ANALYSIS AND DESIGN OF THIRTY FOOT SINGLE SLOPE WOOD TRUSSES

Thesis for the Degree of M. S.

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Philip James Mielock

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SINGLE SLOPE WOOD TRUSSES

by

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AN ABSTRACT

Submitted to the College of Agriculture
Michigan State University of Agriculture and
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requirements for the degree of

MASTER OF SCIENCE

Agricultural Engineering Department
1959

Approved Merle L Esmay

The primary objective of this investigation was to analyze and design single slope trusses suitable for farm construction.

A Review of Literature indicated that 4-foot truss spacings were the most economical from the standpoint of lumber and materials. This spacing also allows the use of lighter trusses that do not require special equipment to lift the truss in place.

Stress diagrams were drawn for various possible truss designs. With the aid of the stress diagrams, two truss designs were selected for further analysis. The designs selected could be fabricated with either glue-nail or ring-bolt fasteners. Both designs selected made possible the construction with 2" x 4" and 2" x 6" lumber.

The theoretical deflections of the panel points of each truss were calculated by the use of Williot-Mohr Diagrams. Theoretical deflections were based on the design load of 30 psf of roof with trusses on 4-foot centers.

In order to compare the actual trusses with the theoretical analysis, three trusses of each type fastening and design were tested. There were a total of thirteen trusses tested. These trusses were constructed of commercial Douglas fir, No. 2 or better. Two and one-half inch split-rings and 5/8" exterior-grade plywood were used for the two types of fasteners.

The trusses were tested in the horizontal position, as this allowed for easier means of controlling lateral movement.

The test method consisted of loading the top chord of the trusses by use of 2-inch hydraulic cylinders placed 2 feet apart. Simulated wall-reaction plates served to hold the truss in place while a motor-driven pump applied pressure to the cylinders. Ames dial gauges were used to measure the deflection of points on the trusses. Two or three loadings were made on each truss before it was taken to the failure point.

The test results indicate a greater difference between the types of fasteners than between the two designs. The glue-nail trusses for both designs compared favorably with the theoretical deflections. The ring-bolt trusses deflected considerably more than either the glue-nail or the calculated values. When maximum loads were reduced to simulate a 2-months loading, all the trusses produced load factors of safety greater than one. Reduced to 2-months load equivalent, the ring-bolt trusses had load factors of safety of 1.12 and 1.57. The glue-nail trusses had load factors of safety of 1.85 and 1.93.

The conclusions drawn from this investigation were:

1. From the standpoint of strength and economy when a light framing construction grade is used, both designs analyzed are suitable for farm construction.

2. The increased strength of the glue-nail trusses can be utilized further by increasing the truss spacing.

3. Both designs are easily fabricated and assembly time is held to a minimum, thereby reducing the need for highly experienced erectors.

4. These basic truss designs, when employing glue-nail construction, lend themselves to spans of more than 30 feet, since stresses are evenly distributed in trusses.

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INTRODUCTION

An ever-increasing number of farm buildings are being constructed without interior pole or post obstructions. This type of construction is commonly referred to as clear-span construction.

Clear-span farm buildings offer many advantages over buildings that require interior support. Perhaps the most important, long range advantage of this construction is that the buildings are readily adapted to other uses. With this flexibility, a farm operator can take advantage of changing market demands by switching to another enterprise. Such a switch in enterprises would not result in costly building alterations, as it has in the past.

In addition to the long range flexibility, there are many immediate benefits in clear-span type construction. All forms of mechanization within the building can be used with greater ease and at less cost. Barns employing clear-span construction can be cleaned easier with tractor loaders. Endless feeders and waterers, as in modern poultry houses, are installed with greater ease and with possibly greater choice of location. Gutter cleaners in barns and large dropping pit cleaners in poultry houses have greater flexibility for location.

In storage buildings, the usable area and accessibility of movement of mechanical equipment are increased. Forced air drying is more adaptable since there is no loss of air around poles or posts. Light and air distribution also are increased when interior posts are eliminated.

Clear-span farm buildings, wider than 20 feet, require trusses to support the roof between wall supports. Although steel is used in prefabricated trusses, commercial lumber is a popular material for clear-span farm buildings. Wood is used because of its low cost, availability, ease of fabrication, and strength characteristics. There are many forms of fasteners for these trusses. In general, glue-nail, ring-bolt, and nails are the most common. For 24- to 40-foot spans, the majority of the farm trusses use glue-nail or ring-bolt fasteners.

Spans of up to 40 feet have been found practical for farm buildings. Spans of greater than 40 feet require more fabrication and erection skill, as well as a higher cost per square foot of building.

Spacing of trusses varies from 2 to more than 12 feet in different plans. The larger spacings require considerably heavier trusses and much larger roof girts. The spacing of 4 feet on center allows the use of 2" x 4" girts for metal-covered buildings. Using 4-foot spacing, 1" x 6" tongue and groove material also can be used, if shingles or roll roofing is desired (14). Spacing of 4 feet offers a light weight truss that is easily fabricated and placed in position without excess labor or special apparatus. This spacing can be utilized on any type wall construction. It also offers more economical use of connectors and lumber (9).

The interest in clear-span, gable roofed buildings has brought up the question of eliminating poles or supports from wider lean-tos and additions to existing buildings. Demand for complete buildings with single-sloped roofs is greatly increasing. Single-slope design can provide a greater amount of light in buildings. Snow and rain water can be diverted to one side of a building. In dairy barns, this diversion relieves a sanitation problem. Cows would not have to walk through mud or snow at the building entrance.

While many tested plans of gable type trusses are available for use in farm buildings, there are no available plans for shed or single-slope type trusses. The lack of and the need for trusses of single slope design for farm buildings have been the reasons for this investigation.

REVIEW OF LITERATURE

The factor of safety involved in construction is dependent upon many things. Homes and buildings used by the public frequently have very high safety factors because of possible loss of human life. It is the author's opinion that farm buildings do not require this same high factor of safety.

Previous tests of wooden trusses indicated that theoretical design loads were generally greatly exceeded. This indicates that some factor of safety generally is applied to allowable stresses given for lumber. The design loads for roofs frequently contain another factor of safety as they are unrealistic loads.

The Review of Literature was, therefore, made in four parts:

1. Stresses in Wood
2. Design Loads for Roofs
3. Fastening of Wood
4. Truss Testing

Stresses in Wood

Allowable unit stresses are available for structural grade lumber. According to Johnson (27), the following principles are carried out in stress grading:

1. Every individual timber must be capable of safely carrying its full design load.

2. The working stresses must be applicable to all conditions of use.
3. Timbers must be capable of carrying the full design load for the life of the structure.
4. It is assumed that workmanlike fabrication and design are reasonably good.

Johnson (27) states further that, in order to arrive at an allowable stress for a species, small clear specimens are tested. ASTM (D143-52) "Standard Methods of Testing Small Clear Specimens of Timber" (7) gives the methods for selecting and testing these samples.

ASTM (D245-49T) "Methods for Establishing Structural Grades of Lumber" (7) states that the basic stresses assume normal variability of material, competent design, good fabrication, reliable grading, and adequate supervision. A nominal factor of safety is then provided that permits an occasional slight temporary overload.

Johnson (27) says that twelve factors are taken into account when the "built in" factor of safety is used. The factors included are variability of wood, indeterminancy of stress analysis, oversize resulting from use of standard sizes, depth factor, efficiency of grading rules, efficiency of inspection, range of defects within grade, offsize, duration of load, imperfections in fabrication, temperature, expected versus actual load, and other conditions of service.

Johnson (27) states further that the factor of safety on the most probable timber under the most probable service conditions, as shown by analysis, is 2-1/2 to 3 in bending and shear and 2-1/4 in compression. One timber in 100 has a factor of safety as low as 1-1/4.

Figure 1 is a diagram presented by Wood (41) that shows the range in bending strength values in small, clear, green Douglas fir.

Basic stresses resulting from these tests of clear specimens must be reduced in actual pieces of lumber to allow variations in factors that affect the strength. The factors that affect strength are given in the Wood Handbook (4) as slope of grain, knots, shakes and checks, wane, pitch pockets, holes, and moisture content. Stress grading takes these factors into account when grading of individual pieces is accomplished according to use.

Wood (41) states that the direct effect of drying of wood is the stiffening and strengthening of the wood fibers. In larger timbers, however, this may be accompanied by checking or splitting that largely offsets the gain in strength of the wood fibers. Some increase in strength in drying is recognized in smaller sizes of structural lumber subjected to bending and in compression members of all sizes.

ASTM (D-245-49T) (7) states that it is common practice to take advantage of the benefit of drying, in material four inches or less in thickness, by increasing permissable sizes of knots or other characteristics.

Wood can carry very high loads for short periods. Johnson (27) relates that engineers have unknowingly used

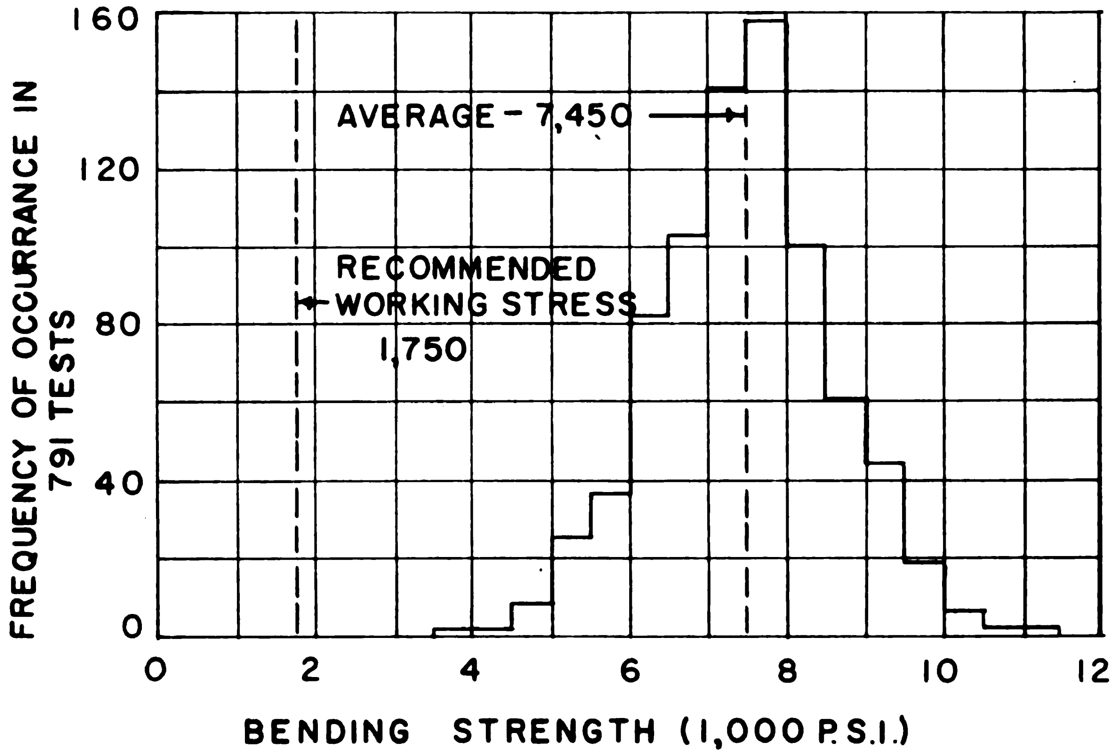


FIGURE I - FREQUENCY DIAGRAM SHOWING THE RANGE IN BENDING STRENGTH VALUES IN SMALL SPECIMENS OF CLEAR, GREEN DOUGLAS FIR.

that characteristic of wood for years by increasing the stresses they have used in designing for wind, impacts, and snow loads. The ASTM (D-245-49T) (7) states that advantage can be taken of this characteristic in many structural designs. Figure 2 taken from ASTM (D-245-49T) (7) shows the relation of basic stress to duration of maximum load. This same relationship holds in converting short-time test loads to longer loading periods.

Design Loads for Roofs

Barre and Sammet (1) recommend a minimum of 20 pounds per square foot for snow load. In northern areas of the United States, these figures should be increased according to figures by the National Bureau of Standards (33). In Michigan these figures vary from 20 pounds per square foot in the southern part of the State to 35 pounds per square foot at the extreme northern part of the Upper Penninsula. These loads are not specified for farm buildings.

Radcliffe and Granum (35) state that they used live snow load of 25 pounds per square foot for trusses for house design because it conformed to standard practice. They state further that such loads probably will not occur and lead to conservative designs.

Wind pressures on roofs vary with wind direction and orientation of any opening. Many different investigators have measured these effects and results show that, for slopes of less than 20°, the pressures are fairly constant over the surface. The National Bureau of Standards (33) gives a force

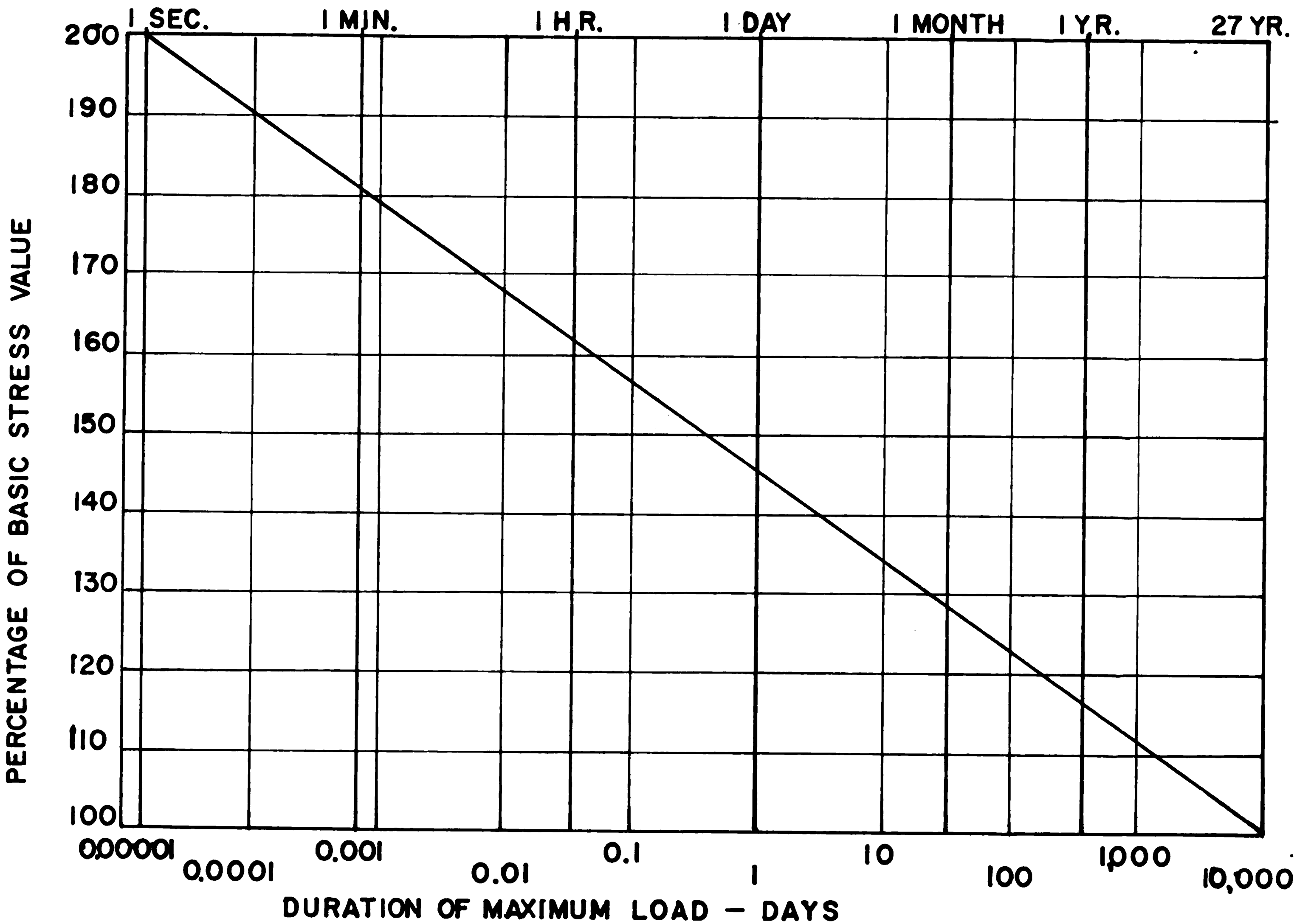


FIGURE 2 RELATION OF BASIC STRESS TO DURATION OF MAXIMUM LOAD

coefficient of -0,6 on the windward side and -0.45 on the leeward side for slopes of 20° or less.

Giese and Henderson (24) state that, because live loads in farm buildings can be determined rather closely, the high factor of safety is not required for farm construction.

Fastening of Wood

While many types of fasteners for wood are known, this review deals chiefly with split-ring connectors and casein and resorcinol resin glues.

The Design Manual for TECO Timber Connectors Construction (39) gives the design material for split-rings. The manual explains that where the wood, not strength of the rings, determines the load capacity the design figures can be increased. The National Design Specifications for Stress-Grade Lumber and its Fastenings (34) gives an allowable increase in stresses for both lumber and the fastenings for short time loading. These increases are 15 percent for two months' duration, as for snow; 25 percent for seven days' duration; 33-1/3 percent for wind and earthquake; and 100 percent for impact.

Markwardt (29) indicates that one of the chief reasons that wood is being used more for structural material is because of the developments in glue and gluing techniques.

Giese (23) states that one of the important reasons for using glue is that the bearing area of joints may be increased to almost any size, thus making possible a more uniform stressing throughout the structure.

The glues most commonly used in farm construction are casein and resorcinol resin. The main difference in these glues is in the ability to withstand moisture.

Resorcinol resin, first introduced in 1943 for use in aircraft and naval vessels, is waterproof. The Forest Product Laboratory Report Number 1336 (20) states that resorcinol resins are as durable as the wood itself. The report points out, however, that manufacturers do not recommend curing resorcinol resins glued below 70°F.

Casein glues have been in existence for many years and are continually being improved. Traux (37) says casein glue produces joints in most of the common species of wood that are equal to or greater than the strength of wood itself.

McLaughlin (30) states that while a casein glue line is weakened by the presence of moisture, it will regain its original strength when again dried out. Kaufert (28) points out, however, that continuous moisture or alternating moist and dry conditions eventually will weaken the joint.

Dunsan (13) found that casein joints, when exposed to bacteria capable of digesting casein, were not materially weakened when the moisture content of the wood was below approximately 55 percent.

As to the durability of casein glue, McLaughlin (30) states that this type of glue has been used in the United States for 30 years and even longer in Europe with satisfying results. One of the conclusions of Giese and Henderson (24) was that casein glue is of ample durability if the joint is protected from direct action of water.

Review indicates a wide variation in shear stress for glue. Boyd (9) notes that a unit stress of 200 psi is practical for farm buildings. Giese and Henderson (24) recommend 430 psi be used as a design stress for casein glue joints loaded parallel to the wood grain for farm construction with Douglas fir. A stress of 215 psi is recommended for joints loaded perpendicular to the grain.

Truss Testing

The ASTM (E-73-52) (7) indicates that the number of tests of like trusses depends on the desired accuracy and reliability of the results and purpose of the test. The article recommends a minimum of three tests for reliable and accurate results.

The article further states that compression chords of the truss should be braced laterally if it does not interfere with the free vertical deflection of the truss. The lateral bracing should duplicate as far as possible the effect of the complete structure.

ASTM (E-73-52) (7) allows load increments of 25 percent of design load up to design load. After design load is reached, additional increments of 50 percent of design load may be used. Rate of loading should be as uniform as possible and each increment should remain on about five minutes so that the truss may come to rest.

Boyd (9) tested a gable truss intensively with strain gauges while using small aircraft cylinders placed 2 feet on center to apply the load. This truss was tested in a horizontal position instead of the common standing position.

Boyd (9) found that secondary stresses caused by plywood gusset plates in a Fink truss could be neglected when using standard practices, as the factors of safety and design procedures make adequate allowances.

While the Williot diagram assumes pinned joints, it is common to use this method for trusses with all types of joints. Grinter (2) indicates that this method is widely used when a graphical layout is preferred. In combination with the Mohr diagram, the Williot diagram gives the deflection of all panel points of the truss.

From results observed in testing gable trusses, this author noted that the heel joint strength could be improved by use of a wedge between the lower and upper chords in the ring-bolt truss.

Acre (6) found, when comparing several heel joint desings, that a birds mouth cut in the upper chord at the reaction could increase the strength of the joint over the conventional method.

THE INVESTIGATION

Preliminary Considerations

Modulus of Elasticity

The Review of Literature indicated moisture content influences the stiffness and strength of wood. Representative samples of the lumber used in trusses were, therefore, tested for modulus of elasticity and moisture content. Each piece was selected to be clear grained and knot free.

The samples were tested with simple center loading. The reactions and loading piece were rounded hardwood blocks, as specified for this test in ASTM (E-73-52) (7). The span between reactions was 2 feet. SR-4 gauges placed on the samples were used to measure strain.

After testing, small pieces of each sample were placed in an oven to determine moisture content.

The values for modulus of elasticity obtained ranged from 1.49×10^6 to 2.55×10^6 psi. Most of these figures were higher than the value of 1.6×10^6 psi given for modulus of elasticity for Douglas fir in the Wood Handbook (4). Part of the high values for moduli of elasticity can be attributed to the lumber being very dry at the time of the test. Moisture contents for the moduli of elasticity samples ranged from 4.5 to 7.2 percent. Other moisture samples, not checked for modulus of elasticity had a range in moisture content from 9.2 to 15.4 percent.

According to the Wood Handbook (4), the modulus of elasticity given in design information is evaluated for longer time intervals then this test used. Wood has the property of deflecting gradually over a period of time after a load is applied. This fact coupled with the low moisture content could appreciably change the value of modulus of elasticity.

Boyd (9), after testing, used 1.96×10^6 psi to determine theoretical truss deflections. However, the author's test showed higher values for the small pieces, larger members containing defects and flaws would reduce the total modulus of elasticity. From this consideration, 1.6×10^6 psi was used for theoretical design and deflections.

Design Stress of Lumber

The lumber recommended on Michigan State plans for trusses is Douglas fir, Number 2 or better (or equivalent). This corresponds to the Construction Light Framing Grade in new grading system. The single slope truss also would carry this specification.

Since the Construction Light Framing Grade does not carry a stress grade, some allowable stress had to be selected for design. Barre and Sammet (1) give allowable unit stresses for stress graded lumber. The stress of 2150 psi is given for select structural joists and planks of Douglas fir in tension parellel to grain and for extreme fiber in bending. Allowable unit stress for compression parallel to grain is 1750 psi.

The 1957 ammendments to National Design Specifications for Stress-grade Lumber and its Fastenings (34) gives ranges of 1200 to 2050 psi for tension parellel to grain or extreme fiber in bending, depending on grade of joist and planks. The values for compression parallel to grain range from 1000 to 1650 psi, again depending on grades.

According to the National Design Specifications for Stress-grade Lumber and its Fastenings (34), these stresses can be increased by 15 percent for snow loads, based on two months of loading.

Considering this allowable increase and realizing that the best yard-grade lumber may not approach stress-grade lumber, a stress of 2000 psi in extreme fiber in bending was used for design. The allowable compressive stress used was 1600 psi. This value used in conjunction with the 30 pounds per square foot roof load still should allow a considerable factor of safety.

Theoretical Analysis

Selection of Truss Design

Many different 30-foot single slope truss designs are possible. This investigation is not intended to cover all the variations. The basis for consideration in design was balanced stressing between members, ease of construction, simplicity of design, ability to use available lengths of lumber without waste, and economy. In addition, designs that allowed the use of both glue-nail and ring-bolt fasteners were used.

Because of the intended use of the trusses, two-point-type support was selected. Indeterminate, three-point support, would complicate problems of additions to existing structures. Two-point support also gives more head room for storage. Frequently a single slope roof covers a hay feeding shed attached to a hay storage barn. This operation requires more room than is possible with a three-point supported truss.

Various possible truss designs were checked with stress diagrams. The load for each member indicated possibilities of balanced design.

Some possible trusses were eliminated because fabrication with rings and bolts was not possible without members crossing planes.

Two 30-foot trusses were selected from the remaining designs. Each of these trusses had the top chord supported in three places, in addition to the end supports. With this span, the stress due to bending plus the axial stress allowed the use of 2" x 6" lumber in the top chord.

The two basic designs selected are shown in Figures 3 to 6 with both types of fasteners. Figure 7 shows the designs after they were constructed.

Roof Design Loads and Loading Points

Twenty-five pounds of live load per square foot were used in design. This, in addition to five pounds dead load, gave a total load of thirty pounds per square foot of horizontal roof projection. This is the loading given for most areas of

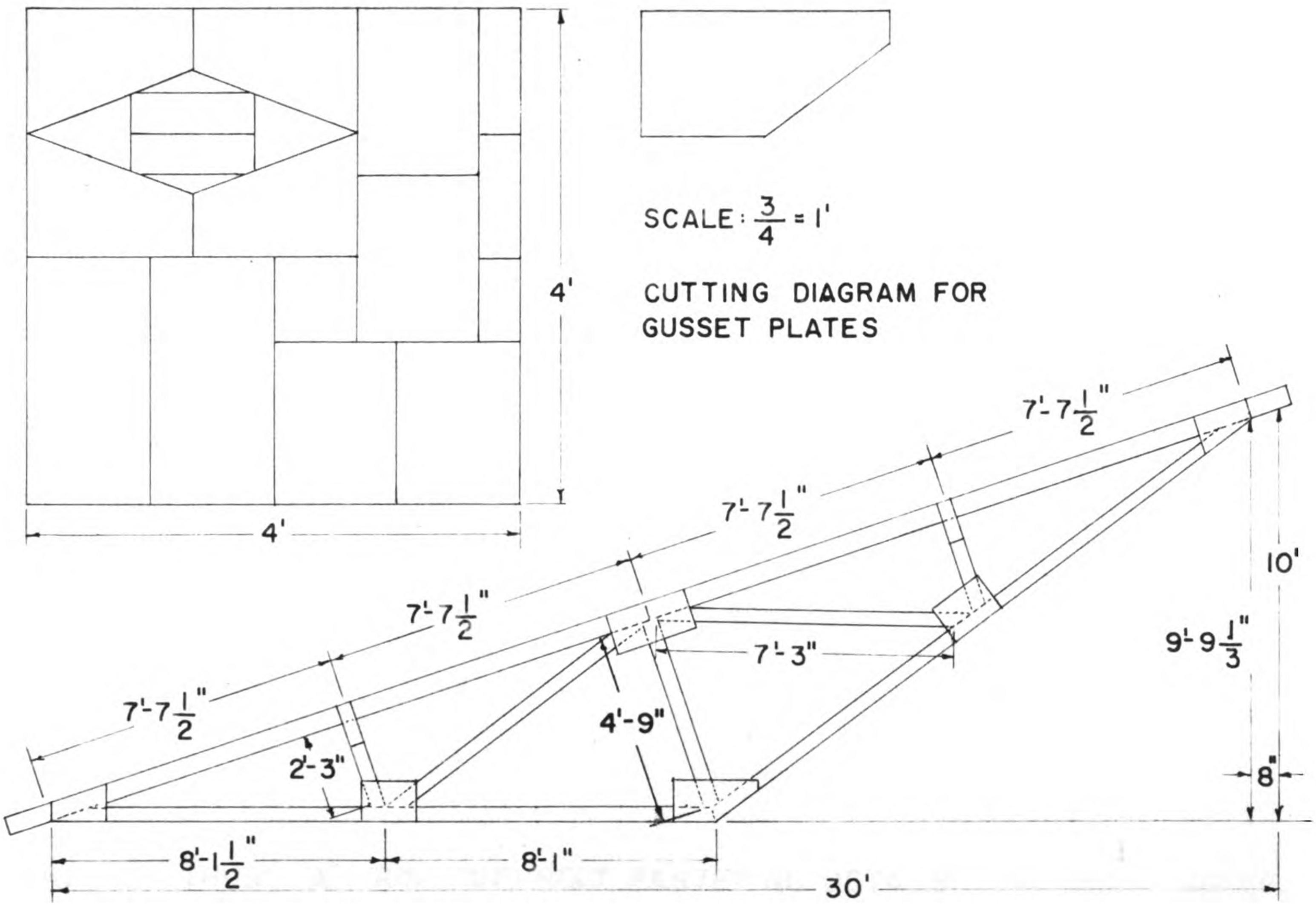


FIGURE 3 TRUSS "A" – GLUE-NAIL FASTENING DETAILS

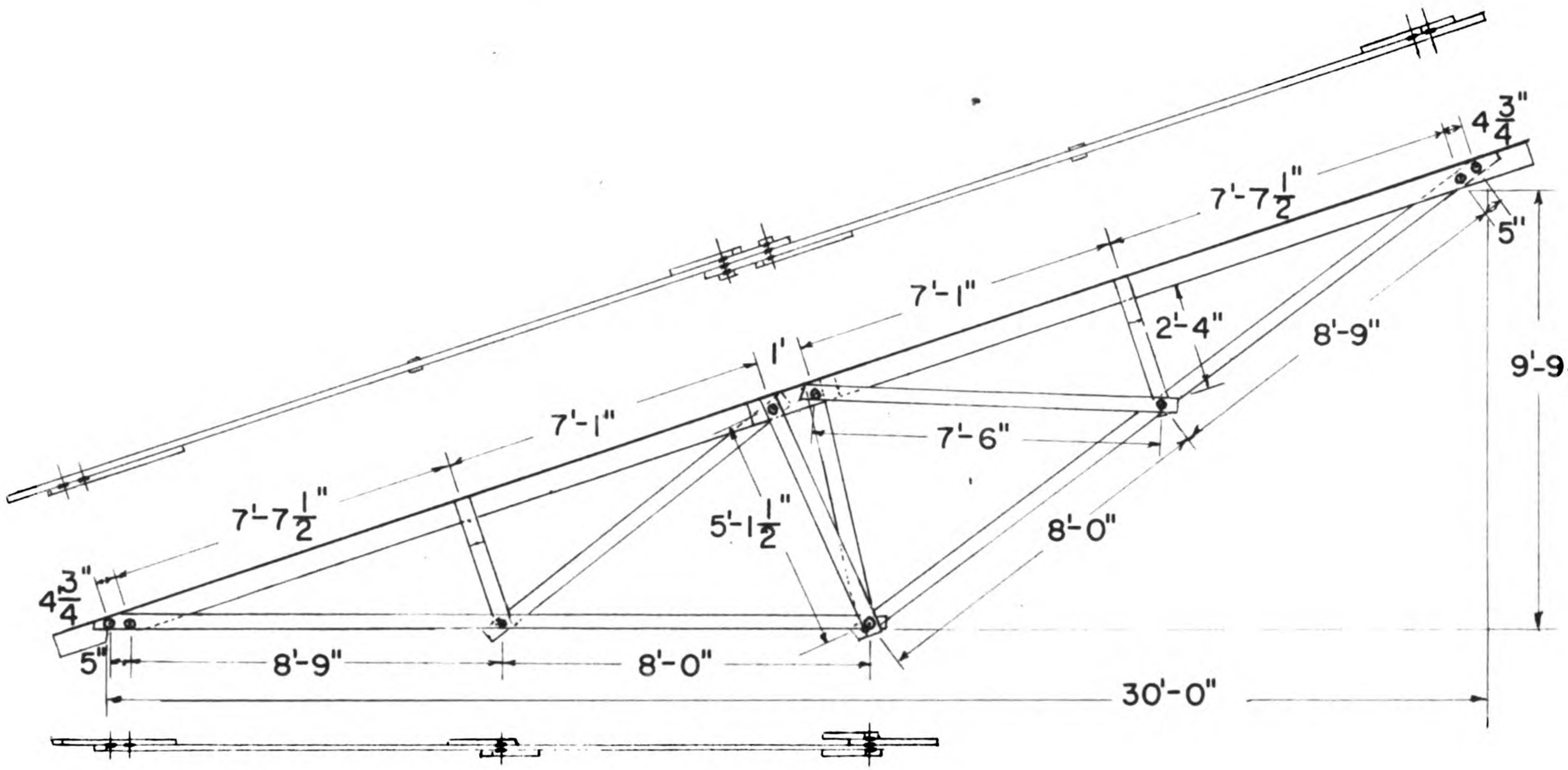
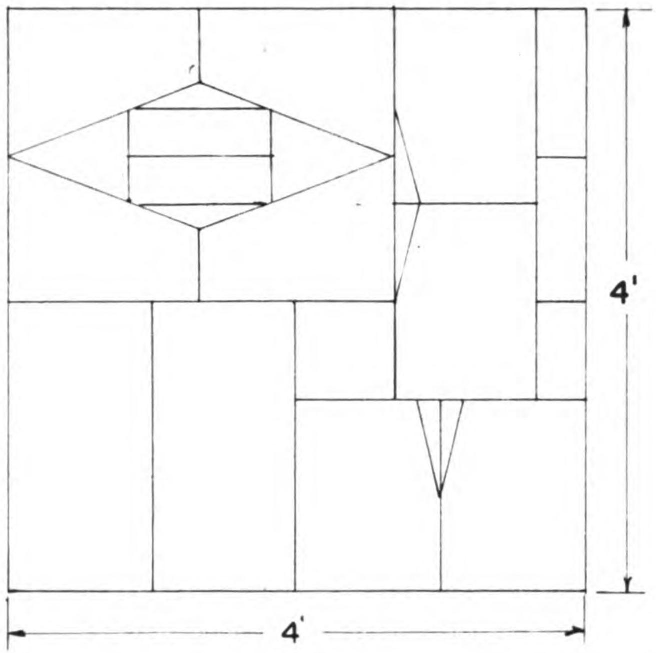


FIGURE 4 TRUSS "A" - RING AND BOLT FASTENING DETAILS



SCALE: $\frac{3''}{4} = 1'$

CUTTING DIAGRAM FOR
GUSSET PLATES

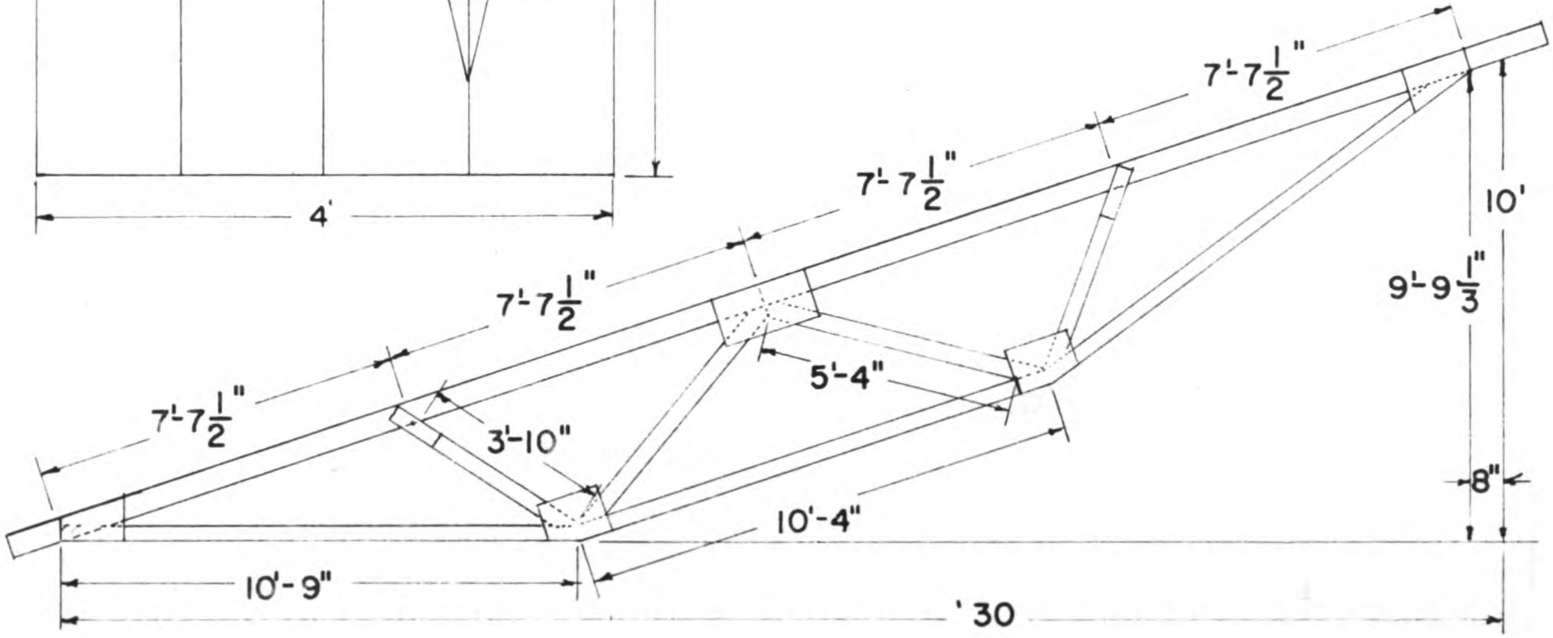


FIGURE 5 TRUSS "B"— GLUE-NAIL FASTENING DETAILS

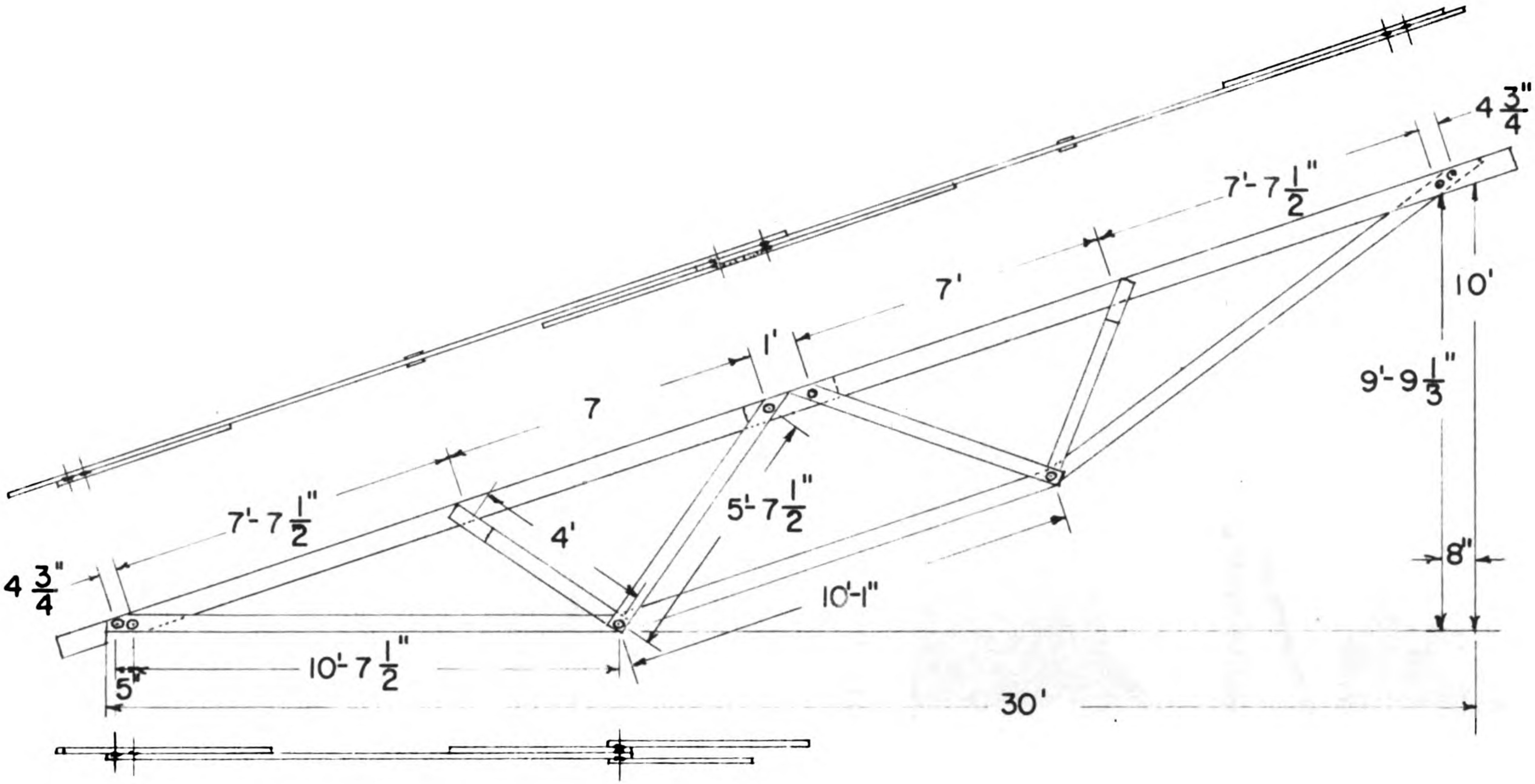


FIGURE 6 TRUSS "B"- RING AND BOLT FASTENING DETAILS

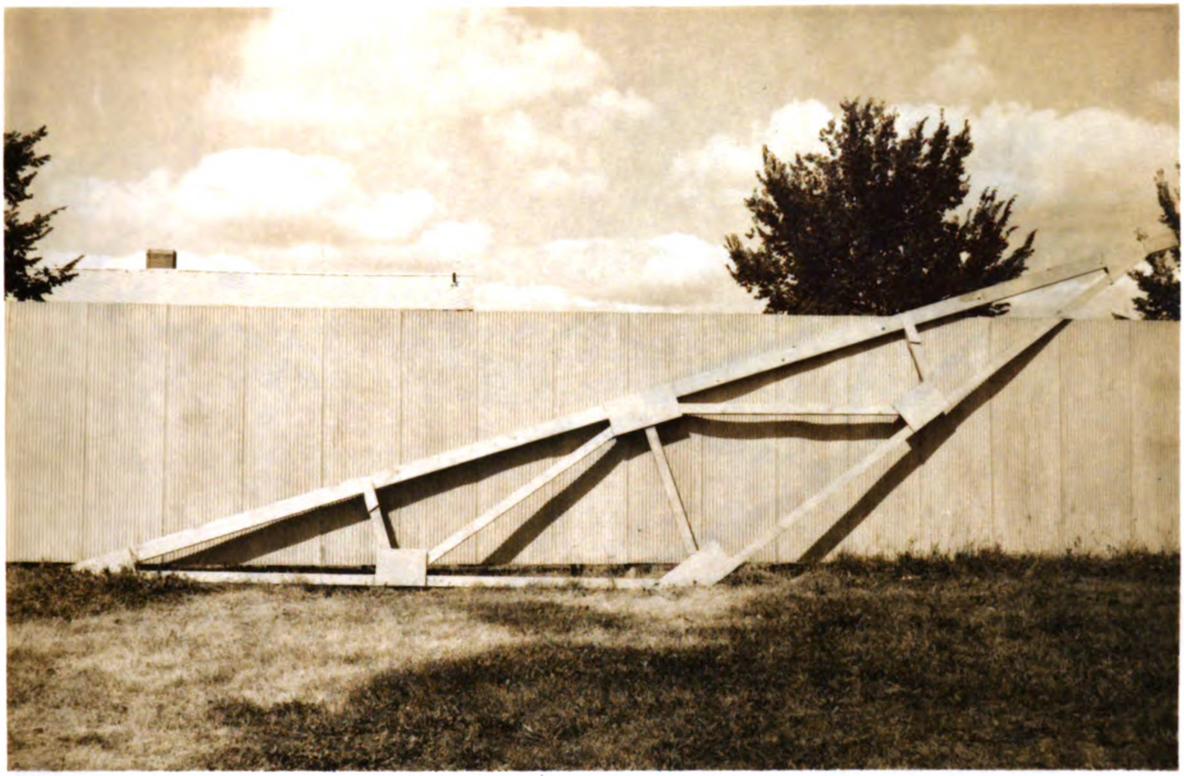


FIGURE 7a - TRUSS "A" - GLUE-NAIL CONSTRUCTION

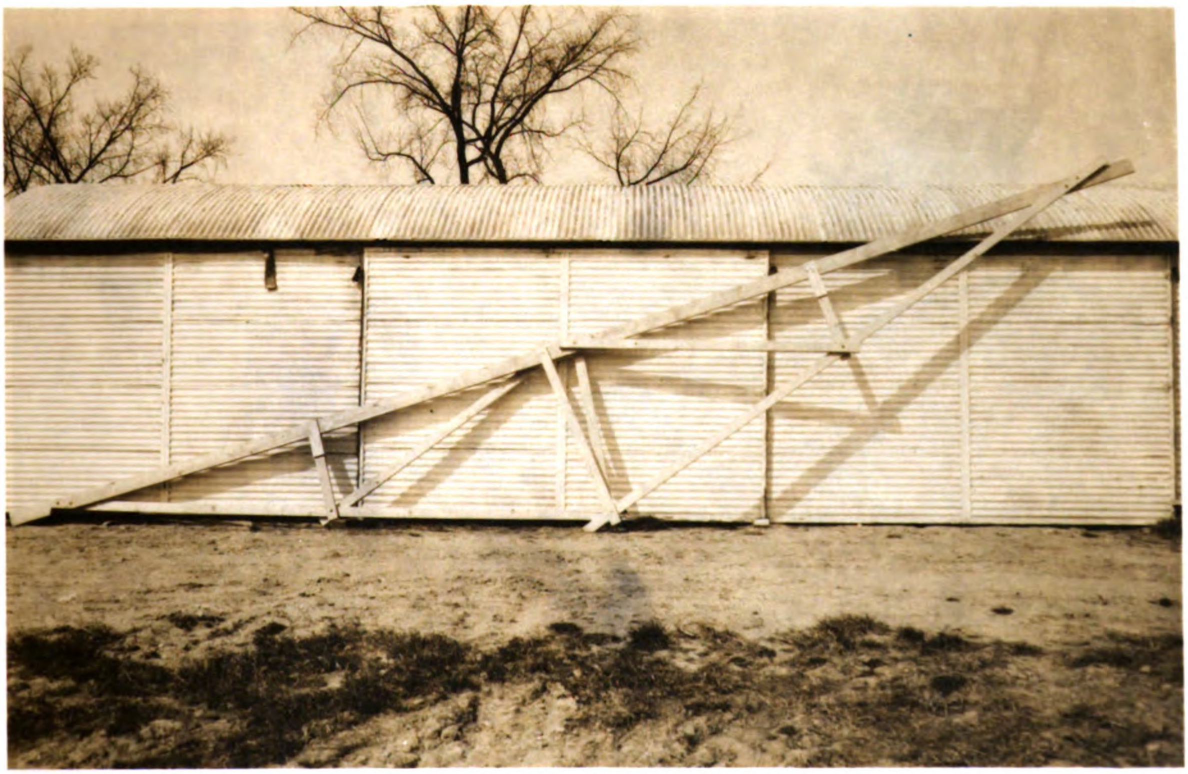


FIGURE 7b - TRUSS "A" - RING-BOLT CONSTRUCTION

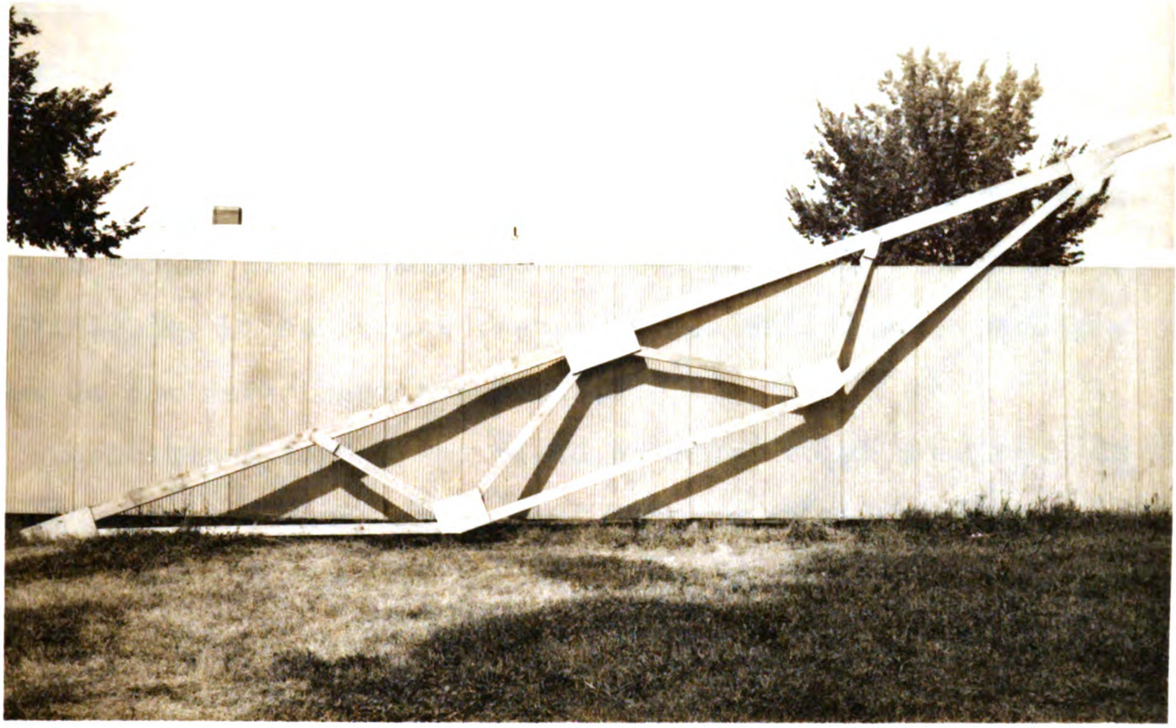


FIGURE 7c - TRUSS "B" - GLUE-NAIL CONSTRUCTION

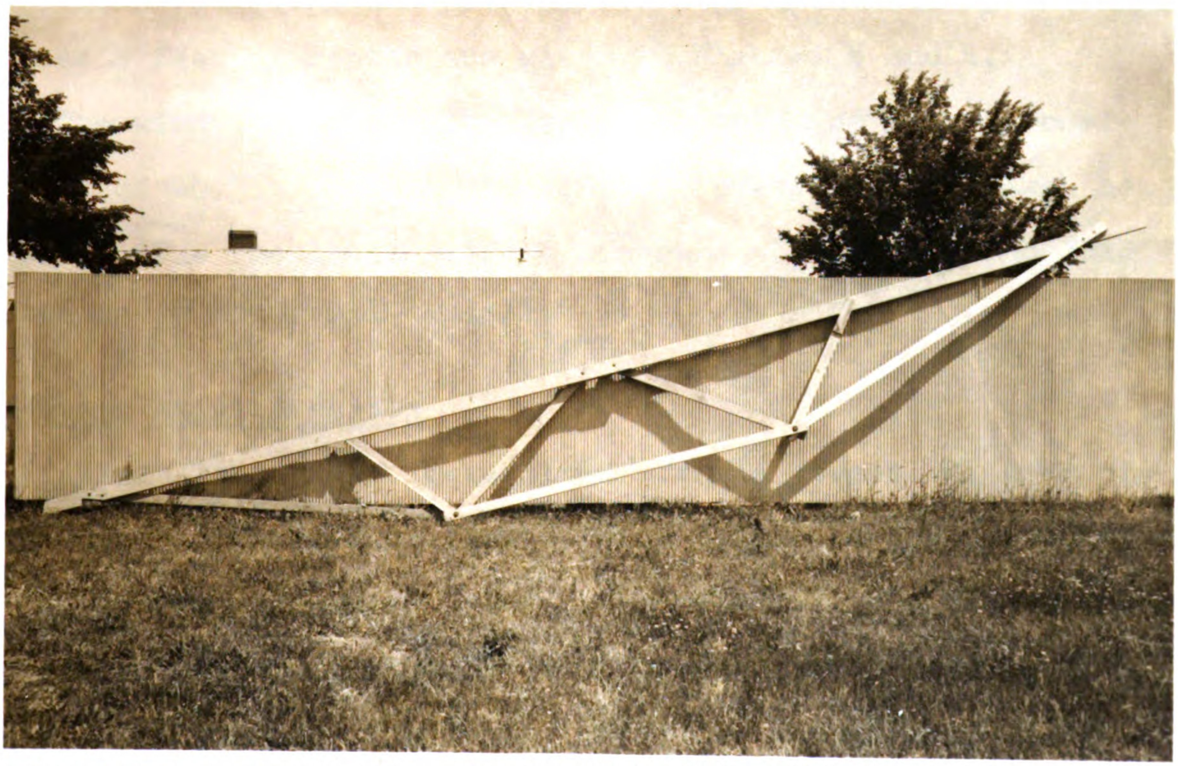


FIGURE 7d - TRUSS "B" - RING-BOLT CONSTRUCTION

Michigan by the National Bureau of Standards (33). These design loads are given for all types of building uses. If allowable stresses given in the Wood Handbook (4) are used in addition to these roof loads, the result will be a building that is rather conservative for farm use. While no information is available, it is apparent that many new buildings on Michigan farms do not meet these standards.

Theoretical design was based on panel point loading. In actual practice with metal roof construction, the load is applied every 2 feet of truss. This would correspond to 2" x 4" girts placed on 2-foot centers. For theoretical purposes, the additional stress due to bending is small, so that the assumption of panel point loading is not in great error.

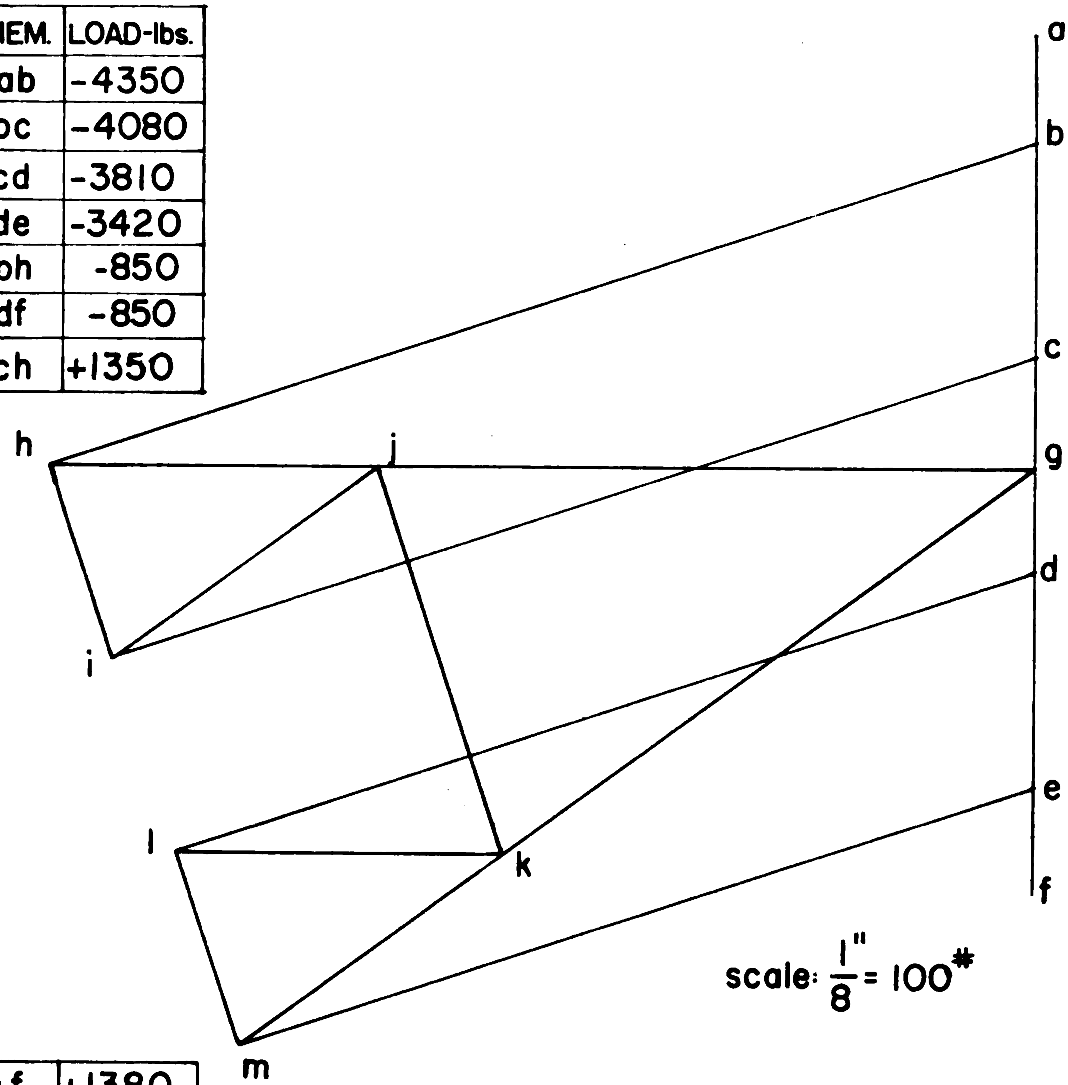
The stress diagrams for the design load of 30 psf with trusses 4 feet on center are given in Figures 8 and 9.

The wind load on the trusses was based on a force coefficient of -0.5. The wind velocity pressure was taken as 20 psf. The resultant stress diagrams from this wind load are shown in Figures 10 and 11.

Computing Axial Stresses

Axial loads were determined from a stress diagram of each truss. The design load carried by each truss was based on 30 pounds per square foot of horizontal projection and truss spacing of 4 feet. The appropriate standard size lumber was selected to fit the loads given by the stress diagram. In the top chord, the additional stresses due to bending also were considered in selecting lumber size.

MEM.	LOAD-lbs.
ab	-4350
bc	-4080
cd	-3810
de	-3420
bh	-850
df	-850
ch	+1350



cf	+1380
cg	-1750
ah	+4160
hg	+2800
gf	+2760
fe	+4160

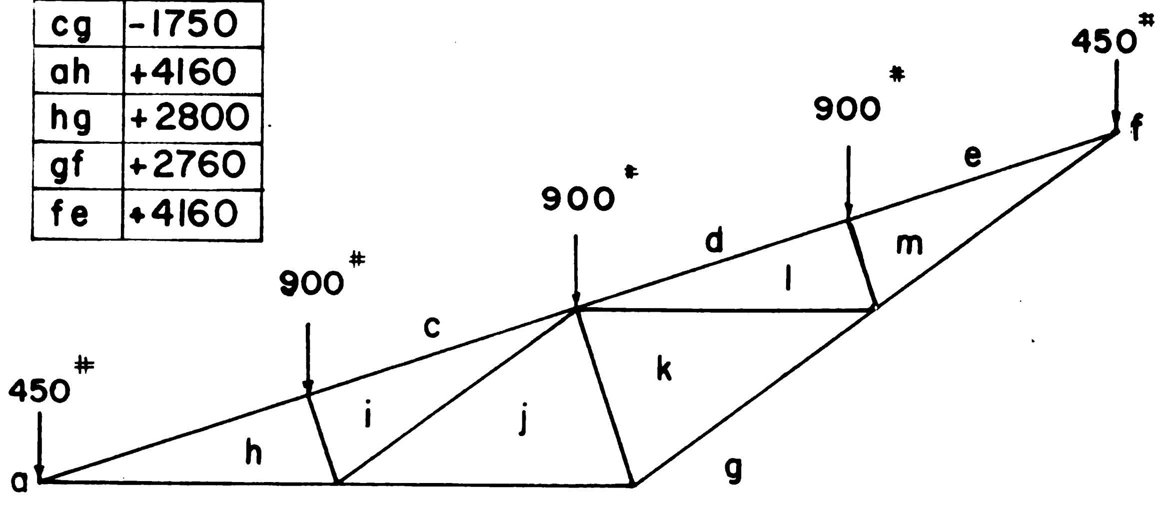
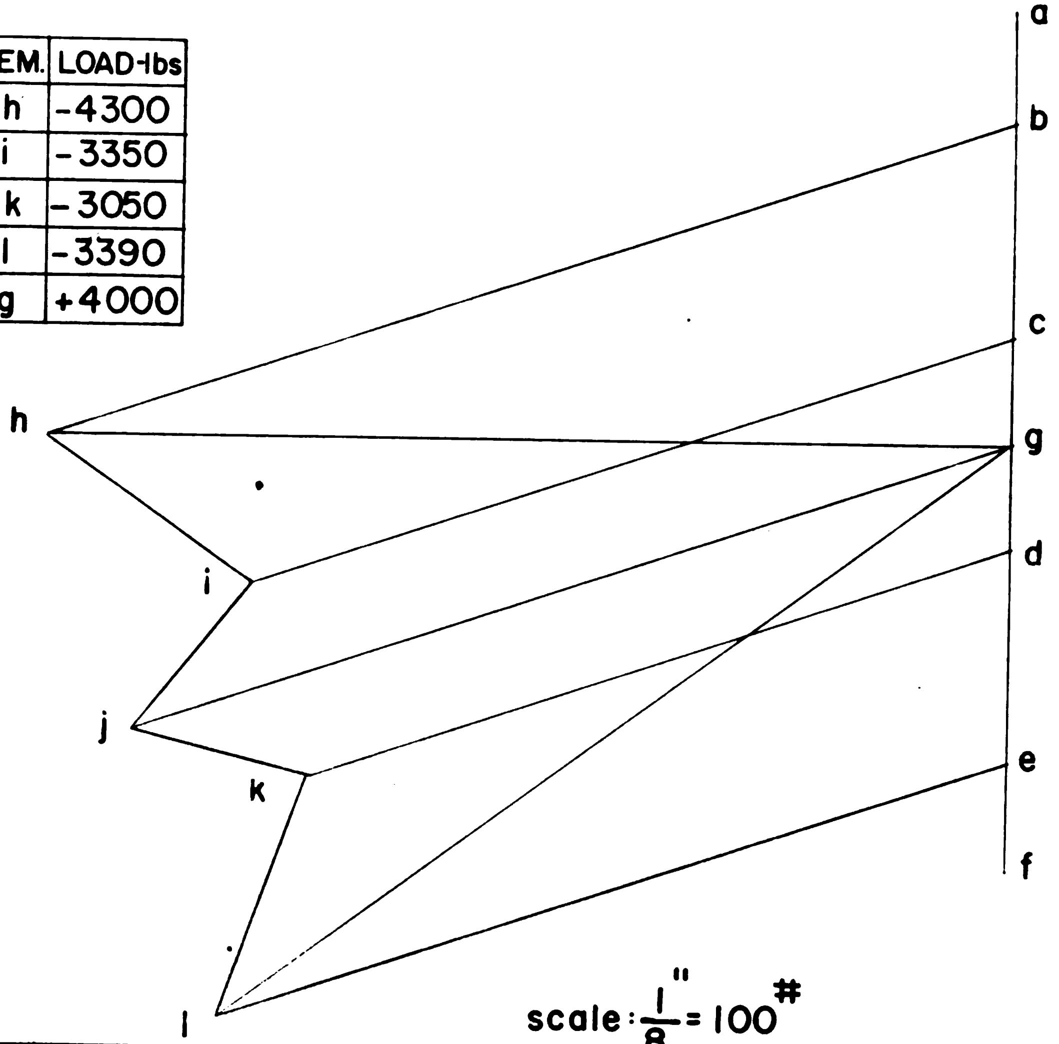


FIGURE 8 STRESS DIAGRAM OF TRUSS "A"
LIVE PLUS DEAD LOAD

MEM.	LOAD-lbs
bh	-4300
ci	-3350
dk	-3050
el	-3390
lg	+4000
jg	+3900
hg	+4050
hi	-1050
ij	-780
jk	-760
kl	-1060



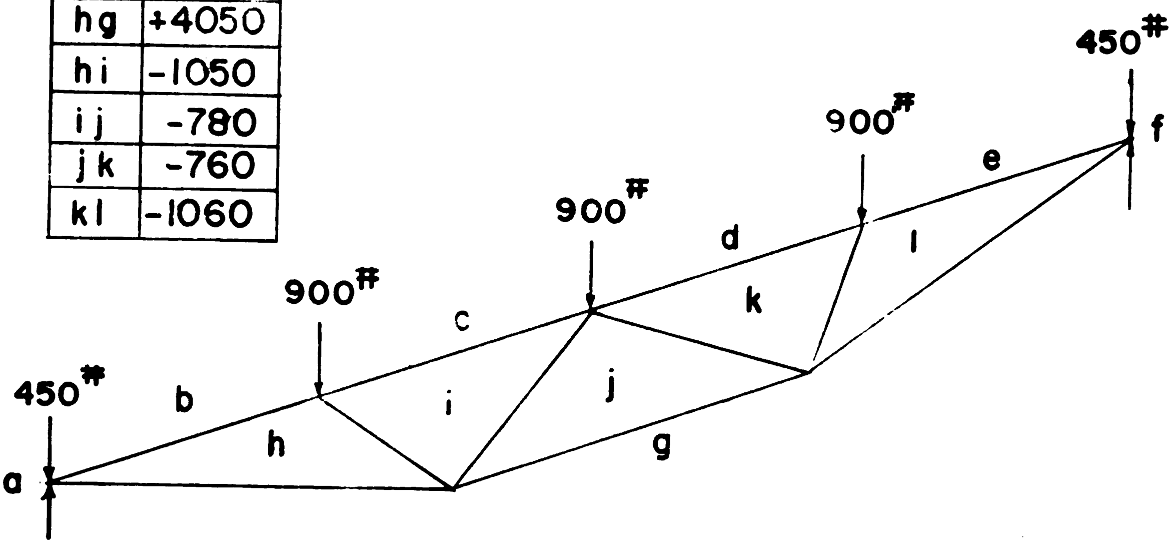
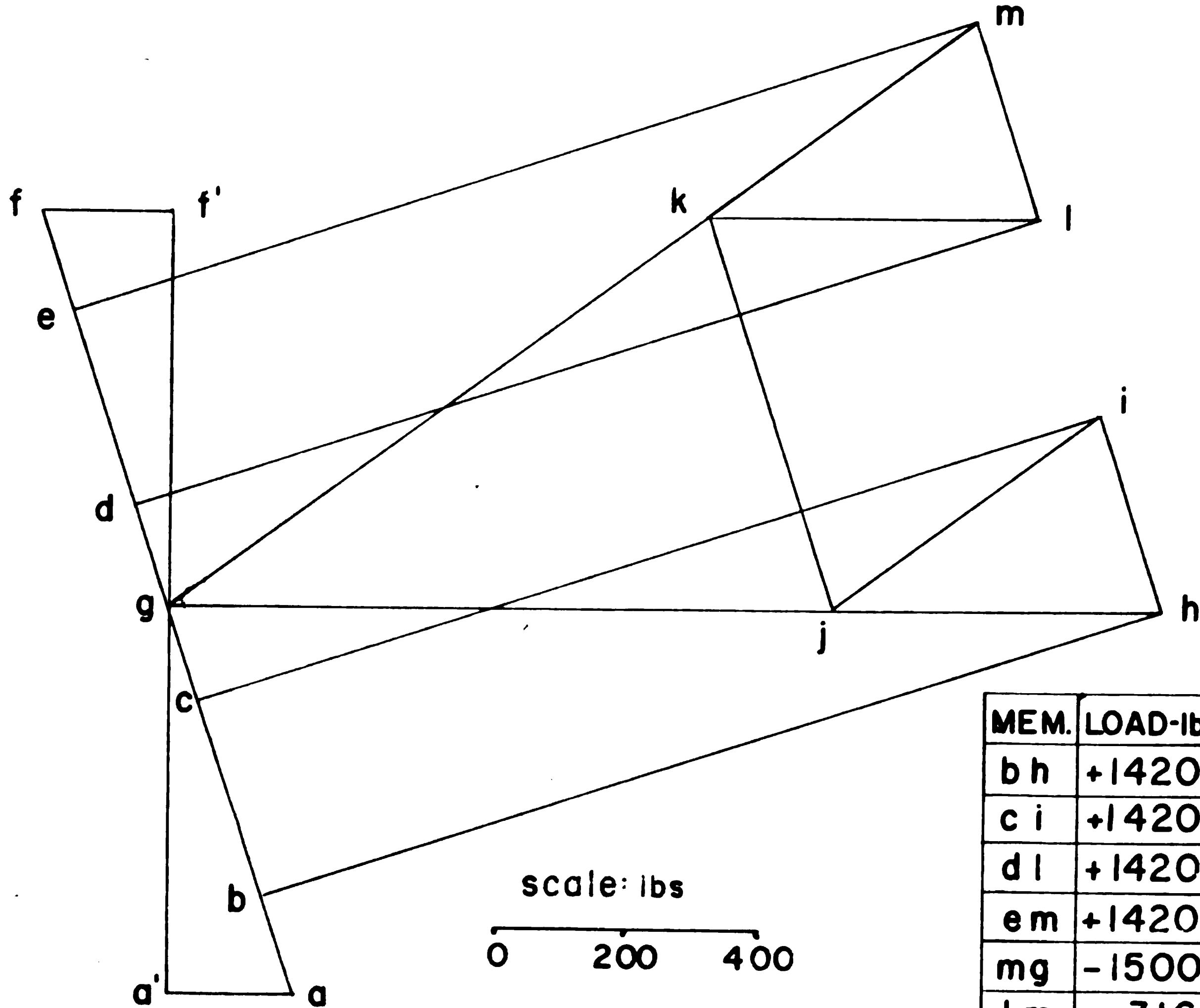


FIGURE 9 STRESS DIAGRAM OF TRUSS "B"
LIVE PLUS DEAD LOAD



MEM.	LOAD-lbs.
b h	+1420
c i	+1420
d l	+1420
e m	+1420
mg	-1500
l m	+316
k l	-490

k g	-1010
j k	+635
j g	-1010
i j	-490
h i	+316
h g	-1500

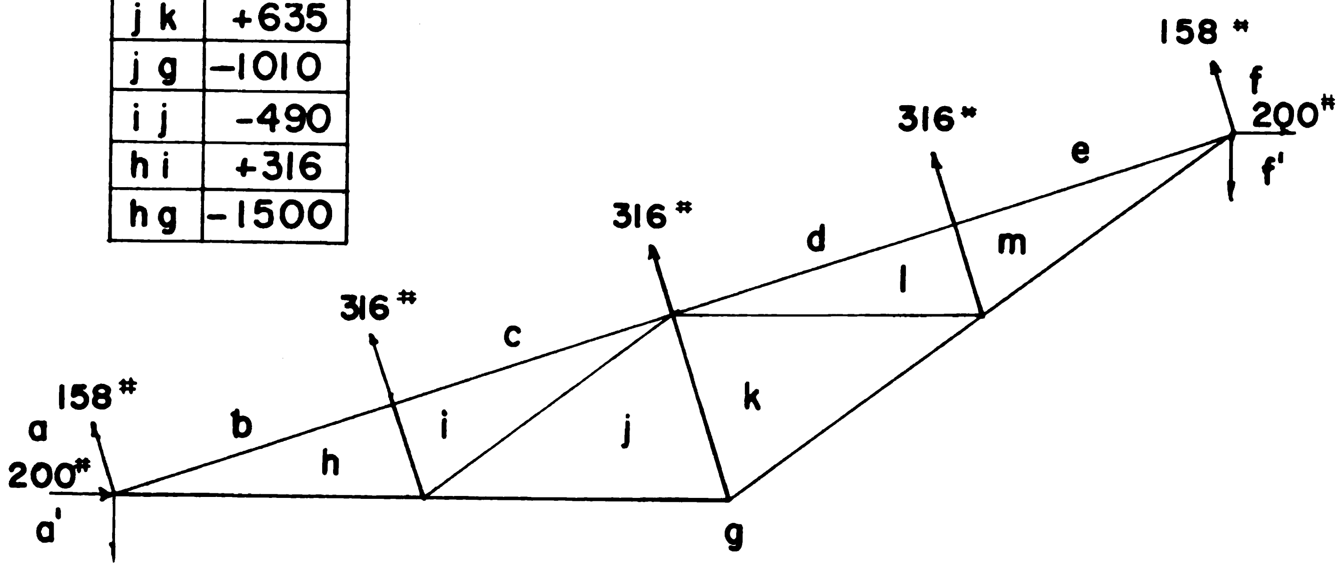
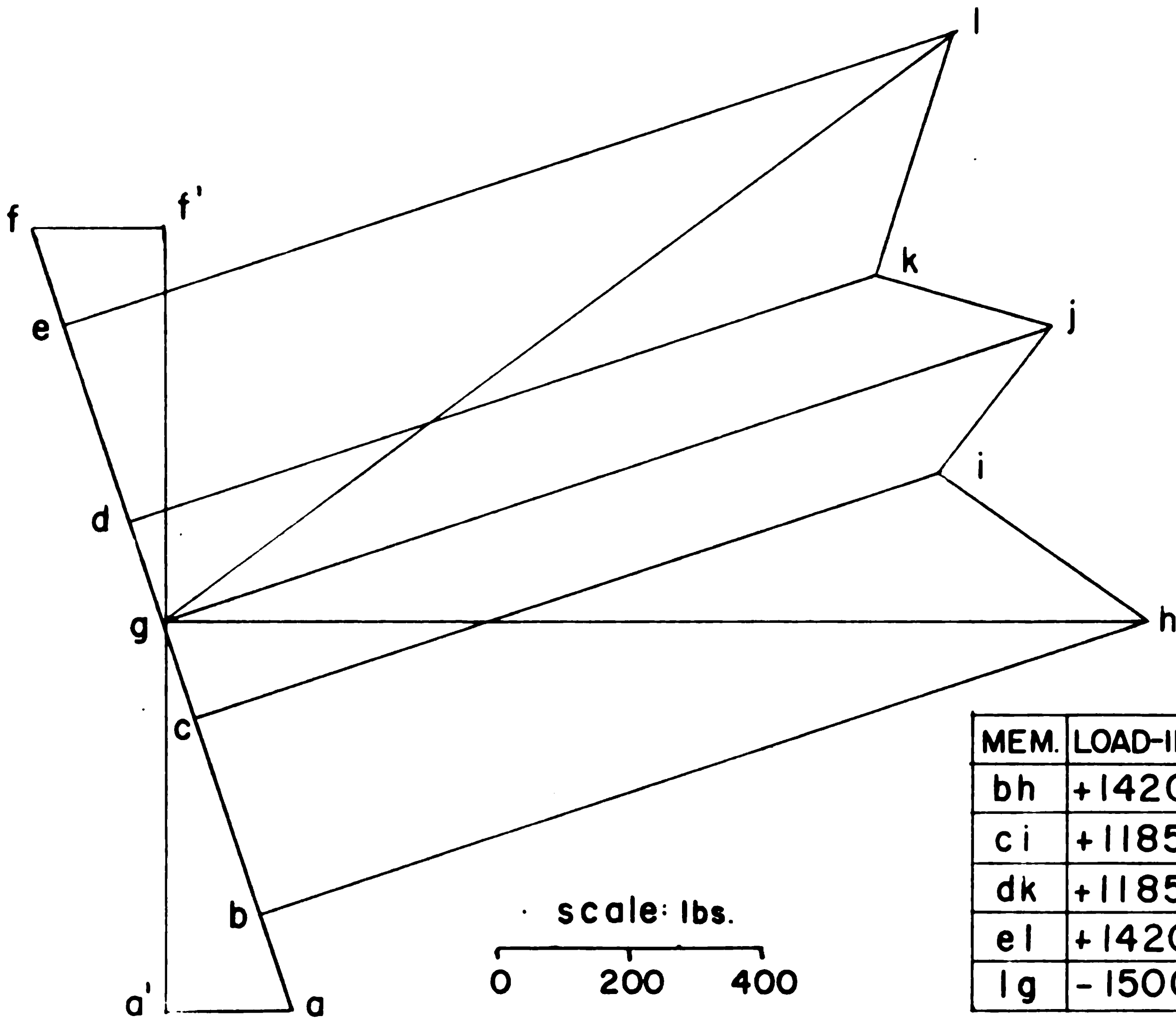


FIGURE 10 STRESS DIAGRAM OF TRUSS "A"- WIND LOAD



MEM.	LOAD-lbs
bh	+1420
ci	+1185
dk	+1185
el	+1420
lg	-1500
lk	+400
kj	+275
jg	-1420
ij	+275
hi	+400
hg	-1500

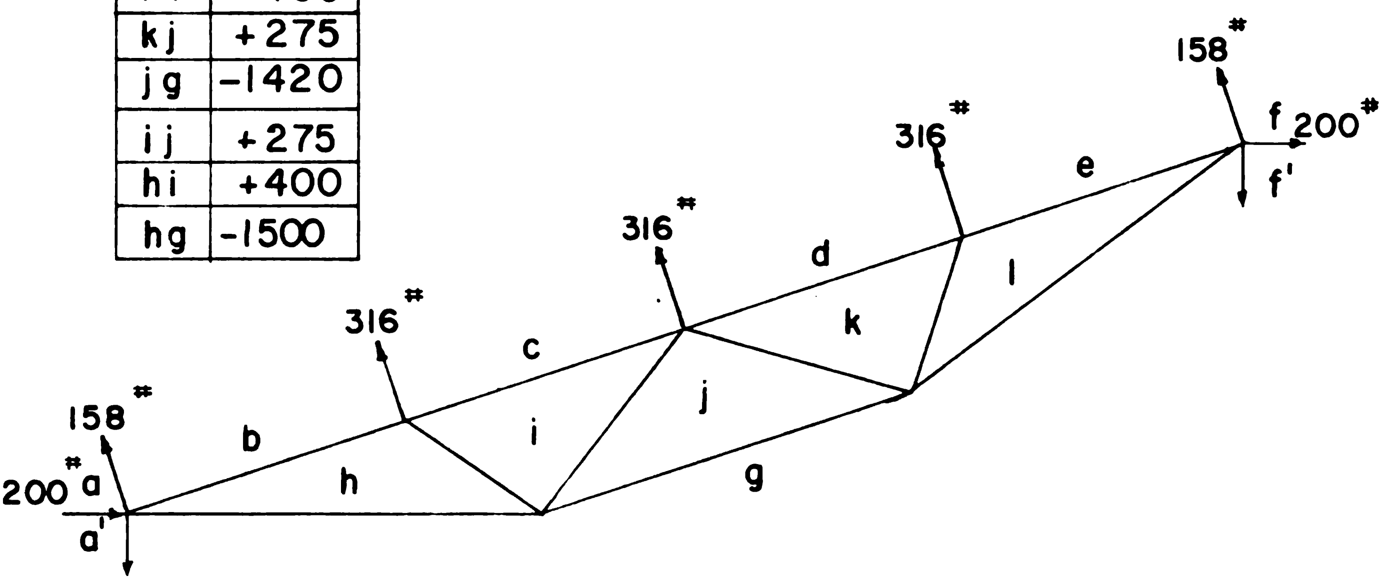


FIGURE 11 STRESS DIAGRAM OF TRUSS "B" - WIND LOAD

It was apparent that balanced stressing could be possible by using non-standard sizes in the glue-nail truss designs. However, this would lead to more labor and to the possibility of the lumber being wasted, regardless.

Glue Areas for Joints

Joint size was determined chiefly by the allowable shear stresses for lumber. Since it is hoped that these designs are practical for farm construction, economy of use and simplicity also were considered. In laying out the gusset plates, it became apparent that a 4' x 4' sheet of plywood would serve one truss. The shapes of the gusset plates then were made so as to minimize the plywood requirement. The outline diagram for the gusset plates for truss "B" appears in Figure 5.

Truss "A" actually required more plywood than a 4' x 4' section. Here again, this additional plywood was layed out so as to maximize use and minimize waste. Figure 3 shows the gusset plate layout for truss "A."

A shear stress of 200 psi was used for the glue line. Although this stress is highly conservative, it allows for variation in application that might occur with inexperienced fabricators. The actual limiting stresses in the glued joints is the shear stress of the lumber and the cross-section shear of the plywood. The design of gussets was, therefore, based on a 95 psi shear stress for lumber and a 200 psi cross-sectional shear of plywood. Table I gives the minimum glue contact area required between gusset plates and the individual member for axial stresses.

TABLE I

MINIMUM GLUE CONTACT AREA REQUIRED BETWEEN THE GUSSET PLATES AND THE INDIVIDUAL MEMBERS FOR AXIAL STRESSES

TRUSS "A"

Member	Area-In.^{2}	Member	Area-In.^{2}
AB	45.8	HA	43.8
BC	43.0	BH	8.95
CD	40.1	CH	14.2
DE	36.0	CG	18.4
EF	43.8	CF	14.5
FG	29.1	DF	8.95
GH	29.5		

TRUSS "B"

AB	45.3	CG	8.2
BC	35.3	CF	8.0
CD	32.1	AG	42.7
DE	35.7	GF	41.0
BG	11.1	FE	42.1
DF	11.2		

Ring-bolt Design and Location

The design manual for TECO Timber Connector Construction (39) was used for design. Advantage was taken of allowable loads for snow loads of 2 months' duration. This increase amounts to 15 percent over the values given in the manual or 25 percent over permanent loads.

Because of loads involved, 2" x 4" members could be used in the trusses. Two and one-half inch split-rings were used throughout the truss since a combination of 2-1/2 and 4-inch rings would complicate fabrication.

Ring location and number are shown in Figures 4 and 6.

Calculating Change in Length of Members

The total strain in a member was determined by the equation $s = \frac{Pl}{AE}$ where

P = load in pounds on the member

l = length of member in inches

A = cross-sectional area of the member ($in.^2$)

E = modulus of elasticity (1,600,000 psi)

The change in length of each member of both designs is given in Table 2.

TABLE 2

DATA USED FOR CALCULATING THE CHANGE IN LENGTH OF TRUSS MEMBERS

TRUSS "A"*

Member	Length (In.)	Area ($In.^2$)	Load (Lbs.)	$\frac{Pl}{AE}$ (Inches)
AB	95	9.14	-4350	-0.0283
BC	95	9.14	-4080	-0.0265
CD	95	9.14	-3810	-0.0247
DE	95	9.14	-3420	-0.0222
BH	32	5.89	- 850	-0.00289
DF	32	5.89	- 850	-0.00289
CH	100	5.89	1350	0.0143
CF	100	5.89	1380	0.0146
CG	63	5.89	-1750	-0.0117
AH	100	5.89	4160	0.0441
HG	100	5.89	2800	0.0297
GF	100	5.89	2760	0.0293
FE	100	5.89	4160	0.0441

* For glue-nail truss

TABLE 2--Continued

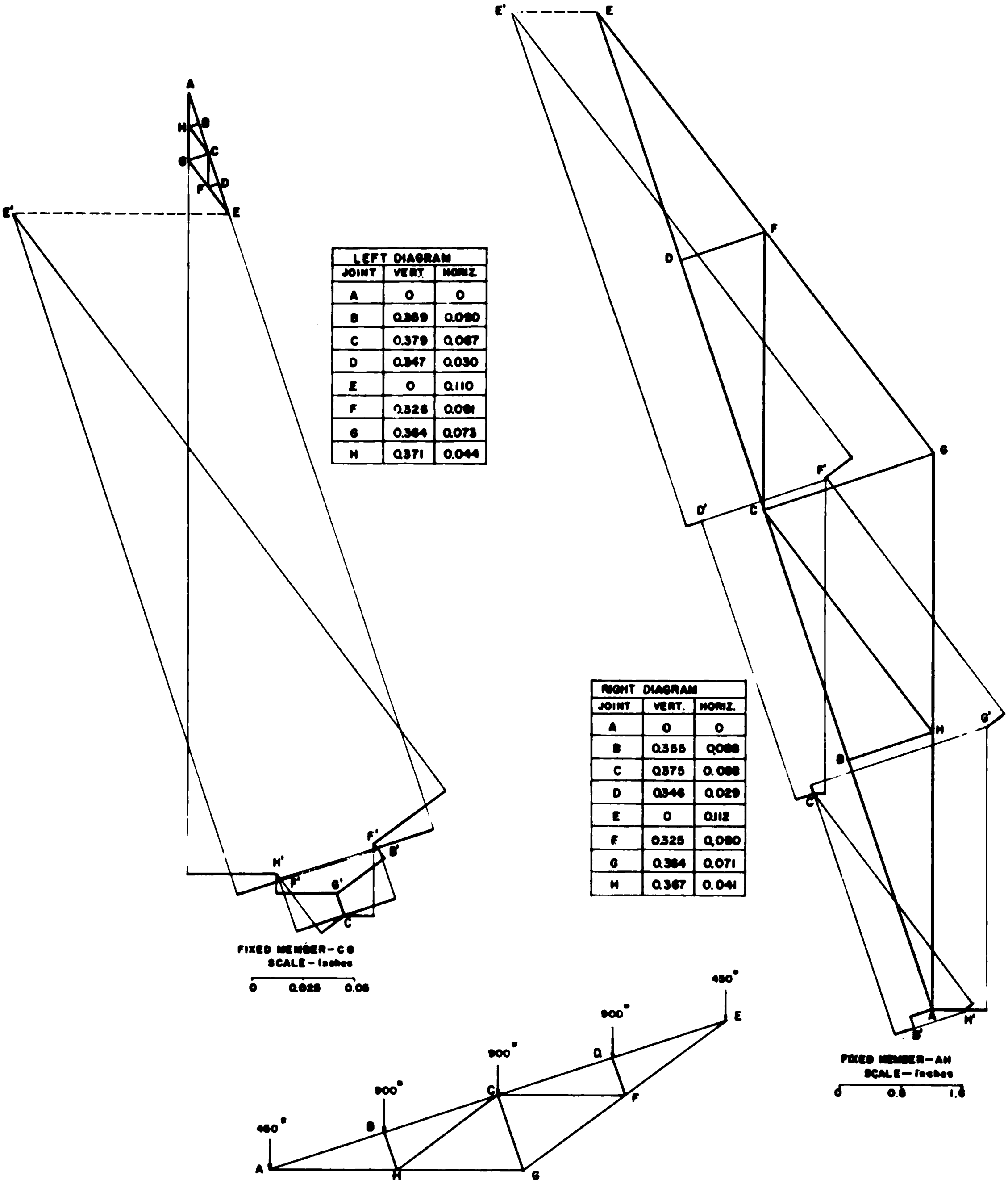
TRUSS "B"

Member	Length (In.)	Area (In.^{2})	Load (Lbs.)	$\frac{Pl}{AE}$ (Inches)
AB	95	9.14	-4300	-0.0279
BC	95	9.14	-3350	-0.0217
CD	95	9.14	-3050	-0.0198
DE	95	9.14	-3390	-0.0220
BG	53	5.89	-1050	-0.0059
DF	53	5.89	-1060	-0.00596
CG	77	5.89	- 780	-0.00637
CF	77	5.89	- 760	-0.00622
AG	134	5.89	4050	0.0576
GF	127	5.89	3900	0.0525
FE	134	5.89	4000	0.0570

Calculating Panel Point Deflection

In order to compare the theoretical design to test results, the Williot-Mohr diagram was used to determine panel point deflection. The change in length of each member is considered in the Williot diagram. This graphical presentation gives the theoretical horizontal and vertical deflection of each of the panel points. Figures 12 and 13 show the Williot-Mohr diagrams and resulting deflections for each truss. Deflections are determined by scaling the horizontal and vertical distance between respective points on the Williot-Mohr diagrams. Two different Williot-Mohr diagrams for each truss served as a check on the accuracy.

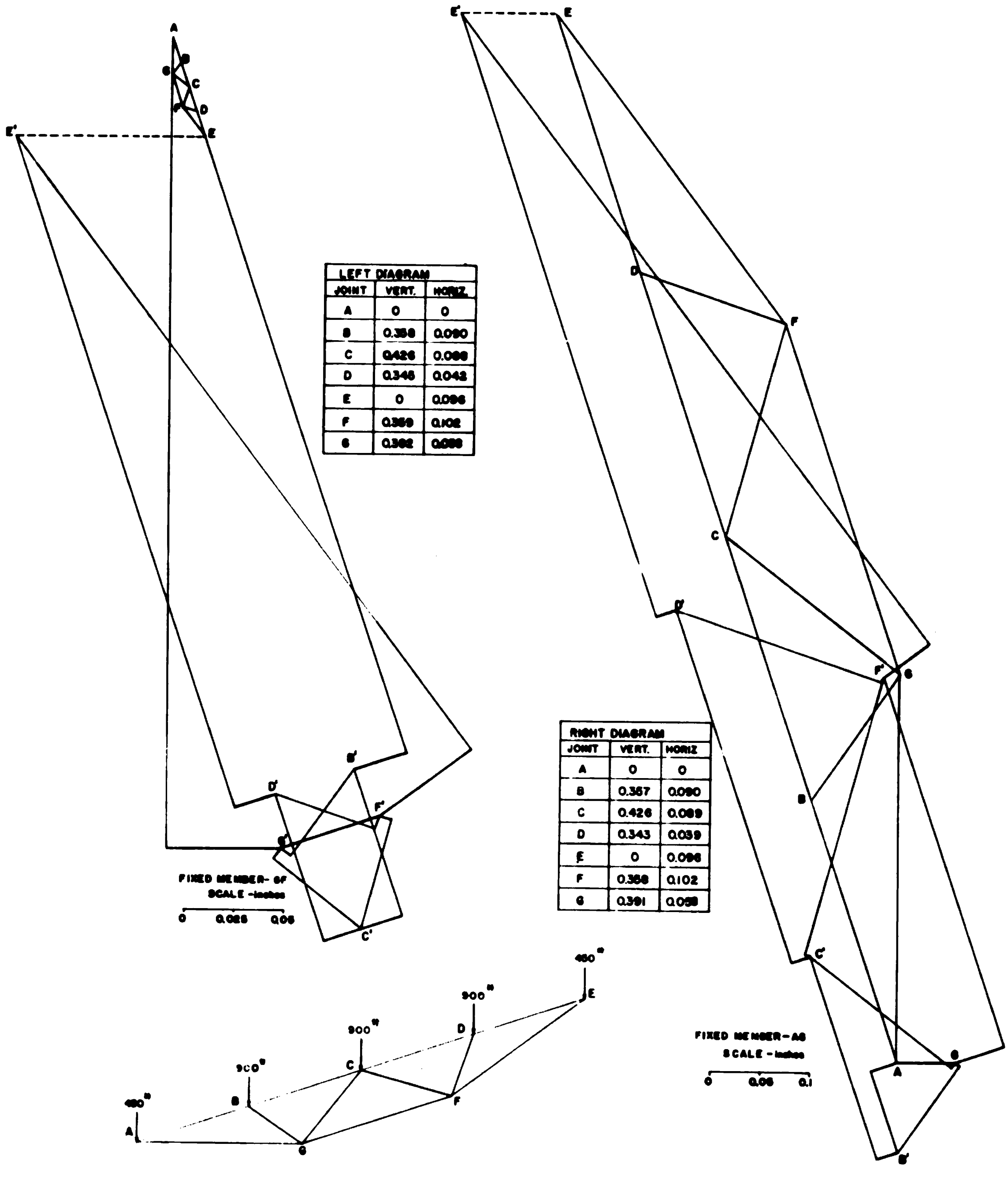
The Williot-Mohr diagram for truss "A" was based on the glue-nail truss. As can be noted, the center diagonal of the



LEFT DIAGRAM		
JOINT	VERT.	HORIZ.
A	0	0
B	0.369	0.090
C	0.379	0.067
D	0.347	0.030
E	0	0.110
F	0.326	0.081
G	0.364	0.073
H	0.371	0.044

RIGHT DIAGRAM		
JOINT	VERT.	HORIZ.
A	0	0
B	0.355	0.088
C	0.375	0.088
D	0.346	0.029
E	0	0.112
F	0.325	0.080
G	0.364	0.071
H	0.367	0.041

FIGURE 12 - WILLOIT-MOHR DIAGRAMS OF TRUSS A



LEFT DIAGRAM		
JOINT	VERT.	HORIZ.
A	0	0
B	0.358	0.090
C	0.426	0.088
D	0.346	0.042
E	0	0.096
F	0.369	0.102
G	0.362	0.058

RIGHT DIAGRAM		
JOINT	VERT.	HORIZ.
A	0	0
B	0.357	0.090
C	0.426	0.089
D	0.343	0.039
E	0	0.096
F	0.358	0.102
G	0.391	0.058

FIGURE 13 - WILLOIT-MOHR DIAGRAMS OF TRUSS B

ring-bolt truss has two 2" x 4"'s, whereas the glue-nail truss has only one 2" x 4".

Comparison of the Two Truss Designs

Truss "A" appears to be slightly stronger than truss "B." At 30 pounds per square foot, truss "A" has a theoretical deflection of 0.375 inches, whereas truss "B" has 0.426-inch deflection at the top center joint. Each truss contains the same number of split-rings. Truss "A" has more board feet of lumber than truss "B." In the glue-nail type connectors, truss "A" utilized 5/8 of a sheet of 4' x 8' plywood, whereas truss "B" used 1/2 a sheet.

EXPERIMENTAL INVESTIGATION

Apparatus

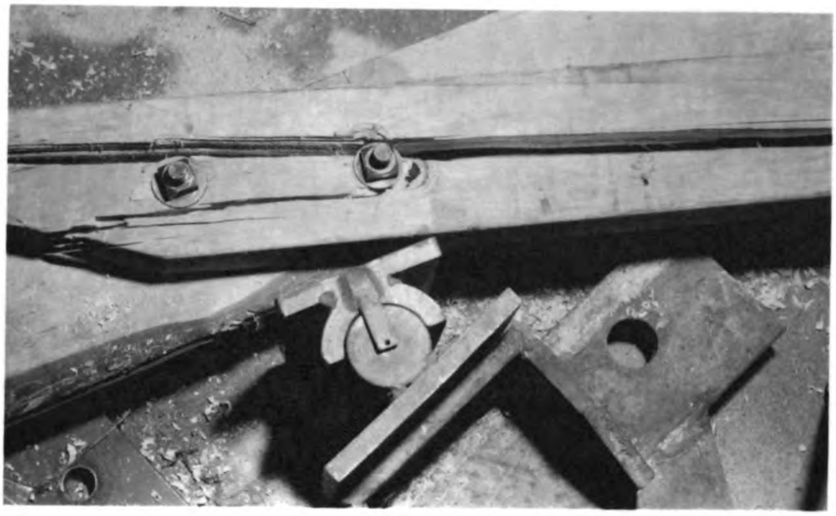
Test Floor

The test floor consists of steel inserts placed 2 feet apart in each direction in the reinforced concrete floor. Two rows of double "I" beams, spaced to allow a bolt to drop through, form the wall reaction supports. A heavy steel plate is bolted across each of these pairs of beams. The simulated wall reaction point is then mounted on the heavy plate. A hinge between the truss and each reaction point serves to allow some lateral movement of the truss while it is under load. Figure 14a shows the hinged upper wall reaction.

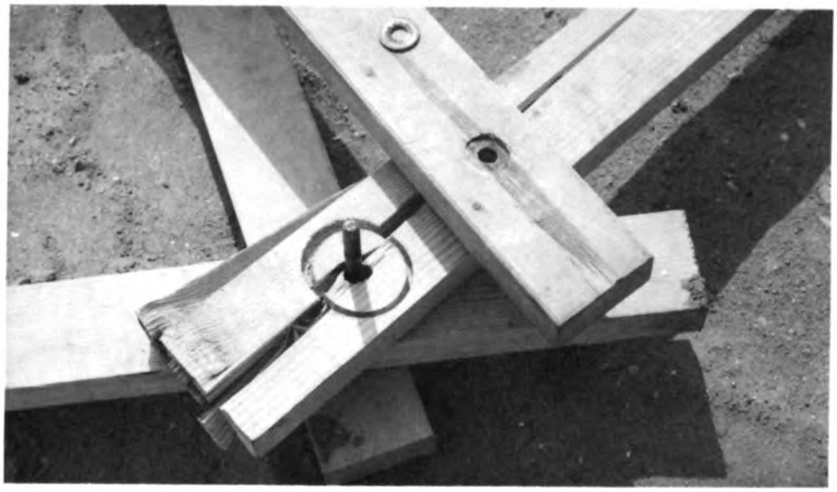
Hydraulic Cylinders and Loading Apparatus

Small, 2-inch diameter hydraulic cylinders were used to apply the load on each truss. The cylinders were placed 2 feet on center and simulated a verticle downward force on the truss. The cylinders were mounted on brackets connected to movable bars fastened to the floor brackets. Since full travel of the pistons was only 3 inches, the movable bars gave a means of taking up the slack before loading. The cylinders and their mounting are pictured in Figure 15.

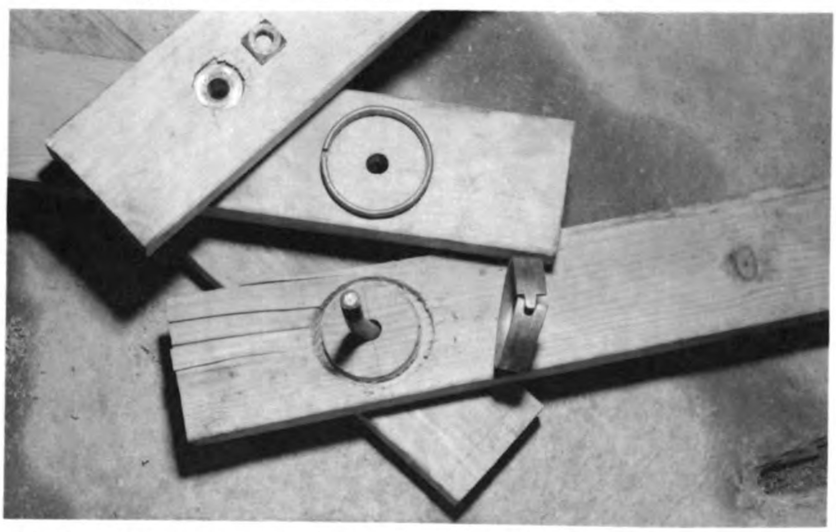
Hydraulic pressure was applied to the cylinders with a two-way hydraulic pump. This pump was motor-driven and could maintain high pressures. In earlier tests, hand pumps did



14a - Hinged upper wall reaction and failure in tension member



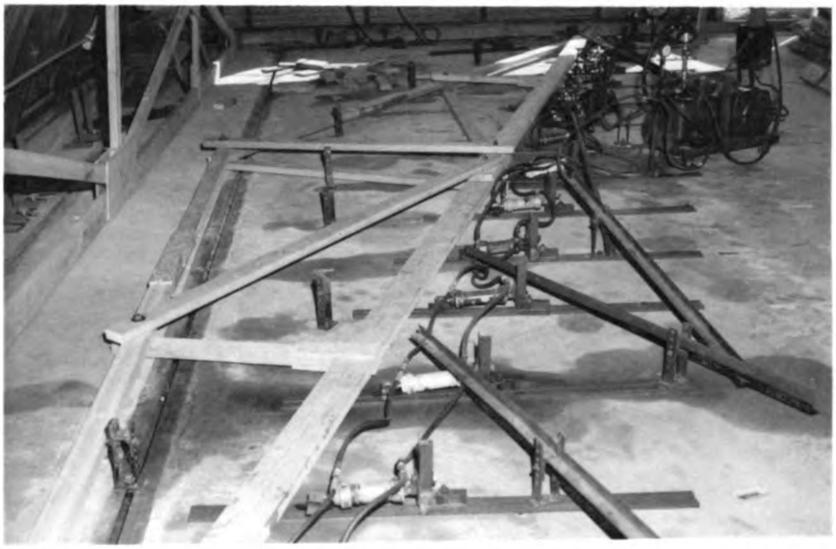
14b - Failure in lower center joint of a type "A" truss



14c - Failure in a lower joint of a type "B" design

FIGURE 14 - TYPICAL FAILURES IN RING-BOLT TRUSSES

15a - Hydraulic pump and control apparatus used to apply pressure to the trusses



15b - Truss type "A" in place before cylinders and hold-down bracket have been applied

15c - Truss type "B" in place and ready for testing

FIGURE 15 - APPARATUS USED FOR LOADING TRUSSES

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not maintain a steady pressure due to small leaks in the system. As the result of pressure leaks, a two-man testing crew was needed when the hand pump was used. While one man read the deflection, it was necessary for the other to pump in order to maintain a steady pressure.

Pressure readings were taken in three positions on the system. Pressure gauges were placed at each end of the bank of cylinders and at the center of the system where the pump was located. With the pump located in the center of the system, there was no noticable pressure drop between the pump and the ends. Figure 16 shows a schematic diagram of the hydraulic system, while Figure 15a shows the pump and control apparatus.

Hold-down Brackets and Rollers

Because of the high compression loads, the top chords had a severe tendency to buckle up or down in the test apparatus. To overcome this problem, rollers were placed at 2 to 3-foot intervals under the truss. These rollers and some additional blocks served to level the truss and to permit free deflection. The hold-down brackets placed over the truss at 2 to 3-foot intervals served to prevent the top chord from buckling up. Rollers on the hold-down brackets also allowed free movement of the truss in the loading plane. Placement of these supports every 2 to 3 feet simulated the roof girts of a building. ASTM (7) indicates that total simulated building effect is preferred as long as free deflection is not hampered. Figure 14b shows a truss before

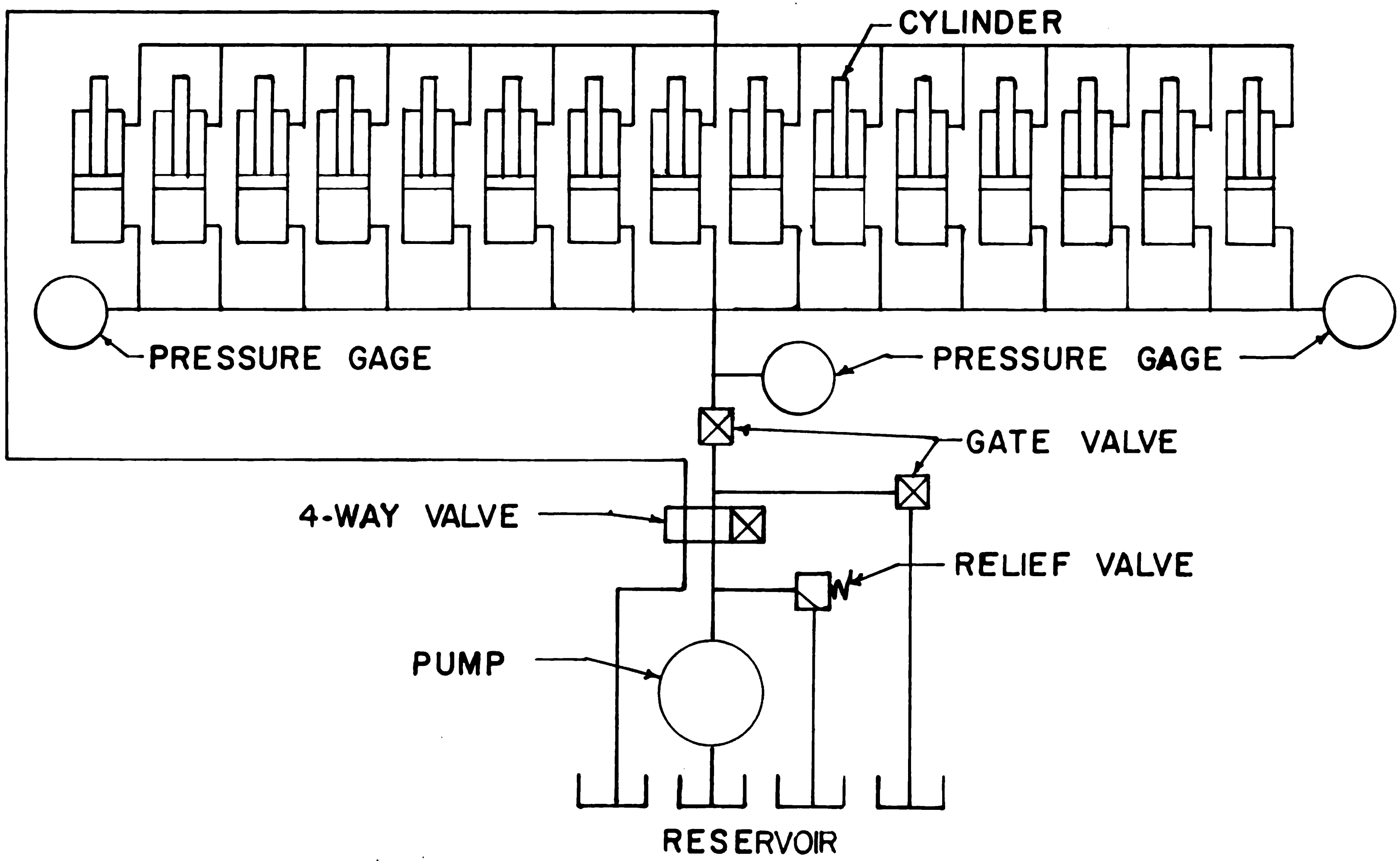


FIGURE 16 SCHEMATIC DIAGRAM OF HYDRAULIC LOADING SYSTEM

hold-down brackets and cylinders have been applied. Figure 14c shows another truss at the start of the test.

Material and Construction of Trusses

The lumber used was Douglas Fir Number 2 or better and was obtained from a local supplier. The supplier indicated that lumber was 80 percent Number 2 and 20 percent Number 1 lumber. This corresponds to the Construction grade in The Light Framing Grades.

No attempt was made to select the lumber to be used in the truss. When obvious defects were noticeable, however, another piece was used. These defects generally were confined to warped pieces or to where large knots were present in the 2" x 4" material. Selection beyond this point probably would not be made by the average farm builder.

Material for the gusset plates was 5/8" exterior grade Douglas Fir plywood. Occasionally 1/2" plywood was used with no apparent difference in ultimate strength of the truss. Aircraft-type casein glue was applied to both the lumber and the gusset plate before the 6d nails were used. The nails served only to hold the gusset to the members until the glue had hardened.

Each glued truss was allowed to lie for 48 hours or more before testing was begun.

The ring-bolt trusses were constructed with the use of a 1/2" drill and a groove cutter. The groove cutter was a four blade instrument with an adjustable depth control. The groove, about 3/8" deep, is cut into the face of both pieces on a two member joint. The 2-1/2" split-ring sets about 3/8' into each member. Washers 1-1/2" in diameter were used on each side of the joint formed with the ring and bolt.

A birds mouth cut was placed in the lower end of the top chord of each ring-bolt truss. This cut resulted in the wall reaction load being placed on both the top and lower chord. Acre (6) found that this increased the strength of his test trusses by 55 percent. Without the birds mouth cut or a block, the load placed on the lower chord tended to produce a twisting movement on the joint. This difficulty is not present in the glued trusses since all the members are in the same plane.

Testing Methods

Loading Method

Some slack always remained after the truss was placed in position and the cylinders attached the first time. In order to eliminate as much of the slack as possible, a small load was applied to each truss. In the ring-bolt trusses, this load tended to set the rings in position and to produce a more uniform loading curve. This load was quickly removed and the truss was allowed to come to rest.

After the truss had returned to an unloaded equalibrium position, the dial gauges were set in location and the test was begun. Loads were applied to the truss as slowly as possible. In the first part of the test, increments of from 15 to 25 percent of design load were applied. After design load had been reached, the increments of loading were increased in some cases. This method was in accordance with ASTM (7).

Each truss was tested two or three times before it was taken to the breaking point. The first test was taken to design load or a small amount beyond before the load was released. In the remaining tests, the final loads were gradually increased until failure occurred.

Measuring Deflections

Seven Ames dial indicators, measuring to the nearest 0.001 inch, were used to measure deflection. The gauges were placed at panel points on the truss. In addition, several gauges were used to record deflection between panel points of the top chord.

Since the range of the gauges was only slightly over one inch, it was necessary to reset the gauges several times in some tests. The gauges were reset when the truss had come to equalibrium with the last increment of load. Each recording of deflection also was made after the truss had reached or nearly reached an equalibrium position.

Figure 14c shows the dial gauges in place ready for testing.

Plotting Deflection of Test Results

The deflection curves for each point on the trusses were drawn from the results of 2 or 3 loadings of each truss. Since the different runs did not always start at the same place, it was necessary to plot the runs individually and then shift the curve to start at zero. This practice, in theory, did away with the variation in slack at the start of the tests.

By using two or three test runs for each truss, it was possible to spot any major errors or problems in a particular test. Test runs that produced radically different results from other tests on the same or similar trusses were examined to see if they were caused by the test apparatus. In case cylinder brackets had slipped or something was holding up the free movement of the truss, the problem was corrected and the test was repeated or discarded.

The best curve representing the several tests of a truss was then drawn to represent the final deflection pattern.

RESULTS AND DISCUSSION

Number of Trusses Tested

Tests were conducted on thirteen different trusses. Seven of these trusses were type "B" and the rest were type "A." There were four tests of type "B" trusses using ring-bolt and three tests with glue-nail construction. Truss "A" had three tests of each type of construction. Table 3 shows the maximum load and type of failure for each truss. The first two letters under type of truss refer to the method of fastening, glue-nail or ring-bolt. The third letter indicates the truss design. Each truss was loaded two or three times before it was loaded to failure.

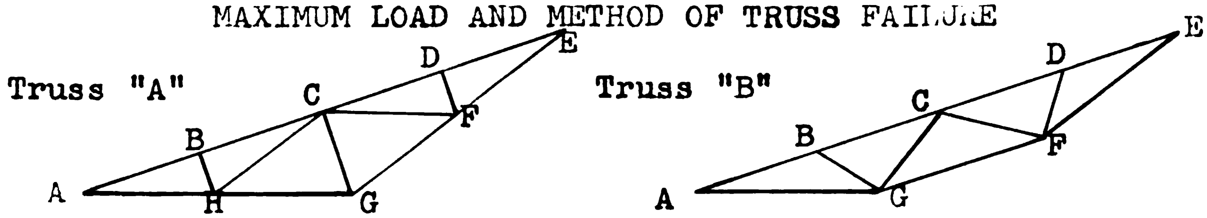
Comparison of Trusses

Comparison of the tested trusses indicates a greater difference between type of fasteners than type of design. The test results for the glue-nail trusses compare very favorably with the theoretical deflections from the Williot-Mohr diagrams. Figure 17 shows the theoretical and experimental deflections for the top center joint of each glue-nail truss. These same deflections for the uppermost joint are shown in Figure 18.

Figures 19 and 20 show the theoretical deflection pattern for both truss designs. Theoretical deflection is based on truss spacing of 4 feet on center and a load of 30 psf. From

TABLE 3

MAXIMUM LOAD AND METHOD OF TRUSS FAILURE



Truss Number	Type*	Maximum Load	Method of Failure
1	RB-A	66.2	Lower joint "G" split out sometime after load had been on truss
2	RB-B	58.8	Lower joint "F" split out
4	RB-B	40.6	Both joints "G" and "F" failed
5	RB-B	37.9	Tension member GF was splitting out of both joints
6	RB-B	40.6	Joint at "G" failed
7	GN-A	69.8	Gusset plate at "F" failed--grain of plywood placed in wrong direction--failure due to rolling shear in plywood
8	GN-A	58.8	Failure in member AH--severe slope to grain at point of failure
9	GN-B	95.6	Failure in member AG near the reaction "A"
10	GN-B	58.8	Failure in member AG near the reaction "A"
11	GN-B	84.6	Gusset plate at "E" failed in the glue line of the interior grade plywood
12	GN-A	91.9	Did not fail at this point
13	RB-A	55.2	Failure occurred at joint "F"
14	RB-A	66.2	Lower reaction "A" was point of failure--bottom chord broke a ring connection

* GN = Glue-nail, RB = Ring-bolt construction
A = Truss type "A" B = Truss type "B"

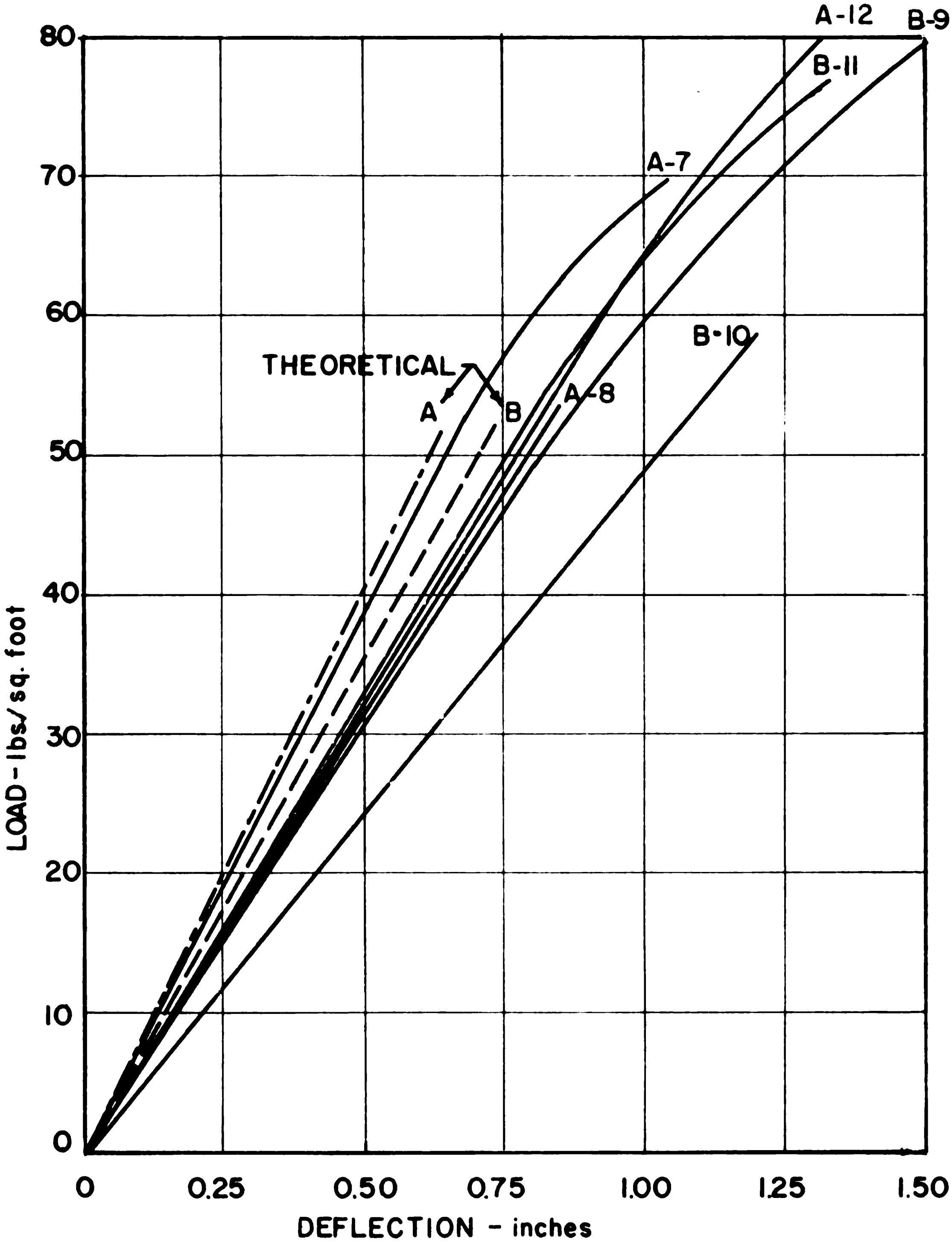


FIGURE 17 COMPARISON OF THEORETICAL AND ACTUAL DEFLECTION OF TOP CENTER JOINT - GLUE-NAIL TRUSSES

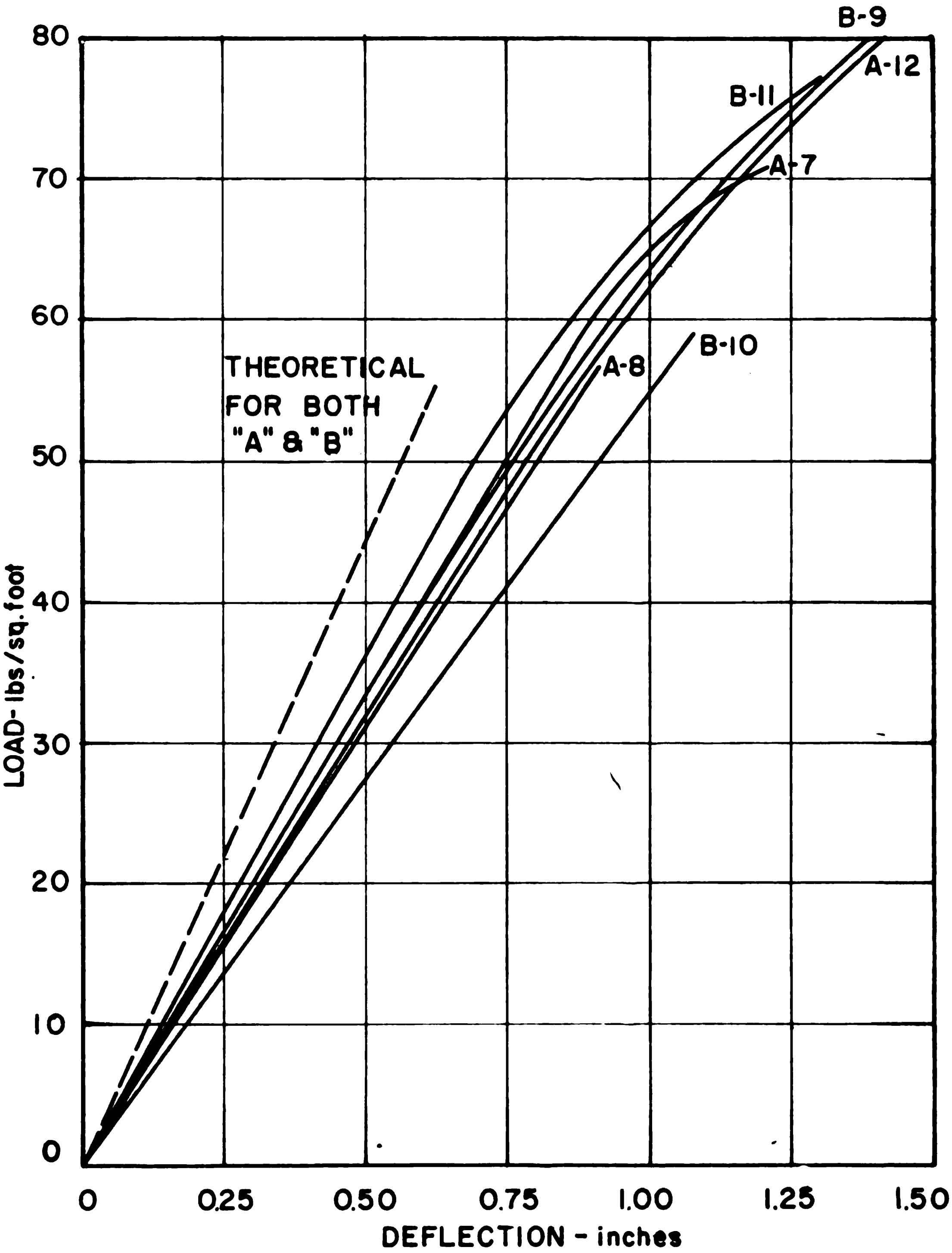


FIGURE 18 COMPARISON OF THEORETICAL AND ACTUAL DEFLECTION OF POINT "D" - GLUE-NAIL TRUSSES

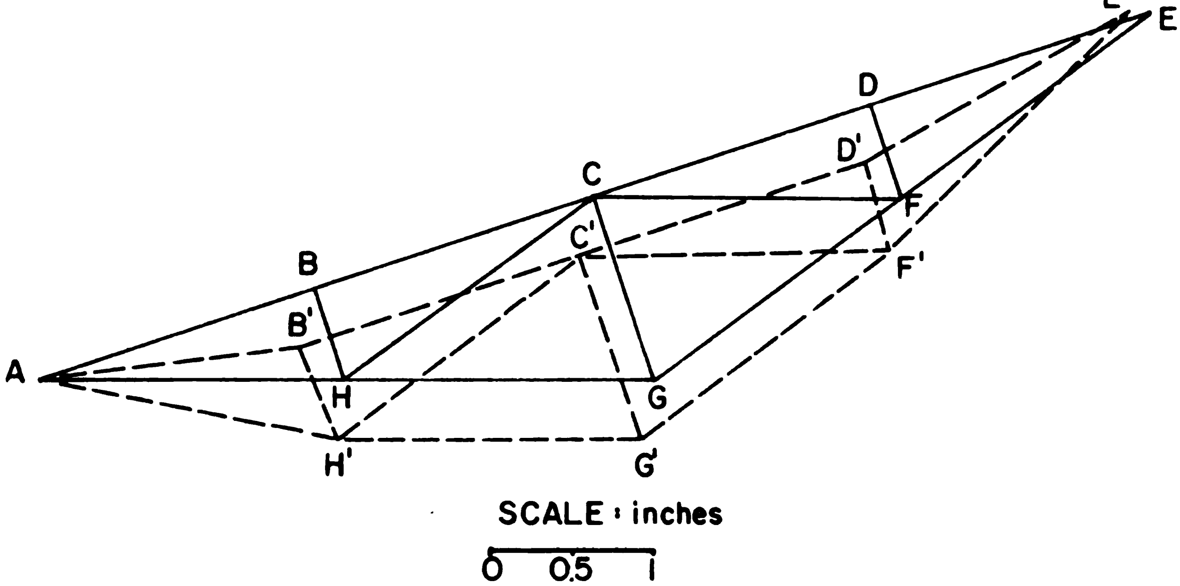


FIGURE 19 THEORETICAL DEFLECTION OF TRUSS "A"-
30 POUNDS PER SQUARE FOOT LOAD ON TRUSS-
TRUSS SPACING IS 4 FEET ON CENTERS.

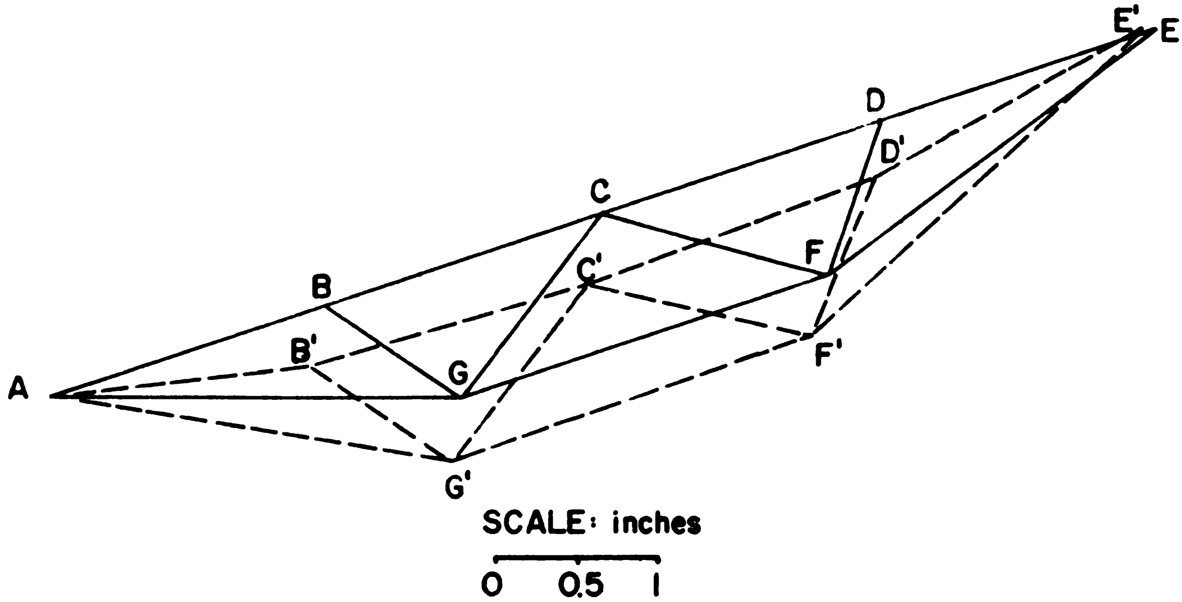


FIGURE 20 THEORETICAL DEFLECTION OF TRUSS "B"-
30 POUNDS PER SQUARE FOOT LOAD ON TRUSS-
TRUSS SPACING IS 4 FEET ON CENTER.

these figures it can be seen that truss "B" has a single smooth curve in the top chord when it is under load. The theoretical deflection pattern for truss "A" has two curves in the top chord. That is, the center joint of the top chord of truss "A" is not deflected as much as truss "B." Comparing the theoretical deflection of point "C" (top center joint) and point "D" (uppermost joint on top chord) in Figures 19 and 20, this characteristic also is apparent. This same fact was present in the actual tests. In the tests, however, the deflection pattern of the top chords was more pronounced.

Comparison of the two designs, as far as other points on the trusses are concerned, is not valid since the other panel points are in different locations relative to one another. The joint on the lower bottom chord of truss "B" is farther from the wall reaction than is the corresponding joint of truss "A." Figure 21 gives an indication of the deflection of various points of truss Number 7.

Figure 22 shows the results of the deflections of the top center joint in the ring-bolt trusses. The deflections for the uppermost joint of the top chord of the same trusses are shown in Figure 23. Deflection results with trusses of ring-bolt fabrication for both designs did not compare well with the theoretical deflections of the Williot-Mohr diagram.

From Figures 22 and 23, it also is apparent that there is not as much consistency in ring-bolt trusses as there is

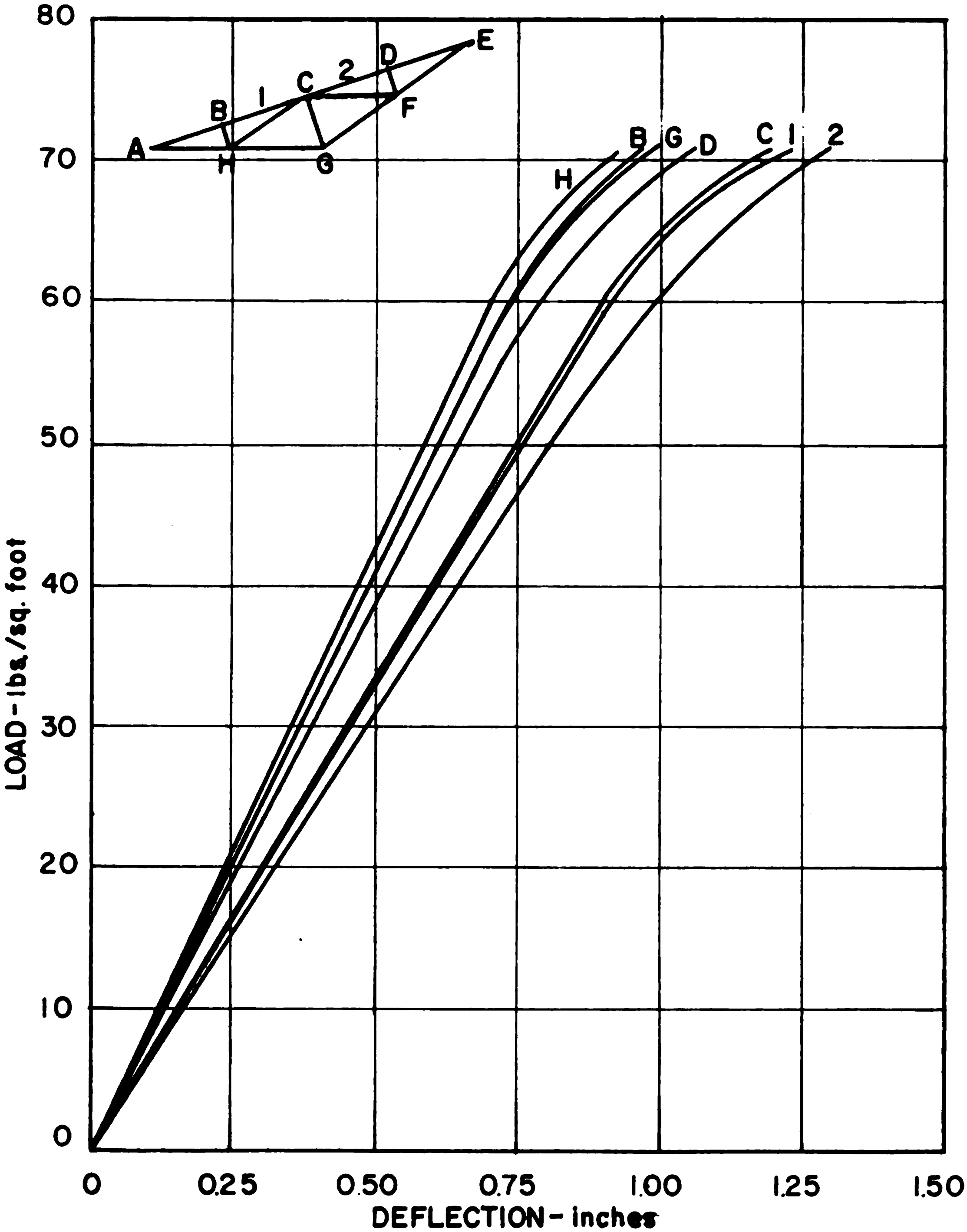


FIGURE 21 COMPARISON OF DEFLECTION OF POINTS ON TRUSS NO. 7 - GLUE-NAIL CONSTRUCTION.

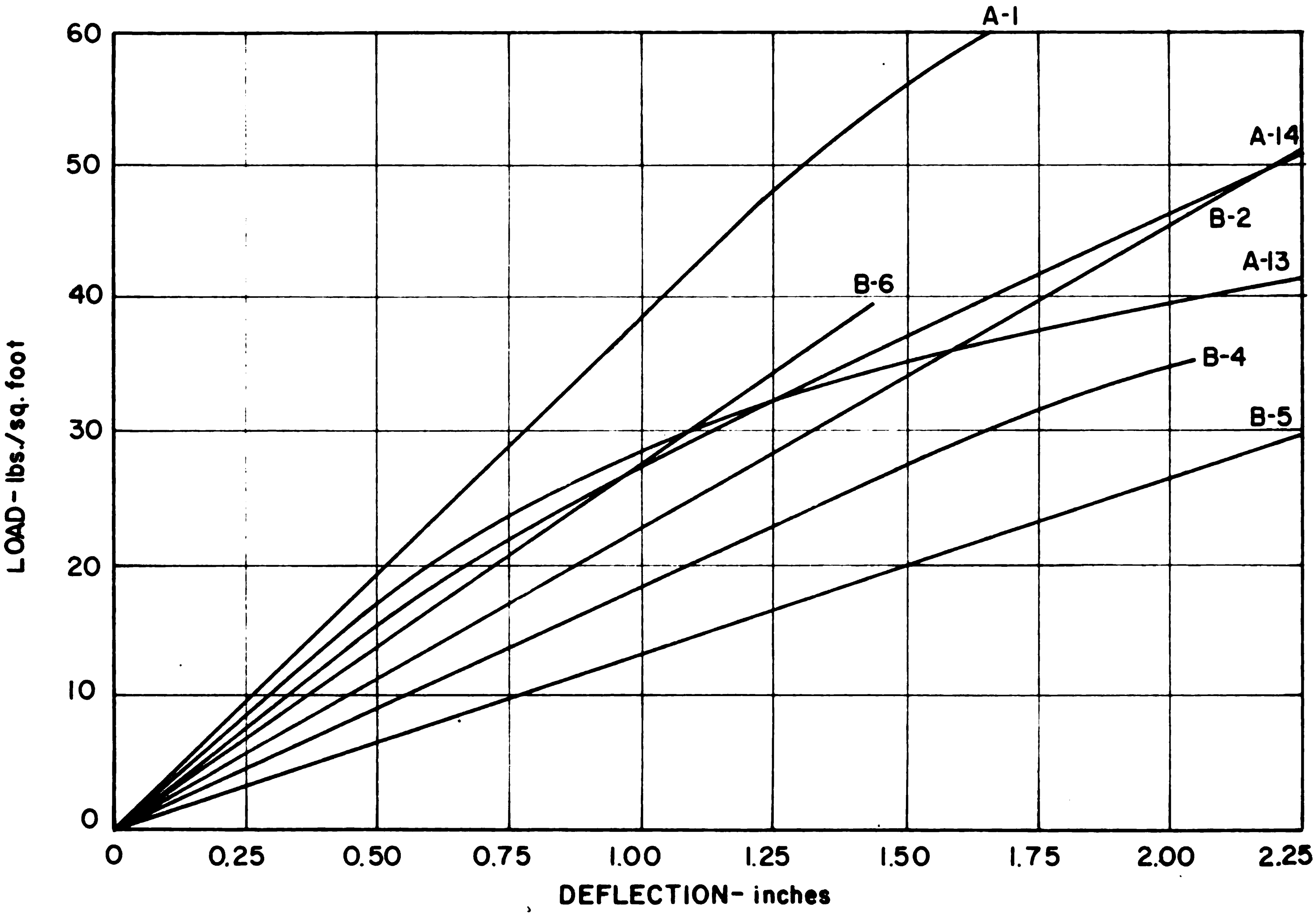


FIGURE 22 DEFLECTION OF PANEL POINT (C) (TOP CENTER) - RING AND BOLT TRUSSES

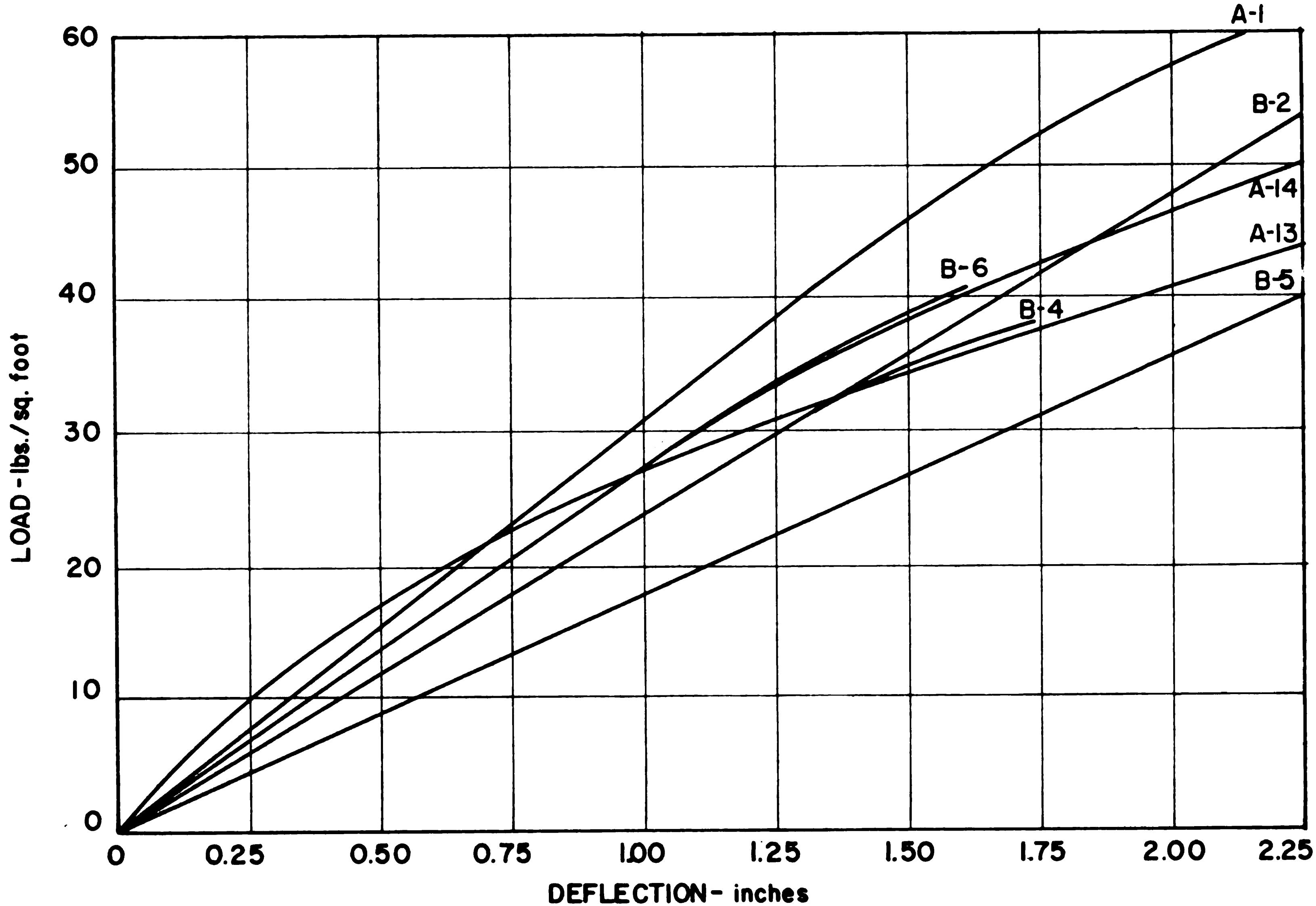


FIGURE 23 DEFLECTION OF PANEL POINT (D) - RING AND BOLT TRUSSES

in the glue-nail trusses. The deflection curves established from the results of two or three tests of a truss design do not correspond well with the curves for other trusses of the same design. In some cases, individual test curves of the same truss did not compare well with other tests.

In plotting the deflection curves for ring-bolt trusses, the points did not readily form a smooth curve. For a particular increment of load, the deflection frequently was less than the preceeding increment. In some cases, a truss would have come almost to equilibrium with the last increment of load, then suddenly deflect an additional amount. This erratic behavior occurred after the design load had been reached.

The points established by the glue-nail trusses always produced easily established curves. Very seldom did one point deviate greatly from the established deflection curve.

Method of Failure

The load at failure point varied with different trusses. The location and method of failure, however, were more uniform within types of fasteners than in truss designs. Table 3 shows the location and method of failure of each truss.

In general, the ring-bolt trusses failed at or very near a joint. The glue-nail trusses failed at weak points in the members.

There were two failures in the plywood gusset plates of the glue-nail trusses. The failure in truss Number 7 was due to the surface grain of the plywood running the wrong way in respect to the stress on the gusset. Failure was due to rolling shear within the gusset plate. The other failure in a gusset occurred in truss Number 11. This truss had interior plywood that did not have a grade mark. Failure in this case was due to glue line shear between plys of the gusset.

The other failures in glue-nail trusses occurred in the bottom chords where deflects, such as knots or cross-grain, were present. The lowermost member of the bottom chord in the trusses was the most frequent point of failure.

In the ring-bolt trusses, failures occurred at the joints along the lower chords of the trusses. Most of the "B" type, ring-bolt trusses failed at one or both of the lower joints along the underside of the truss. One of the "B" trusses failed at the joint near the upper reaction. The ring-bolt, type "A" trusses all failed at different locations, although, the lower center joint of this truss had the most cracking--even when failure occurred at some other location.

A great deal of the failure in ring-bolt trusses could be due to the eccentricity of the joints. There was a severe tendency of the lower joints to pull apart. The washers on the end of the bolts would be pulled into the wood. The result of this was that the total bearing area of the rings was not in use. The washers also were pulled in considerably when 2-inch washers were used in some of the later tests.

No failures occurred in the top chords of the trusses. The center diagonals did not have any failures, except in the ring-bolt trusses where the lower joints failed.

Maximum Loads and Load Factor of Safety

Load at failure was always well above the design load for all the trusses. The test loads were, however, short-time loads. Failure probably would have occurred sooner if the duration of load had been increased.

Since it would not be proper to compare the loads at failure with the design loads, a reduction was made for longer loading. The basis of this reduction came from Figure 2. The test load was taken as being from 5 to 10 minutes in duration. Test loads were reduced to loads of 2 months' duration. Considering the test load as 165 percent of a permanent load and the 2 months' load as 125 percent of a permanent load, the tests were reduced by 125/165. This information is taken from allowable stresses and applies to individual members, rather than to a complete truss. The effect should be nearly the same except where strength of the rings or bolts are effected. In any event, it more nearly signifies the load factor of safety than the test results alone would do.

When the maximum loads are averaged for both methods of fastening in each truss design and the reduction factor is applied, the trusses still are stronger than the design load. The average load factor of safety is given for different fasteners and designs in Table 4.

TABLE 4

LOAD FACTOR OF SAFETY BASED ON TWO MONTHS DURATION OF LOAD

Truss Type	Truss Number	Maximum Load #/ft.^{2}	Average Maximum Load #/ft.^{2}	Reduced Load For Longer Duration 125/165	Load Factor of Safety
RB-A	1	66.2			
	13	55.2			
	14	66.2			
			62.5	47.3	1.57
RB-B	2	58.8			
	4	40.6			
	5	37.9			
	6	40.6			
			44.5	33.7	1.12
GN-A	7	69.8			
	8	58.8			
	12	91.9*			
			73.5	55.7	1.85
GN-B	9	95.6			
	10	58.8			
	11	84.6			
			76.3	57.8	1.93

* Truss Number 11 had not failed at this load.

CONCLUSIONS

1. Both 30-foot span, single slope, truss designs analyzed are practical for farm construction. Either of the two designs tested in this research work will support 30 psf and still have a factor of safety greater than one.

For the designs tested, the glue-nail trusses are superior to the ring-bolt trusses. In each basic design, the glue-nail trusses will support about half again as much load as will the ring-bolt trusses. The deflection of the glue-nail trusses is not as great as the ring-bolt trusses, even when they are carrying greater loads. At an equivalent load of 50 psf, the glue-nail trusses are deflected about 3/4 of an inch. With the same load, the ring-bolt trusses show deflections in the range of 2 inches and more. This same deflection pattern holds true with smaller loads.

2. The factor of safety remaining after the test loads were reduced to correspond with a 2-months' loading ranged from 1.12 to 1.93. A factor of safety of 1.12 may be a little low for some areas of northern Michigan; however, a roof load of 30 psf is unrealistically high for most of Michigan. The higher factors, 1.85 and 1.93, in the glue-nail designs may be unjustifiably large for farm purposes. In this case, the truss spacing could be increased to take advantage of the increased strength of the glue-nail trusses.

3. Both designs are easily fabricated and assembly time is held to a minimum. A farm builder or a farmer, with

a small amount of building experience, can erect either the ring-bolt or glue-nail trusses. Due to the spacing of 4 feet on center, the trusses also are light; therefore, they are easily put in place without costly equipment.

The most critical part of construction with rings and bolts is locating the bolt holes so that the joints do not have to be forced together. In the glue-nail trusses, the shear strength of the glue line is several times greater than the shear strength of the wood itself. This fact nearly eliminates any possibility of weakness due to glue starvation in the joints.

Full strength advantage of glue-nail fastening will result if some additional time is spent in selection of the lower truss members.

4. Larger span, single slope, glue-nail trusses can use either of these basic designs. Both designs do a very good job of distributing the stresses evenly. An extension of the ring-bolt trusses should not be made without re-designing the joints. The joints made with ring-bolt fasteners are the weak joints in the 30-foot trusses. Larger spans would require larger rings and members. This would result in much higher building costs than can be justified for farm purposes.

SUMMARY

The major objective of this investigation was to analyze and design 30-foot, single slope trusses that are suitable for farm construction. In the preliminary planning, consideration was given to truss spacing and the economy of using readily available commercial grade lumber. In addition, designs that allowed both glue-nail and ring-bolt fasteners were used.

Stress diagrams were drawn for many different truss designs. With the aid of the stress diagrams, two designs were selected for further analysis. Both designs selected made construction with 2" x 4" and 2" x 6" material possible. The theoretical deflections of the panel points of both trusses were calculated by the use of Williot-Mohr diagrams. The deflections were based on the design load of 30 psf of roof with the trusses spaced 4 feet on center.

In order to compare the actual trusses with the theoretical analysis, three trusses of each type fastening and design were tested. The total number of trusses tested were thirteen.

The test method consisted of loading the top chord of the trusses by use of 2-inch hydraulic cylinders placed 2 feet apart. Simulated wall reaction plates served to hold the truss in place while a motor-driven pump applied pressure to the cylinders. Ames dial gauges were used to measure the deflection of points on the trusses. Two or three loadings were made on each truss before it was taken to the failure point.

The test results indicate a greater difference between type of fasteners then between the two designs tested.

The glue-nail trusses for both designs compared favorably with the theoretical deflections. The ring-bolt trusses deflected considerably more than either the glue-nail trusses or the calculated values. When maximum loads were reduced to simulate a two months' loading, all the trusses produced load factors of safety larger than one. The averaged results, reduced to correspond with a two-month roof load, appear in Table 4.

SUGGESTIONS FOR FURTHER STUDY

1. Investigate the possibilities of using different types of materials for use as gusset plates.

2. Investigate the effect of larger washers or steel plates on the ring-bolt trusses.

3. Investigate designs of single slope trusses that have the lower wall rigidly connected to the truss.

4. Investigate the possibility of basing design of farm buildings on frequency of occurrence of snow storms for different areas of Michigan. This would be similar to water retaining walls that are designed for a number of years based on past precipitation data.

5. In conjunction with Item 4, investigate the amount of snow that accumulates on unheated metal roof farm buildings in different areas of Michigan.

6. Investigate the effect of long-time loads on the single slope trusses.

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