PERFORMANCE OF FOUR RAISED-HEEL OR CANTILEVERED WOOD ROOF TRUSS DESIGNS

Research Report 82-1

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Small Homes Council-Building Research Council

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University of Illinois at Urbana-Champaign
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Saint Louis, Missouri

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INTRODUCTION

With the ever-increasing concern over energy costs, home owners are demanding fully-insulated structures, including the requirement for a minimum R-value of 30 in the ceiling. This will usually require at least 12 inches in added height in the attic above the exterior walls of the house. Two approaches are used to obtain this added height; 1) a cantilevered truss, or 2) a raised-heel truss.

This report covers the results of testing four different heel configurations typical of those used in residential roof truss construction. These test results are intended to verify the cantilever design procedure proposed by the Truss Plate Institute (TPI). The study was done in two phases. The first was sponsored and funded in part by the Cantilever Committee of TPI (John Denny, Chairman), and the second by Lumbermate Company of St. Louis, Missouri, and the Small Homes Council-Building Research Council (SHC-BRC) of the University of Illinois at Urbana-Champaign. The tests were conducted at the hydraulic test facility of the TPI.

The study was done in two phases. The first was sponsored and funded by the Cantilever Committee of TPI (John Denny, Chairman), and the second by Lumbermate Company of St. Louis, Missouri, and the Small Homes Council-Building Research Council (SHC-BRC) of the University of Illinois at Urbana-Champaign. The tests were conducted at the hydraulic test facility of the SHC-BRC.

OBJECTIVES

Phase I was conducted to provide information on the relative performance and structural adequacy of two types of end conditions on a residential truss. A standard design was included as a control for purposes of comparison. Three end conditions were tested: a normal end-bearing design; a 24" cantilever truss containing heel wedge blocks; and a 24" cantilever design with top-chord reinforcing members. The information obtained in this phase was developed to verify a proposed TPI cantilever design procedure.

Phase II was conducted to investigate the performance of a non-engineered, but commonly used, "energy-saver" truss. Three end conditions were studied: a normal end-bearing design; a raised-heel truss with a 12" overall end-height and no diagonal struts; and a similar but engineered raised-heel truss with diagonal struts to complete triangulation.

MATERIALS

Lumber. Following the study plans, all of the chord lumber was selected using a portable "E-Tester," to match the published "E" values for the grade. This selection process was used to eliminate variation in the modulus of elasticity (MOE) of the lumber. The lumber used in the chords was 2x4 #1-KD Southern Pine. The measured MOE averaged 1.8x10^6 and the moisture content ranged between 8 and 15% with an average of 12%. The web lumber was 2x4 #3-KD Southern Pine, but was not sorted for MOE. (The web lumber was shorter than the required 10-foot length needed for the "E-tester.") The specific gravity was measured for each piece of lumber, with the top chords averaging 0.57, the bottom chords 0.58, and the webs 0.52.

Connectors The metal-plate connectors were 20 gauge (min. 0.036"), Lumbermate series 680, a typical residential roof truss connector plate. All of the test trusses were fabricated by Okaw Buildings of Chesterville, Illinois, under the supervision of the authors.

TRUSS DESIGNS

Phase I. Three separate types, numbers I, II, and III, Figure 1, were developed for Phase I of the study. Each design had a basic overall span of 30'-0", 4/12 slope, and a "W" or Fink web configuration. Three units were fabricated for each design. Plating details are shown on the engineering drawings in Appendix A.

Phase II. Three separate types, numbers IV, V, and VI, Figure 2, were developed for Phase II, with each design having an overall span of 26'-0", 4/12 slope, and a "W" or Fink web configuration. Three units were fabricated for each design. Plating details are shown on the engineering drawings in Appendix A.

An 80-pound-per-linear-foot (plf) load was selected as the design load for purposes of truss design and plate selection. (The original intent of this study was to hold lumber and basic configuration constant, thereby resulting in some unavoidable over-design.) The entire design load was applied to the top chords to facilitate testing. Previous testing experience and analysis of residential trusses has indicated there is no significant difference in deflection between placing the entire load on the top chords and dividing the load between the top and bottom chords. Specifications and details of these truss configurations are shown on the engineering drawings in Appendix A.

TEST PROCEDURES

The truss tests were conducted in a hydraulic truss testing facility located at the University of Illinois at Urbana-Champaign. Each truss was positioned horizontally between roller bearings and allowed to move freely against calibrated compression load cells at the reaction points. The loads were applied by hydraulic cylinders in parallel, exerting point-loads at one-foot intervals along the top chord, as shown in Figure 3. Roller hold-down brackets were positioned along the top chord to prevent lateral buckling during the tests. Deflection measurements were taken by the taut wire method with mirrored scales placed on the bottom chord of the truss at the centerline and quarter-points. On cantilevered trusses, dial gauges were positioned at each end of the bottom chord overhang.
TYPE I. This truss was a standard 30-foot end-bearing design. The plate sizes and locations are shown on the engineering drawings in Appendix A.

TYPE II. This truss was a standard 30-foot design with 2x8 wedge blocks added at the heel location. The reactions were set back 24 inches at each end to form cantilevers.

TYPE III. This truss was a standard 30-foot design with 2x6 reinforcing members fabricated to the top chords at the heel locations. The reactions were set back 24 inches at each end to form cantilevers.

Figure 1. Truss Configurations for Phase I.
TYPE IV. This truss was a standard 26-foot design. The sizes and locations of the plates are shown in the engineering drawings in Appendix A.

TYPE V. This truss was fabricated as a 30-foot nominal span, following industry practice for web location. Then 24 inches were cut from each end to form a 26-foot, raised-heel configuration sometimes incorrectly called an "energy truss." Vertical members were added at each end to provide a 12-inch clearance for insulation.

TYPE VI. This truss configuration was identical to Type V except that diagonal members were also added at each end to complete triangulation in accordance with recommended engineering practice.

Figure 2. Truss Configurations for Phase II.
The test loads were applied in 20-plf increments at 5-minute intervals with deflections recorded at each increment. Deflection readings were initiated at 20 plf due to the nature of the test procedures and difficulty in obtaining accurate readings at zero load. The data was later extrapolated to obtain an estimated reading for zero load. Loads were applied until failure occurred. Deflection was recorded up to failure in most cases. Ultimate loads and location and type of failure were also recorded.

RESULTS AND DISCUSSION

The test results are summarized in tabular and graphic form. Tables 1 and 2 list ultimate load data and centerline deflection at several selected load levels. Figures 4 and 6 plot loads versus centerline deflection. Average deflection values for each of the six designs were used for plotting the load-deflection curves.

The results of each test phase will be covered separately, and comparisons between the two phases will be made later. The two sets of end-bearing control trusses performed as expected from past research and testing experience.

Phase I. Table 1 shows the deflection and ultimate load results for the truss types tested in Phase I: end-bearing; wedge-block; and reinforcing-member. All the test loads are presented in pounds per lineal foot (plf) with deflection in inches.

Figure 4 shows the load-deflection curves using the average deflection for each type. Under the same load, the centerline deflection was smallest for the end-bearing trusses and largest for the wedge-block cantilevered trusses. For the cantilevered trusses, the reinforcing member provided considerably more stiffness than did the wedge-block. Both types of cantilevered trusses failed at higher loads than the end-bearing trusses. Higher axial forces due to the longer clear span probably accounted for the lower failure loads in the end-bearing trusses.

The end-bearing trusses performed as expected to the point of failure and exhibited minimal deflection or distortion at the design load. The cantilevered wedge-block trusses exhibited excessive deflection and chord curvature even at low loads. The first panels of the top chord bowed slightly upward because the upward bending due to the eccentric moment more than cancelled the downward bending due to the uniform load. The second panels of the top chord had large downward curvatures because bending from both the eccentric moment and uniform load acted in the downward direction. The cantilevered reinforcing-member trusses distorted similarly, but to a lesser degree. The end panels were stiffened by the reinforcing members and exhibited minimal distortion, even near the ultimate load. However, the top chord second panels deflected nearly as much as those in the cantilevered wedge-block trusses. Diagrams showing the characteristic patterns of deflection are shown in Appendix B.

As mentioned earlier, the cantilevered trusses had a higher ultimate load at failure than the end-bearing trusses. This was primarily because the clear span of the end-bearing trusses was four feet more than the clear span of the cantilevered trusses and both were carrying the same total load. In general, the moment to be resisted by the truss increases with the square of the clear span. While the cantilevered trusses carried more load, in spite of the large eccentric moments created at the heel, the end-bearing trusses had the lowest deflection to-load ratio. The eccentric moments increased deflection and distortion without reducing ultimate load capacity. It should be noted that no large strength-reducing characteristics were located in the chord section with eccentric moments. Figure 5 shows some details of the tests and several examples of failure types.

Phase II. Table 2 shows the deflection and ultimate load results for the truss types tested in Phase II: end-bearing; raised-heel with no diagonal strut; and raised-heel with strut. Figure 6 shows the load-deflection curves using the average deflection for each type. The end-bearing and raised-heel with strut truss types performed similarly and failed at about 3.5 times the design load, and exhibited no excessive deflection or chord curvature. The raised-heel
Figure 4. Phase I, Load-Deflection Data
(averaged for three trusses of each type)

TRUSS TYPES
I  •  end bearing
II  +  wedge blocks
III  ▪  top chord reinforcing members

Table 1. Centerline Deflection and Ultimate Load Data, Phase I

<table>
<thead>
<tr>
<th>Truss Type</th>
<th>Load-Deflection (in.)</th>
<th>Ultimate Load pf</th>
<th>Failure Location, Type</th>
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<tbody>
<tr>
<td></td>
<td>40</td>
<td>80</td>
<td>120</td>
</tr>
<tr>
<td>I end-bearing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>.145</td>
<td>.319</td>
<td>.484</td>
</tr>
<tr>
<td>2</td>
<td>.115</td>
<td>.287</td>
<td>.441</td>
</tr>
<tr>
<td>3</td>
<td>.128</td>
<td>.295</td>
<td>.461</td>
</tr>
<tr>
<td>ave</td>
<td>.130</td>
<td>.300</td>
<td>.462</td>
</tr>
<tr>
<td>II wedge blocks</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>.362</td>
<td>.728</td>
<td>1.067</td>
</tr>
<tr>
<td>5</td>
<td>.362</td>
<td>.724</td>
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<tr>
<td>6</td>
<td>.338</td>
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<tr>
<td>ave</td>
<td>.342</td>
<td>.708</td>
<td>1.070</td>
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<td>III top chord reinforcing</td>
<td></td>
<td></td>
<td></td>
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<td>7</td>
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<td>.346</td>
<td>.551</td>
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<td>8</td>
<td>.180</td>
<td>.366</td>
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</tr>
<tr>
<td>9</td>
<td>.170</td>
<td>.358</td>
<td>.563</td>
</tr>
<tr>
<td>ave</td>
<td>.170</td>
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a. Compression ring at each reaction to monitor forces

b. Bottom chord failure in end-supported control truss

c. Typical deflection pattern of cantilevered trusses

d. Truss No. 7 showing 2x6 reinforcing top chord

e. Bottom chord failure in Truss No. 4. Note wedge block

f. Top chord bending failure of Truss No. 6

Figure 5. Selected Photographs of Phase I Testing
Figure 6. Phase II, Load-Deflection Data (averaged for three trusses of each type)

TRUSS TYPES
IV ● end bearing
V ▲ raised heel, no struts
VI ■ raised heel, struts added

Table 2. Centerline Deflection and Ultimate Load Data, Phase II

<table>
<thead>
<tr>
<th>Truss Type</th>
<th>Load-Deflection (in.)</th>
<th>Ultimate Load (plf)</th>
<th>Failure Location, Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>40</td>
<td>80</td>
<td>120</td>
</tr>
<tr>
<td>IV end bearing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>.120</td>
<td>.280</td>
<td>.430</td>
</tr>
<tr>
<td>11</td>
<td>.100</td>
<td>.200</td>
<td>.320</td>
</tr>
<tr>
<td>12</td>
<td>.115</td>
<td>.220</td>
<td>.370</td>
</tr>
<tr>
<td>ave.</td>
<td>.112</td>
<td>.233</td>
<td>.373</td>
</tr>
<tr>
<td>V raised heel no struts</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>.550</td>
<td>1.420</td>
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<td>14</td>
<td>.585</td>
<td>1.970</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>.525</td>
<td>1.620</td>
<td></td>
</tr>
<tr>
<td>ave.</td>
<td>.553</td>
<td>1.670</td>
<td></td>
</tr>
<tr>
<td>VI raised heel struts added</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>.160</td>
<td>.280</td>
<td>.430</td>
</tr>
<tr>
<td>17</td>
<td>.180</td>
<td>.320</td>
<td>.470</td>
</tr>
<tr>
<td>18</td>
<td>.140</td>
<td>.260</td>
<td>.410</td>
</tr>
<tr>
<td>ave.</td>
<td>.160</td>
<td>.287</td>
<td>.437</td>
</tr>
</tbody>
</table>
Figure 7. Selected Photographs of Phase II Testing

a. Plate shear failure in Truss No. 12, end-supported

b. Rotation of raised-heel truss without strut

c. Top chord, tension-bending failure, strut added

d. Typical deflection and failure of raised-heel without strut

e. Top chord bending failure in Truss No. 18, with strut

f. Typical top chord bending failure of raised-heel with strut
with no diagonal strut-type, however, failed at the design load and exhibited tremendous deflection and chord curvature.

At very low loading, the raised-heel with no strut-type behaved as though the connections to the vertical members at both ends were rigid. The eccentric moment created by the chord separation over the bearing was indicated by the upward bending of the chords inside the end verticals. At about 60 plf, the connections between the chords and the verticals could no longer carry the eccentric moment and began to slip and fail. At this point, there was a sudden increase in deflection and a complete change in chord curvature. The vertical acted as a pinned-end member, and the chords exhibited large downward bending in their first panels, with the maximum moment occurring at the first interior panel points on both top and bottom chords. The vertical connections then pulled loose entirely and the rapidly increasing deflection at constant load prevented the test from continuing. If the vertical connections had not rotated, failure would have occurred at the first interior panel point of the top chords. Figure 7 shows several examples of failure types in Phase II.

When a strut was added to the raised-heel configuration, triangulation was achieved and the eccentric moment was avoided. The raised-heel with diagonal strut type was very stiff under load and showed no unusual chord curvature. The struts, which were in compression and braced in the weak direction, buckled somewhat in the strong direction under heavier loading. In all three trusses of this type tested, the diagonal struts buckled toward the top chord. Joint fixity and eccentricity on both ends of the struts may have caused this buckling.

Of the six truss types examined in this study, three were statically determinate - the two end-bearing types and the raised-heel with diagonal strut-type. The statically determinate types, selected primarily as controls, carried three to four times design load, exhibited minimal deflection, and failed at the location of a strength-reducing characteristic in a chord or in a critical joint. The statically indeterminate types - wedge-block, reinforcing member, and raised-heel with no diagonal strut-type, were statically unstable because of the four-sided heel panels created by the top and bottom chord separation over the bearing. However, the joints in the heel panel were "semi-fixed" by chord continuity and/or connector plate rigidity, achieving some stability but forming large eccentric moments that affected the joints and the adjacent members.

For these statically indeterminate trusses, joint fixity over the bearing related directly to load capacity. The wedge-block and reinforcing-member trusses had very good joint fixity and carried large loads. The raised-heel with no diagonal strut-type had poor joint fixity and poor load capacity. Chord stiffness and joint fixity in the heel panel related directly to good deflection performance. Only the reinforcing-member truss-type exhibited good deflection performance. This is because the reinforcing member stiffened the top chord, preventing curvature, and, along with the joint fixity, stiffened the heel panel and, thereby, the whole truss.

In this study, deflection performance was an important criteria for truss-type evaluation. In typical residential roof trusses, initial deflection can be expected to double over several years as a result of creep if stressed near design loads. In wood members stressed below the proportional limit, creep may continue at a decreasing rate over a period of years, and at some point level off. In wood members stressed above the proportional limit, creep may continue at an increasing rate until failure occurs. This is rarely a consideration in residential roof truss design because the trusses are seldom called upon to support the design load for extended periods.

From analysis of the six truss types in this study, it is known that several chord members in the wedge-block and raised-heel with no diagonal strut truss-types were severely overstressed at the selected design load. The raised-heel with no diagonal strut truss-type failed at design load and probably would have failed at half that load over a period of time as a result of creep. The wedge-block trusses failed at loads far above design load, but if they were loaded to design load only, creep would probably occur at a constant or increasing rate, resulting in very large long-term deflections and possibly truss failure after a period of time. The deflections shown in this report do not reflect the influence of creep. Since trusses are usually sheathed with panel sheathing on the top chords, thereby contributing to an equal distribution of the load and transfer of part of the load to the supporting walls, the initial distortion and deflection of the trusses in actual use would be less, but would increase substantially with time due to creep.
CONCLUSIONS

The results of the first phase of this study indicate the cantilevered trusses equalled the strength of the end-bearing trusses, but deflected and distorted more.

A raised-heel truss without struts (Phase II) is similar to any truss with two webs cut out—it does not function properly. The performance of the raised-heel truss type was improved dramatically with the introduction of a diagonal strut at each end.

Whenever the top and bottom chords are separated over the bearing, eccentric moments are created, and must be either resisted by reinforcing the heel joints and one or more chords or eliminated by restoring triangulation through the addition of additional diagonal struts.

REFERENCES


3. Design Specification for Metal Plate Connected Wood Trusses, 1978. Truss Plate Institute, 100 W. Church St., Frederick, MD 21701.


Typical truss plates
TRUSS T-3 30'-0" SPAN T43 4/12
A-01 TPI CANT TEST

DES. BY: JMD

MAY 1, 1980
CODE TPI 78

LUMBERMATE COMPANY
SAINT LOUIS, MISSOURI

TRUSS PLATE: SERIES 680 20GA 200 PSI NET

LOAD DURATION ADJUSTMENT 15% TRUSS SPACING 2.00 FEET ON CENTERS
REPETITIVE MEMBER BENDING STRESS USED IN THIS DESIGN

REACTION = 1173# MIN. BRG = 3.50 IN

MEMBER FORCE MEMBER FORCE MEMBER FORCE MEMBER FORCE
1- 2 -2752 1- 7 +2611 2- 7 -622 3- 7 +622
2- 3 -2289 7- 6 +1731

* Note: This number indicates the truss plate specified by Lumbermate Co. The number 203x9 indicates 20 gauge, 3" x 9". Other manufacturers may use other notation systems.
TRUSS 4-6 30' - 0" SPAN T43 4/12
A-01 TPI CANT TEST
MAY 1, 1980
CODE TPI 7B

DES. BY: JD

TOP CHORD LIVE LOAD 40.0 PSF
TOP CHORD 2X4 SOU PINE #1 KD
TOP CHORD DEAD LOAD 0.0 PSF
BOT CHORD 2X4 SOU PINE #1 KD
BOT CHORD DEAD LOAD 0.0 PSF
WEBS 2X4 SOU PINE #3 KD
TOTAL UNIFORM LOAD 40.0 PSF

LOAD DURATION ADJUSTMENT 15%
TRUSS SPACING 2.00 FEET ON CENTERS
REPETITIVE MEMBER BENDING STRESS USED IN THIS DESIGN

REACTION= 1173# MIN. BRG= 3.50 IN
PLATES: SERIES 680 20GA 200 PSI NET
MEMBER FORCE MEMBER FORCE MEMBER FORCE MEMBER FORCE
1- 2 -2752 1- 7 +2611 2- 7 -622 3- 7 +622
2- 3 -2289 7- 6 +1731

LUMBERMATE COMPANY
SAIN T LOUIS, MISSOURI

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DESIGN BASED ON CRITERIA ESTABLISHED BY THE TRUSS PLATE INSTITUTE AND "HIDE" BY THE NATIONAL FOREST PRODUCTS ASSOCIATION.

CUT MEMBERS TO BEAR, LATERALLY SUPPORT CHORDS.
LUMBERMATE TRUSS PLATES OF GALVANIZED STEEL ARE INDICATED BY GAUGE AND SIZE. PRESS PLATES SECURELY ON BOTH SIDES OF JOINTS. CENTER PLATES ON JOINTS UNLESS NOTED.

MEMBER FORCE

204 x 4½
203 x 2½
203 x 9
203 x 11½
4½"
2 x 8 WEDGE
24"
30'-0"

© 1980 LUMBERMATE COMPANY
TRUSS 7-9  30'-0'' SPAN T43  4/12
A-01  TPI CANT TEST  DES. BY: JMD  MAY 1, 1980  CODE TPI 7E

TOP CHORD LIVE LOAD  40.0 PSF  TOP CHORD 2X4  SOUTHERN PINE #1 KD
TOP CHORD DEAD LOAD  .0 PSF  BOT CHORD 2X4  SOUTHERN PINE #1 KD
BOT CHORD DEAD LOAD  .0 PSF  WEBS 2X4  SOUTHERN PINE #3 KD
TOTAL UNIFORM LOAD  40.0 PSF

LOAD DURATION ADJUSTMENT  15%  TRUSS SPACING  2.00 FEET ON CENTERS
REPETITIVE MEMBER BENDING STRESS USED IN THIS DESIGN

REACTION= 1173#  MIN. BRG= 3.50 IN  PLATES: SERIES 680 20GA 200 PSI NET

MEMBER FORCE  MEMBER FORCE  MEMBER FORCE  MEMBER FORCE
1-  2   -2752  1-  7   +2611  2-  7   -622  3-  7   +622
2-  3   -2289  7-  6   +1731

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CUT MEMBERS TO BEAM, LATERALLY SUPPORT CHORDS.
LUMBERMATE TRUSS PLATES OF GALVANIZED STEEL ARE INDICATED BY GAUGE AND SIZE. PRESS PLATES SECURELY ON BOTH SIDES OF JOINTS. CENTER PLATES ON JOINTS UNLESS NOTED.
TRUSS 10-12 26'-0" SPAN T43 4/12  ENGR JD  OCT 12, 1981  CODE TP1 7B

TOP CHORD LIVE LOAD 40.0 PSF  TOP CHORD 2X4  SOU PINE #1 KD
TOP CHORD DEAD LOAD 0.0 PSF  BOT CHORD 2X4  SOU PINE #1 KD
BOT CHORD DEAD LOAD 0.0 PSF  WEBS 2X4  SOU PINE #3 KD
TOTAL UNIFORM LOAD 40.0 PSF
TRUSS SPACING 2.00 FT CIRS  LOAD DURATION ADJUSTMENT 15%

REACTON AT 1 = 1013  MIN BRG= 4.00 IN
MEMBER FORCE MEMBER FORCE MEMBER FORCE
1-2 -2373  2-7 -537  3-7 +537
2-3 -1972
1-7 +2251
7-6 +1491

LM PLATE SERIES:
680 20GA 200 PSI NE-1

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SAINT LOUIS, MISSOURI

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CUT MEMBERS TO BEAR. LATERALLY SUPPORT CHORDS.
LUMBERMATE TRUSS PLATES OF GALVANIZED STEEL ARE INDICATED BY GAGE AND SIZE. PRESS PLATES SECURELY ON BOTH SIDES OF JOINTS. CENTER PLATES ON JOINTS UNLESS NOTED.
Forces are not listed because this configuration is statically indeterminate. This design was a standard 30-foot truss with the ends cut off.
TRUSS 16-18 30'-0" T43 4/12
A-01

DES. BY: JMD

FEB 11, 1982
TP-178 ANALYSIS

LM PLATE SERIES:
680 20GA 200 PSI NET

LOAD DURATION FACTOR 15%

TRUSS SPACING 2.00 FT. CENTERS

REATIONS:
JOINT VERT. HOR. MIN.BRG.
1 1040 0 4.00 IN.
7 1040 0 4.00 IN.

WEB AXIAL MEMBER FORCE LBR WEB AXIAL MEMBER FORCE LBR
1 - 2 -236 W1 1 - 3 -1953 W1
3 - 9 -319 W1 3 - 4 -328 W1
5 - 6 -236 W1

WEB AXIAL MEMBER FORCE LBR WEB AXIAL MEMBER FORCE LBR
1 - 2 -236 W1 1 - 3 -1953 W1
3 - 9 -319 W1 3 - 4 -328 W1
5 - 6 -236 W1

LUMBERS:
T1 2X4 SOU PINE 1 KD 15
B1 2X4 SOU PINE 1 KD 15
W1 2X4 SOU PINE 3 KD 15

TOP AXIAL MEMBER FORCE LBR
2 - 3 0 T1 1 - 9 1780 B1
3 - 4 -1653 T1 9 - 8 1344 B1
4 - 5 -1653 T1 8 - 7 1780 B1
5 - 6 -0 T1
APPENDIX B
Examples of Deflection Configuration

Ave. $\Delta = 2.63''$
at 240 plf

Typical deflection configuration for wedge-block

Ave. $\Delta = 1.39''$
at 240 plf

Typical deflection configuration for reinforced top chord

Ave. $\Delta = 1.67''$
at 80 plf

Typical deflection configuration for raised-heel-type without diagonal members added