plates (Figure 2); and heavy-gauge, long-tooth plates (Figure 3). The joints for the framing lumber were tested in three conditions: assembled dry and tested dry; assembled green and tested green; and assembled green and tested dry. Three tests were conducted on each plate and for each condition to obtain average results. Test results of the joints are shown in Figures 4, 5, and 6. Tests on joints were conducted generally in the manner recommended by the U.S. Truss Plate Institute (3). The joints were tested in a testing machine which loaded the joints in tension in increments, with each increment applied for 5 minutes before displacements were recorded. Displacements were measured from wood member to wood member; the values plotted in Figures 4, 5, and 6 are half these displacements to represent the movement between plate and member only, or d.

The connectors were tested in only one direction, although it would have been preferable in the case of toothed plates to have included tests where the load was applied at various tooth orientations to the direction of load to obtain a better assessment of the factor d for use in calculating over-all truss deflections. It is hoped that this will be investigated in future testing. In joint tests on the light-gauge plates, one spiral nail per eight teeth was used, similar to the plate manufacturer's directions for the use of these plates in trusses. This was found to be approximately the proportions used in the trusses that were tested which had been fabricated by the truss plate manufacturer.

TOTAL DEFLECTION

The total deflection at the centre line of a Howe or the third point of a "W" truss may be calculated by adding formulae (3) and (5):

$$\Delta = C_1 C_2 \frac{WL^2}{E} + C_3 d \qquad \ldots\ldots (6)$$

where

Δ = total deflection, in.

W = the total load on a truss (ceiling plus roof), lb per lineal ft of truss

L = span of truss, ft

E = modulus of elasticity under axial loading, psi

d = displacement of plate relative to the member at the loading being considered (Figures 4, 5, and 6), in.

(Values for C_1, C_2, and C_3 are given in Table III.)

Although this formula gives the third point deflections of the bottom chord in a "W" truss, the centre line deflection of "W" trusses is usually what is desired. It is difficult to determine this deflection in purely theoretical terms because it occurs between panel points. The third point deflections were plotted against centre line deflections from the results of a number of tests on "W" trusses (Figure 7). The relationship between the third point and centre line deflection appears to be practically a straight line that varies little with roof slope or span and corresponds approximately to the relationship:

centre line deflection = 1.30 times the panel point deflection.

The centre line deflection of a "W" truss can, therefore, be determined using the formula:

$$\Delta = 1.30\left[C_1 C_2 \frac{WL^2}{E} + C_3 d\right]. \qquad \ldots\ldots (7)$$

COMPARISON OF MEASURED AND CALCULATED RESULTS

A number of tests on "W" trusses with nailed plywood gusset plates had been previously completed by the Division of Building Research (5) (Figure 1) so that the measured deflections for these trusses of different slopes, spans, and member sizes could be checked against calculated values using formula (7). The trusses in Figure 1a represent the type for which the constant C_3 in Table III for "W" trusses would apply because of the typical arrangement of gusset plates. A fairly large number of tests had also been completed on a slightly different type of "W" truss (Figure 1b) incorporating 1- by 8-in.-long diagonals rather than 2- by 4-in. members. The difference between these two types of trusses is not great, however, and the same constants were applied to the theoretical calculation of deflections for both types of trusses in order to check a wider spectrum of spans and slopes for which test information exists.

Additional tests were conducted on 4/12 slope trusses of 28-ft span made with the two types of metal connector plates described in Figures 2 and 3. Trusses made with the plate described in Figure 2 were "W" type (3 tests) and Howe truss (1 test). Trusses made with plates shown in Figure 3 were "W" type (3 tests). The trusses were tested in pairs in accordance with the method described (5) using hydraulic tension jacks to supply the roof load to the top chords, and lead weights to supply the ceiling load to the lower chord (Figure 8).

The axial modulus of elasticity of white spruce has been found to average 1,270,000 psi for the green condition and 1,550,000 psi for the air dry condition (4). In these calculations the value of 1,500,000 psi was assumed to be representative of the lumber (nearly dry) at the time of the test.

The values of d used in calculations for the nailed trusses were those for trusses manufactured dry and tested dry; those for the metal-plate-connected trusses were for trusses made green and tested dry. These conditions were considered representative for these trusses. In selecting the value of d to be used in these calculations from Figures 4, 5, and 6 the dead weight of the truss and loading equipment (approximately 5 lb/sq ft) was taken into account. The load testing procedure was such that the zero deflection was recorded with the dead weight of the truss and test equipment in place, so that subsequent measured deflections recorded only that deflection caused by the applied roof and ceiling load.

The values of d selected from Figures 4, 5, and 6, therefore, were determined by calculating the load per unit of connector caused by the applied load plus the dead load and subtracting the slip caused by the dead load only. In calculating the deflection caused by axial strain this procedure was not necessary since this deflection has a straight line relationship with the applied load whereas the connector slip has not. The calculated deflections in Table IV are those caused by the addition of the ceiling and roof loads, after the dead load of 5 psf was applied.

In calculating the deflections for the nailed plywood trusses, the constants (C_2 in Table III) were divided by an additional factor of 1.10 which is approximately the ratio of the cross-sectional area of lumber cut to the older size (Eastern Canadian) which was used in these trusses, to that of the current Canadian Lumber Standard sizes for which the values of C_2 in Table IV were calculated. Since CLS-size lumber was used in the metal-plate-connected trusses, the constant C_2 used in calculating their deflections was the same as shown in Table III.

DISCUSSION

The measured deflections in Table IV are based on three tests except where otherwise noted. Table IV shows that in most cases the calculated results agree relatively closely with the calculated deflections, when one takes into consideration the natural variability of the properties of wood.

The errors obtained for the metal-plate-connected trusses in Table IV are also relatively small except for the "W" truss made with light-gauge plates at a total load of 65 psf (50 psf snow load plus ceiling load and dead load). This maximum error was 0.17 in. The calculated deflection error for the same truss at a total load of 35 psf, however, was only 0.06 in.

It was noted that in the tests on these trusses, under higher loads the heel plates buckled in compression which forced a number of teeth out of the wood thus considerably reducing the joint stiffness (Figure 9). This probably led to substantially greater movements at these joints than was determined by calculation. The Howe trusses, which did not exhibit this plate buckling tendency to the same degree even though the same type of plates was used, showed close agreement between measured and calculated deflections. It was noted that the heel plates of the "W" trusses with the light-gauge plates were placed so that they extended below the bottom chord of the truss and therefore bore the weight of the truss at the end supports. This probably contributed to the premature plate buckling that led to the greater-than-expected deflections for these trusses. Plate buckling at the heel joint may also be caused by the trusses being made of very green lumber which, on drying, shrinks and causes the plate to buckle. In addition, this shrinkage would cause the truss members at the heel joint to separate so that the vertical load at the heel joint from the top chord has to be transferred to the bottom chord through the thin walls of the plate which are not able to withstand high loads without buckling.

In determining the joint slip characteristics of connectors by simple joint tests, one should be sure that the joint has the same history of fabrication as the truss for which deflections are to be calculated. That is, if the trusses are to be manufactured from green lumber, the joint test specimens should also be manufactured from green lumber and all joint tests should preferably be conducted after the wood has dried to about 10 to 12 per cent which is close to the final moisture content of the wood in service.

Figures 4, 5, and 6 show the substantial differences in connector displacement depending upon the condition of the wood at the time of assembly and at the time of test. When joints are assembled with dry wood and are tested dry, the connectors display their greatest stiffness. Joints assembled green display greater displacement under a given load.

It should be emphasized that these calculated results are valid only for short-term tests (5 minutes' duration). For longer-term loading the calculated results should be increased approximately 5 per cent for 1-hr loading, 25 per cent for 24-hr loading, 55 per cent for a one-week loading, and 100 per cent for a one-month loading. These increases are based on observations of nailed "W" trusses (5) but until further information is obtained for toothed connectors, it is suggested that these values be applied to tooth-plate-connected trusses as well.

There are, of course, roof trusses used in residential construction of other geometries than those tested. These other types, however, have not been load tested by DBR/NRC and a comparison of theoretical deflections with measured deflections has not been made for trusses other than as described in this paper.

CONCLUSION

It would appear that this method of computing centre line deflections is reliable for the kinds of trusses tested and could be used for the design of trusses for deflections where test results for full-size trusses are not available. Because the displacement of the toothed-plate connectors was determined on the basis of tests in one loading direction the agreement between the calculated and measured deflections is very good. In determining the joint slip characteristics of connectors by simple joint tests, it is essential that the joint has the same history of fabrication as the truss for which the deflection is to be calculated.

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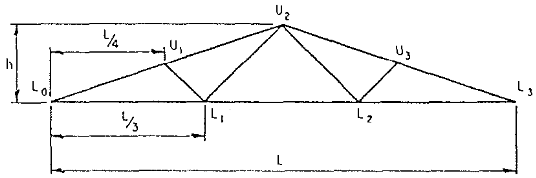
TABLE I

VALUES OF S, U, AND ℓ FOR W TRUSSES



Member	S	U	ℓ
L_0L_1	$+\frac{L^2}{2h}\left(\frac{3}{8}W_1+\frac{1}{3}W_2\right)$	$+1/3\,\frac{L}{h}$	$L/3$
L_1L_2	$+\frac{L^2}{h}\left(\frac{1}{8}W_1+\frac{1}{9}W_2\right)$	$+1/6\,\frac{L}{h}$	$L/3$
L_2L_3	$+\frac{L^2}{2h}\left(\frac{3}{8}W_1+\frac{1}{3}W_2\right)$	$+1/6\,\frac{L}{h}$	$L/3$
L_0U_1	$-\frac{L}{2h}\left(\frac{3}{8}W_1+\frac{1}{3}W_2\right)\sqrt{L^2+4h^2}$	$-\frac{1}{3h}\sqrt{L^2+4h^2}$	$1/4\sqrt{L^2+4h^2}$
U_1U_2	$-\frac{L}{h}\left(\frac{5}{32}W_1+\frac{1}{6}W_2\right)\sqrt{L^2+4h^2}$	$-\frac{1}{3h}\sqrt{L^2+4h^2}$	$1/4\sqrt{L^2+4h^2}$
U_2U_3	$-\frac{L}{h}\left(\frac{5}{32}W_1+\frac{1}{6}W_2\right)\sqrt{L^2+4h^2}$	$-\frac{1}{6h}\sqrt{L^2+4h^2}$	$1/4\sqrt{L^2+4h^2}$
U_3L_3	$-\frac{L}{2h}\left(\frac{3}{8}W_1+\frac{1}{3}W_2\right)\sqrt{L^2+4h^2}$	$-\frac{1}{6h}\sqrt{L^2+4h^2}$	$1/4\sqrt{L^2+4h^2}$
L_1U_2	$+\frac{L}{h}\left(\frac{1}{32}W_1+\frac{1}{18}W_2\right)\sqrt{36h^2+L^2}$	$+\frac{1}{6h}\sqrt{36h^2+L^2}$	$1/6\sqrt{36h^2+L^2}$
U_1L_1	$-\frac{L}{32h}W_1\sqrt{36h^2+L^2}$	0	$1/12\sqrt{36h^2+L^2}$
U_2L_2	$+\frac{L}{h}\left(\frac{1}{32}W_1+\frac{1}{18}W_2\right)\sqrt{36h^2+L^2}$	0	$1/6\sqrt{36h^2+L^2}$
L_2U_3	$-\frac{L}{32h}W_1\sqrt{36h^2+L^2}$	0	$1/12\sqrt{36h^2+L^2}$

Where S = axial force on member

U = force caused by unit load applied vertically at third point of lower chord

ℓ = length of member

W_1 = load per unit length of truss along top chord

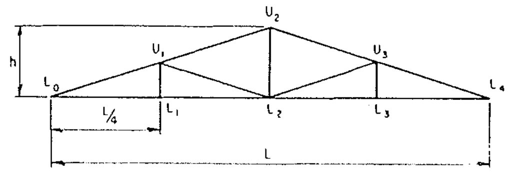
W_2 = load per unit length of truss along bottom chord

L = span of truss

h = height of truss at peak

TABLE II

VALUES OF S, U, AND ℓ FOR HOWE TRUSSES



Member	S	U	ℓ
L_0L_1	$\frac{3L^2}{16h}(W_1+W_2)$	$1/4\,\frac{L}{h}$	$L/4$
L_1L_2	$\frac{3L^2}{16h}(W_1+W_2)$	$1/4\,\frac{L}{h}$	$L/4$
L_2L_3	$\frac{3L^2}{16h}(W_1+W_2)$	$1/4\,\frac{L}{h}$	$L/4$
L_3L_4	$\frac{3L^2}{16h}(W_1+W_2)$	$1/4\,\frac{L}{h}$	$L/4$
L_0U_1	$\frac{3L}{16h}(W_1+W_2)\sqrt{L^2+4h^2}$	$\frac{\sqrt{L^2+4h^2}}{4h}$	$\frac{\sqrt{L^2+4h^2}}{4}$
U_1U_2	$\frac{L}{8h}(W_1+W_2)\sqrt{L^2+4h^2}$	$\frac{\sqrt{L^2+4h^2}}{4h}$	$\frac{\sqrt{L^2+4h^2}}{4}$
U_2U_3	$\frac{L}{8h}(W_1+W_2)\sqrt{L^2+4h^2}$	$\frac{\sqrt{L^2+4h^2}}{4h}$	$\frac{\sqrt{L^2+4h^2}}{4}$
U_3L_4	$\frac{3L}{16h}(W_1+W_2)\sqrt{L^2+4h^2}$	$\frac{\sqrt{L^2+4h^2}}{4h}$	$\frac{\sqrt{L^2+4h^2}}{4}$
U_1L_1	$\frac{L}{4}W_2$	0	$\frac{h}{2}$
U_1L_2	$\frac{L}{16h}(W_1+W_2)\sqrt{L^2+4h^2}$	0	$\frac{\sqrt{L^2+4h^2}}{4}$
U_2L_2	$\frac{L}{4}(W_1+2W_2)$	1	h

Where S = axial force on member

U = force caused by unit load applied vertically at the centre of the lower chord

ℓ = length of member

W_1 = load per unit length of truss along top chord

W_2 = load per unit length of truss along bottom chord

L = span of truss

h = height of truss at peak

TABLE III

VALUES OF C_1, C_2, C_3 FOR "W" AND HOWE TRUSSES

Coefficient	Slope	Chord Member Size	Type of Truss	
			W	Howe
C_1	2/12	All Chord Sizes	24.4	25.8
	3/12		11.2	11.8
	4/12		6.6	6.9
	5/12		4.5	4.7
C_2	All Slopes	2 x 4 top and bottom	1.00	1.00
		2 x 6 top and 2 x 4 bottom	0.81	.84
		2 x 4 top and 2 x 6 bottom	0.86	.83
		2 x 6 top and bottom	0.67	0.66
C_3	2/12	All Chord Sizes	25.0	21.1
	3/12		17.5	15.1
	4/12		13.8	12.2
	5/12		11.7	10.4

TABLE IV

Comparison of Calculated and Measured Deflections of Various Spruce Trusses

Type of Truss	Span, ft	Slope	Top Chord Size, nominal	Bottom Chord Size, nominal	Deflections, in.		
					Calculated	Measured	Difference
W type - Nailed plywood gussets (see Fig. 1a)	26	4/12	2x4	2x4	.25	.29	- .04
					.55	.62	- .07
	28	4/12	2x4	2x4	.28	.31	- .03
					.61	.65	- .04
	28 (Single Test)	4/12	2x6	2x4	.24	.34	- .10
					.52	.61	- .09
	28 (Single Test)	4/12	2x4	2x6	.25	.24	+ .01
					.54	.52	+ .02
	28 (Single Test)	4/12	2x6	2x6	.20	.19	+ .01
					.45	.40	+ .05
	28	3/12	2x6	2x4	.38	.41	- .03
					.82	.75	+ .07
	28	3/12	2x4	2x4	.46	.51	- .05
					.98	.95	+ .03
W type - Nailed plywood gussets (see Fig. 1b)	24	5/12	2x4	2x4	.15	.18	- .03
					.35	.39	- .04
	24	4/12	2x4	2x4	.21	.20	+ .01
					.46	.44	+ .02
	26	5/12	2x4	2x4	.17	.20	- .03
					.39	.44	- .05
	26	4/12	2x4	2x4	.25	.22	+ .03
					.55	.48	+ .07
	28	5/12	2x4	2x4	.20	.21	- .01
					.44	.46	- .02
	28	4/12	2x4	2x4	.28	.34	- .06
					.61	.67	- .06
W type with glued plywood gussets	24	5/12	2x4	2x4	.12	.09	+ .03
					.24	.17	+ .07
W type with light-gauge metal connectors	28	4/12	2x4	2x4	.33	.39	- .06
					.72	.89	- .17
W type with heavy-gauge metal connectors	28'8" (Single Test)	4/12	2x4	2x4	.33	.35	- .02
					.70	.76	- .06
Howe type with light-gauge metal connectors	28	4/12	2x4	2x4	.29	.28	+ .01
					.67	.65	+ .02

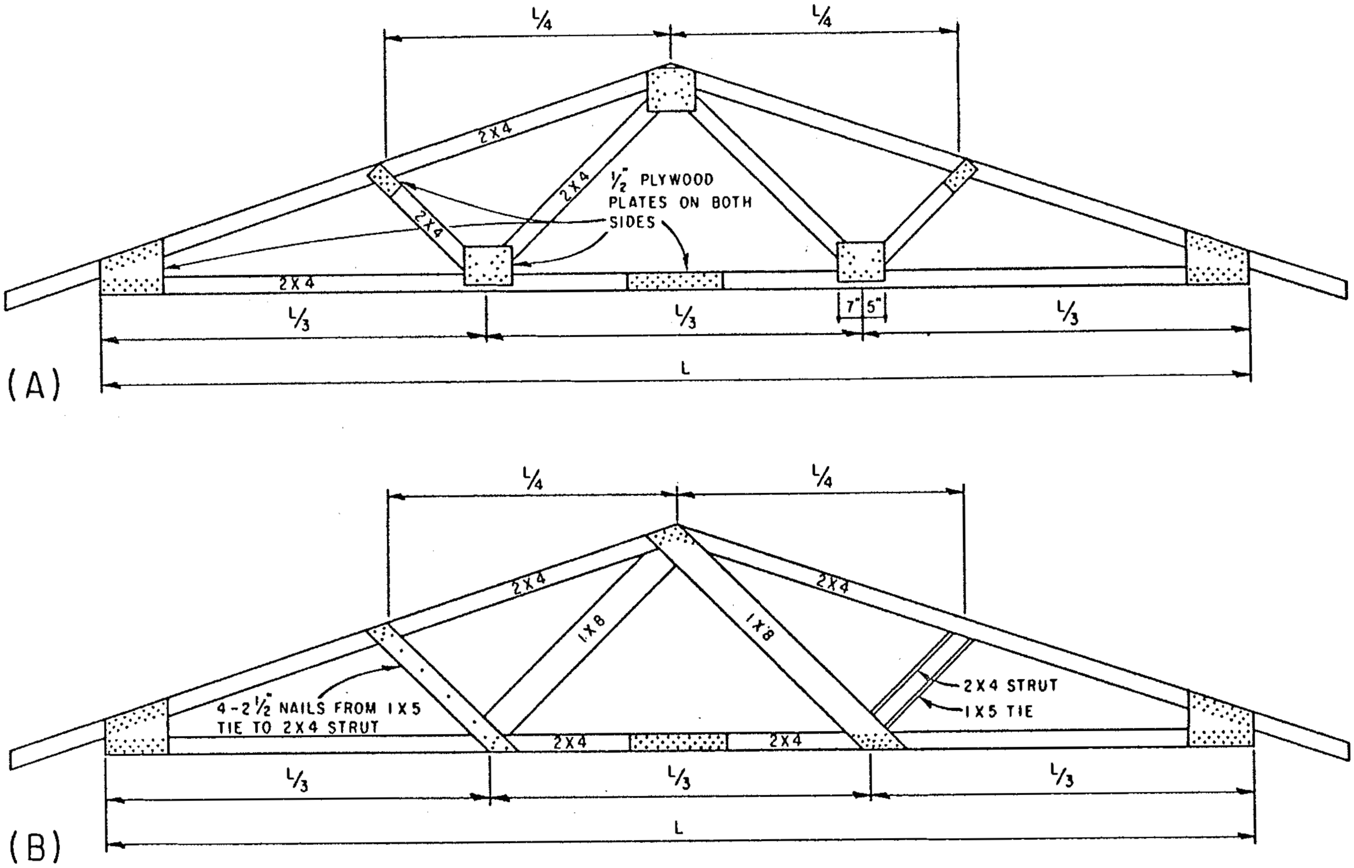


FIGURE 1 NAILED "W" TRUSSES WITH PLYWOOD GUSSETS

BR 3277-2

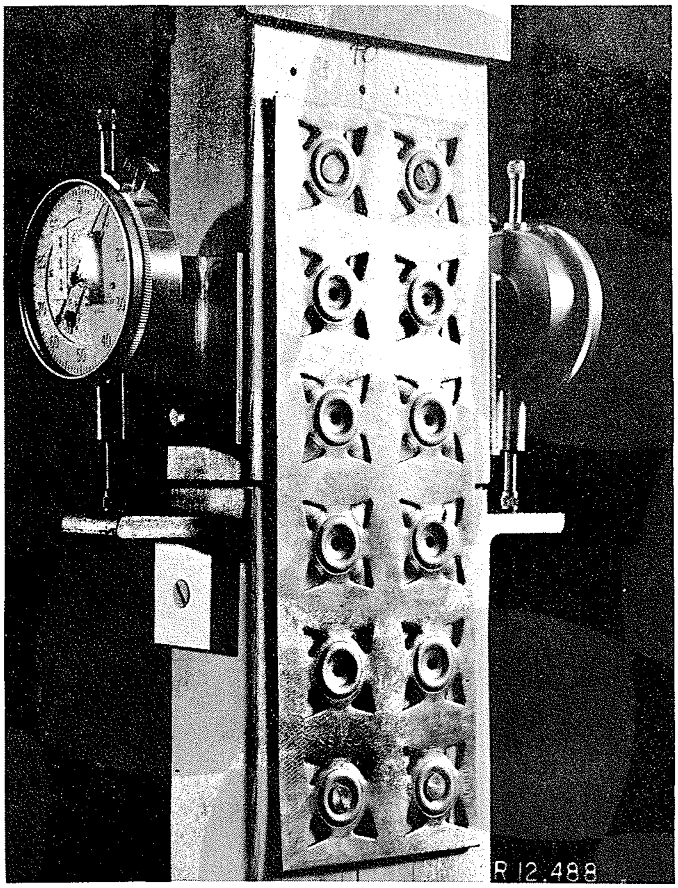


Figure 2 Light-gauge toothed plates in tension test

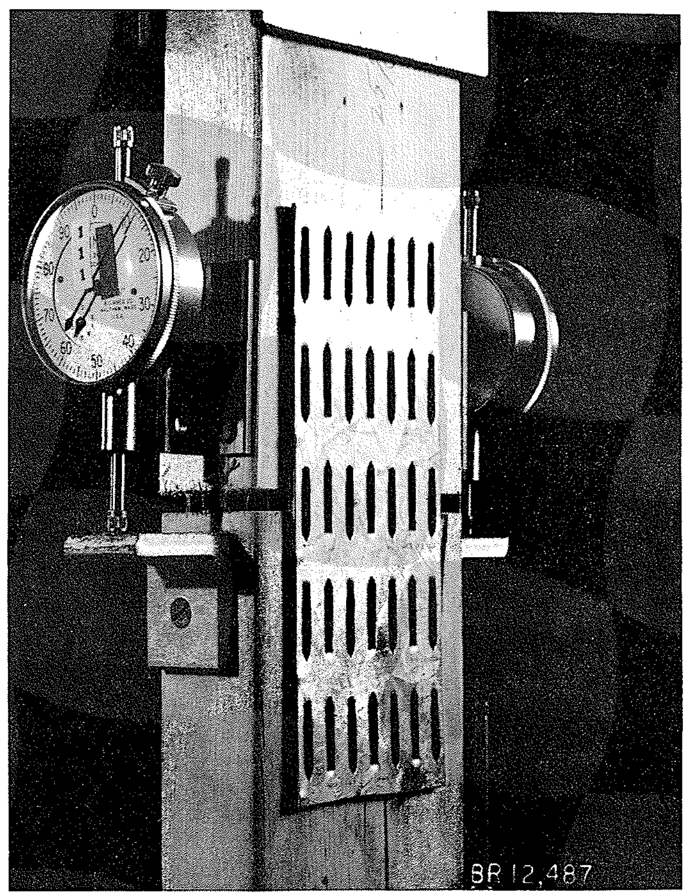


Figure 3 Heavy-gauge toothed plates in tension test

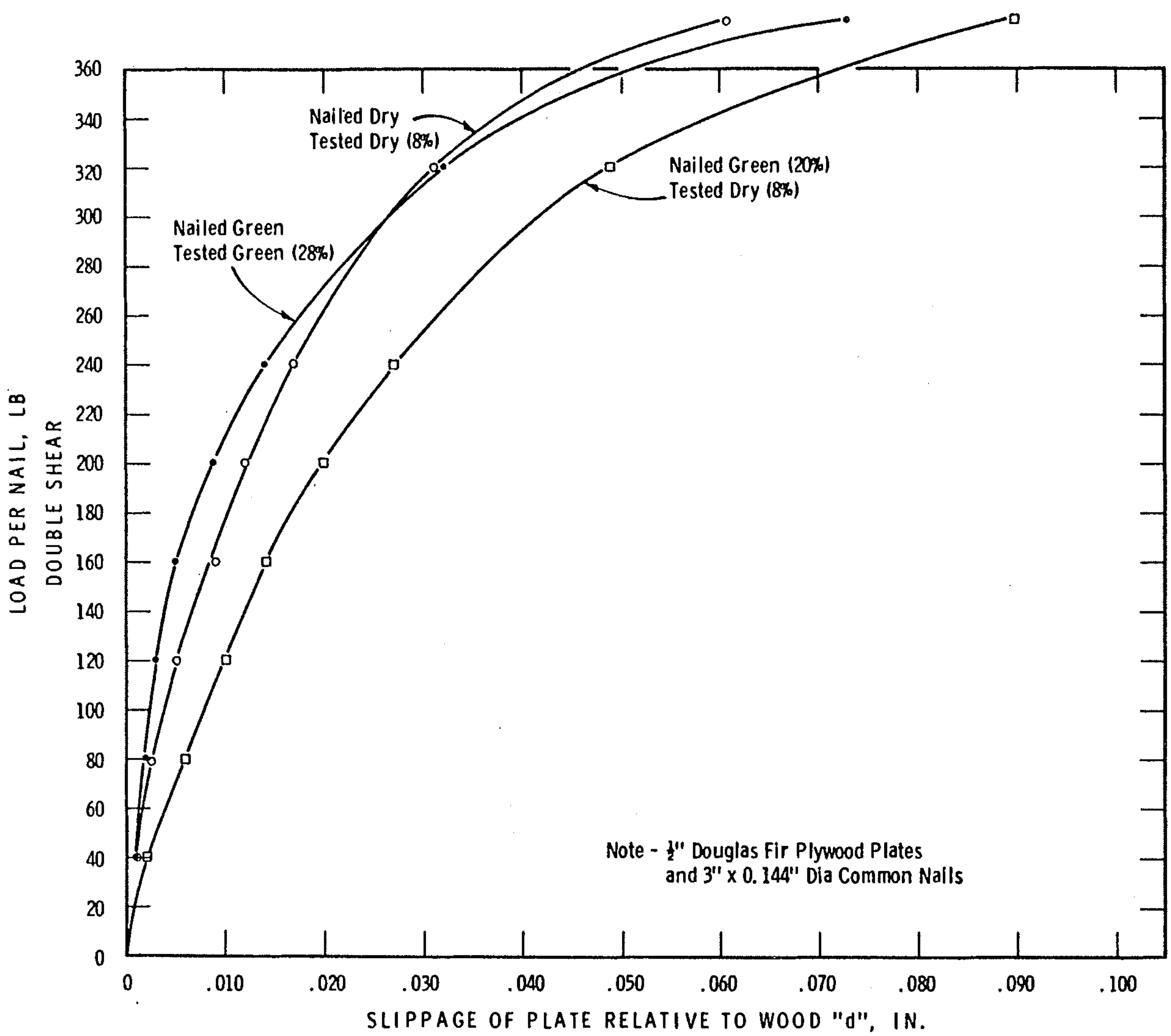


FIGURE 4

SLIP "d" OF PLYWOOD PLATES NAILED TO SPRUCE

6R 3277-3

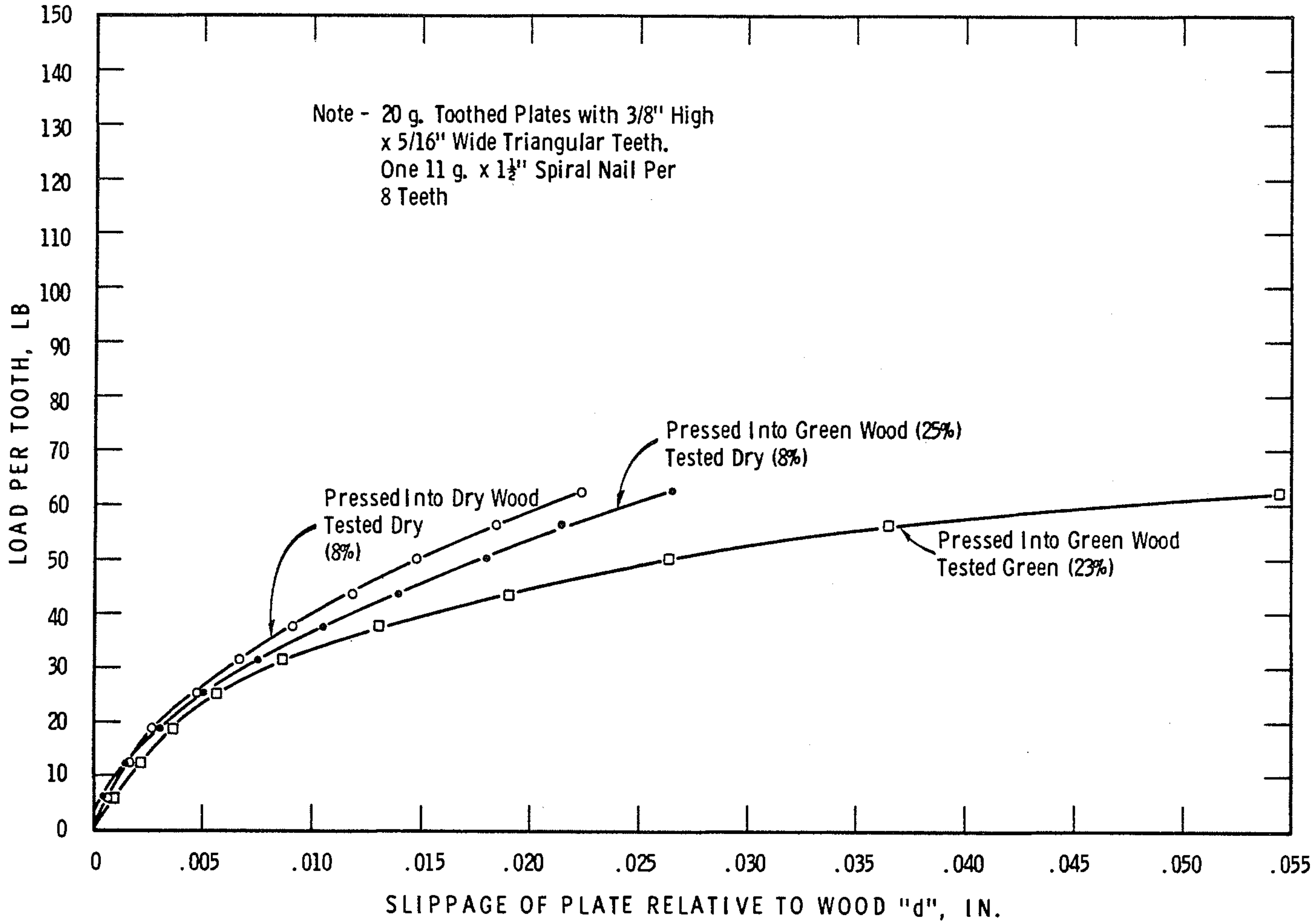


FIGURE 5

SLIP "d" OF LIGHT GAUGE TOOTHED PLATES IN SPRUCE

8R3277-4

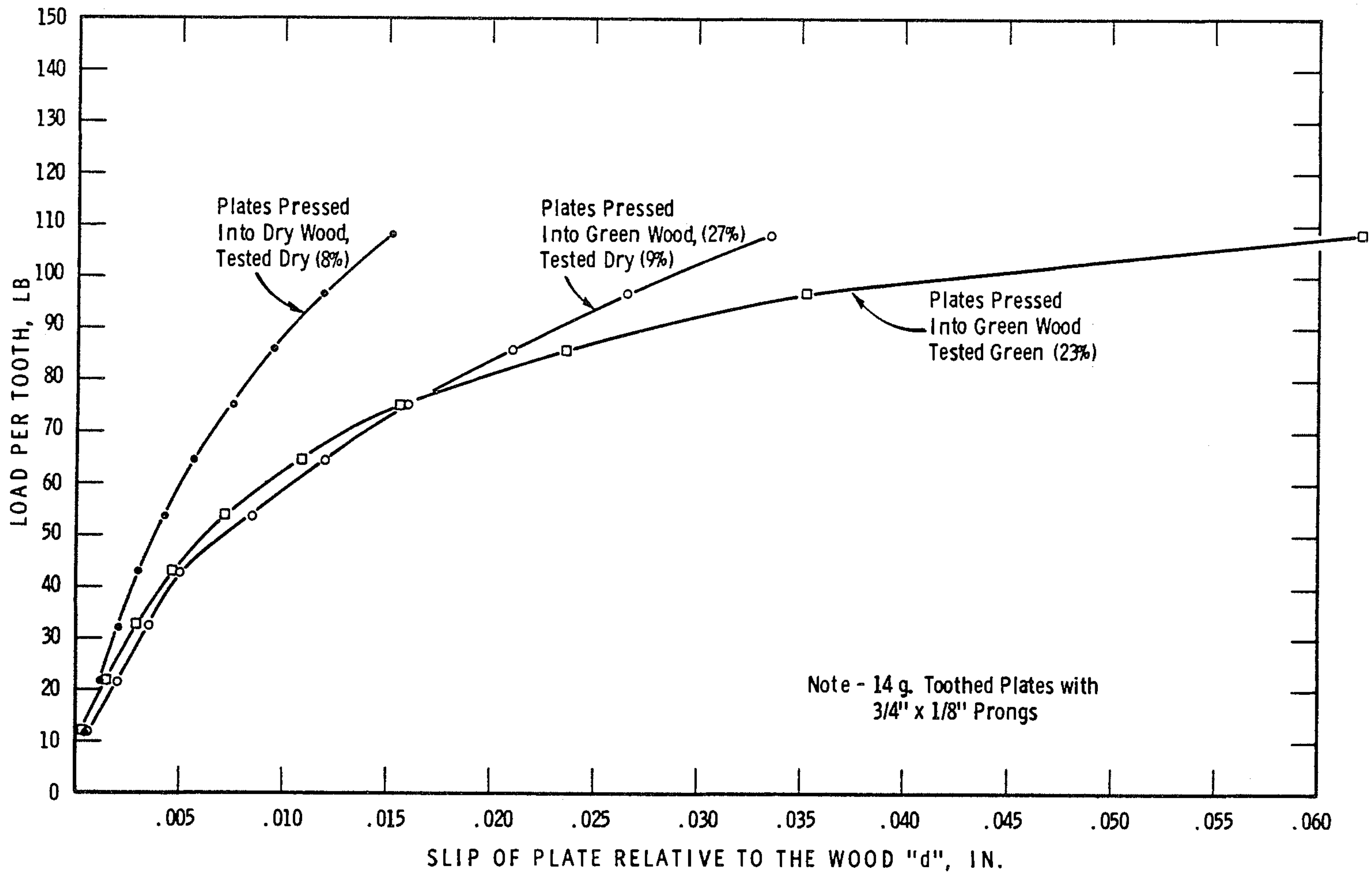


FIGURE 6

SLIP "d" OF HEAVY GAUGE TOOTHED PLATES IN SPRUCE

BR 3277-5

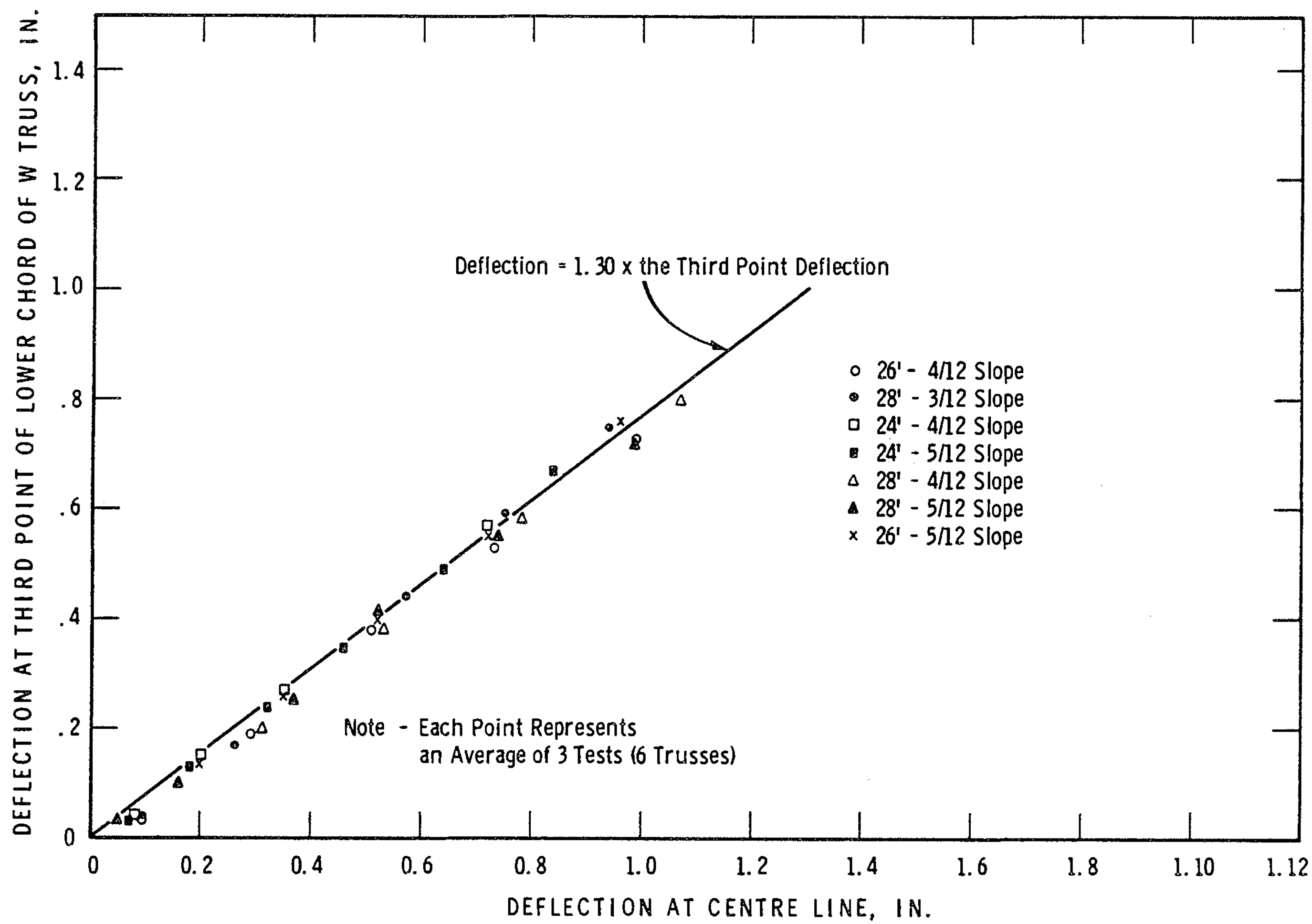


FIGURE 7

RELATIONSHIP BETWEEN THIRD POINT DEFLECTIONS AND MID SPAN DEFLECTIONS FOR NAILED W TRUSSES

BR 3277-6

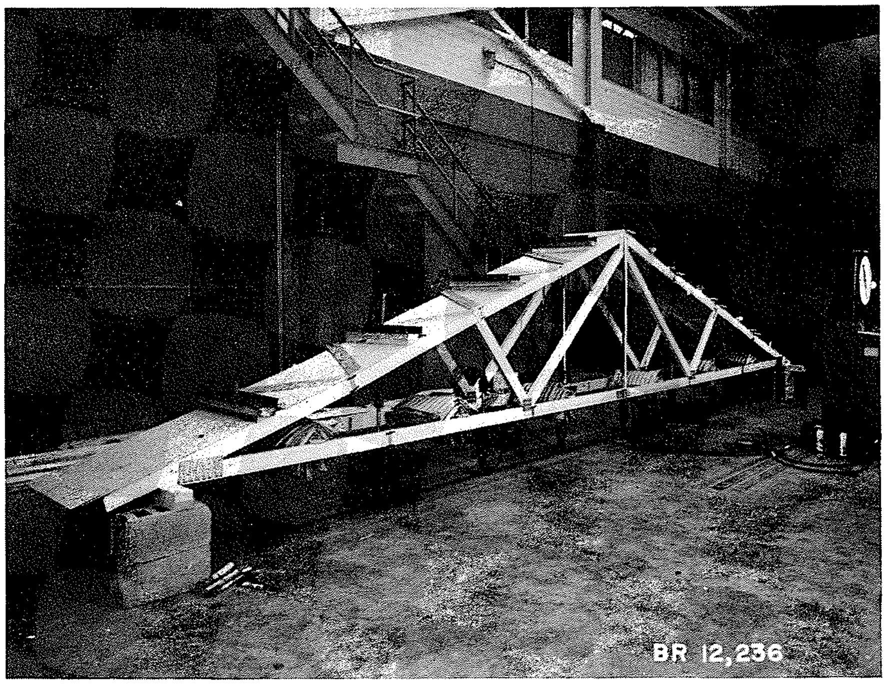


Figure 8 Method of testing complete trusses

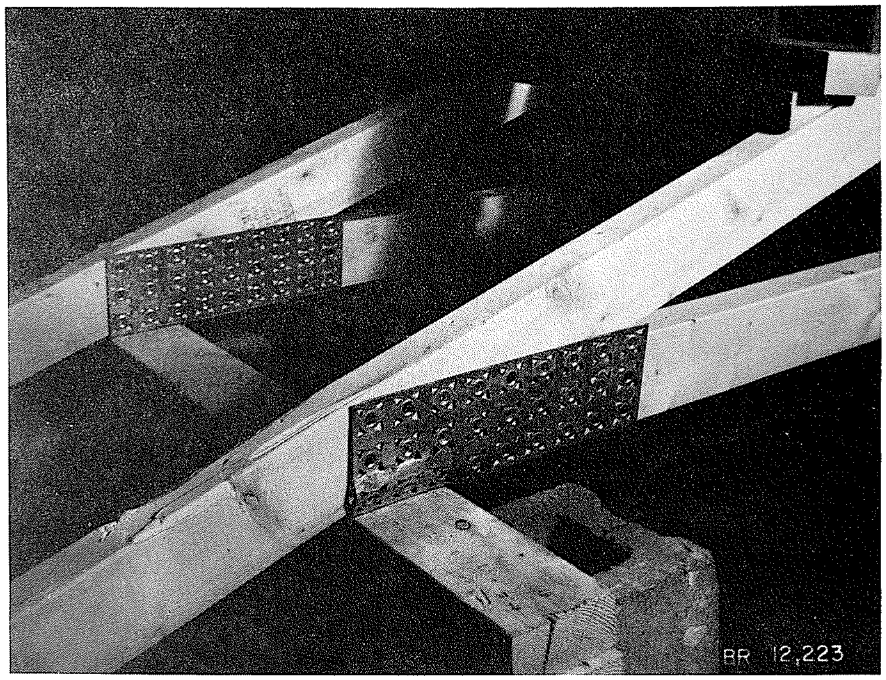


Figure 9 Plate failure at heel of "W" trusses made with light-gauge plates