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A STUDY OF ROOF TRUSSES

BY

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UNIVERSITY OF ILLINOIS
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A STUDY OF ROOF TRUSSES

By N. Clifford Ricker, D. Arch., Professor of Architecture

The investigation described in this bulletin had for its original object the determination of a formula for the weight of roof trusses more accurate than those now in existence. As the investigation progressed, however, other topics arose and some interesting results were secured, which it is believed will be of value to architects and engineers. Very little study has been devoted to roof trusses in comparison with the thorough treatment of bridge trusses by eminent writers. The chief result of the work has been the devising of a method to save time and labor by presenting data in a form most convenient for comparison. This system will be found convenient in calculating and designing roof trusses to satisfy given conditions, whether constructed of wood and steel, or entirely of steel.

I. METHOD OF INVESTIGATION

In the determination of weights, general mathematical methods may be readily applied to most forms of bridge trusses, especially those with parallel chords; these are, however, less valuable for roof trusses where far more varied conditions must be arbitrarily limited in order to make such methods applicable. The results are then of doubtful worth. A more practical method of investigation was therefore chosen. For a single common type of truss, Fig. 2, nearly fifty trusses of varied span, rise, and distance apart were calculated and designed in the same general
Next the weight of each truss was carefully computed, and if this materially differed from the assumed weight of the truss, the necessary corrections were made in the sectional dimensions and weight of members.

The verticals were steel rods with upset ends; all other members were long leaf pine timbers. Splices in the tie-beam and its connection with the principal were made with vertical steel fish plates and through bolts. A purlin rested on each apex of the principal and supported the rafters on which was laid \( \frac{1}{8} \)-in. matched sheathing covered by a painted tin roof.

In accordance with the usual custom of engineers the roof was assumed to support a snow load and wind pressure at the same time, although the writer believes that this extreme condition rarely occurs. The assumption, however, provides some surplus strength for contingencies, such as unusual snowfall, very violent winds, etc.

II. CONDITIONS ASSUMED

The spans of the trusses were assumed to vary by 20-ft. intervals from 20 to 200 ft.

The rise of the trusses was varied by 5-ft. intervals from \( \frac{1}{10} \) to \( \frac{1}{4} \) the span.

The distance between trusses was varied by 5-ft. intervals from 10 to 30 ft.

The horizontal panel length was varied from 10 to 25 ft.

The number of purlins per panel was varied from 1 to 5.

III. LOADS SUPPORTED BY A TRUSS

1. Permanent loads

The following are the permanent loads assumed:

- Painted tin covering.................. 2 lb. per sq. ft.
- Long leaf pine lumber.............. 4 lb. per sq. ft. B. M.
- Steel (see Cambria, etc.).......... 480 lb. per cu. ft.
- Cast iron .......................... 450 lb. per cu. ft.
- Weight of truss..................... Assumed

The weights of trusses were first assumed in accordance with Merriman and Jacoby's formula; but the following formula was deduced from the results of this investigation and is found to agree more closely with the computed weights of the trusses examined:
2. Snow load

The snow load varies with latitude, but was here assumed at 20 lb. per sq. ft. of horizontal projection of roof for location of Chicago. Denoting by $i$ the angle of inclination of roof surface with the horizontal, we have $20 \cos i =$ snow load in lb. per sq. ft. of inclined roof surface.

3. Wind pressure normal to roof

The formula for the normal wind pressure most commonly employed in England and the United States is that of Hutton, viz.:

$$P_n = P \sin i \left(1.84 \cos i - 1\right)$$

in which $P$ denotes the pressure on a vertical surface and $P_n$ the normal pressure.

A review of Hutton's apparatus and experiments (Hutton's mathematical papers) casts serious doubts upon the accuracy of this formula. The complex form is also objectionable. Other formulas are proposed by different authors. In Fig. 1 are shown the values of $P_n$ given by various formulas reduced to a common basis of 30 lb. per sq. ft. on a vertical plane. A comparison of these values shows that different formulas give widely different results.

![Fig. 1](image-url)
Müller of Breslau is the greatest living authority on graphic statics and probably on the theory of bridges and roofs; his formula, however, gives the smallest values for normal wind pressures. It is evident that very little is certainly known concerning the relation between horizontal and normal wind pressures; hence the following empirical formulas were here adopted as being sufficient and convenient in use:

Taking the angle \( i \) in degrees,

\[
P_n = \frac{1}{2} i, \text{ for } P = 30 \text{ lb. per sq. ft. horizontal pressure.}
\]

\[
P_n = \frac{1}{3} i, \text{ for } P = 40 \text{ lb. per sq. ft. horizontal pressure.}
\]

\[
P_n = \frac{1}{4} i, \text{ for } P = 50 \text{ lb. per sq. ft. horizontal pressure.}
\]

These formulas are applicable for values of \( i \) less than 45° for higher inclinations, the normal and horizontal pressures are equal. They were believed to be original, but it has since been found that similar formulas had already been published.

IV. APPLICATION OF THE METHOD TO A TRUSS

The treatment of a specimen truss will most clearly explain the method employed.

1. Assumptions

Let the following program be assumed:

Span of truss = 200 ft.; rise = 50 ft.; trusses set 20 ft. apart on centers; 20 panels of 10 ft. each; \( i = 26.5^\circ \). Roof covered with painted tin, laid on matched sheathing, supported by rafters resting on one purlin fixed on each apex of principal. Vertical rods, splice plates, and bolts to be of steel; other members to be long leaf pine timbers.

The following values are readily found:

Panel length of principal = 11.18 ft.

Panel area of roof surface = 223.6 sq. ft.

Snow load on roof surface = \( 20 \cos i = 17.9 \text{ lb. per sq. ft.} \)

Normal wind pressure on roof surface = \( \frac{1}{2} i = 17.7 \text{ lb. per sq. ft.} \)

2. Sheathing

To determine the maximum safe length \( L \) of sheathing between rafters we proceed as follows:

Vertical load on sheathing = 2 lb. (tin) + 4 lb. (sheathing) + 17.9 lb. (snow) = 23.9 lb. per sq. ft.
This load is resolved into a normal component, acting at right angles to surface of sheathing, and into a component parallel to the surface. The latter may be neglected, since it is resisted amply by the edgewise strength of the sheathing. The normal component is $23.9 \cos \theta = 21.4$ lb. per sq. ft. Adding to this the normal wind pressure, we get for the total normal load on the sheathing $21.4 + 17.7 = 39.1$ lb. per sq. ft.

Let $L =$ distance in ft. between centers of rafters

$t =$ thickness of sheathing in inches.

For safety against breaking the sheathing we have

$$L = 43.2 \frac{t}{\sqrt{w}} = \frac{43.2 \times 0.875}{\sqrt{39.1}} = 6.05 \text{ ft.}$$

For deflection limited to $\frac{1}{390}$ of length,

$$L = 12.9 \frac{t}{\sqrt{w}} = \frac{12.9 \times 0.875}{\sqrt{39.1}} = 3.33 \text{ ft.}$$

Hence the rafters can not be spaced more than 3.33 ft. between centers.

3. Rafters

Assuming 2-in. by 6-in. rafters, the maximum safe distance between their centers is determined as follows:

Let $e =$ distance between centers of rafters in inches;

$$I = \text{section modulus} = 12 \text{ for 2-in. by 6-in. cross-section.}$$

$$I = \text{moment of inertia} = 36 \text{ for 2-in. by 6-in. cross-section.}$$

The values of the section modulus and moment of inertia may be found without calculation by means of Tables 1 and 2.

The vertical load on rafters is 2 lb. (tin) + 4 lb. (sheathing) + 2 lb. (rafters) + 17.9 lb. (snow) = 25.9 lb. per sq. ft.

Hence the normal load on the rafters is $25.9 \cos \theta + 17.7 \text{(wind)} = 40.9$ lb. per sq. ft.; and the load parallel to the rafters acting lengthwise and producing longitudinal compression in them is $25.9 \sin \theta = 11.6$ lb. per sq. ft.

For safety against breaking,

$$e = \frac{11200 I}{wL^2c} = \frac{11200 \times 12}{40.9 \times 11.1^2} = 26.3 \text{ in. between centers of rafters.}$$
### Table 1

**Table of Values of Section Modulus $\frac{I}{c}$**

For Rectangular Wooden Timbers

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### Table 2

**Table of Values of Section Moment of Inertia $I$**

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For safe limit of deflection,

$$e = \frac{30223 I}{wL^3} = \frac{30223 \times 36}{40.9 \times 11.18^3} = 19.2 \text{ in. between centers of rafters.}$$

Since the rafters are dressed smaller than 2 in. by 6 in., it is best to space them 16 in. between centers instead of 19.2 in., thus making 15 equal spaces per bay of the roof.

The deflection $\Delta$ of the rafters at the middle of their length is

$$\Delta = \frac{3 wL^3}{2720000 \cdot I} = \frac{3 \times 40.9 \times 11.18^4 \times 16}{2720000 \times 86} = .315 \text{ in.}$$
The total longitudinal load on one rafter is
\[ 11.6 \times 1\frac{1}{2} \times 11.18 = 173 \text{ lb.} \]
One-half this, or 86.5 lb., acts on the cross-section at mid-length of the rafter. If the rafter is straight, this compression would be uniformly distributed over the cross-section, thus giving
\[ \frac{86.5}{12} = 7.2 \text{ lb. per sq. in. compression.} \]
But after the rafter has deflected .315 in., the compression in the upper fibers of the cross-section is increased to
\[ p' = p \left(1 + \frac{6 \Delta}{d}\right) = 7.2 \left(1 + \frac{6 \times 0.315}{6}\right) = 9.5 \text{ lb. per sq. in.,} \]
where \( p \) = uniform compression in lb. per sq. in. at mid-length, and \( d \) = depth of rafter in inches.

If the load is expressed in tons, we have
\[ p' = 9.5 \text{ lb. per sq. in.} = .0048 \text{ ton per sq. in. of cross-section.} \]
The compression \( p' \) must be deducted from the safe fiber stress of long leaf pine timbers in tons per sq. in. to obtain the safe value for this case. Taking 0.7 tons per sq. in. as this safe stress, we obtain, therefore,
\[ .7000 - .0048 = .6952 \text{ ton per sq. in. as the net safe fiber stress for long leaf pine timbers.} \]
Let this be denoted by \( F; \) then for safety against breaking and compression:
\[
\Theta = \frac{16000 \cdot F \cdot I}{wL^3c} = \frac{16000 \times .6952 \times 12}{40.9 \times 11.18^3} = 26.0 \text{ in. = maximum distance between centers of rafters in inches.} \]
It is evident from this analysis that the parallel loading and the longitudinal compression thereby caused in rafters may generally be neglected except for very steep roofs.

4. **Purlins**

The weight of the purlin being unknown is at first neglected. The total loading on a purlin may be resolved into two components, one normal to the roof, the other parallel thereto.

Denoting these components by \( W \) and \( W' \) respectively, we have
\[
W = \frac{223.6 \times 40.9}{2000} = 4.57 \text{ T., the normal loading,} \]
\[
W' = \frac{223.6 \times 11.6}{2000} = 1.30 \text{ T., the parallel loading.} \]
For normal loading
\[ \frac{I}{c} = 2.14 \times WL = 2.14 \times 4.57 \times 20 = 195.2 \]
\[ I = .795 \times WL^2 = .795 \times 4.57 \times 20^2 = 1453 \]

For parallel loading
\[ \frac{I}{c} = 2.14 \times WL = 2.14 \times 1.30 \times 20 = 55.6 \]
\[ I = .795 \times WL^2 = .795 \times 1.30 \times 20^2 = 413 \]

From Tables 1 and 2 we select a cross-section for the purlin such that the values for \( \frac{I}{c} \) and \( I \) (edgewise for normal loading and flatwise for parallel loading) are not less than those just found. A cross-section 8 in. x 14 in. is found to satisfy the four conditions just determined.

The weight of the purlin is therefore
\[ \frac{8 \times 14 \times 4 \times 20}{12} = 747 \text{ lb.} = .374 \text{ T.} \]

The normal component of the weight is
\[ .374 \cos i = .33 \text{ T.} \]

and the parallel component of the weight is
\[ .374 \sin i = .17 \text{ T.} \]

Adding these to previous values, we find \( W = 4.90 \text{ T.} \) and \( W' = 1.47 \text{ T.} \)

Using these new values of \( W \) and \( W' \), we obtain for \( \frac{I}{c} \) and \( I \) the following results:

For normal loading
\[ \frac{I}{c} = 2.14 \times 4.90 \times 20 = 209.3 \]
\[ I = .795 \times 4.90 \times 20^2 = 1558 \]

For parallel loading
\[ \frac{I}{c} = 2.14 \times 1.47 \times 20 = 59.6 \]
\[ I = .795 \times 1.47 \times 20^2 = 448 \]

Referring again to Tables 1 and 2, we find that a purlin 8 in. x 14 in. is still amply sufficient to support both its load and its own weight.
5. Weight of truss

Using the formula given in Art. 3, we obtain for the weight of truss

$$\frac{S}{25} + \frac{S^2}{6000} = \frac{200}{25} + \frac{200^2}{6000} = 14\frac{3}{5} \text{ lb. per sq. ft. of horizontal projection of roof.}$$

6. Computation of apex loads

Permanent loads:

- Covering, sheathing and rafters, $223.6 \times 8 = 1789$ lb.
- Purlin, as before: $747$ lb.
- Truss: $200 \times 14\frac{3}{5} = 2933$ lb.
- Snow load: $200 \times 20 = 4000$ lb. = 2.000 T.
- Wind load: $223.6 \times 17.7 = 3958$ lb. = 1.979 T.

7. Computation of loading on one-half truss

- Permanent loading: $2.735 \times 9\frac{1}{2} = 25.98$ T.
- Snow loading: $2.000 \times 9\frac{1}{2} = 19.00$ T.
- Wind loading: $1.979 \times 9\frac{1}{2} = 18.80$ T.

8. Truss diagram, stress diagrams, and stress sheet

Having the loads, the truss diagram is drawn, and the stress diagrams are then drawn for permanent and wind loadings.
<table>
<thead>
<tr>
<th>Member</th>
<th>Permanent Stress</th>
<th>Snow Stress</th>
<th>Wind Stress</th>
<th>Maximum Stress</th>
<th>Center Length</th>
<th>Sectional Dimensions</th>
<th>Center Length Weight, lb.</th>
<th>Weight of Connections</th>
<th>Excess of Perm. Stress</th>
<th>Corrected Maximum Stress</th>
<th>Corrected Sectional Dimensions</th>
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<tr>
<td>X 14</td>
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<td>22.2</td>
<td>12.5</td>
<td>65.0</td>
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<td>1460</td>
<td>+ 229 lb. wood</td>
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<td>1224</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ - 761 \text{ lb. wood} \]

\[ + 1648 \text{ lb. steel} \]

Total weight = 51342 + 3212 lb. = 54554 lb. Less 174 lb.
The magnitude of each stress is measured and written in the proper column and space of the stress sheet.

Since the diagram for snow stresses would be exactly similar to that for permanent stresses, it need not be drawn, but the snow stresses can be quickly found with a good slide rule, using the following proportion:

$$25.98 : 19.00 = \text{permanent stress} : \text{snow stress}.$$  

It is best to tabulate the permanent, snow and wind stresses separately, to permit taking any combination of them as the maximum stress on the members.

Maximum stresses are here found by taking the sum of the stresses on each member.

9. **Sections of members**

Principals and struts are in compression, and their sectional dimensions are most conveniently found by the graphical table, Fig. 5, based on a modified form of Stanwood's formula for yellow pine columns, viz.:

$$p = 0.700 - 0.084 \frac{L}{d}$$

where

- $p =$ safe compression in tons per sq. in. 
- $L =$ free length of timber in feet 
- $d =$ least side in inches

(a) **Principals**

The length of the principal is 11.18 ft.; and from the stress sheet the maximum compression in $X_1$ is 125.5 tons. Hence by the table (Fig. 5) the section of principal should be 14 in. x 14 in. For convenience in construction and better appearance, the principals are made of uniform section throughout their entire length.

(b) **Struts**

The struts are most conveniently framed and look best if made the same breadth as principal, so as to be flush on each side. They are here 14 in. wide, and their safe thickness or depth is found by the table.

(c) **Tie-beam**

The tie-beam is to be made the same breadth as principals and struts. The safe resistance of long leaf pine to tension is
taken as 0.6 ton per sq. in. Then if \( b \) is the breadth of the tie-beam in inches and \( d \) the depth, we have

\[
\frac{d}{0.6 \times b} = \frac{117.8}{0.6 \times 14} = 14.0 \text{ in.}
\]

Assuming that 3 horizontal rows of 1-inch bolts are to be used in splices, we should make the total depth \( 14 + 3 = 17 \) in., or say 18 in.

(d) Vertical steel rods

The table for rods, nuts, washers, etc., is based on a safe tensile stress of 7.5 tons per sq. in. for net section of rods, whose ends are upset to standard diameters. The bearing areas and diameters of washers are based on a maximum safe pressure of \( \frac{1}{3} \) ton per sq. in. on long leaf pine.

**TABLE 4**

**TABLE OF STEEL RODS FOR LONG LEAF PINE TIMBERS**

<table>
<thead>
<tr>
<th>Diameter of Rod inches</th>
<th>Safe Strength tons</th>
<th>Upset Ends inches</th>
<th>Sq. Nuts inches</th>
<th>Cast iron Round Washers inches</th>
<th>Area of Washers sq. in.</th>
<th>Drop inches</th>
<th>Weight 1 in. Rod pounds</th>
<th>No. Two Ends, Nuts and Washers</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{1}{8} )</td>
<td>1.47</td>
<td>3 by 41</td>
<td>( \frac{3}{8} ) by 18</td>
<td>( \frac{3}{8} ) by 21</td>
<td>4.93</td>
<td>2 ( \frac{3}{4} )</td>
<td>.67</td>
<td>3.48</td>
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<tr>
<td>( \frac{3}{32} )</td>
<td>1.87</td>
<td>3 by 41</td>
<td>( \frac{3}{8} ) by 18</td>
<td>( \frac{3}{8} ) by 21</td>
<td>6.13</td>
<td>2 ( \frac{3}{4} )</td>
<td>.85</td>
<td>3.96</td>
</tr>
<tr>
<td>( \frac{7}{32} )</td>
<td>2.30</td>
<td>3 by 41</td>
<td>( \frac{3}{8} ) by 18</td>
<td>( \frac{3}{8} ) by 21</td>
<td>7.59</td>
<td>2 ( \frac{3}{4} )</td>
<td>1.04</td>
<td>5.76</td>
</tr>
<tr>
<td>( \frac{1}{8} )</td>
<td>2.78</td>
<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
<td>9.23</td>
<td>2 ( \frac{3}{4} )</td>
<td>1.26</td>
<td>7.70</td>
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<tr>
<td>( \frac{3}{32} )</td>
<td>3.22</td>
<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
<td>10.85</td>
<td>2 ( \frac{3}{4} )</td>
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</tr>
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<td>1 by 18</td>
<td>1 by 31</td>
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<td>1 by 18</td>
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<td>5.18</td>
<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
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<td>2 ( \frac{3}{4} )</td>
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<td>5.89</td>
<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
<td>19.27</td>
<td>2 ( \frac{3}{4} )</td>
<td>2.67</td>
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<td>( \frac{7}{32} )</td>
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<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
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<td>1 by 18</td>
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<td>1 by 18</td>
<td>1 by 31</td>
<td>29.84</td>
<td>2(\frac{3}{4})</td>
<td>4.17</td>
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<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
<td>33.03</td>
<td>2(\frac{3}{4})</td>
<td>4.60</td>
<td>44.70</td>
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<td>11.14</td>
<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
<td>36.00</td>
<td>2(\frac{3}{4})</td>
<td>5.05</td>
<td>47.40</td>
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<tr>
<td>( \frac{7}{32} )</td>
<td>12.17</td>
<td>1 by 43</td>
<td>1 by 18</td>
<td>1 by 31</td>
<td>39.46</td>
<td>2(\frac{3}{4})</td>
<td>5.52</td>
<td>55.56</td>
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<td>2 by 4</td>
<td>2 by 7(\frac{1}{2})</td>
<td>43.09</td>
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<td>2 by 4</td>
<td>2 by 7(\frac{1}{2})</td>
<td>46.51</td>
<td>2(\frac{3}{4})</td>
<td>6.52</td>
<td>69.78</td>
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<td>2 by 4</td>
<td>2 by 8(\frac{3}{8})</td>
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<td>( \frac{1}{8} )</td>
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<td>2 by 4</td>
<td>2 by 8(\frac{3}{8})</td>
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<td>2 by 4</td>
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<td>2 by 8(\frac{3}{8})</td>
<td>58.32</td>
<td>2(\frac{3}{4})</td>
<td>8.18</td>
<td>95.06</td>
</tr>
</tbody>
</table>
Divide the maximum tension in a member by the number of rods composing it to find the stress in one rod; the diameter of the rod is then found in the table.

10. Splices in tie-beam and connections with principals

Splices in principals being in compression, they are simple halved splices 1 foot long with two or four 1-inch bolts each, and are made as near the apex as possible.

Splices in tie-beams and connections of tie-beams with principals are calculated by the following formulas:

- Let $t$ = thickness of one fish plate in inches.
- $R$ = number of horizontal rows of bolts.
- $D$ = diameter of bolt in inches.
- $b$ = breadth of tie-beam in inches.
- $d$ = depth of the tie-beam in inches.
- $a$ = distance in inches between centers of bolts or to end of timber.
- $N$ = total number of bolts in both ends of splice.
- $Z$ = maximum longitudinal stress in member at splice or connection.

(a) Connection of tie-beam with principal.

For the thickness of the fish plate, we have

$$ t = \frac{Z}{15 \left( d - R \frac{D}{3} \right)} = \frac{125.5}{15 (18 - 3 \times 1)} = .56 \text{ in. to resist tension.} $$

$$ t = .525 \cdot D = .525 \times 1 = .53 \text{ in. to resist crushing against bolt.} $$

Therefore we make these fish plates each $\frac{5}{6}$ in. thick.

$$ a = 5 \cdot D = 5 \text{ in. between centers of bolts or to end of timber.} $$

The end of the fish plate is to be 2 in. outside the center of the bolt.

For the number of bolts, we have

$$ N = \frac{Z}{.375 \cdot Db} = \frac{125.5}{.375 \times 14 \times 1} = 24 \text{ one-in. bolts in the connection.} $$

These bolts must be symmetrically arranged about the axis of the member, so that it becomes necessary to put some $\frac{3}{4}$-in. bolts in 1-in. holes to prevent the fish plates from buckling outwards, being under compression.

(b) Splice in the member Y 5.

Applying the same formulas, we find $\frac{5}{6}$-in. fish plates as before, and 20 one-in. bolts are required.

(c) Splice in Y 11.
Fish plates must be \( \frac{3}{4} \) in. thick, and 16 one in. bolts are needed.

\( (d) \) Splice at middle of truss.

Fish plates should be \( \frac{9}{16} \) in. thick, and 12 one-in. bolts are needed.

11. Calculation of center length weights of members

The center length of each member is easily computed or measured on the truss diagram and it is then written on the stress sheet. This center length is then multiplied by the weight per linear foot of the member, which is doubled to include both sides of the truss (except for the middle vertical 18 18). These weights are then written in the proper column of the stress sheet.

12. Calculation of weights of joints and connections

The differences between the actual and center length weights of each member are next computed and written on the stress sheet.

The table of rods, etc., gives for rods their weight in pounds per linear foot, also the weight of the two upset ends, of two nuts, and of two cast-iron washers, but it does not include the portions of rod between the intersection of the center lines of the truss members and the upset end of rod. This weight must be computed and added to weight of rod ends, etc.

These weights of connections of members are written in the stress sheet.
18. Computed weight of truss

From the data presented in the preceding paragraphs, the total weight of the truss may now be calculated. We have

- Total center length weight of members: 51342 lb.
- Total net weight of connections: 3160 lb.

Total computed weight of truss: 54554 lb.

Assumed weight of truss: 58660 lb.

Deduct computed weight: 54554 lb.

Excess of assumed weight over computed weight: 4106 lb.

We have, therefore, excess per apex of principal:

\[
\frac{4106}{20} = 205.3 \text{ lb.}
\]

Whence the excess in permanent loading on one-half the truss is

\[
\frac{205.3 \times 9\frac{1}{2}}{2000} = 0.975 \text{ T.}
\]

This excess is \( \frac{0.975}{25.98} = 0.03\frac{3}{4} \) or \( 3\frac{3}{4} \) per cent of the permanent load; hence permanent stresses in the stress sheet are \( 3\frac{3}{4} \) per cent in excess. This excess is computed, written in the stress sheet, and deducted from the maximum stress. The corrected maximum stresses are thus found, since the snow and wind stresses on the members are unchanged.

The sectional dimensions of members are next revised for these corrected maximum stresses, the result being a slight reduction in two members only. The corresponding reduction in weight is found to be 174 pounds.

14. Final Summary

- Net weight of wood in truss: 45429 lb.
- Net weight of metal, nearly all steel: 8777 lb.

Total net weight of truss: 54380 lb.

Hence the weight of the wood is \( 83\frac{1}{2} \) per cent and the weight of the metal is \( 16\frac{1}{2} \) per cent of the total weight of truss.

The weight of the connections is \( 6\frac{1}{2} \) per cent of the center length weight of the truss.
Weight per square foot of horizontal projection of roof:

- Roof covering: 2.12 lb.
- Sheathing: 4.24 lb.
- Rafters: 2.12 lb.
- Purlins: 3.74 lb.
- Truss: 13.58 lb.
- Permanent load: 25.80 lb.
- Snow load: 20.00 lb.
- Wind load: 17.70 lb. on inclined roof.

V. Weights of Trusses of Different Spans

Ten trusses were designed for spans of 20, 40, 60, 80, 100, 120, 140, 160, 180, and 200 ft. respectively. The trusses were set 20-ft. apart with a uniform rise of \( \frac{1}{4} \) span, the panel lengths being 10 feet. Their weights were computed and plotted in Fig. 7, and the points connected by the broken line \( D \). The increasing slope of this line shows that the weight of the truss increases faster than the span. The total permanent weight of the roof also increases with the span. The additional weight of the connections, however, diminishes very rapidly for spans less than 60 ft. and varies irregularly for greater spans. Other weights are constant.

VI. Formula for Weight of Truss

The following empirical formula for weights of long leaf pine and steel trusses is represented by the dotted line in Fig. 7, which agrees well with the computed line \( D \).

\[
W = \frac{S}{25} + \frac{S^2}{6000}
\]

where \( S \) = span in feet
and \( W \) = weight of truss in lb. per sq. ft. of horizontal projection of roof.

VII. Weight of White Pine and Steel Trusses

A series of trusses constructed of white pine timbers and steel verticals was also designed and computed. The results of these computations are shown in Fig. 8. The preceding formula
Fig. 8. Trusses 20 ft. on Centers—Rise—Span+4
White Pine with Steel Verticals

Span in Feet

B—Weight of purlin.
E—Per cent of total center length weights to be added for weight of connections.
C—Weight of covering, sheathing and rafters.
A—Weight of truss.
D—Total permanent weight.
F—Weight of truss by Ricker's formula.
is represented by the dotted line $F$, which here gives weights somewhat exceeding those found by computation. In Fig. 9 is shown a comparison of the weights of trusses and also of the total weights of roofs constructed of the two kinds of wood. White pine makes a somewhat lighter truss and roof than long leaf pine. However, in designing, it will be most convenient and accurate to apply the same formula and make the necessary reductions on the stress sheet.

**VIII. Weight of Steel Trusses**

Several steel trusses of different spans were also designed and computed. Their weights for spans of 100 and 200 feet were found to be about the same as those of long leaf pine and steel trusses. Therefore the formula is also applicable to steel trusses. It is very probable, however, that for short spans, steel trusses are somewhat heavier than those of wood and steel given by the formula; their connections are far more complex and certainly require the addition of a larger per cent to the center length weights of truss. But these variations are taken into account on the stress sheet.
IX. Most Economical Distance Between Trusses

Trusses of 200-ft. span and 50-ft. rise spaced 10, 15, 20, 25 and 30 ft. apart respectively were designed and their weights were computed. The results are shown in Fig. 10. The weight of covering, sheathing, and rafters remains constant; the weight of the purlins and connections increases, and the weight of the truss diminishes as the distance between the trusses increases, being a minimum for a spacing of 25 ft. The total weight of the roof, however, is a minimum when the trusses are 15 ft. apart.
X. Most Economical Size of Panels

Trusses of 200-ft. span, 50-ft. rise, set 20 ft. apart, were divided into 8, 10, 12, 14, 16, 18 and 20 panels to determine the best length of panel. The results are given in Fig. 11 and show the following: (1) that the weight of covering, sheathing and rafters increases with the panel length; (2) the weight of the purlins diminishes; (3) the weight of the truss, and very nearly that of the entire roof, is a minimum for a panel length of 20 feet. Therefore 20-ft. panels appear to be most economical.

XI. Best Number of Purlins Per Panel

For a series of trusses of like dimensions with panel lengths of 25 ft. from 1 to 5 purlins were used on each panel length of the principal, which was therefore required to resist safely the...
longitudinal compression and the stresses caused by the weights of the purlins and their loads. The results are plotted in Fig. 12. These show (1) that the weight of the purlins increases with their number; (2) the weight of covering, sheathing, and rafters decreases; (3) the additional weight of the connections slightly diminishes; (4) the weight of truss is increased, but is least with one purlin per panel. The total weight of the roof is least for 2, 3 or 4 purlins. No advantage results from the use of more than 2 purlins per panel of 25 ft. or of more than one for panels of ordinary size.

![Fig. 12. Trusses 200-ft. Span—50 ft.-Rise-20 ft. on Centers L. L. Pine with Steel Verticals; Eight Panels](image_url)
XII. Effect of Raising Lower Chord at Center of Span

Trusses of like dimensions were designed, excepting that the lower chord was raised 0, 5, 10, 15, 20 and 25 ft. respectively at the center of span, the rise of upper chord being 50 ft.

The results are given in Fig. 13. The dotted line $D$ represents the weights of trusses of equal depth at the center, but having a horizontal lower chord and supporting smaller normal wind pressures on account of their lesser inclinations.

The weight of covering, sheathing, rafters, and purlins remains constant; the weights of trusses and of roofs both increase rapidly with the rise of lower chord. A comparison of curves $D$
and $E$ shows clearly that this raising or cambering of the lower chord is not economical and is done only for effect.

XIII. MOST ECONOMICAL RATIO OF RISE TO SPAN OF TRUSS

A series of trusses of 200-ft. span and set 20 ft. apart was also designed and computed for rises of 20, 25, 30, 35, 40, 45 and 50 ft. respectively in order to determine the best proportion of rise to span. The results are plotted in Fig. 14. As the rise increases, the weights of covering, sheathing, rafters, and purlins slightly increase; also the additional weight of the connections: the weight of trusses and that of the roof diminish, each being a minimum for a rise of 35 ft. which is practically $\frac{1}{3}$ the span,
identical with the ratio for ordinary bridge trusses. Hence the best rise is \( \frac{1}{6} \) the span.

XIV. SUMMARY OF RESULTS

(a) Comparison of formulas for normal wind pressure.
(b) System of calculation and design.
(c) Form of stress sheet.
(d) Formula for weight of truss.
(e) Comparative weights of trusses of other materials.
(f) Economical distance between trusses.
(g) Economical length of panels.
(h) Economical number of purlins per panel.
(i) No advantage results from cambering lower chord.
(j) Economical ratio of rise to span of roof trusses.
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