THE DESIGN OF

SIMPLE ROOF-TRUSSES

IN WOOD AND STEEL.

WITH AN INTRODUCTION TO THE ELEMENTS
OF GRAPHIC STATICS.

BY

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FIRST EDITION.

FIRST THOUSAND.

PREFACE.

VERY little, if anything, new will be found in the follow-
ing pages. The object in writing them has been to bring
together in a small compass all the essentials required in
properly designing ordinary roof-trusses in wood and steel.

At present this matter is widely scattered in the various
comprehensive treatises on designing and in manufacturers'
pocket-books. The student who desires to master the ele-
ments of designing simple structures is thus compelled to
procure and refer to several more or less expensive books.

Students in mechanical and electrical engineering, as
a rule, learn but little of the methods of designing em-
ployed by students in civil engineering. For this reason
the writer has been called upon for several years to give a
short course in roof-truss design to all students in the Junior
class of the Rose Polytechnic Institute, and in order to do
so he has been compelled to collect the data he has given
in this book.

The tables giving the properties of standard shapes are
based upon sections rolled by the Cambria Steel Company.
Standard sections rolled by other manufacturers have
practically the same dimensions.

<div style="text-align: right">MALVERD A. HOWE.</div>

TERRE HAUTE, IND., September, 1902.
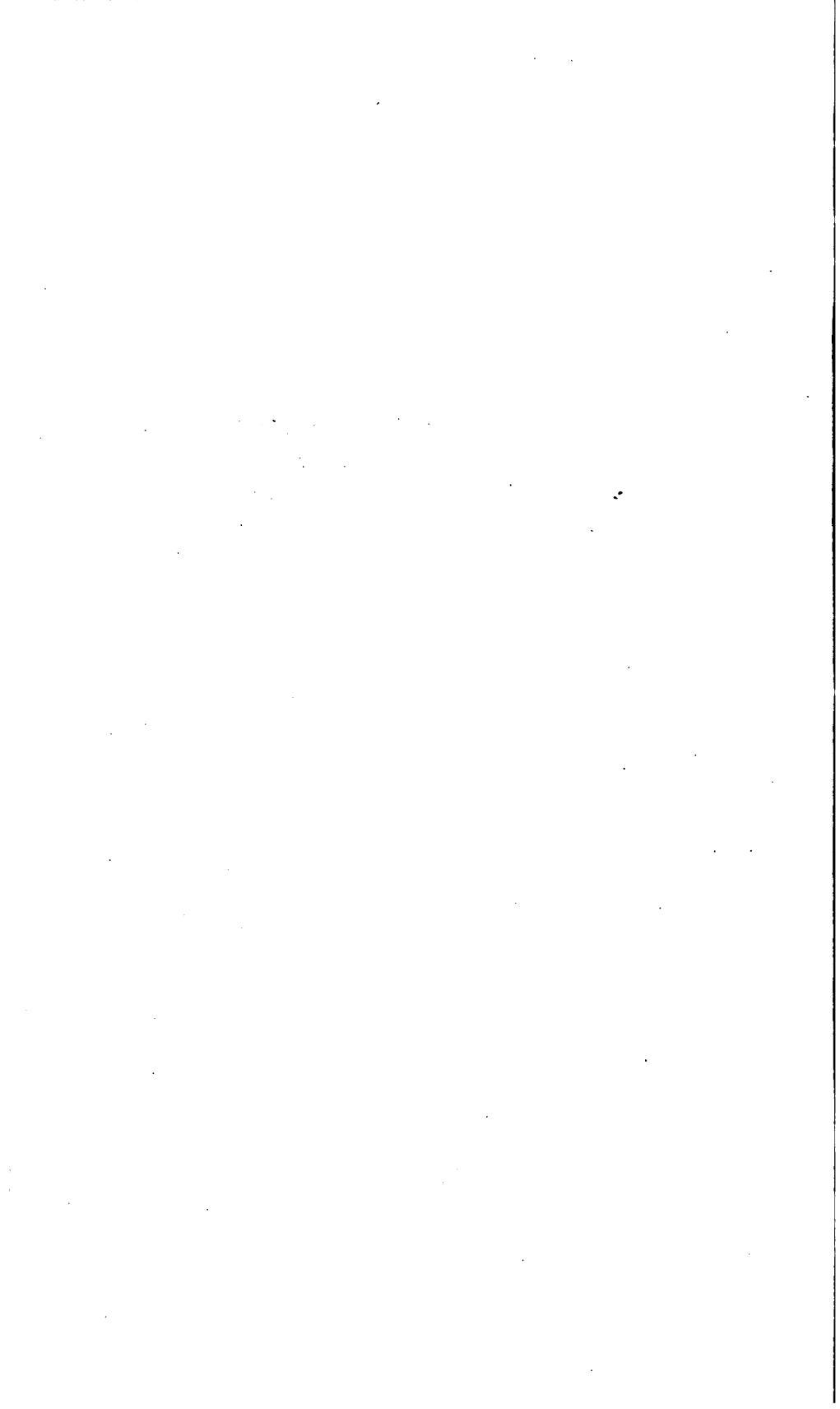
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CHAPTER I.

GENERAL PRINCIPLES AND METHODS.

1. Equilibrium.—Forces acting upon a rigid body are in equilibrium when the body has neither motion of translation nor rotation.

For forces which lie in the same plane the above conditions may be stated as follows:

(a) There will be no motion of translation when the algebraic sums of the components of the forces resolved parallel to any two coordinate axes are zero. For convenience the axes are usually taken vertical and horizontal, then the vertical components equal zero and the horizontal components equal zero.

(b) There will be no motion of rotation when the algebraic sum of the moments of the forces about any center of moments is zero.

2. The Force Polygon.—Let AB, BC, CD, and DA, Fig. 1, be any number of forces in equilibrium. If these forces are laid off to a common scale in succession, parallel to the directions in Fig. 1, a closed figure will be formed as shown in Fig. 1a. This must be true if the algebraic sums of the vertical and horizontal components respectively equal zero and there is no motion of translation. Such a figure is called a force polygon.

Conversely, if any number of forces are laid off as explained above and a closed figure is formed, the forces are
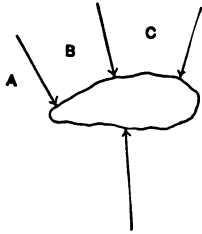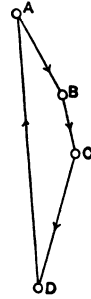
FIG. 1.   FIG. 1*a*.

in equilibrium as far as motion of translation is concerned. Motion of rotation may exist, however, when the above condition obtains.

3. **Forces Not in Equilibrium.**—In case a number of forces, not in equilibrium, are known in direction and magnitude, the principle of the force polygon (Art. 2) makes it possible to at once determine the magnitude and direction of the force necessary to produce equilibrium.
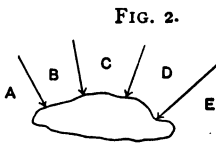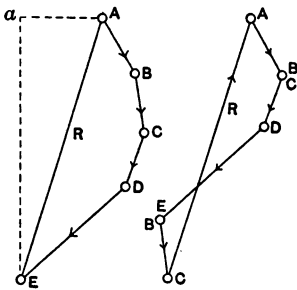
FIG. 2.

Let AB, BC, ..., DE be forces not in equilibrium, Fig. 2. According to Art. 2, lay them off on some convenient scale, as shown in Fig. 2*a*. Now in order that the sum of the vertical components shall equal zero a force must be introduced

FIG. 2*a*.   FIG. 2*b*.

having a vertical component equal to the vertical distance between E and A, and in order that the horizontal components may equal zero the horizontal component of this force must equal the horizontal distance between E and A. These conditions are satisfied by the force EA. If this force acts in the direction shown by the arrow-head in Fig. 2a, it will keep the given forces in equilibrium (Art. 2). If it acts in the opposite direction, its effect will be the same as the given forces, and hence when so acting it is called the *resultant*.

Fig. 2b shows the force polygon for the above forces drawn in a different order. The magnitude and direction of R is the same as found in Fig. 2a.

4. Perfect Equilibrium.—Let the forces AB, BC, . . . , DE, Fig. 2, act upon a rigid body. Evidently the force R, found above (Art. 3), will prevent motion, either vertically or horizontally, wherever it may be applied to the body. This fulfills condition (a) (Art. 1). For perfect equilibrium condition (b) (Art. 1) must also be satisfied. Hence there must be found a point through which R may act so that the algebraic sum of the moments of the forces given and R, may be zero. This point is found by means of the equilibrium polygon.

5. The Equilibrium Polygon.—Draw the force polygon (Art. 2) $ABCDE$, Fig. 3a, and from any convenient point P draw the lines $S_1, S_2, . . . , S_5$. If S_1 and S_2 be measured with the scale of the force polygon, they represent the magnitudes and directions of two forces which would keep AB in equilibrium as far as translation is concerned, for they form a closed figure with AB (Art. 2). Likewise S_2 and S_3 would keep BC in equilibrium, etc. Now in Fig. 3 draw

S_1 parallel to S_1 in Fig. 3a, S_2 parallel to S_2 in Fig. 3a, etc.,
as shown.   If forces be assumed to act along these lines
having the magnitudes shown in Fig. 3a, respectively, the
points 1, 2, 3, and 4 will be *without motion*, since the forces

FIG. 3.
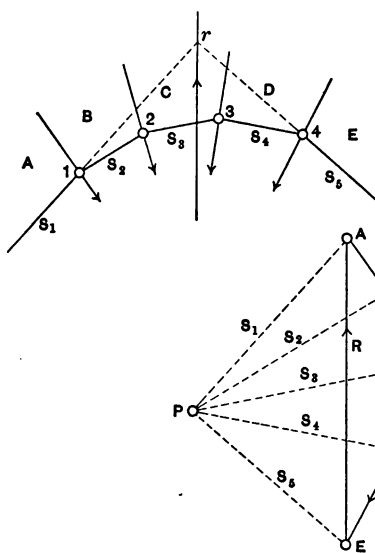
FIG. 3a.

meeting at each point are in equilibrium against translation
by construction, and, since they meet in a point, there can
be no rotation.

In Fig. 3a, S_1 and S_5 form a closed figure with R; there-
fore if, in Fig. 3, S_1 and S_5 be prolonged until they intersect
in the point r, this point will be free of all motion under the
action of the forces S_1, S_5, and R.

Since the points 1, 2, 3, 4, and r in Fig. 3 have neither
motion of translation nor rotation, if the forces AB, BC, CD,
and DE and the force R be applied to a rigid body in the
relative positions shown in Fig. 3, this body will have no

motion under their action.   The forces S_1 and S_5 keep the system $ABCD$ in equilibrium and can be replaced by R.

The lines S_1, S_2, etc., in Fig. 3a are for convenience called *strings*, and the polygon S_1, S_2, S_3, etc., in Fig. 3 is called the *equilibrium polygon*.

The point P in Fig. 3a is called the *pole*.

6. Application of the Equilibrium Polygon in Finding Reactions.—Let a rigid body be supported at K and K', Fig. 4, and acted upon by the forces AB, BC, CD, and

FIG. 4.
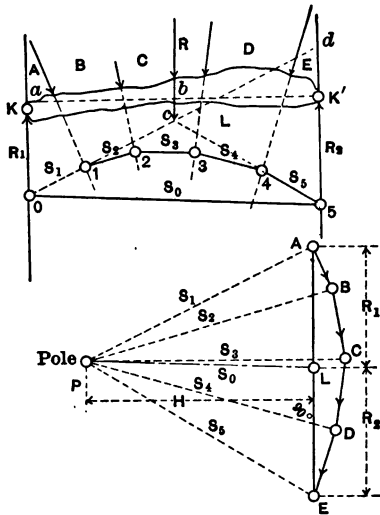
FIG. 4a.

DE.   Then, if equilibrium exists, it is clear that two forces, one at each support, must keep the forces AB, BC, etc., in equilibrium.   These two forces are called *reactions*.   For convenience designate the one upon the left as R_1, and the one upon the right as R_2.   The magnitudes of R_1 and R_2 can be found in the following manner: Construct the force

polygon and draw the strings S_1, S_2, etc., as shown in Fig. 4a, and then construct the equilibrium polygon (Art. 5) as shown in Fig. 4. Unless some special condition is introduced the reactions R_1 and R_2 will be parallel to EA, Fig. 4a, and their sum equal the magnitude of EA, or the resultant of the forces AB, BC, CD, DE. Draw through K and K' lines parallel to R, and, if necessary, prolong the line S_1 until it cuts oK, Fig. 4, and S_5 until it cuts $5K'$. Connect o and 5, and in Fig. 4a, draw the string S_0 parallel to $o5$, Fig. 4, until it cuts EA in L. Now, since S_1, S_0, and AL form a closed figure in Fig. 4a, the point o in Fig. 4 will be in equilibrium under the action of these three forces. For a like reason the point 5 will be in equilibrium under the action of the three forces S_0, S_5, and EL. Therefore the reaction $R_1 = AL$ and $R_2 = LE$, and the body M will be in equilibrium under the action of the forces AB, BC, CD, DE, R_1 and R_2.

It may not be perfectly clear that no rotation can take place from the above demonstration, though there can be no translation since $R_1 + R_2 = EA$, the force necessary to prevent translation under the action of the forces AB, BC, CD, and DE.

To prove that rotation cannot take place let the forces AB, BC, etc., be replaced by their *resultant* R, acting downward, as shown in Fig. 4.

If no rotation takes place (Art. 1),

$$R(bK') = R_1(aK') \quad \text{or} \quad R_1 = \frac{bK'}{aK'}R.$$

From the similar triangles $od5$, Fig. 4, and PAL, Fig. 4a,

$$d5 : aK' :: R_1 : H \quad \text{or} \quad R_1 aK' = H(d5).$$

From the similar triangles cd_5, Fig. 4, and PAE, Fig. 4a,

$$d_5 : bK' :: R : H \quad \text{or} \quad R(bK') = H(d_5).$$

$$\therefore \quad R_1(aK') = RbK' \quad \text{or} \quad R_1 = \frac{bK'}{aK'} R,$$

or the value of R_1 by the above construction fulfills the condition that no rotation takes place.

7. Parallel Forces.—In case the forces AB, BC, etc., had been parallel the force polygon would become a straight line and the line $ABCD \ldots E$ would coincide with EA. All of the constructions and conclusions given above apply to such an arrangement of forces. See Figs. 9 and 9a.

8. The Direction of One Reaction Given, to Find the Magnitude and Direction of the Other.—Let the direction of R_2 be assumed as vertical, then the horizontal compo-
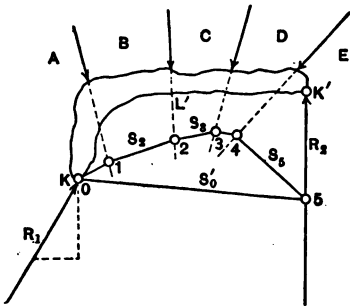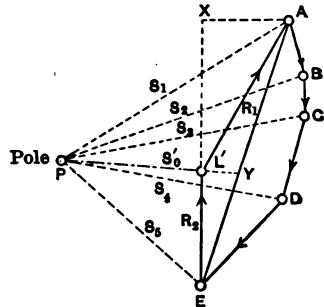
FIG 5. FIG. 5a.

nent, if any, of all the forces acting must be applied at K. The force polygon (Art. 2) becomes $ABCDEX$, as shown in Fig. 5a. Assume any pole P, and draw the strings S_1, S_2, etc. In Fig. 5, construct the equilibrium polygon (Art. 5) as shown, *starting with S_1, passing through K, the only point on R_1 which is known.* Draw the closing line S_0', and in

Fig. 5a the string PL' parallel to S_0' of Fig. 5. Then EL' is the magnitude of the vertical reaction R_2, and $L'A$ the magnitude and direction of the reaction R_1.

To show that there will be no rotation under the action of the above forces let AB, BC, etc., be replaced by their resultant Fig. 6. Then, in Fig. 5a, it is seen that R_1, R_2, and $EA = R$ form a closed figure. In Fig. 6, ED represents the direction and the relative position of the resultant R. If no rotation takes place DE prolonged will pass through the point B where R_1 and R_2 intersect. Since in Fig. 6 the sum of angles
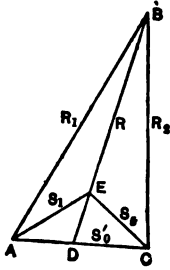
FIG. 6. BAE, EAD, and EDA equals the sum of the angles PAL', APY, and AYP in Fig. 5a, the angle ABD between R_1 and ED produced in Fig. 6 must equal the angle $L'AY$ in Fig. 5a. In like manner the angle CBD in Fig. 6 equals the angle AEL' in Fig. 5a. Consequently the sum of the angles ABD, DBC, BCD, and BAD equals 180° and DE produced passes through the point B.

9. Application of the Equilibrium Polygon in Finding Centers of Gravity.—Let $abc \ldots k$ be an unsymmetrical body having the dimension normal to the paper equal unity. Divide the area into rectangles or triangles whose centers of gravity are readily determined. Compute the area of each small figure, and assume that this area multiplied by ' the weight of a unit mass is concentrated at the center of gravity of its respective area. These weights may now be considered as parallel forces P_1, P_2 and P_3, acting as shown in Fig. 7. The resultant of these forces must pass through the center of gravity of the entire mass, and hence lies in the lines R and R' formed by constructing two equilibrium

GENERAL PRINCIPLES AND METHODS. 9

polygons for the forces P_1, P_2, and P_3, first acting vertically and then horizontally. The intersection of the lines R and R' is the center of gravity of the mass.

The load lines in Fig. 8 and Fig. 8a are not necessarily at right angles, but such an arrangement determines the point of intersection of R and R' with a maximum degree of accuracy, since they intersect at right angles.

FIG. 7.

FIG. 8a.

FIG. 8.

In the above constructions the weight of a unit mass is a common factor, and hence may be omitted and the areas alone of the small figures be used as the values of P_1, P_2, and P_3.

10. Application of the Equilibrium Polygon in Finding Moments of Parallel Forces.—Let AB, BC, ..., EF be any number of parallel forces, and M' and N' two points through which R_1 and R_2 pass (Fig. 9). Construct the force

polygon Fig. 9a, and select some point P as a pole, so that the perpendicular distance H from the load line is 1000, 10000, or some similar quantity. Construct the equilibrium polygon Fig. 9 as explained in previous articles.

Suppose the moment of AB, BC, and CD about M' as a center of moments is desired. The moment equals $AB(a_1) + BC(a_2) + CD(a_3) = M_m$. Prolong the lines S_3,

FIG. 9.
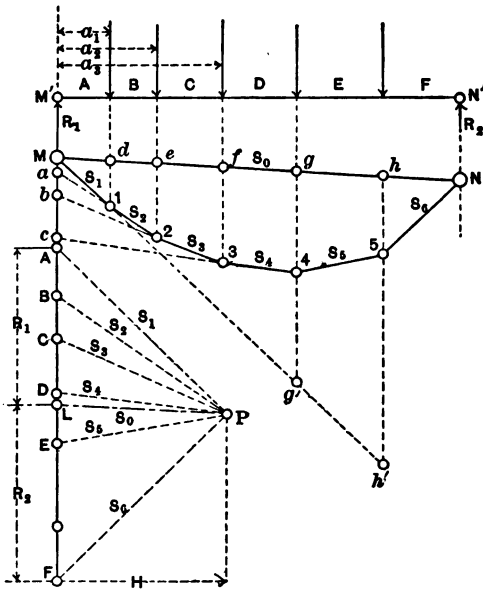
FIG. 9a.

S_3, and S_4 until they cut a line through M' parallel to AB, BC, etc.

From the triangles $MA1$, Fig. 9, and ABP, Fig. 9a,

$$aM : a_1 :: AB : H \quad \text{or} \quad AB(a_1) = H(aM).$$

From the triangles $ab2$, Fig. 9, and BCP, Fig. 9a,

$$ab : a_2 :: BC : H \quad \text{or.} \quad BC(a_2) = H(ab).$$

From the triangles $bc3$, Fig. 9, and CDP, Fig. 9a,

$$bc : a_3 :: CD : H \quad \text{or} \quad CD(a_3) = H(bc).$$

Or

$$AB(a_1) + BC(a_2) + CD(a_3) = M_m = H(aM + ab + bc) = H(Mc).$$

From this it is seen that the moment of any force equals the ordinate measured on a line passing through the center of moments, and parallel to the given force, which is cut off between the two sides of the equilibrium polygon which are parallel to the two strings drawn from the pole P (prolonged if necessary until they cut this line) to the extremities of the load in Fig. 9a; multiplied by the *pole distance* H. For a combination of loads the ordinate to be multiplied by H is the *algebraic* sum of the ordinates for each load; the loads acting downward having ordinates of one kind, and those acting upward of the opposite kind.

To illustrate, let the moment of R_1, AB, BC, and CD about g be required. In Fig. 9a the strings S_1 and S_0 are drawn from the extremities of R_1, hence in Fig. 9 the ordinate gg' multiplied by H is the moment of R about g as a center of moments.

The strings S_1 and S_4 are the extreme strings for AB, BC, CD, and hence the ordinate $g'4$ multiplied by H is the moment of these forces. Now since the reaction acts upward and the forces AB, BC, and CD act downward, the ordinate $g4$ multiplied by H is the moment of the combination.

The above property of the equilibrium polygon is very convenient in finding the moments of unequal loads spaced at unequal intervals, as is the case where a locomotive stands upon a girder bridge.

11. Graphical Multiplication.—Let the sum of the products a_1b_1, a_2b_2, etc., be required. The method of the previous article can be readily applied in the solution of this problem. Let b_1, b_2, etc., be taken as loads and a_1, a_2, etc., as the lever-arms of these loads about any convenient point as shown in Fig. 10. Then $H(ab) = a_1b_1$, $H(bc) = a_2$,
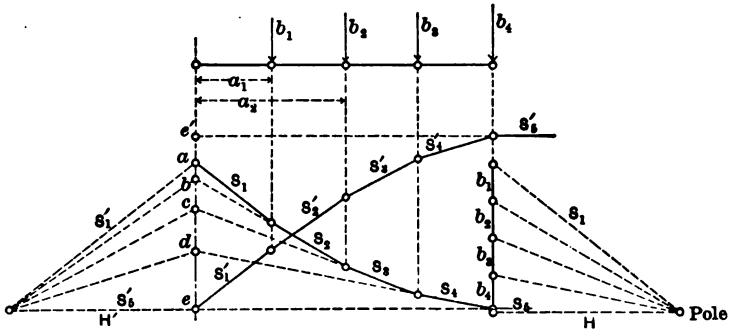
FIG. 10.   FIG. 10a.

b_2, etc., and finally $H(ae) = \Sigma(ab)$, or the algebraic sum of the products a_1b_1, a_2b_2, etc.

In case $\Sigma(ba^2)$ is desired, the ordinates ab, bc, etc., can be taken as loads replacing b_1, b_2, etc., in Fig. 10. For convenience take a pole distance H' equal to that used before and draw the polygon S_1', S_2', etc., then $(ee')H^2 = \Sigma(ba^2)$.

12. To Draw an Equilibrium Polygon through Three Given Points.—Given the forces AB, BC, CD, and DE, it is required to pass an equilibrium polygon through the points X, Y, and Z. Construct the force polygon Fig. 11a, and through X and Y draw lines parallel to EA. Then, starting with S_5, passing through Y, construct the equilibrium polygon Fig. 11, drawing the closing line S_0. In Fig. 11a there result the two reactions R_1 and R_2 when a line is drawn through P parallel to S_0 of Fig. 11. Since the values

of R_1 and R_2 remain constant for the given loads, the pole from which the strings in Fig. 11a are drawn must lie upon a line drawn from L parallel to a line S_0'' connecting X and Y in Fig. 11. That is, S_0'' is the position of the closing line for all polygons passing through X and Y, and the pole can be taken anywhere upon the line $P'L$ in Fig. 11a. In order that the polygon may also pass through Z take the loads upon the right of Z and find their resultant EB, and through Z draw a line parallel to EB. Assume Z and Y to be two
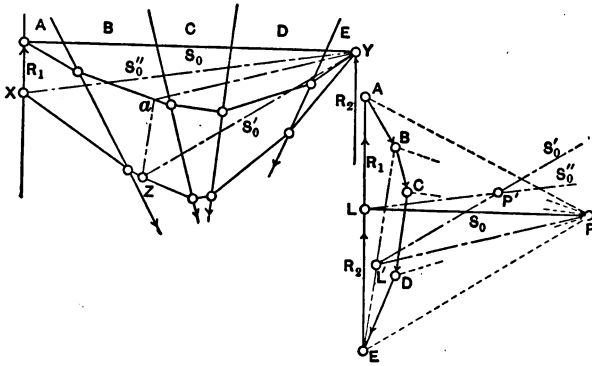
FIG. 11.   FIG. 11a.

points through which it is desired to pass an equilibrium polygon. Proceeding as in the first case, the pole must lie somewhere upon the line $L'P'$, Fig. 11a, drawn parallel to aY, Fig. 11. Then if a polygon with its pole in LP' passes through X and Y, and one with its pole in $L'P'$ passes through Z, the polygon with a pole at the intersection of these lines in P' will pass through the three points X, Y, and Z.

ROOF-TRUSSES.

CHAPTER II.

BEAMS AND TRUSSES.

13. **Vertical Loads on a Horizontal Beam: Reactions and Moments of the Outside Forces.**—Let the beam XY support the loads AB, BC, etc., Fig. 12, and let the ends of

FIG. 12.

FIG. 12a.
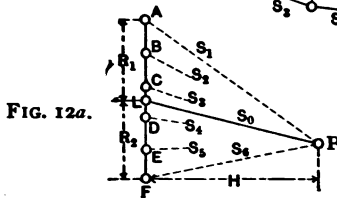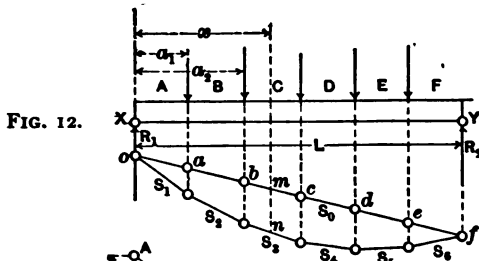
the beam rest upon supports X and Y. *Required the reactions* R_1 *and* R_2, neglecting the weight of the beam. In order that the beam remains in place free from all motion the outside forces AB, BC, etc., with R_1 and R_2 must fulfill the conditions of Art. 1. Proceeding according to Art. 6, the force polygon $ABCDEF$ is constructed, any point P taken as a pole, and the strings $S_1 \ldots \ldots S_5$ drawn, Fig. 12a. Then, in Fig. 12, the equilibrium polygon is constructed,

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the closing line S_0 drawn, and, parallel to this line, LP is drawn in Fig. 12a, cutting the line AF into two parts; LA being the value of R_1, and LF the value of R_2.

The moment about any point in the vertical passing through any point x is readily found by Art. 10:

$$M_x = R_1 x - AB(x - a_1) - BC(x - a_2) = (mn)H$$
$$= \text{the moment of the } \textit{outside forces}.$$

14. Vertical Loads on a Simple Roof-truss: Structure Considered as a Whole.—In this case the method of procedure is precisely that given in Art. 10. The reactions R_1 and R_2 will of course be equal if the loads are equal and
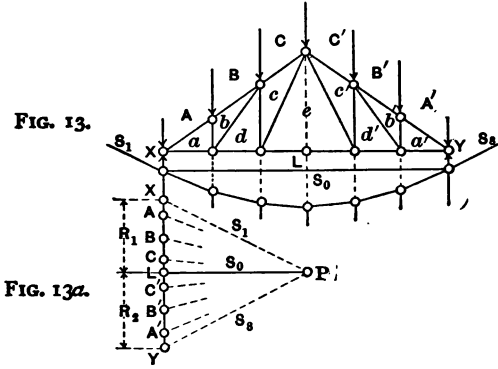
FIG. 13.

FIG. 13a.

symmetrically placed about the center of the truss. This being known, the pole P may be taken on a horizontal line drawn through L, Fig. 13a, and then the closing line S_0 in Fig. 13 will be horizontal. The closing line may be made horizontal in any case by taking the pole P horizontally opposite L, which divides the load line into the two reactions.

It is evident from what precedes that the particular shape of the truss or its inside bracing has no influence

upon the values of R_1, R_2, and the ordinates to the equilib-
rium polygon. However, the internal bracing must have
sufficient strength to resist the action of the outside forces
and keep each point of the truss in equilibrium.

**15. Inclined Loads on a Simple Roof-truss: Structure
Considered as a Whole.**—The case shown in Fig. 14 is that
usually assumed for the action of wind upon a roof–truss,
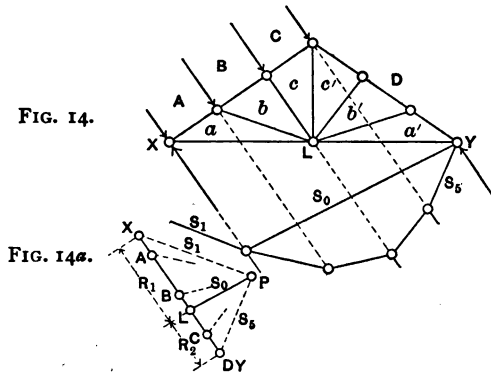
Fig. 14.

Fig. 14a.

the truss being supported at X and Y. The directions of
R_1 and R_2 will be parallel to AD of Fig. 14a. The deter-
mination of the values of R_1 and R_2 is easily accomplished
by Art. 10, as shown in Figs. 14 and 14a.

**16. Inclined Loads on a Simple Roof-truss, One
Reaction Given in Direction: Structure Considered as a
Whole.**—Suppose the roof-truss to be supported upon
rollers at Y. Then the reaction R_2 is vertical if the rollers
are on a horizontal plane. The only point in R_1 which is
known is the point of support X through which it must
pass. Drawing the equilibrium polygon through this point,
S_5 cuts the direction of R_2 in Y', and XY' is the closing line,
Fig. 15. At Y', which is by construction in equilibrium,

there are three forces acting having the directions S_6, S_5, and R_2, and these forces must make a closed figure; hence, in Fig. 15a, DL is the magnitude of R_2. Since R_1 must close the force polygon, LX is the magnitude and direction of R_1.

FIG. 15.
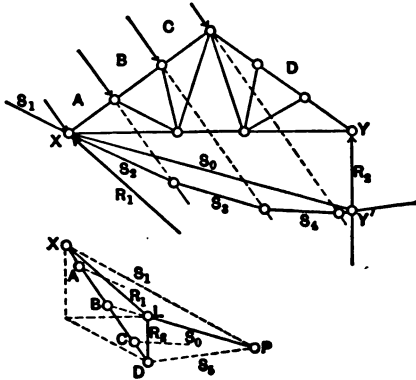
FIG. 15a.

If the rollers had been at X instead of Y, the method of procedure would have been quite similar. The equilibrium polygon would have passed through Y and ended upon a vertical through X, and the string S_0 would have cut off the value of R_1 on a vertical drawn through X, Fig. 15a.

17. Relation between the Values of R_2 in Articles 15 and 16.—In Article 15, R_2 can be replaced by its vertical and horizontal components without altering the existing equilibrium. If the supports are in a horizontal plane, the horizontal component can be applied at X instead of Y without in any way changing the equilibrium of the structure as a whole. Therefore the vertical component of R_2, as found in Art. 15, is the same in value as the R_2 found in

Art. 16. This fact makes it unnecessary to go through the constructions of Art. 16 when those of Art. 15 are at hand. The constructions necessary to determine R_1 and R_2 of Art. 16 are shown by the dotted lines in Fig. 15a.

18. **Internal Equilibrium and Stresses.**—As previously stated (Art. 14), although the structure as a whole may be in equilibrium, it 'is necessary that the internal framework shall have sufficient strength to resist the stresses caused
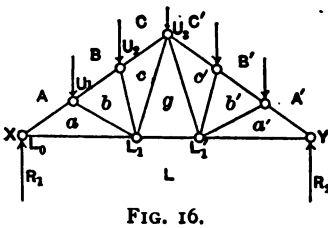
FIG 16c.
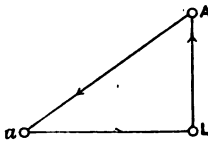
FIG. 16.
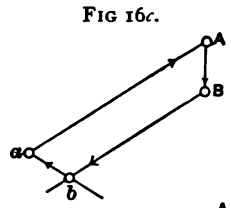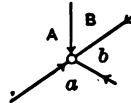
FIG. 16a.   FIG. 16b.   FIG. 16d.
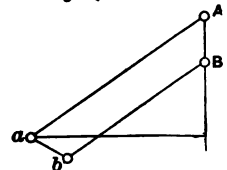
by the outside forces. For example, in Fig. 16, at the point X, R_1 acts upward and the point is kept in equilibrium by the forces transmitted by the pieces Aa and La, parts of the frame. Suppose for the moment that these pieces be replaced by the stresses they transmit, as in Fig. 16a. The angular directions of these forces are known, but their magnitudes and character are as yet unknown. Now, since X is in equilibrium under the action of the forces R_1, Aa, and La, these forces must form a closed figure (Art. 2). Lay off R_1 or LA, as shown in Fig. 16b, and then through A draw a line Aa parallel to Aa, Fig. 16 or 16a, and through

L a line parallel to *La*, Fig. 16 or 16*b*; then *La* and *Aa* are the magnitudes of the two stresses desired. Since in forming the closed figure Fig. 16*b* the forces are laid off in their true directions, one after the other, the directions will be as shown by the arrow-heads. If these arrow-heads be transferred to Fig. 16*a*, it is seen that *Aa* acts toward *X*, and consequently the piece *Aa* in the frame Fig. 16 is in compression, and in like manner the piece *La* is in tension.

Passing to point U_1, Fig. 16, and treating it in a similar manner, it appears that there are four forces acting to produce equilibrium, two of which are known, namely, the outside force *AB* and the inside stress in *Aa*.

Fig. 16*c* shows the closed polygon for finding the magnitudes and directions of the stresses in *ab* and *Bb*.

Since Fig. 16*b* contains some of the lines found in Fig. 16*c*, the two figures can be combined as shown in Fig. 16*d*.

In finding the actual directions of the stresses, the forces acting around any given point must be considered independently in their own closed polygon. Although Fig. 16*d* contains all the lines necessary for the determination of the stresses around *X* and the point U_1, yet the stress diagram for one point is independent of that for the other, for Figs. 16*b* and 16*c* can be drawn to entirely different scales if the diagrams are not combined.

The remaining points of the truss can be treated in the manner outlined above and the stress in each member found. Separate stress diagrams may be constructed for each point, or a combination diagram employed. Since, in case of the inside stresses, the forces meet in a point and there can be no revolution, there remain but two conditions of equilibrium, namely, the sum of the vertical com-

ponents of all the forces must equal zero, and the same condition for the horizontal components. This being the case, if there are *more than two unknowns* among the forces acting at any point being considered, the problem cannot be solved by the above method.

19. Inside Forces Treated as Outside Forces.—Suppose the truss shown in Fig. 17 is cut into two parts along the line *aa*, then the left portion remains in equilibrium as long as the pieces *Dd*, *dg*, and *gL* transmit to the frame the stresses
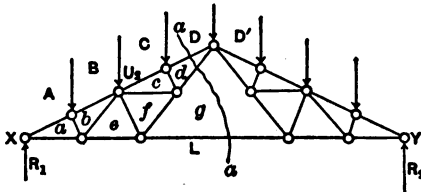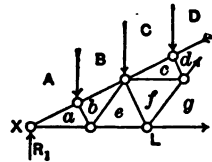
FIG. 17.   FIG. 17*a*.

which actually existed before the cut was made. This condition may be represented by Fig. 17*a*. The stresses *Dd*, *dg*, and *gL* may now be considered as outside forces, and with the other outside forces they keep the structure as a whole in equilibrium, consequently the internal arrangement of the frame will have no influence upon the magnitudes of these forces. Equilibrium would still exist if the frame were of the shape shown in Fig. 17*b* and 17*b'*.

Fig. 17*c* shows the stress diagrams for the two cases shown, and also for the original arrangement of the pieces as shown in Fig. 17.

20. More than Two Unknown Forces Meeting at a Point.—Taking each point in turn, commencing with X, the stress diagrams are readily formed until point U_2 of Fig. 17 is reached. Here *three* unknowns are found, and hence the

problem becomes indeterminate by the usual method. If now the method of Art. 19 is adopted, the bracing changed,

FIG. 17*b*.
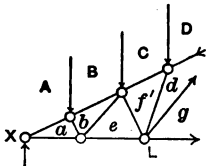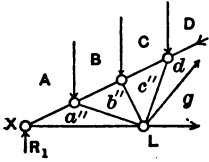
FIG. 17*b'*.
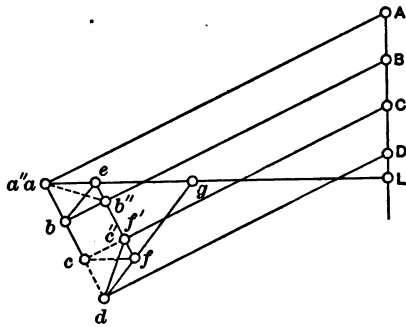
FIG. 17*c*.

and the stresses in Dd, gd, and Lg found, the problem can be solved by working back from these stresses to the point U_2, as shown in Fig. 17*c*.

CHAPTER III.

STRENGTH OF MATERIALS.

21. Wood in Compression: Columns or Struts.—When a piece of wood *over fifteen diameters* in length is subject to compression, the total load or stress required to produce failure depends upon the kind of wood and the ratio of the least dimension to its length. If the strut is circular in cross-section, then its least dimension is the diameter of this section; if rectangular in section, then the least dimension is the smaller side of the rectangular section. The above statements apply to the usual forms of timber which are uniform in cross-section from end to end.

A piece of oak 6″ × 8″ × 120″ long requires about twice the load to produce failure that a similar piece 300″ long requires.

A piece of oak 3″ × 8″ × 120″ requires but about one third the load that a piece 6″ × 8″ × 120″ requires for failure.

The actual ultimate strengths of the various woods used in structures have been determined experimentally and numerous formulas devised to represent these results. One of the later formulas, based upon the formula of A. L. Johnson, C.E., U. S. Department of Agriculture, Division of Forestry, is

$$P = F \times \frac{700 + 15c}{700 + 15c + c^2},$$

where $P =$ the ultimate strength in pounds per square
inch of the cross-section of a strut or column;

$F =$ the ultimate strength per square inch of wood
in short pieces;

$$c = \frac{l}{d} = \frac{\text{length of column in inches}}{\text{least dimension in inches}}.$$

A table of the values of P is given on page 24.

The factor of safety to be used with this table depends
upon the class of structure in which the wood is employed.

The following statements are made in Bulletin No. 12,
U. S. Department of Agriculture, Division of Forestry:

"Since the strength of timber varies very greatly with
the moisture contents (see Bulletin 8 of the Forestry Divi-
sion), the economical designing of such structures will neces-
sitate their being separated into groups according to the
maximum moisture contents in use.

MOISTURE CLASSIFICATION.

"Class A (moisture contents, 18 per cent.)—Structures
freely exposed to the weather, such as railway trestles, un-
covered bridges, etc.

"Class B (moisture contents, 15 per cent.)—Structures
under roof but without side shelter, freely exposed to out-
side air, but protected from rain, such as roof-trusses of
open shops and sheds, covered bridges over streams, etc.

"Class C (moisture contents, 12 per cent.)—Structures
in buildings unheated, but more or less protected from out-
side air, such as roof-trusses or barns, enclosed shops and
sheds, etc.

"Class D (moisture contents, 10 per cent.)—Structures
in buildings at all times protected from the outside air,

ULTIMATE STRENGTH OF COLUMNS. VALUES OF P.

$\dfrac{l}{d}$	ULTIMATE STRENGTH IN POUNDS PER SQUARE INCH.				
	Douglas, Oregon and Washington Yellow Fir or Pine.	Southern, Long-leaf or Georgia Yellow Pine, Canadian (Ottawa) White Pine, Canadian (Ontario) Red Pine.	White Oak.	Northern or Short-leaf Yellow Pine, Red Pine, Norway Pine, Spruce and Eastern Fir, Hemlock, Cypress, Cedar, California Redwood, California Spruce.	White Pine.
	$F = 6000$	$F = 5000$	$F = 4500$	$F = 4000$	$F = 3500$
1	5992	4993	4494	3994	3495
2	5967	4973	4475	3978	3481
3	5928	4940	4446	3952	3458
4	5876	4897	4407	3918	3428
5	5813	4844	4359	3875	3391
6	5739	4782	4304	3826	3347
7	5656	4713	4242	3770	3299
8	5566	4638	4174	3710	3247
9	5469	4558	4102	3646	3190
10	5368	4474	4026	3579	3132
11	5264	4386	3948	3509	3070
12	5156	4297	3867	3438	3008
13	5047	4206	3785	3365	2944
14	4937	4114	3703	3291	2880
15	4826	4022	3620	3217	2815
16	4716	3930	3537	3144	2751
17	4606	3838	3455	3071	2687
18	4498	3748	3373	2998	2624
19	4391	3659	3293	2927	2561
20	4286	3571	3214	2857	2500
21	4183	3486	3137	2788	2440
22	4082	3402	3061	2721	2381
23	3983	3320	2988	2656	2324
24	3888	3240	2916	2592	2268
25	3794	3162	2846	2529	2213
26	3703	3086	2777	2469	2160
27	3615	3013	2711	2410	2109
28	3529	2941	2647	2353	2059
29	3446	2872	2585	2298	2010
30	3366	2805	2524	2244	1963
32	3212	2677	2409	2142	1874
34	3068	2557	2301	2046	1790
36	2934	2445	2200	1956	1711
38	2808	2340	2106	1872	1638
40	2690	2241	2017	1793	1569
42	2579	2149	1934	1719	1505
44	2476	2063	1857	1650	1444
46	2379	1982	1784	1586	1388
48	2288	1907	1716	1525	1335
50	2203	1835	1652	1468	1285

heated in the winter, such as roof-trusses in houses, halls, churches, etc."

Based upon the above classification of structures, the following table has been computed.

Safety factors to be used with the table on page 24 in order to obtain the safe loads for struts.

Class.	Yellow Pine	All Others
Class A	0.20	0.20
" B	0.23	0.22
" C	0.28	0.24
" D	0.31	0.25

All struts considered in this article are assumed to have square ends.

EXAMPLE.—A white-pine column in a church is 12 feet long and 12 inches square; what is the safe load per square inch? $\frac{l}{d} = \frac{12 \times 12}{12} = 12$, and from the table on page 24 $P = 3008$ pounds per. square inch. Churches belong to structures in Class D, and hence the factor of safety is 0.25 and the safe load per square inch $3008 \times 0.25 = 752$ pounds. $752 \times 144 = 108300$ pounds is the total safe load for the column.

22. Metal in Compression: Columns or Struts.—Steel is practically the only metal used in roof-trusses at the present time, and, unless they are very heavy, angles are employed to the exclusion of other rolled shapes. The load required to cripple a steel column depends upon several things, such as the kind of steel, the length, the value of the least radius of gyration for the shape used (this is usually designated by the letter *r*, and

STRENGTH OF STEEL COLUMNS OR STRUTS

For Various Values of $\dfrac{L}{r}$ in which L = Length in Feet and r = Radius of Gyration in Inches.

P = ultimate strength in lbs. per square inch.

FOR SOFT STEEL.

Square Bearing.

$$P = \dfrac{45,000}{1 + \dfrac{(12\ L)^2}{36,000r^2}}.$$

Pin and Square Bearing.

$$P = \dfrac{45,000}{1 + \dfrac{(12\ L)^2}{24,000r^2}}.$$

Pin Bearing.

$$P = \dfrac{45,000}{1 + \dfrac{(12\ L)^2}{18,000r^2}}.$$

To obtain safe unit stress:

For quiescent loads, as in buildings, divide by 4.
For moving loads, as in bridges, divide by 5.

$\dfrac{L}{r}$	ULTIMATE STRENGTH IN POUNDS PER SQUARE INCH.			$\dfrac{L}{r}$	ULTIMATE STRENGTH IN POUNDS PER SQUARE INCH.		
	Square.	Pin and Square.	Pin.		Square.	Pin and Square.	Pin.
3.0	43437	42694	41978	12.0	28553	24142	20911
3.2	43230	42395	41593	12.2	28207	23771	20542
3.4	43011	42081	41190	12.4	27863	23406	20179
3.6	42782	41754	40773	12.6	27522	23046	19823
3.8	42543	41412	40340	12.8	27185	22693	19474
4.0	42294	41058	39893	13.0	26850	22343	19133
4.2	42035	40693	39435	13.2	26524	22005	18797
4.4	41765	40317	38966	13.4	26189	21662	18469
4.6	41488	39930	38485	13.6	25864	21329	18148
4.8	41203	39534	37998	13.8	25543	21002	17833
5.0	40910	39130	37500	14.0	25224	20680	17523
5.2	40608	38807	36997	14.2	24909	20363	17221
5.4	40299	38300	36488	14.4	24598	20052	16925
5.6	39984	37874	35975	14.6	24290	19746	16634
5.8	39663	37443	35457	14.8	23985	19445	16350
6.0	39335	37006	34938	15.0	23684	19148	16071
6.2	39003	36566	34416	15.2	23387	18858	15799
6.4	38665	36122	33894	15.4	23093	18572	15532
6.6	38323	35676	33371	15.6	22803	18288	15270
6.8	37976	35219	32849	15.8	22516	18015	15105
7.0	37616	34776	32328	16.0	22234	17744	14764
7.2	37272	34324	31809	16.2	21954	17478	14518
7.4	36914	33872	31292	16.4	21678	17216	14279
7.6	36554	33419	30779	16.6	21406	16960	14043
7.8	36193	32966	30268	16.8	21137	16708	13812

STRENGTH OF STEEL COLUMNS OR STRUTS—*Continued.*

$\dfrac{L}{r}$	ULTIMATE STRENGTH IN POUNDS PER SQUARE INCH.			$\dfrac{L}{r}$	ULTIMATE STRENGTH IN POUNDS PER SQUARE INCH.		
	Square.	Pin and Square.	Pin.		Square.	Pin and Square.	Pin.
8.0	35828	32514	29762	17.0	20872	16459	13584
8.2	35462	32064	29260	17.2	20611	16216	13366
8.4	35095	31615	28763	17.4	20353	15977	13150
8.6	34727	31169	28272	17.6	20098	15742	12938
8.8	34358	30724	27787	17.8	19847	15512	12731
9.0	33988	30282	27306	18.0	19599	15286	15258
9.2	33611	29844	26832	18.2	19351	15063	12329
9.4	33249	29408	26364	18.4	19114	14845	12135
9.6	32880	28977	25903	18.6	18878	14630	11944
9.8	32511	28549	25448	18.8	18644	14420	11757
10.0	32143	28125	25000	19.0	18418	14218	11579
10.2	31776	27706	24559	19.2	18185	14010	11394
10.4	31411	27290	24125	19.4	17961	13811	11219
10.6	31054	26879	23698	19.6	17740	13616	11048
10.8	30684	26474	23279	19.8	17519	13422	10877
11.0	30324	26072	22866	20.0	17308	13235	10715
11.2	29965	25675	22460	20.2	17096	13050	10553
11.4	29608	25285	22063	20.4	16888	12868	10434
11.6	29247	24899	21671	20.6	16682	12690	10249
11.8	28903	24517	21288	20.8	16480	12515	10087

the values are given in the manufacturers' pocket-books), the manner in which the ends are held, etc.

If a column has its end sections so fixed that they remain parallel, the column is said to be *square-ended.* If both ends are held in place by pins which are parallel, the column is said to be *pin-ended.* A column may have one square end and one pin end.

The above table contains the ultimate strength per square inch of SOFT-STEEL columns or struts.

To obtain the safe unit stress for MEDIUM STEEL:

For quiescent loads, as in buildings, divide by 3.6

For moving loads, as in bridges, divide by 4.5

EXAMPLE.—What load will cripple a square-ended column of soft steel made of one standard $6'' \times 6'' \times \frac{1}{2}''$ angle if the length of the strut is 10 feet?

From any of the pocket-books or the table at end of book the value of r is 1.18 inches, then $\dfrac{L}{r} = \dfrac{10}{1.18} = 8.5$, and from the table on page 27 $P = 34800$ pounds per square inch. The area of the angle is 5.75 square inches, hence the crippling load is $5.75 \times 34800 = 200100$ pounds. The safe load in a roof-truss is $200100 \div 4 = 50025$ pounds. If medium steel had been used, the safe load becomes $200100 \div 3.6 = 55600$ pounds.

23. End Bearing of Wood.—When a stress is transmitted to the ends of the fibers there must be a sufficient number to carry the load without too much compression or bending over. To illustrate, let a load P be transmitted through a metal plate to the end of a wooden column, then the area $b \times d$ must be such that no crushing takes place.

In Fig. 18b the load is transmitted to the wooden strut by means of a casting and round pin. The area of the fibers taking the entire load P is $b \times d$.

The following table gives the safe end-bearing values for various woods:

1100	1200	1400	1600 .	Lbs. per Sq. In.
White Pine	Northern or Short-leaf Yellow Pine, Red Pine, Norway Pine, Spruce, Cypress, Cedar	White Oak	Southern Long-leaf Pine or Georgia Yellow Pine, Douglas, Oregon and Yellow Fir	The values in this table have a factor of safety of 5

EXAMPLE.—In Fig 18 let $b = 12$ inches, $d = 4$ inches, and suppose the wood to be white oak; what is the safe load P? $P = 4 \times 12 \times 1400 = 67200$ pounds.
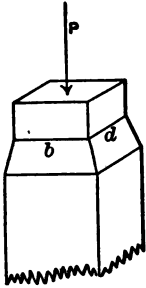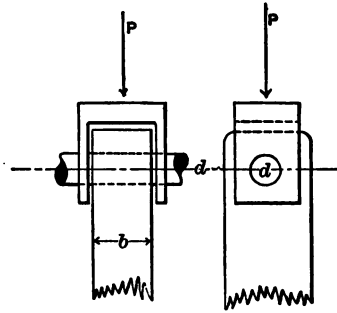
FIG. 18.   FIG. 18*b*.

24. Bearing of Steel.—All that has been said concerning the end bearing of wood applies to steel whenever rivets or pins are harder than the steel through which they pass. In roof-trusses the rivets and pins are seldom harder than the angles or plates they connect. The bearing value used in designing should be that for the softer material.

For soft or soft-medium steel the *safe* bearing value may be taken as 20000 pounds per square inch.

TABLE OF SAFE BEARING VALUES.

Diameter of Rivet.	Area in Sq. Inches.	BEARING VALUE FOR DIFFERENT THICKNESSES OF PLATE IN INCHES AT 20,000 POUNDS PER SQUARE INCH.					
		$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$
$\frac{3}{8}$	.1105	1875	2344	2813			
$\frac{1}{2}$	.1964	2500	3125	3750	4375	5000	
$\frac{5}{8}$	.3068	3125	3906	4688	5469	6250	7031
$\frac{3}{4}$	.4418	3750	4688	5625	6563	7500	8438
$\frac{7}{8}$	.6013	4375	5469	6563	7656	8750	9844
1	.7854	5000	6250	7500	8750	10000	11250

TABLE OF SAFE BEARING VALUES—*Continued.*

Diameter of Rivet.	Area in Sq. Ins.	BEARING VALUE FOR DIFFERENT THICKNESSES OF PLATE IN INCHES AT 20,000 POUNDS PER SQUARE INCH.						
		⅜	7⁄16	½	9⁄16	⅝	11⁄16	1
⅜	.1105							
½	.1964							
⅝	.3068	7813						
¾	.4418	9375	10313	11250				
⅞	.6013	10938	12031	13125	14219	15313	16406	
1	.7854	12500	13750	15000	16250	17500	18750	20000

25. Bearing Across the Fibers of Wood.—If a load P, Fig. 19, be transmitted through a wooden corbel to a column, the area $b \times d$, bearing directly upon the support, must be sufficient to resist crushing. This is a point very often overlooked in construction. In Fig. 19a the same
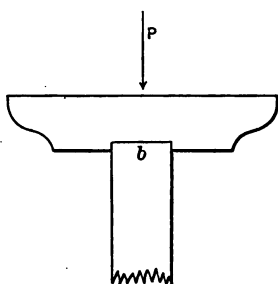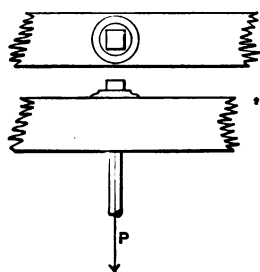
FIG. 19.   FIG 19a.

TABLE OF SAFE BEARING VALUES.

150	200	250	300	350	500	Lbs. per Sq. In.
Hemlock	White Pine, Red Pine, Norway Pine, Spruce, Eastern Fir, Cypress, Cedar, California Redwood	Northern or Short-leaf Yellow Pine, Chestnut	Douglas Fir, Oregon Fir, Yellow Fir	Southern Long-leaf orGeorgia Yellow Pine	White Oak	The values in this table have a factor of safety of 4

conditions obtain. The washer must be of such a size that the area bearing upon the wood shall properly distribute the stress transmitted by the rod. A table of safe bearing values is given on page 30.

26. Bearing Across the Fibers of Steel. See Art. 24.

27. Longitudinal Shear of Wood.—In Fig. 20 let the piece *A* push against the notch in *B*, then the tendency is to push the portion above *ba* along the plane *ba*, or to shear lengthwise a surface *b* in length and *t* in width.

A similar condition exists in Fig. 20*a*. The splice may fail by the shearing along the grain the two surfaces *abc* and *a'b'c'*. A table of safe longitudinal shearing values is given on page 32.
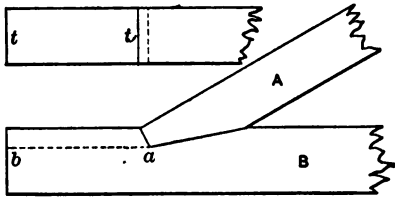
FIG. 20.
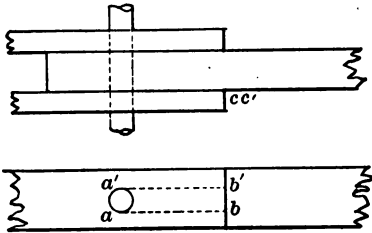
FIG. 20*a*.

28. Longitudinal Shear of Steel.—For all structures considered in this book the longitudinal shear of steel is fully provided for by the practical rules governing the spacing of rivets, etc. See Table III.

TABLE OF SAFE LONGITUDINAL SHEARING VALUES.

100	150	200	Pounds per Sq. In.
White Pine, Northern or Short-leaf Yellow Pine, Canadian White Pine, Canadian Red Pine, Spruce, Eastern Fir, Hemlock, California Redwood	Southern Long-leaf or Georgia Yellow Pine, Red Fir, Chestnut	White Oak	The values in this table have a factor of safety of 4

29. Transverse Strength of Wood.—When a beam supported at the ends is loaded with concentrated loads, as shown in Fig. 21, the maximum moment is readily found by means of the equilibrium polygon. Let this moment be called M, then for rectangular beams
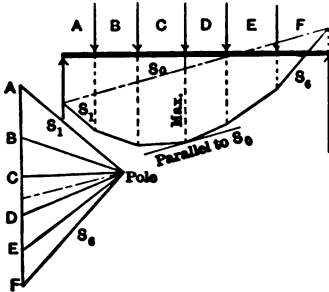
FIG. 21.

$$M = \tfrac{1}{6}Rbd^2,$$

where M = the maximum moment in inch-pounds;

b = the breadth of the beam in inches;

d = the depth of the beam in inches;

R = the allowable or safe stress per square inch in the extreme fiber.

If M is given in foot-pounds, then the second member of the above equation becomes $\tfrac{1}{72}Rbd^2$.

For a uniformly distributed load

$$M = \tfrac{1}{8}wl^2 = \tfrac{1}{6}Rbd^2,$$

where w = the load per linear inch of span;

l = the span in inches.

EXAMPLE.—An oak beam 6 inches deep has a span of 10 feet and carries a load of 100 pounds per linear foot. What must be the breadth of the beam to safely carry the load?

$$M = \tfrac{1}{8}wl^2 = \tfrac{1}{8} \times 100 \times 10 \times 10 = 1250 \text{ ft.-lbs. or } 15000$$
in.-lbs.

or
$$M = \tfrac{1}{6}Rbd^2 = 15000 = \tfrac{1}{6} \times 1000 \times b \times 6 \times 6,$$

$$b = \frac{15000}{6000} = 2\tfrac{1}{2} \text{ inches.}$$

Hence a $2\tfrac{1}{2}'' \times 6''$ white-oak beam will safely carry the load; but the weight of the beam has been neglected, and consequently the breadth must be increased to, say, 3 inches. A second calculation should now be made with the weight of the beam included.

TABLE OF SAFE VALUES OF R FOR WOOD.

600	700	750	800
Hemlock	White Pine, Norway Pine, Spruce, Eastern Fir	California Red-wood	Red Fir, Red Pine, Cypress, Cedar, Chestnut, California Spruce

1000	1100	1200
White Oak, Northern or Short-leaf Yellow Pine	Washington Fir or Pine (Red Fir)	Southern Long-leaf or Georgia Yellow Pine

The above values are pounds per square inch. Factor of safety 6.

30. Transverse Strength of Steel Beams.—In the case of steel beams

$$M = \frac{RI}{v} = RS,$$

where M = the maximum moment in inch-pounds;

I = the moment of inertia (given in the manufac-
turers' pocket-books);

v = the distance of the outermost fiber from the
neutral axis;

R = the safe stress in pounds per square inch in
the outermost fiber;

$S = \dfrac{I}{v}$ is given in the manufacturers' pocket-books
for each shape rolled for the conditions
usually obtaining in practice.

The safe value of R for soft steel may be taken as 16000 pounds.

EXAMPLE.—Suppose the oak beam in Article 29 is re-
placed by a steel channel. What must be its size and weight?

$$M = 15000 = RS = 16000S; \therefore S = 0.94$$

From any of the manufacturers' pocket-books, a 3-inch channel weighing 4 pounds per linear foot has $S = 1.1$. The moment due to the weight of the channel is $\frac{1}{8}wl^2 = \frac{1}{8} \times 4 \times 10 \times 10 = 50$ ft.-lbs. or 600 in.-lbs.; hence the total moment is 15600 inch-pounds, and $S = \dfrac{15600}{16000} = 0.98$, which is greater than the value required. This being the case, a 3-inch channel weighing 4 pounds per foot will be safe. (See Tables at end of book.)

31. Special Case of the Bending Strength of Metal Pins.

—Where pins are used to connect several pieces, as in Fig.
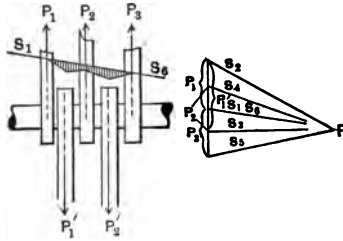
FIG. 22.

22, the moments of the outside forces can be determined in the usual way.

This moment $M = \dfrac{RI}{v} = R(0.098d^3)$,

where d = the diameter of the pin in inches;

R = the safe stress in the outer fiber in pounds per square inch.

The table on page 36 gives the safe values of M for various sizes of bolts or pins. For wrought iron use $R = 15000$, and for steel use $R = 25000$.

32. Shearing Across the Grain of Bolts, Rivets, and Pins.

—For wrought-iron bolts use 7500 pounds per square inch, and for steel 10000 pounds. The safe shearing values of rivets and bolts are given on page 36.

MAXIMUM BENDING MOMENTS ON PINS WITH EXTREME FIBER STRESSES,

VARYING FROM 15000 TO 25000 POUNDS PER SQUARE INCH.

Diameter of Pin in Inches.	Area of Pin in Square Inches.	MOMENTS IN INCH-POUNDS FOR FIBRE STRESSES OF				
		15000 Lbs. per Sq. In.	18000 Lbs. per Sq. In.	20000 Lbs. per Sq. In.	22500 Lbs. per Sq. In.	25000 Lbs. per Sq. In.
1	.785	1470	1770	1960	2210	2450
1⅛	.994	2100	2520	2800	3150	3490
1¼	1.227	2900	3450	3830	4310	4790
1⅜	1.485	3830	4590	5100	5740	6380
1½	1.767	4970	5960	6630	7460	8280
1⅝	2.074	6320	7580	8430	9480	10530
1¾	2.405	7890	9470	10520	11840	13150
1⅞	2.761	9710	11650	12940	14560	16180
2	3.142	11780	14140	15710	17670	19630
2⅛	3.547	14130	16960	18840	21200	23550
2¼	3.976	16770	20130	22370	25160	27960
2⅜	4.430	19730	23670	26300	29590	32880
2½	4.909	23010	27610	30680	34510	38350
2⅝	5.412	26640	31960	35520	39960	44400
2¾	5.940	30630	36750	40830	45940	51040
2⅞	6.492	34990	41990	46660	52490	58320
3	7.069	39730	47680	52970	59600	66220
3⅛	7.670	44940	55930	59920	67410	74900
3¼	8.296	50550	60660	67400	75830	84250
3⅜	8.946	56610	67940	75480	84920	94350
3½	9.621	63140	75770	84180	94710	105230
3⅝	10.321	70150	84180	93530	105220	166910
3¾	11.045	77660	93190	103540	116490	129430
3⅞	11.793	85690	102820	114250	128530	142810
4	12.566	94250	113100	125660	141370	157080

SAFE SHEARING VALUES OF RIVETS AND BOLTS.

Diam. of Rivet.	Area in Square Inches.	Single Shear at 7500 lbs.	Double Shear at 15000 lbs.	Single Shear at 10000 lbs.	Double Shear at 20000 lbs.
⅜	.1105	828	1657	1105	2209
½	.1964	1473	2945	1964	3927
⅝	.3068	2301	4602	3068	6136
¾	.4418	3313	6627	4418	8836
⅞	.6013	4510	9020	6013	12026
1	.7854	5891	11781	7854	15708

33. Shearing Across the Grain of Wood.

SAFE TRANSVERSE SHEARING VALUES.

400	500	600	Lbs. per Sq. In.
Cedar, Chestnut	White Pine	Hemlock	Factor of safety 4

750	1000	1250	Lbs. per Sq. In.
Spruce, Eastern Fir	White Oak, North-ern or Short-leaf Yellow Pine	Southern Long-leaf or Georgia Yellow Pine	Factor of safety 4

34. Wood in Direct Tension.

SAFE TENSION VALUES.

600	700	800	Lbs. per Square Inch.
Hemlock, Cypress	White Pine, Cali-fornia Redwood	Spruce, Eastern Fir, Cedar	Factor of safety 10

900	1000	1200	Lbs. per Square Inch.
Northern or Short-leaf Yellow Pine, Red Pine, Chestnut	White Oak, Washington Fir or Pine, Cana-dian White Pine and Red Pine	Southern Long-leaf or Georgia Pine, Douglas Fir, Oregon Fir, Yel-low Fir	Factor of safety 10.

35. Steel and Wrought Iron in Direct Tension.—For wrought iron use 12000 pounds per square inch, for steel use 16000 pounds per square inch.

CHAPTER IV.

ROOF-TRUSSES AND THEIR DESIGN.

36. Preliminary Remarks.—Primarily the function of a roof-truss is to support a covering over a large floor-space which it is desirable to keep free of obstructions in the shape of permanent columns, partitions, etc. Train-sheds, power-houses, armories, large mill buildings, etc., are examples of the class of buildings in which roof-trusses are commonly employed.

The trusses span from side wall to side wall and are placed at intervals, depending to some extent upon the architectural arrangement of openings in the walls and upon the magnitude of the span. The top members of the trusses are connected by members called purlins, running usually at right angles to the planes of the trusses. The purlins support pieces called rafters, which run parallel to the trusses, and these carry the roof covering and any other loading, such as snow and the effect of wind.

The trusses, purlins, and rafters may be of wood, steel, or a combination of the two materials.

37. Roof Covering.—This may be of various materials or their combinations, such as wood, slate, tin, copper, clay tiles, corrugated iron, flat iron, gravel and tar, etc.

The weights given for roof coverings are usually per *square*, which is 100 square feet.

38

Tables I and II give the weights of various roof coverings.

38. Wind Loads.—The actual effect of the wind blowing against inclined surfaces is not very well known. The formulas in common use are given below:

Let θ = angle of surface of roof with direction of wind;

F = force of wind in pounds per square foot;

A = pressure normal to roof, $= F \sin a^{1.84 \cos \theta - 1}$;

B = pressure perpendicular to direction of the wind

$= F \cot \theta \sin \theta^{1.84 \cos \theta}$;

C = pressure parallel to the direction of the wind

$= F \sin \theta^{1.84 \cos \theta}$.

(Carnegie.)

Angle θ	5°	10°	20°	30°	40°	50°	60°	70°	80°	90°
$A = F \times$	0.125	0.24	0.45	0.66	0.83	0.95	1.00	1.02	1.01	1.00
$B = F \times$	0.122	0.24	0.42	0.57	0.64	0.61	0.50	0.35	0.17	0.00
$C = F \times$	0.010	0.04	0.15	0.33	0.53	0.73	0.85	0.96	0.99	1.00

39. Pitch of Roof.—The ratio of the rise to the span is
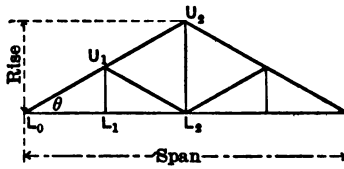
Fig. 23

called the pitch, Fig. 23 The following table gives the angles of roofs as commonly constructed:

Pitch.	Angle θ.		Sin θ.	Cos θ.	Tan θ.	Sec θ.
1/2	45°	0′	0.70711	0.70711	1.00000	1.41421
1/3	33°	41′	0.55460	0.83212	0.66650	1.20176
$\frac{1}{2\sqrt{3}}$	30°	0′	0.50000	0.86603	0.57735	1.15470
1/4	26°	34′	0.44724	0.89441	0.50004	1.11805
1/5	21°	48′	0.37137	0.92849	0.39997	1.07702
1/6	18°	26′	0.31620	0.94869	0.33330	1.05408

40. Transmission of Loads to Roof-trusses.—Fig. 24 shows a common arrangement of trusses, purlins, and rafters, so that all loads are finally concentrated at the apexes B, C,
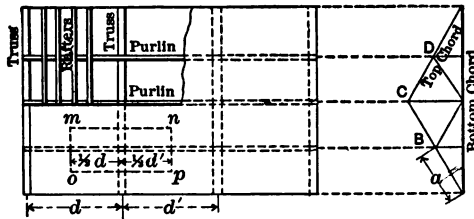
FIG. 24.

D, etc., of the truss. Then the total weight of covering, rafters, and purlins included by the dotted lines mn, np, po, and om will be concentrated at the vertex B. The total wind load at the vertex B will be equal to the normal pressure of the wind upon the area $mnop$.

41. Sizes of Timber.—Although any size of timber can be obtained on a special order, yet it is more economical so to design structures that only commercial sizes will be required.

Commercial timber is commonly cut in even inches in cross-section and even feet in length. For example, $2'' \times 12''$, $4'' \times 4''$, $4'' \times 12''$, $6'' \times 8''$ are commercial sizes, and these can be obtained in lengths of 8′. 10′, 12′, 14′,

etc. Certain sizes are cheaper when in particular lengths; extra-long pieces are usually expensive. See Timber Table XV.

42. Steel Shapes.—Only such shapes should be employed as are marked *standard* in the manufacturers' pocket-books. These are readily obtained and cost less per pound than the "special" shapes.

Ordinarily all members of steel roof-trusses are composed of two angles placed back to back, sufficient space being left between them to admit a plate for making connections at the joints. See Tables IX–XII.

43. Round Rods.—In wooden trusses the vertical tension members, and diagonals when in tension, are made of round rods. These rods should be upset at the ends so that when threads are cut for the nuts, the diameter of the rod at the root of the thread is a little greater than the diameter of the body of the rod. It is common practice to buy stub ends—that is, short pieces upset—and weld these to the rods. Unless an extra-good blacksmith does the work the upsets should be made upon the rod used, without welds of any kind. Very long rods should not be spliced by welding, but connected with sleeve-nuts or turnbuckles.

Upset ends, turnbuckles, and sleeve-nuts are manufactured in standard sizes and can be purchased in the open market. See Table VII.

44. Bolts.—The sizes of bolts commonly used in wooden roof-trusses are $\frac{3}{4}''$ and $\frac{7}{8}''$ in diameter. Larger sizes are sometimes more economical if readily obtained. $\frac{3}{4}''$ and $\frac{7}{8}''$ bolts can be purchased almost anywhere. Care should be taken to have as many bolts as possible of the same size,

as the use of several sizes in the same structure usually causes trouble or delay.   See Tables V and VI.

45. Rivets.—The rivets in steel structures should be of uniform diameter if possible.   The practical sizes for different shapes are given in the manufacturers' pocket-books.   See Tables III, IV, and V.

46. Local Conditions.—In making a design local markets should be considered.   If material can be purchased from local dealers, although not of the sizes desired, it will often happen that even when a greater amount of the local material is used than required by the design, the total cost will be less than if special material, less in quantity, had been purchased elsewhere.   This is especially true for small structures of wood.

CHAPTER V.

DESIGN OF A WOODEN ROOF-TRUSS.

47. Data.

Wind load = 40 pounds per square foot of vertical projection of roof.

Snow load = 20 pounds per square foot of roof.

Covering = slate 14″ long, ¼″ thick = 9.2 pounds per square foot of roof.

Sheathing = long-leaf Southern pine, 1⅛″ thick = 4.22 pounds per square foot of roof.

Rafters = long-leaf Southern pine, 2″ thick.

Purlins = long-leaf Southern pine.

Truss = long-leaf Southern pine, for all members except verticals in tension, which will be of soft steel.

Distance c. to c. of trusses = 10 feet.

Pitch of roof = ⅓.

Form of truss as shown in Fig. 25.

Span 60′   Rise 20′
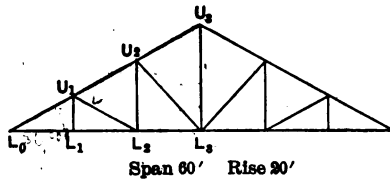
FIG. 25.

48. Allowable Stresses per Square Inch.

SOUTHERN LONG-LEAF PINE.

Tension with the grain................. Art. 34,   1200 lbs.
End bearing........................ Art. 23,   1600 lbs.
End bearing against pins....................   2500 lbs.
Compression across the grain....... Art. 25,   350 lbs.
Transverse stress—extreme fiber stress, Art. 29,   1200 lbs.
Shearing with the grain............. Art. 27,   150 lbs.
Shearing across the grain........... Art. 33,   1250 lbs.
Columns and Struts.   Values given in Art. 21.

STEEL.

Tension with the grain............. Art. 35,   16000 lbs.
Bearing for rivets and bolts........ Art. 24,   20000 lbs.
Transverse stress—extreme fiber stress, Art. 30,   16000 lbs.
Shearing across the grain.......... Art. 32,   10000 lbs.
Extreme fiber stress in bending (pins), Art. 31,   25000 lbs.

49. **Rafters.**—The length of each rafter c. to c. of purlins is $10 \times \sec \theta = 10 \times 1.2 = 12$ feet, and hence the area *mnop*, Fig. 24, is $12 \times 10 = 120$ square feet.

VERTICAL LOADS.

Snow $= 20.00 \times 120 = 2400$ lbs.
Slate $= 9.20 \times 120 = 1104$ lbs.
Sheathing $= \underline{4.22 \times 120 = 506}$ lbs.
                $33.42 \times 120 = 4010$ lbs.

The normal component of this load is $4010 \times \cos \theta$, or $4010 \times 0.832 = 3336$ pounds.

The normal component of the wind is (Art. 38) about $40 \times 0.70 = 28$ lbs. per square foot, and the total, $28 \times 120 = 3360$ lbs.

The total normal load supported by the rafters, exclusive of their own weight, $= 3336 + 3360 = 6696$ lbs. $6696 \div 12 = 558$ lbs. per linear foot of span of the rafters.

Since the thickness of the rafters has been taken as $2''$, either the number of the rafters or their depth must be assumed.

Assuming the depth as $8''$, the load per linear foot which each rafter can safely carry is (Art. 29)

$$\tfrac{1}{8}wl^2 = \tfrac{1}{6}Rbd^2.$$

$$12 \times \tfrac{1}{8}w \times 12 \times 12 = \tfrac{1}{6} \times 1200 \times 2 \times 8 \times 8,$$

$$\text{or} \quad w = \frac{25600}{216} = 118 \text{ lbs.}$$

$558 \div 118 = 5 =$ the number of $2'' \times 8''$ rafters required.

To allow for the weight of the rafters and the component of the vertical load which acts along the rafter, six rafters will be used. If a rafter is placed immediately over each truss, the spacing of the rafters will be $10 \times 12 \div 6 = 20$ inches c. to c.

The weight of the rafters is $[2 \times \tfrac{8}{8} \times 12]6 \times 3.75 = 360$ lbs.

50. Purlins.—The total load normal to the roof carried by one purlin, exclusive of its own weight, is $6696 + 360 \times 0.832 = 6996$ lbs. This is concentrated in loads of $6996 \div 6 = 1166$ lbs., spaced $20''$ apart, as shown in Fig. 26.

The moment can be determined graphically (Art. 13), or algebraically, as follows:

Moment at center = $2915 \times 60 - 1166 \times 40 - 1166 \times 20 = 104940$ inch-pounds. Assume the purlin to be 10″ deep, then for each inch in thickness the maximum moment
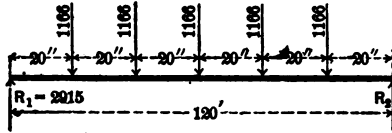
FIG. 26.

which it can safely resist is $\frac{1}{6}Rbd^2 = \frac{1}{6} \times 1200 \times 1 \times 10 \times 10 = 20000$ inch-pounds.

$\frac{104940}{20000} = 5.2″ =$ the required width of purlin. To allow for the weight of the purlin and any force acting along the rafters, the next commercial size will be used, or 6″ × 10″.

The weight of the purlin is $5 \times 10 \times 3.75 = 188$ lbs.

51. Loads at Truss Apexes.—Exclusive of the weight of the truss the vertical load at each apex, U_1, U_2, U_3, U_4, and U_5, Fig. 25, is

Snow, slate, sheathing.............	Art. 49,	4010 lbs.
Rafters...........................	Art. 49,	360 lbs.
Purlins...........................	Art. 50,	188 lbs.
		4558 lbs.

The weight in pounds of the truss may be found from the formula $W = \frac{3}{4}dL(1 + \frac{1}{10}L)$, where d is the distance in feet c. to c. of trusses, and L the span in feet. Substituting for d and L,

$$W = \frac{3}{4} \times 10 \times 60(1 + \frac{1}{10} \times 60) = 3150 \text{ lbs.}$$

The full apex load is $\frac{3150}{6} = 525$ lbs., and hence the

total vertical load at each apex U_1–U_5, inclusive, is 4558 + 525 = 5083 lbs. In case the top chords of the end trusses are cross-braced together to provide for wind pressure, etc., this load would be increased about 75 or 100 lbs.

For convenience, and since the roof assumed will require light trusses, the apex loads will be increased to 6000 lbs. In an actual case it would be economy to place the trusses about 15 feet c. to c.

The load at the supports is $\frac{9000}{3}$ = 3000 lbs.

Wind.—The wind load for apexes U_1 and U_2 is 3360 lbs. (Art. 49), and at apexes L_0 and U_4 the load is $\frac{3360}{2} = 1680$ lbs. For the determination of stresses let the wind apex load be taken as 3400 lbs., and the half load as 1700 lbs.

In passing, attention may be called to the fact that the weight of the truss is less than 10 per cent. of the load it has to support exclusive of the wind; hence a slight error in assuming the truss weight will not materially affect the stresses in the several members of the truss.

52. Stresses in Truss Members.—Following the principles explained in Chapter II, the stress in each piece is readily determined, as indicated on Plate I.

Having found the stresses due to the vertical loads, the wind loads when the wind blows from the left and when it blows from the right, these stresses must be combined in the manner which will produce the greatest stress in the various members. The wind is assumed to blow but from one direction at the same time; that is, the stress caused by the wind from the right cannot be combined with the stress due to the wind from the left.

For convenience of reference the stresses are tabulated here.
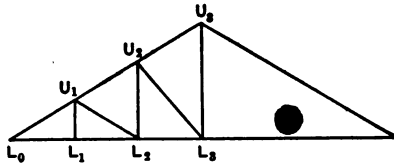
STRESSES.

	Vertical Loads.	Wind Left.	Wind Right.	Maximum Stresses.
L_0U_1	+27200	+7300	+5600	+34500
U_1U_2	+21700	+5800	+5600	+27500
U_2U_3	+16300	+4400	+5600	+20700
L_0L_1	−22600	−8700	−2600	−31300
L_1L_2	−22600	−8700	−2600	−31300
L_2L_3	−18100	−5600	−2600	−23700
U_1L_1	o	o	o	o
U_2L_2	−3000	−2000	o	−5000
U_3L_3	−12000	−4100 +	−4100	−16100
U_1L_2	+5400	+3700	o	9100
U_2L_3	+7600	+5100	o	12700

+ signifies compression.

53. Sizes of Compression Members of Wood.

Piece L_0U_1.   Stress = + 34500.

This piece has the greatest stress of all the upper chords. Since the apex U_1 is held in position vertically by the truss members, and horizontally by the purlins, the *unsupported* length of L_0U_1 as a column is 12 feet.

To determine the size a least dimension must be assumed and a trial calculation made. This will be better explained by numerical calculations.

Let the least dimension be assumed as 4″, then $\dfrac{l}{d} =$

$\dfrac{12 \times 12}{4} = 36$, and from Art. 21, $P = 2445$ lbs. per square

inch. The safe or allowable value is $\dfrac{P}{4} = \dfrac{2445}{4} = 610$ lbs. per square inch. Hence $34500 \div 610 = 55.5 =$ number of square inches required. If one dimension is $4''$, the other must be $14''$, or a piece $4'' \times 14'' = 56$ square inches, $12'$ long, will safely carry the stress 34500 lbs. This shape, however, is not economical. The more nearly square the strut is the more economical will it be if rectangular shapes are used, provided the size is a commercial size.

Let $d = 6''$, then $\dfrac{l}{d} = \dfrac{12 \times 12}{6} = 24$ and $P = 3240$, making the safe load per square inch $3240 \div 4 = 810$ lbs. $\frac{34500}{810} = 42.6 =$ number of square inches in section required. A piece $6'' \times 6''$ is too small, hence a $6'' \times 8''$ timber must be used. Note that a piece $4'' \times 14'' = 56$ square inches, and a piece $6'' \times 8'' = 48$ square inches, a gain of 8 square inches.

NOTE.—The sectional dimensions of commercial lumber are nominal, and all are slightly smaller when measurements are taken of the actual timber. This, as well as shrinkage, makes it impracticable to select timber which may nominally have the required section. The next larger size should always be taken.

$$\text{Pieces} \qquad U_1U_2 \quad \text{and} \qquad U_2U_3.$$
$$\text{Stresses} \quad + 27500 \quad \text{and} \quad + 20700.$$

Letting $d = 6''$, $27500 \div 810 = 34$ square inches required. Now $6'' \times 6'' = 36$ square inches is the minimum number which can be used when $d = 6''$; hence a $6'' \times 6''$ piece can be used. However, a change in size requires a splice, and usually the cost of bolts and labor for the splice exceeds the cost of the extra material used in continuing

the piece L_0U_1 past the point U_2. For this reason, and because splices are always undesirable, the top chords of roof-trusses are made uniform in size for the maximum lengths of commercial timber, and, excepting in heavy trusses, the size of the piece L_0U_1 is retained throughout the top chord, even when one splice is necessary.

To illustrate the method of procedure when the size is changed, suppose U_2U_3 is of a different size from U_1U_2. To keep one dimension uniform the piece must be either 6″ or 8″ on one side. Try the least d as 4″, then $\dfrac{l}{d} = 36$, and

$$\frac{P}{4} = \frac{2445}{4} = 610 \text{ lbs.} \quad 20700 \div 610 = 34 \text{ square inches}$$

required. $34 \div 4$ indicates that a 4″ × 10″ piece is necessary; but the other dimension was taken as 6″, therefore, in order to retain this dimension a 6″ × 6″ piece must be used.

Since U_1U_2 can also be 6″ × 6″, L_0U_1 can be 6″ × 8″, and the remainder of the rafter 6″ × 6″ in case a splice is necessary. 6″ × 8″ will be used throughout.

Piece U_1L_2. Stress $= + 9100$.

The unsupported length of this piece is 12 feet. Try least $d = 4″$, then $\dfrac{P}{4} = 610$ and $9100 \div 610 = 15 =$ the number of square inches required; hence a piece 4″ × 4″ $= 16$ square inches can be used. For appearance and stiffness a piece 4″ × 6″ will be used.

Piece U_2L_3. Stress $= + 12700$.

The unsupported length $= 10 \times 1.6667 = 16.67$ feet,

$$\frac{l}{d} = \frac{16.67 \times 12}{4} = 50.01, \quad \frac{P}{4} = \frac{1835}{4} = 460 \text{ lbs.}$$

12700 ÷ 460 = 28 = number of square inches required, or a piece 4″ × 8″ must be used if $d = 4″$.

Try $d = 6″$, then $\dfrac{l}{d} = 33 +$, $\dfrac{P}{4} = \dfrac{3100}{4} = 775$.

12700 ÷ 775 = 17 square inches required. The smallest size where $d = 6″$ is $6″ × 6″ = 36$ square inches.

In this case a 4″ × 8″ is more economical in material by 4 square inches of section, but the extra material used in a 6″ × 6″ piece is more than balanced by its greater strength and *stiffness*.

54. Sizes of Tension Members of Wood.

Pieces L_0L_1 and L_1L_2. Stress = − 31300.

From Art. 34 the allowable stress per square inch for Southern long-leaf pine is 1200 lbs.

31300 ÷ 1200 = 26.1 = the net number of square inches required. In order to connect the various pieces at the apexes, considerable cutting must be done for notches, bolts, etc., and where the fibres are cut off their usefulness to carry tensile stresses is destroyed. Practice indicates that in careful designing the net section must be increased by about ⅝, or in this case the area required is 23 + 18 = 41 square inches, therefore, a piece 6″ × 8″ = 48 square inches must be used.

Piece L_2L_3. Stress = − 23700.

In a similar manner this member can be proportioned, but since splices in tension members are very undesirable, owing to the large amount of material and labor required in making them, the best practice makes the number a

minimum consistent with the market lengths of timber,
and, consequently, in all but very large spans the bottom
chord is made uniform in size from end to end.

55. Sizes of Steel Tension Members.

Piece U_1L_1. Stress = 0.

Although there is no stress in U_1L_1, yet, in order that
the bottom chord may be supported at L_1, a round rod $\frac{3}{4}''$
in diameter will be used.

Piece U_2L_2. Stress = -5000.

The number of square inches required is (Art. 35), 5000 ÷
16000 = 0.31 square inches. A round rod $\frac{5}{4}$ inch in diam·
eter is required, exclusive of the material cut away by the
threads at the ends. The area at the root of the threads
of a $\frac{7}{8}''$ round rod is 0.42 square inches, hence a $\frac{7}{8}''$ round rod
will be used. (Table VII.)

Piece U_3L_3. Stress = -16100.

16100 ÷ 16000 = 1.06 square inches. A $1\frac{1}{4}''$ round rod
has area of 1.227 square inches. This rod upset (Table
VII) to $1\frac{3}{8}''$ at the ends will be used.

If the rod is not upset a diameter of $1\frac{3}{8}''$ must be used,
having an area of 1.057 square inches at the root of the
threads.

Note that the above rods have commercial sizes.

56. Design of Joint L_0—With $\frac{7}{8}''$ Bolts.—A common
form of joint at L_0 is shown in Fig. 26. The top chord rests
in a notch db in the bottom chord, and, usually, altogether
too much reliance is put in the strength of this detail. The
notch becomes useless when the fibres fail along db, or when

the bottom chord shears along ab. The distance ab is quite variable and depends upon the arrangement of rafters, gutters, cornice, etc. Let about $12''$ be assumed in this case, then it will safely resist a longitudinal shearing force of $12 \times 6 \times 150 = 10800$ lbs. (Art. 27). The area of the end fibers due to the notch db equals $1\frac{1}{2} \times 6 = 9$ square inches, if $dc = 1\frac{1}{2}''$. This will safely resist $1\frac{1}{2} \times 6 \times 1600 = 14400$
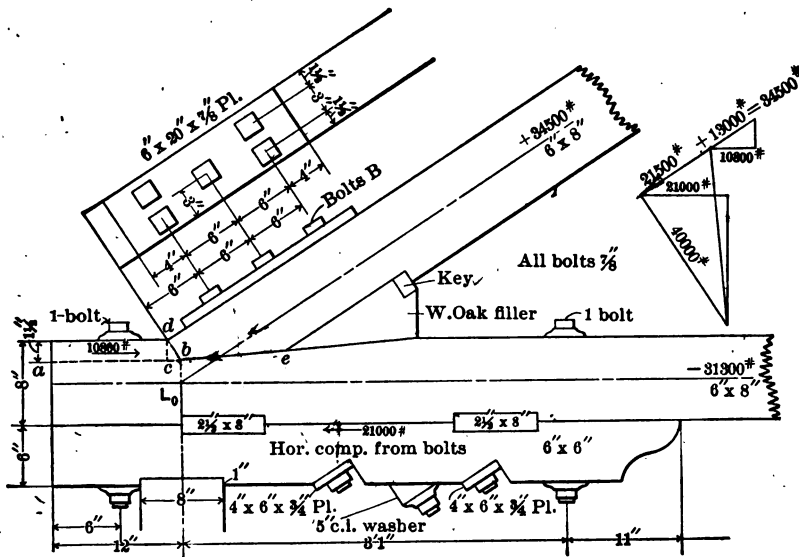
FIG. 26.

lbs. (Art. 23); but the wood would shear along ab before this force could be realized, hence the value of the notch is but 10800 lbs., leaving $34500 - (10800)1.2 = 21500$ lbs. to be held in some other manner, in this case by $\frac{7}{8}''$ bolts.

To save cutting the bottom chord for washers, and also to increase the bearing upon the supports of the truss it is customary to use, a corbel or bolster, as shown in Fig. 26.

Let a single $\frac{7}{8}''$ bolt be placed $6''$ from the end of the bot-

tom chord. This will prevent the starting of a crack at b, and also assist in keeping the corbel in place.

If it is assumed that the bolt holes are slightly larger than the bolts, the instant that any motion takes place along be the bolts B will be subjected to tension. If friction along be, and between the wood and the metal-plate washer be neglected, the tension in the bolts may be determined by resolving $34500 - 10800 \sec \theta = 21500$ into two components, one normal to the plane be, and the other in the direction of the bolts. Doing this the tension in the bolts is found to be about 40000 lbs. See Fig. 26.

Since all friction has been neglected, the allowable tensile strength per square inch of the bolts may be taken somewhat larger than 16000 lbs., or, say, 20000 lbs. Then the total area required is $\frac{40000}{20000} = 2.00 \ \square''$. The area at the root of a thread of a $\frac{7}{8}''$ bolt is 0.42 $\square''$, hence $\frac{2.00}{0.42} = 5 =$ the number of $\frac{7}{8}''$ bolts required.

Each bolt resists a tension of $\frac{40000}{5} = 8000$ lbs., and hence the area of the washer bearing across the fibers of the wood must be $\frac{8000}{350} = 23.0 \ \square''$ (Art. 25). As the standard cast-iron washer has an area of but 11.0 $\square''$, a single steel plate will be used for all the bolts. The total area including $5 - 1''$ holes for bolts will be $5(23 + 0.785) = 119 \ \square''$, and as the top chord is 6″ wide, the plate will be $6'' \times 20'' = 120 \ \square''$.

The proper thickness of this plate can be determined approximately as follows:

The end of the plate may be considered as an overhanging beam fastened by the nuts or heads on the bolts and loaded with 350 lbs. per square inch of surface bearing against the wood.

The distance from the end of the plate to the nuts is about $3\frac{1}{4}''$, and the moment at the nuts is $350 \times 6 \times 3\frac{1}{4} \times 3\frac{1}{4} \times \frac{1}{2} = 11100$ inch-pounds. This must equal $\frac{1}{6} Rbd^2 = \frac{1}{6} Rbt^2 = \frac{1}{6} \times 16000 \times 6 \times t^2$, or $t^2 = \frac{11100}{16000} = 0.70$, and hence $t = 0.84'' = \frac{7}{8}''$ about. A $\frac{7}{8}''$ plate will be used (Art. 30).

The tension in the bolts must be properly transferred to the corbel through adequate washers. Each bolt carries a tension of $\frac{40000}{5} = 8000$ lbs.

The bolts in pairs transfer their stress partially against the end fibers of the wood over an area of $4 \times 6 = 24 \square''$, which will be considered adequate. The washer of the single bolt bears across the grain and should have an available area of $\frac{8000}{350} = 23 \square''$. A cast-iron washer $5''$ in diameter, and beveled as shown in Fig. 26, will be used.

The horizontal component of the tension in the bolts having been transferred to the corbel, must now be transferred to the bottom chord. This is done by two white oak keys $2\frac{1}{2}'' \times 8''$ long. Each key will safely carry an end fiber stress (Art. 23), of $1\frac{1}{4} \times 6 \times 1400 = 10500$ lbs., and two keys 2×10500, or 21000 lbs., or the total horizontal component of the stress in the bolts.

The safe longitudinal shear of each key is (Art. 27), $6'' \times 8 \times 200 = 9600$, and for both keys $2 \times 9600 = 19200$ lbs., a little less than the stress to be transferred.

The bearing of the keys against the end fibers of the corbel and the bottom chord is safe, as the safe value for long-leaf Southern pine is greater than for white oak.

The safe longitudinal shear in the end of the bottom chord is about $6 \times 12 \times 150 = 7200$ lbs. exclusive of the $\frac{7}{8}''$ bolt. The safe strength at the right end of the corbel is

about the same. Between the keys there is ample shear-
ing surface without any assistance from the bolts in both
the corbel and the bottom chord. The actual holding
power of the two bolts near the ends of the corbel is an un-
certain quantity, but it will be safe to assume they will
reinforce the action of the keys sufficiently to resist a stress
of 21000 lbs.

If the oak keys be replaced by metal keys, $2\frac{1}{2}'' \times 2\frac{1}{2}''$,
one placed at the right end of the first key and the other
at the left end of the second key. The two bolts men-
tioned above will simply prevent the keys from rolling
out of their seats, and the 21000 lbs. be provided for.

In order to prevent bending, and also to give a large
bearing surface for the vertical component of 34500 lbs.,
a white oak filler is placed as shown in Fig. 26, and a small
oak key employed to force it lightly into place.

The net area of the bottom chord must be $\frac{31300}{1200} =$
26.1 $\Box''$ which inspection shows is exceeded at all sections
in Fig. 26.

The form of joint just considered is very common, but
almost always lacking in strength. In addition to the
notch, usually but one or two $\frac{7}{8}''$ bolts are used where
five $\frac{7}{8}''$ bolts are required. The writer has even seen trusses
where the bolts were omitted entirely.

The joint as designed would probably fail before either
the top or bottom chords gave out. If tested under a ver-
tical load, the top chord would act as a lever with its ful-
crum over the oak filler; this would throw an excessive
tension upon the lower pair of bolts, and they would fail
in the threads of the nuts.

Whenever longitudinal shear of wood must be depended

upon, as in Fig. 26, bolts should always be used to bring an initial compression upon the shearing surface, thereby preventing to some extent season cracks.

56a. Design of Joint L_0—Bolts and Metal Plates.— The horizontal component of 34500 lbs. is 28700 lbs., which is transferred to the bottom chord by the two metal teeth let into the chord as shown in Fig. 27. Let the first
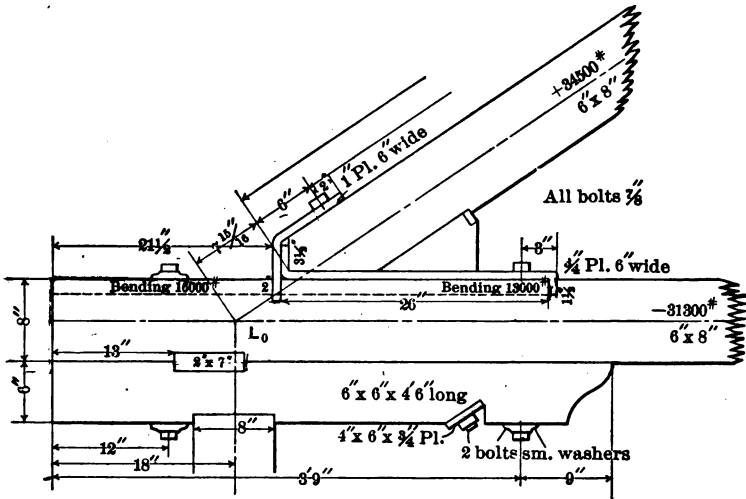
FIG. 27.

plate be 1″ thick and the notch 2″ deep, then the safe moment at the point where it leaves the wood is $\frac{1}{6}Rbt^2 = \frac{1}{6} \times$ 16000 × 6 × 1 × 1 = 16000 inch-pounds.

A load of 16000 lbs. acting 1″ from the bottom of the tooth gives a moment of 16000 × 1 = 16000 inch-pounds. This load uniformly distributed over the tooth $= \dfrac{16000}{2 \times 6} = 1330$ lbs. per square inch; as this is less than 1600 lbs., the safe bearing against the end fibers of the wood, the value of the tooth is 1330 × 12 = 16000 lbs. The shearing surface

ahead of the tooth must be at least $\frac{10000}{111}=108\ \square''$; and since the chord is 6" thick, the length of this surface must be at least $\frac{108}{6}=18''$, which is exceeded in Fig. 27.

-- In like manner the value of the second tooth $\frac{3}{4}''$ thick is found to be 13000 lbs., and hence the value of both teeth is $16000+13000=29000$ lbs., which exceeds the total horizontal component of 34500 lbs. or 28700 lbs.

The horizontal component 28700 lbs. is transferred to the metal through the vertical plates at the end of the top chord, and these are held in place by two $\frac{3}{4}''$ bolts as shown. Since less than two square inches of section are required in the plates to resist by tension, it is evident that there is ample provision for this.

To shear the plates a surface of $1+\frac{3}{4} \times 6=10.5\ \square''$ must fail. The safe strength is $10.5 \times 10000=105000$ lbs.; or over three times 28700 lbs. (Art. 32).

The net area of the bottom chord is 30 $\square''$, which exceeds the amount required.

The corbel is not absolutely necessary in this detail, but it simplifies construction.

To keep the $\frac{3}{4}''$ plate in place two $\frac{3}{4}''$ bolts are employed. They also keep the tooth in its proper position.

The teeth should usually be about twice their thickness in depth, as then the bending value of the metal about equals the end bearing against the wood. This allows for a slight rounding of the corners in bending the plates.

Fig. 28 shows another form of joint using one $\frac{3}{4}''$ plate. The bolt near the heel of the plate resists any slight lifting action of the toe of the top chord, and also assists somewhat in preventing any slipping towards the left.

FIG. 28.

FIG. 29.

The vertical component of 32000 lbs. is resisted by the pin bearing across the fibers of the bottom chord, and by the usual rules the area is entirely too small. Experience shows, however, that when the fibers are confined at the ends, as in this case, if the detail is safe in other
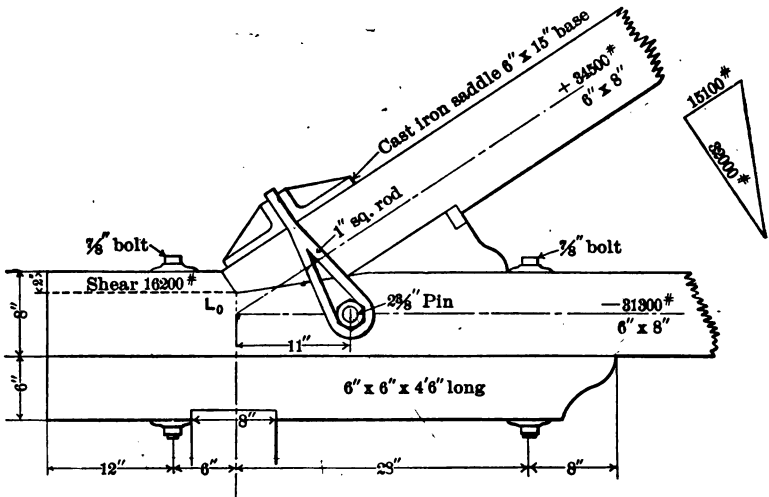
FIG. 31.

particulars, failure will not take place by the crushing of these fibers. Failure usually takes place by the splitting of the bottom chord through the pin-hole.

The cast-iron saddle is large, as 32000 lbs. must be distributed over the surface of the top chord, so that the load per square inch does not exceed 350 lbs.

This joint has the bad feature that no adjustment is readily made after it is assembled, other than by driving the saddle up until it is tight. The probability is that the notch carries the entire load until enough distortion has taken place to bring the stirrup into action.

60. Design of Joint L_0—Plate Stirrup and Pin.—Fig. 32.
—The method pursued in proportioning this type of joint

+ 34500 #
6" x 8"

34500 #

22000 #

⅞" bolt

⅞" bolt

⅝" Pl. 6" wide

L_0

2⅞" Pin

—31300 #
6" x 8"

8"

6"

30"

6" x 6" x 3'6" long

8"

12"   6"   11"   13"
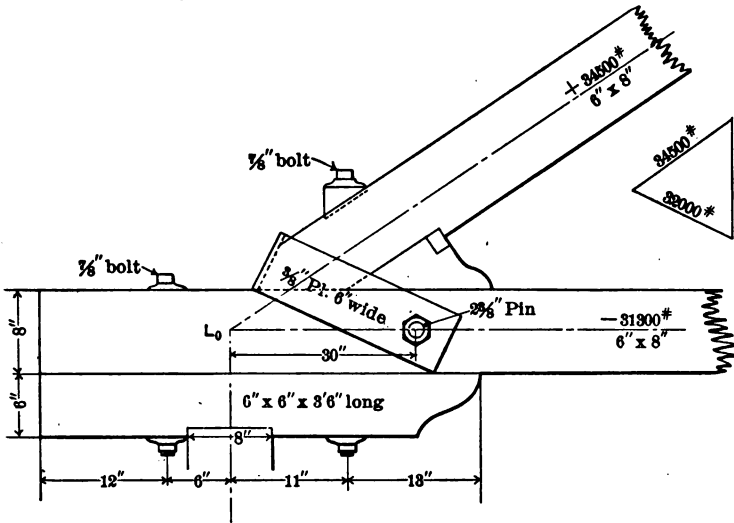
FIG. 32.

is the same as that followed in Art. 59. In this case the stirrup takes the entire component of 34500 lbs., the $\frac{7}{8}$"-bolt merely keeps the members in place.

Note that the bearing against the pin is about 2000 lbs. per square inch of the wood fibers. It requires a 3"-pin to reduce this to 1600 lbs.

61. Design of Joint L_0—Steel Angle Block.—Fig. 33.— This joint needs no explanation. Its strength depends upon the two hooks and the shearing resistance in the bottom chord. The diagonal $\frac{7}{8}$" bolt is introduced to hold the block in its seat, and to reinforce the portion in direct compression. The top chord is kept in position by the top plate, and a 1"-round steel pin driven into the end and passing through a hole in the block.

62. Design of Joint L_0—Cast-iron Angle Block.—At the right, in Fig. 33, is shown a cast-iron angle block made of $\frac{3}{4}''$ or $1''$ metal. It is held in place by two $\frac{7}{8}''$-inch bolts. The top chord is held in position by a cast-iron lug in the centre
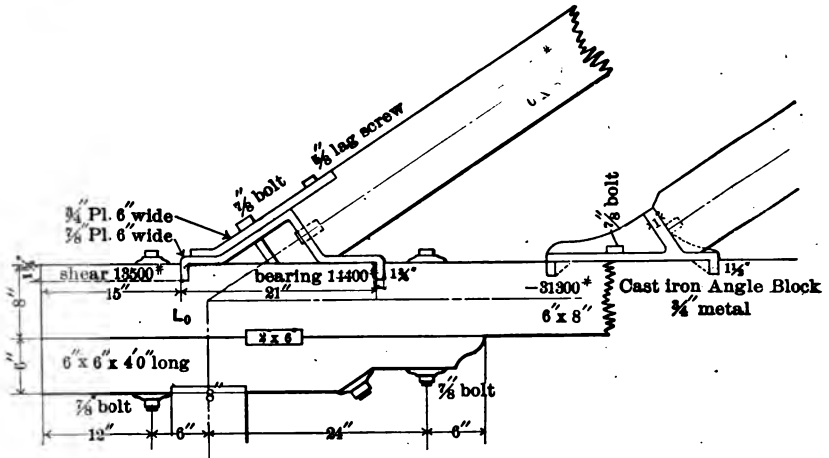
Fig. 33.

of the block used to strengthen the portion of the block at its right end.

In all angle block joints care must be taken to have sufficient bearing surface on top of the bottom chord to safely carry the vertical component of the stress in the top chord.

63. Design of Joint L_0—Special.—It sometimes happens that trusses must be introduced between walls and the truss concealed upon the outside. In this case the bottom chord rarely extends far beyond the point of intersection of the center lines of the two chords. The simplest detail for this condition is a flat plate stirrup and a pin, as shown in Fig. 34. As in Art. 60, a $3''$-pin is required
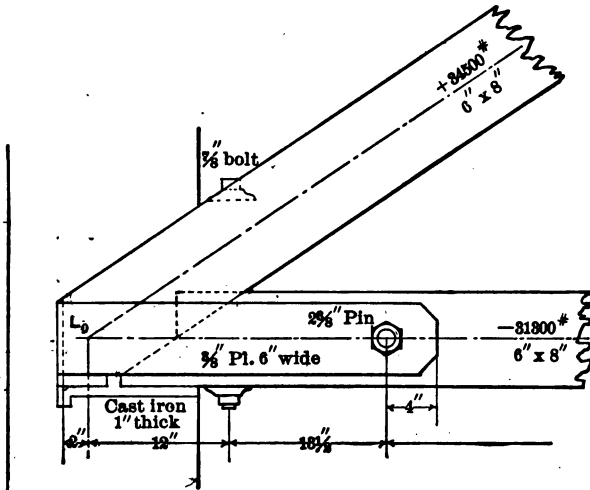
FIG. 34.
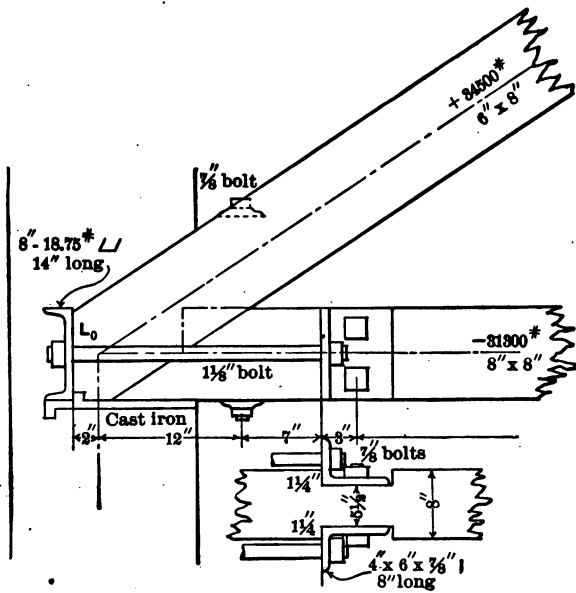
FIG. 35.

if the end bearing of the wood fibers is about 1600 lbs. per square inch. Confined fibers can safely carry 2000 lbs. per square inch.

Fig. 35 shows another type of joint which has some advantages over that shown in Fig. 34, as it can be adjusted. The bottom chord, however, must be heavier on account of the deep notches required.

64. Design of Joint L_0—Plank Members.—Figs. 36 and
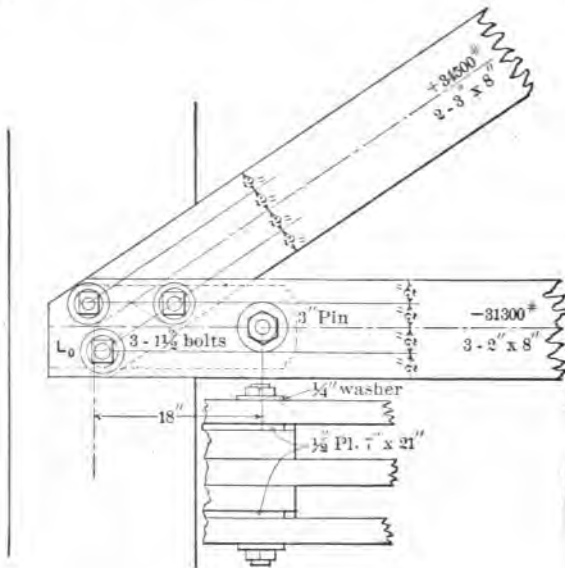
FIG. 36.

37 show two types of joints when the members of the truss are made up of plank. Large bolts are used as they are subjected to bending. In case large bolts cannot be readily obtained, then a larger number of small bolts may be employed.

65. Design of Wall Bearing.—In designing joint L_0 above, no consideration of the reaction at the support has been considered. The vertical and horizontal forces at
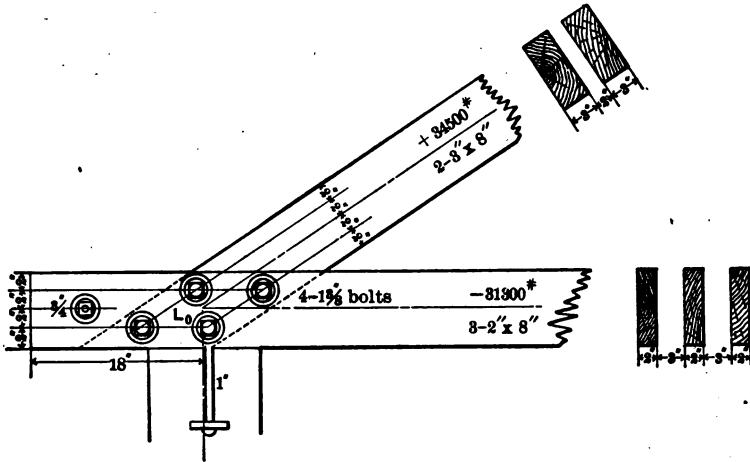
FIG. 37.

L_0 are shown on Plate I. The horizontal component is 3700 lbs., but about 1000 lbs. of this is due to the 1700 lbs. at L_0, leaving 2700 lbs., which is the difference between the horizontal component of the stress in the top chord and the stress in the bottom chord. The corbel, or bolster, must be so fastened to the support that 3700 lbs. may be safely resisted. In most cases friction is sufficient, but as an extra precaution it is best to notch the bolster, as shown in the various designs.

The total vertical load is 23500 lbs., and the bearing area required is $\frac{23500}{370} = 63$ □″. As the bolster is 6″ thick the bearing should be 11″ long for long-leaf Southern pine and 8″ long for oak. Usually there is ample bearing on masonry walls. In frame buildings there may be a lack of bearing surface, but if the stress does not exceed 500 or

600 lbs. per square inch, no trouble will result for the harder woods.

66. Remarks Concerning the Designs for Joint L_0.—In all of the above designs little or no account has been taken of the friction between the various surfaces. This is justifiable, because after the truss has been in position a few months all nuts become loose, owing to the shrinkage of the wood. In twelve inches of wood this often amounts to more than one-half an inch.

All bolts and pins should have a driving fit and the nuts on bolts be screwed up very tight. Whenever possible the nuts should be tightened at the end of three months after erection, and again at the end of a year.

It will be noticed that the full sizes of the timbers have been used in dimensioning. These will not obtain in market sizes, which will be from $\frac{3}{8}''$ to $\frac{1}{2}''$ scant in size. A good carpenter will take this into account in framing.

67. Design of Joint U_2.—As the rafter is continuous by this joint it will be necessary to consider only the vertical rod and the inclined brace.

Since the stress of the rod is comparatively small, the standard size of cast-iron washer can be employed to transfer it to the rafter. Two forms of angle washers are shown in Figs. 38 and 40. In Figs. 39 a bent plate washer is shown which answers very well if let into the wood or made sufficiently heavy so that the stress in the rod cannot change the angles of the bends.

Where the inclined member is so nearly at right angles with the top chord as in this case, a square bearing, as shown in Fig. 40, is all that is required if there is sufficient bearing area. In this case there are 36 □ ″, which has a

safe bearing value of $36 \times 350 = 12600$ lbs., which is within 100 lbs. of being sufficient.
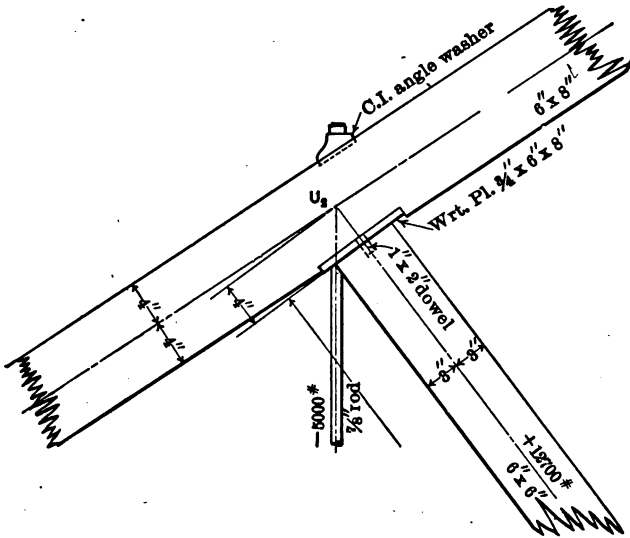
FIG. 38.
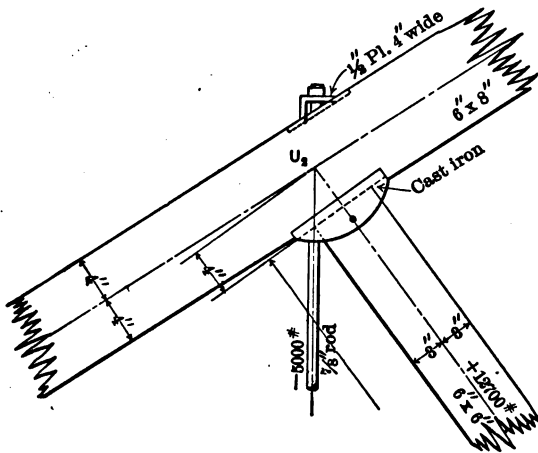
FIG. 39.

Fig. 38 shows a method of increasing the bearing area by means of a wrought plate, and Fig. 39 the same end

reached with a cast-iron block. In all cases the strut should
be secured in place either by dowels, pins or other device.
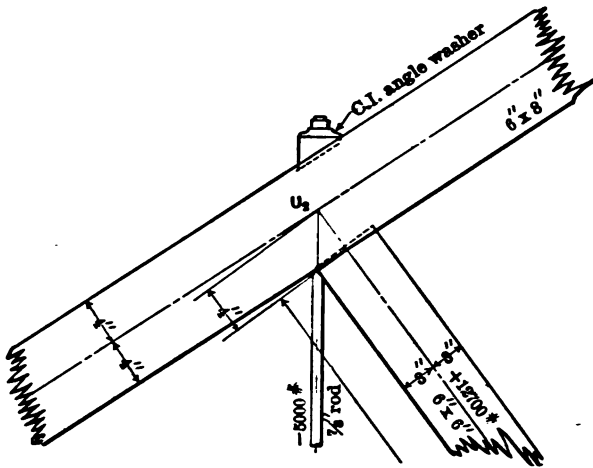
FIG. 40.

68. Design of Joint U_1.—The disposition of the $\frac{3}{4}''$ rod
is evident from the Figs. 41, 42, and 43.
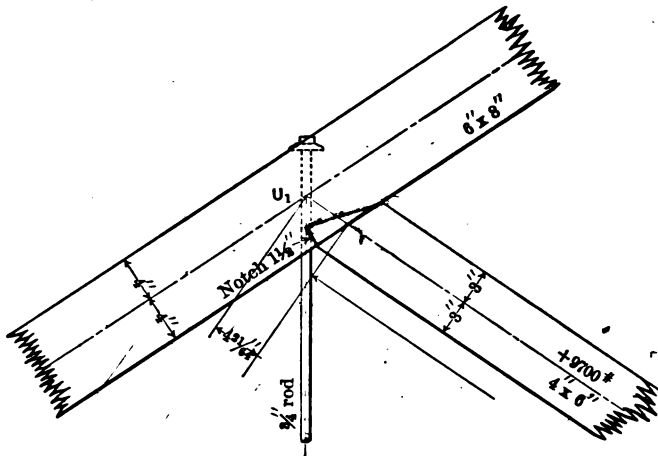
FIG. 41.

Fig. 41 shows the almost universal method employed
by carpenters in framing inclined braces, only they seldom

take care that the center lines of all pieces meet in a point as they should.

If the thrust 9700 lbs. be resolved into two components
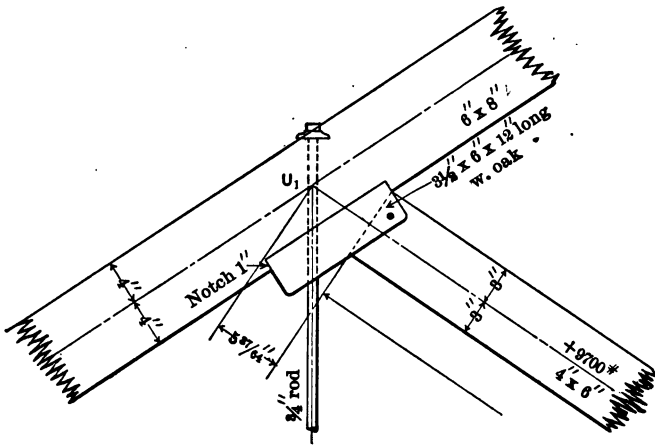
FIG. 42.
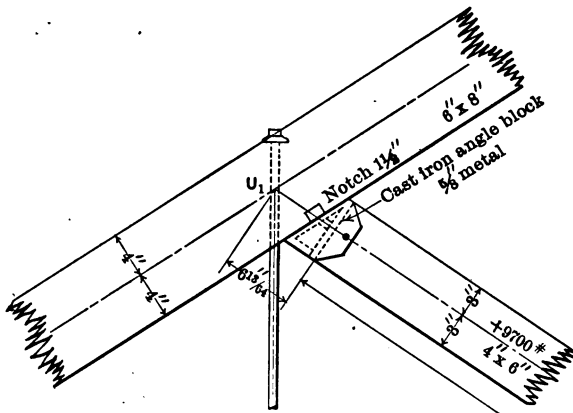
FIG. 43.

respectively normal to the dotted ends, it is found that a notch 1½″ deep is required to safely take care of the component parallel to the rafter. The component nearly normal to the rafter is safely carried by about 20 □″.

Figs. 42 and 43 show the application of angle-blocks, which really make much better connections, though somewhat more expensive, than the detail first described.

69. Design of Joint L_2.—Fig. 44 shows the ordinary method of connecting the pieces at this joint. The hori-
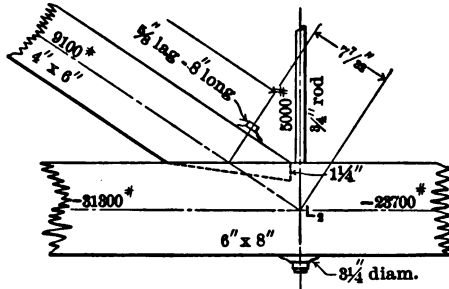
FIG. 44.

zontal component of 9100 lbs. is taken by a notch $1\frac{1}{4}''$ deep and $4''$ long. The brace is fastened in place by a $\frac{5}{8}''$ lag-screw $8''$ long. The standard cast-iron washer, $3\frac{1}{4}''$. in diameter, gives sufficient bearing area against the bottom chord for the stress in the vertical rod.
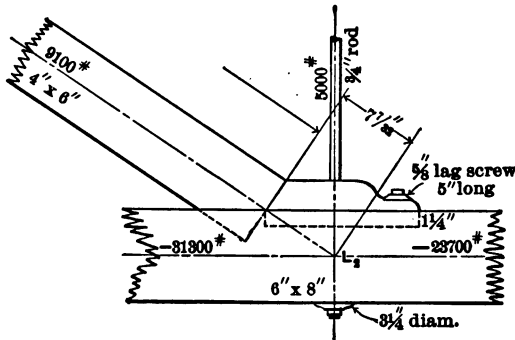
FIG. 45.

Fig. 45 shows a wooden angle-block let into the bottom chord $1\frac{1}{4}''$. The dotted tenon on the brace need not be

over 2″ thick to hold the brace in position. The principal
objection to the two details just described is that the end
bearing against the brace is not central, but at one side,
thereby lowering the safe load which the brace can carry.

Fig. 46 shows the application of a cast-iron angle-block.
The brace is cut at the end so that an area 4″×4″ trans-
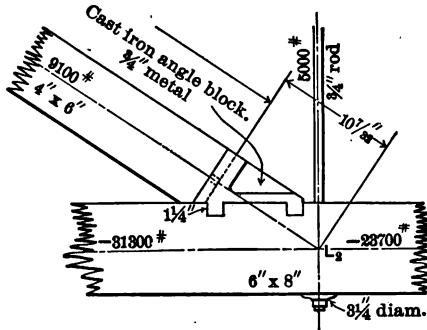
FIG. 46.

mits the stress to the angle-block. If the lugs on the
bottom of the block are 1¼″ deep, the horizontal component
of the stress in the brace will be safely transmitted.
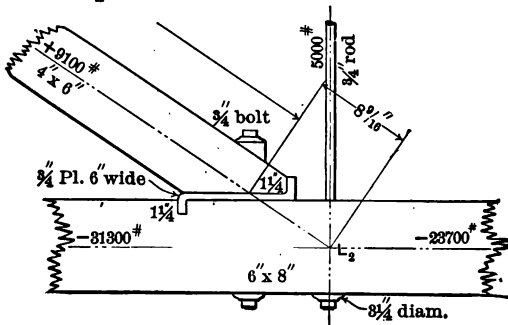
FIG. 47.

In Fig. 47 a ¾″ bent plate is employed. This detail
requires a ¾″ bolt passing through the brace and the bottom

chord to make a solid connection. The use of the bolt makes the end of the brace practically fixed, so that the stress may be assumed to be transmitted along the axis or center line.

70. Design of Joint L_3 and Hook Splice.—A very common method of securing the two braces meeting at L_3 is shown in Fig. 48, though they are rarely dapped into the lower chord. This method does fairly well, excepting
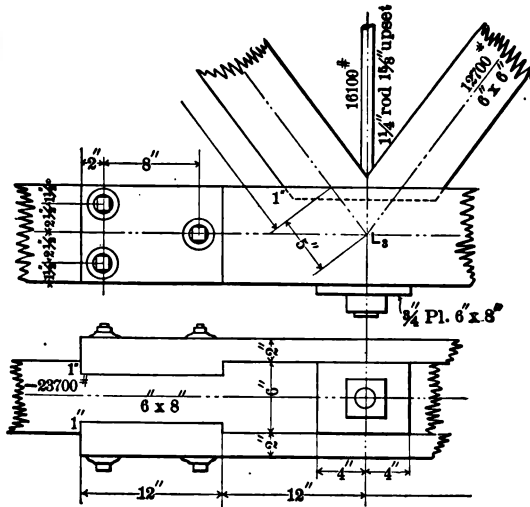
FIG. 48.

when the wind blows and one brace has a much larger stress than the other. In this case the stresses are not balanced, and the struts are held in place by friction and the stiffness of the top chord.

The washer for the $1\frac{1}{4}''$ rod upset to $1\frac{5}{8}''$ must have an area of $\frac{16100}{360} = 45 \ \Box''$, which is greater than the bearing area of the standard cast-iron washer, so a $\frac{3}{4}''$ plate, $6'' \times 8''$, will be used.

It is customary to splice the bottom chord at this joint when a splice is necessary. The net area required is $\frac{23700}{1200} = 20$ $\square''$. The splice shown in Fig. 48 is one commonly used in old trusses, and depends entirely upon the longitudinal shear of the wood and the end bearing of the fibers.

The total end bearing required is $\frac{23700}{1600} = 15$ $\square''$, which is obtained by two notches, each $1''$ deep as shown. The total shearing area required is $\frac{23700}{150} = 158$ $\square''$. Deducting bolt-holes, the area used is $2(8 \times 12) - 2(3) = 186$ $\square''$. The three bolts used simply hold the pieces in place and are not intended to carry any stress.

Fig. 49 shows a similar splice where metal keys are used. The end–bearing area of the wood is the same as
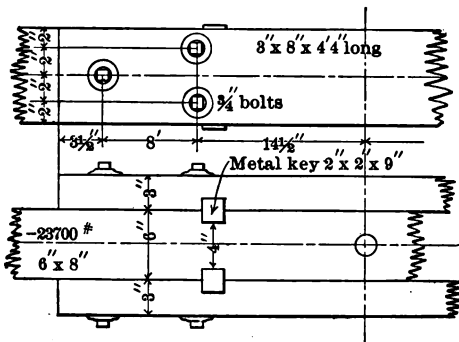
FIG. 49.

before, and the available area of the wood for longitudinal shear is sufficient, as shown by the dimensions given. The net area of the side pieces is $2(2 \times 8) = 32$ $\square''$, while but 20 $\square''$ are actually required.

71. Design of Joint L_3 and Fish-plate Splice of Wood, —In this case the braces are held in position by dowels and a wooden angle-block. The details of the vertical rod

need no explanation, as they are the same as in Art. 70. The splice is made up of two fish-plates of wood each $2'' \times 8'' \times 46''$ long and four $1\frac{1}{2}''$ bolts. The net area of the fish-plates is $2(2 \times 8) - 2(2 \times 1\frac{1}{2}) = 24 \;\square''$, while but $20 \;\square''$ are required.

Each bolt resists in bending $\frac{23700}{8}(1 + \frac{3}{4}) = 7400$ inch-

pounds, which is less than the safe value, or 8280 inch-pounds (Art. 31).
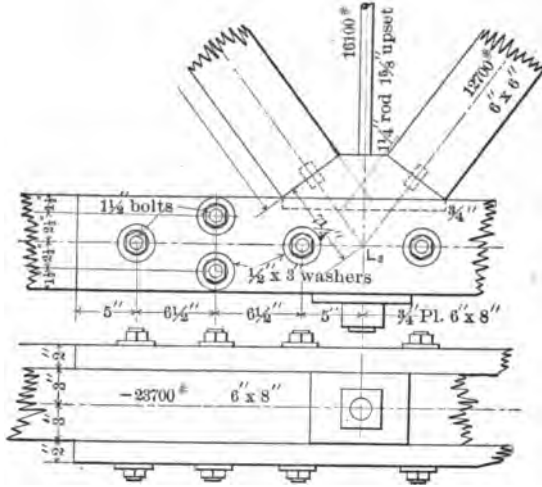
The total end bearing of the wood fibers is $2(4 \times 2 \times 1\frac{1}{2}) = 24 \;\square''$, and that required $\frac{23700}{1600} = 15 \;\square''$.

The longitudinal shearing area of the wood and the transverse shearing area of the bolts are evidently in excess of that required.

The nuts on the bolts may be considerably smaller than the standard size, as they merely keep the pieces in place. The cast-iron washer may be replaced by the small plate washer, to make sure that no threads are in the wood;

otherwise washers are not needed. The bolts should have a driving fit.

72. Design of Joint L_3 and. Fish-plate Splice of Metal. —This detail, differing slightly from those previously given, requires little additional explanation. A white-oak washer
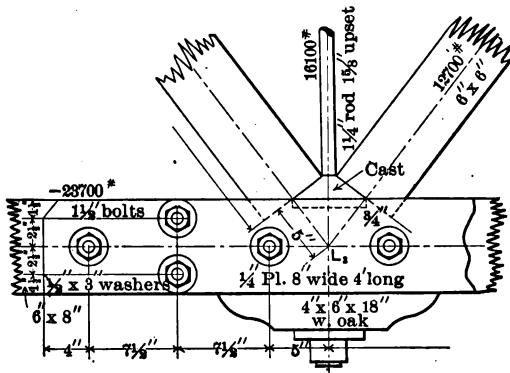
FIG. 51.

has been introduced so that a smaller washer can be used for the vertical rod.
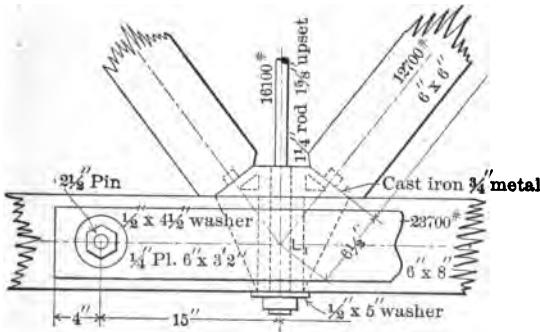
FIG. 52.

A small cast-iron angle-block replaces the wooden block of the previous article. The splice is essentially the same, with metal fish-plates. Contrary to the usual practice,

plate washers have been used under the nuts.   This is to make certain that the fish-plates bear against the bolt proper and not against threads.   If recessed bridge-pin nuts are used, the washers can be omitted.

Fig. 52 shows another metal fish-plate splice where four bolts have been replaced by one pin 2½″ in diameter.
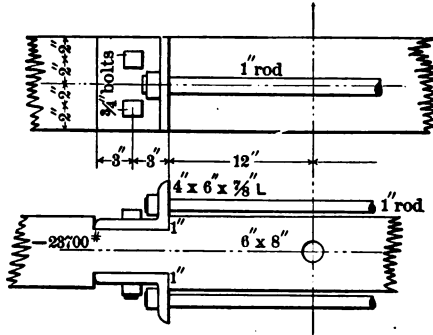
FIG. 53.
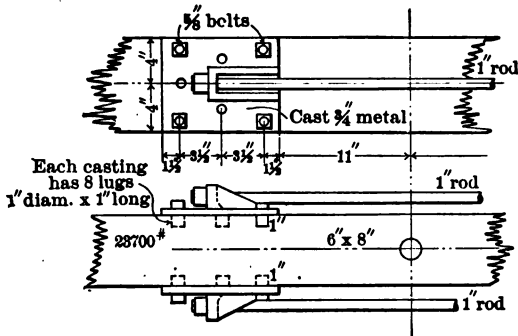
FIG. 54.

The struts bear against a cast-iron angle-block, with a "pipe" for the vertical rod, which transmits its stress directly to the block.   Two pins in the centre of the block keep the bottom chord in position laterally.

73. Metal Splices: for Tension Members of Wood.—Figs. 53 and 54 show two types of metal splices which have the great advantage over all the splices described above that they can be adjusted.
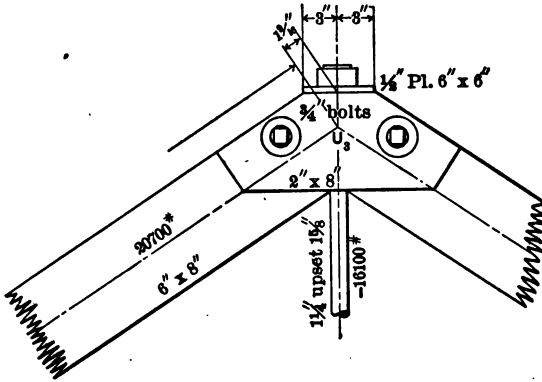
FIG. 55.
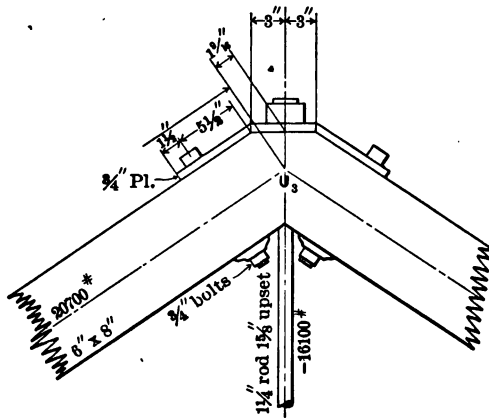
FIG. 56.

74. General Remarks Concerning Splices.—There are a large number of splices in common use which have not been considered, for the reason that most of them are

faulty in design and usually very weak. In fact certain scarf-splices are almost useless, and without doubt the truss is only prevented from failing by the stiffness of its supports.

75. Design of Joint U_3.—The design of this joint is clearly shown in Figs. 55–58. No further explanation seems necessary.
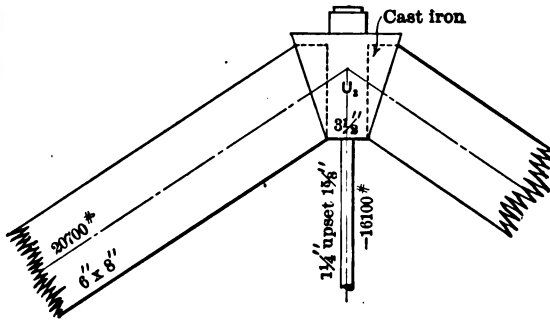
FIG. 57.
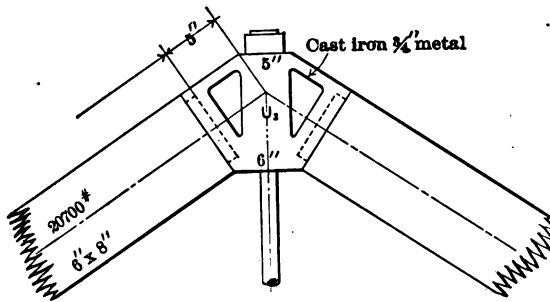
FIG. 58.

76. The Attachment of Purlins.—The details shown (Figs. 59–63) are self-explanatory. In all cases the adjacent purlins should be tied together by straps as shown. This precaution may save serious damage during erection, if at no other time.

The patent hangers shown in Figs. 64. 65, 66, and 67 can be employed to advantage when the purlins are placed between the top chords of the trusses.

77. **The Complete Design.**—Plate I shows a complete design for the roof-truss, with stress diagrams and bills of
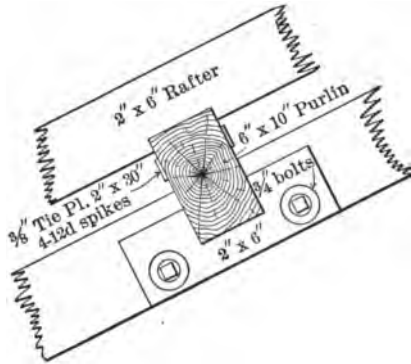
F G. 59.
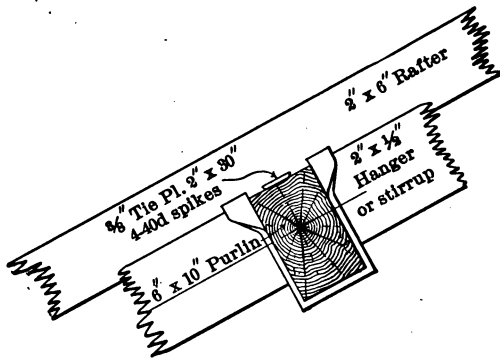
FIG. 60.

material. The weight is about 100 lbs. less than that assumed. In dimensioning the drawing a sufficient number of dimensions should be given to enable the carpenter to lay off every piece, notch, bolt-hole, etc., without scaling

from the drawing. To provide for settlement or sagging due to shrinkage and the seating of the various pieces when
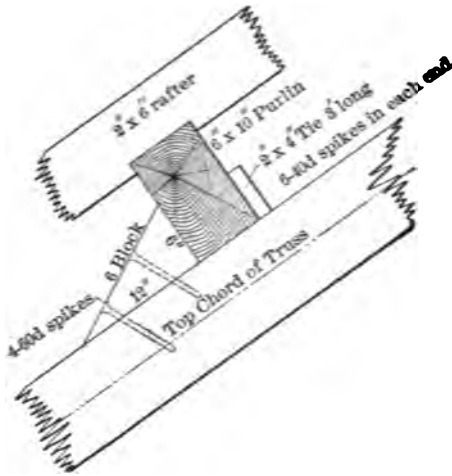
FIG. 61.
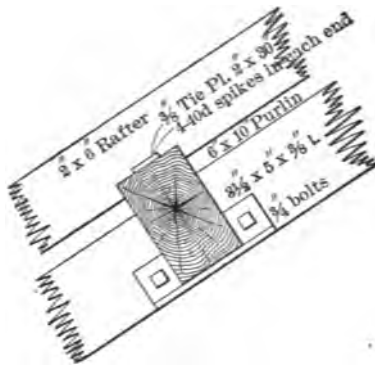
FIG. 62.

the loading comes upon the new truss, the top chord is made somewhat longer than its computed length. From $\frac{1}{2}''$ to $\frac{3}{4}''$ for each $10'$ in length will be sufficient in most cases. A truss so constructed is said to be cambered.

. In computing the weights of the steel rods they have been assumed to be of uniform diameter from end to end, and increased in length an amount sufficient to provide metal for the upsets. See Table VII.

The lengths of small bolts with heads should be given from under the head to the end of the bolt, and the only fraction of an inch used should be ⅛.
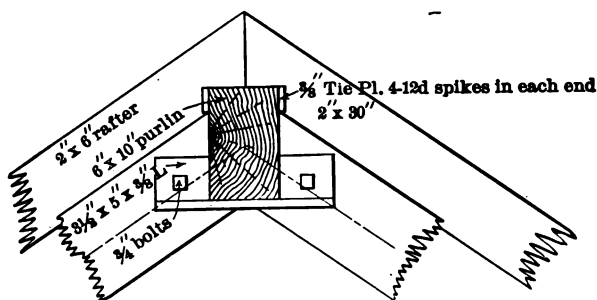
FIG. 63.

FIG. 64.   FIG. 65.   FIG. 66.   FIG. 67.

Plate II shows another arrangement of the web bracing which has some advantages. The compression members are shorter, and consequently can be made lighter. The bottom chord at the centre has a much smaller stress, permitting the use of a cheap splice. On account of the increase of metal the truss is not quite as economical as that shown on Plate I. For very heavy trusses of moderate span the second design with the dotted diagonal is to be preferred.

CHAPTER VI.

DESIGN OF A STEEL ROOF-TRUSS.

78. Data.—Let the loading and arrangement of the various parts of the roof be the same as in Chapter V, and simply replace the wooden truss by a steel truss of the shape shown on Plate III. Since there is but little difference between the weights of wooden and steel trusses of the same strength, the stresses may be taken as found in Chapter V and given on Plate III.

79. Allowable Stresses per Square Inch.

SOFT STEEL.

Tension with the grain.............. Art. 35, 16000 lbs.
Bearing for rivets and bolts........... Art. 24, 20000 lbs.
Transverse stress—extreme fiber stress. Art. 30, 16000 lbs.
Shearing across the grain........... Art. 32, 10000 lbs.
Extreme fiber stress in bending (pins).. Art. 31, 25000 lbs.

For compression use table, page 27, with a factor of safety of 4.

80. Sizes of Compression Members.

Piece $L_0 U_1$.   Stress $= +34500$ lbs.

The ordinary shape of the cross-section of compression members in steel is shown on Plate III. Two angles are placed back to back and separated by $''$ or $\frac{3}{8}''$ to admit gusset-plates, by means of which all members are connected

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at the apexes. Generally it is more economical to employ unequal leg angles with the larger legs back to back.

Let the gusset-plates be assumed $\frac{3}{8}''$ thick, then from Table XIII the least radii of gyration of angles placed as explained above can be taken.

Try two $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angles. From Table XIII the least radius of gyration (r) is 1.09. The unsupported length of the piece $L_0 U_1$ in *feet* is 12, and hence $\dfrac{L}{r} = \dfrac{12}{1.09} = 11.0$. From Art. 22, $P = 30324$ lbs. for square-ended columns when $\dfrac{L}{r} = 12.4$. $30324 \div 4 = 7581$ lbs. $=$ the allowable stress per square inch. $\frac{34500}{7581} = 4.55 =$ number of square inches required. The two angles assumed have a total area of 2.88 square inches, hence another trial must be made. An inspection of Table XIII shows that 1.09 is the least radius of gyration for any pair of $3\frac{1}{2}'' \times 2\frac{1}{2}''$ angles placed $\frac{3}{8}''$ apart, as shown; hence if any pair of $3\frac{1}{2}'' \times 2\frac{1}{2}''$ angles gives sufficient area, the pair will safely carry the load.

Two $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$ angles have an area of $2 \times 2.43 = 4.86$ square inches.

Angles with $2\frac{1}{2}''$ legs do not have as much bearing for purlins as those with longer legs, and sometimes are not as economical. In this case, two $4'' \times 3'' \times \frac{5}{16}''$ angles having an area of 4.18 square inches will safely carry 34500 lbs., making a better and more economical combination than that tried above. This combination will be used.

Thus far it has been assumed that the two angles act as one piece. Evidently this cannot be the case unless they are firmly connected. The least radius of gyration of a single angle is about a diagonal axis as shown in

Table XII, and for a $4'' \times 3'' \times \tfrac{5}{16}''$ angle its value is 0.63.
If the unsupported length of a single angle is l, then in
order that the single angle shall have the same strength
as the combination above, $\dfrac{l}{0.63}$ must equal $\dfrac{L}{1.30} = 9.2$, or
$l = 5'.8$. Practice makes this length not more than $\tfrac{3}{4}(5.8)$,
or about 4 feet. Hence the angles will be rigidly con-
nected by rivets every 4 feet.

<p style="text-align:center">Pieces $U_1 U_2$ and $U_2 U_3$.</p>

Owing to the slight differences in the stresses of the
top chords the entire chord is composed of the same com-
bination, or two $4'' \times 3'' \times \tfrac{5}{16}''$ angles, having an area of 4.18
square inches.

<p style="text-align:center">Piece $U_2 L_2$.   Stress $= + 10100$.</p>

Although it is common practice to employ but one
angle where the web stress is small, yet it is better prac-
tice to use two in order that the stress may not be trans-
mitted to one side of the gusset-plate.

The unsupported length of this piece is $13'.3$. The
least radius of gyration of two $2\tfrac{1}{2}'' \times 2'' \times \tfrac{1}{4}''$ angles is 0.94.
$\dfrac{L}{r} = \dfrac{13.3}{0.94} = 14.1$, and, from Art. 22, $P =$ about 25100. $\tfrac{25100}{4}$
$= 6270 =$ the allowable stress per square inch. $\tfrac{10100}{6300} = 1.60$
square inches required.

Two $2\tfrac{1}{2}'' \times 2'' \times \tfrac{1}{4}''$ angles have an area of 2.12 square
inches, and hence are safe according to the strut formula.
*For stiffness no compression member should have a dimen-
sion less than $\tfrac{1}{50}$ of its length.*

$\dfrac{13.3 \times 12}{50} = 3''.2$, or the long legs of the angles should

be $3''.2$, and the sum of the short legs not less than this amount. Hence two $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angles, having an area of 2.88 square inches, must be used. Tie-rivets will be used once in every four feet about.

Piece L_2U_1 will be the same as L_2U_2.

Piece U_1L_2. Stress $= +9100$ lbs.

Two $3'' \times 2'' \times \frac{1}{4}''$ angles $= 2.38$ square inches can evidently be used, as the dimensions and stresses are slightly less than for U_2L_2.

The least radius of gyration of a single $3'' \times 2'' \times \frac{1}{4}''$ angle is 0.43, hence they must be riveted together every $\frac{3}{8}(0.43)(13.2) = 3.78$ feet. Note that $2''$ legs can be used here, as they will receive no rivets, while in the top chord both angle legs will receive rivets as shown on Plate III.

81. Sizes of Tension Members.

Piece L_0L_2. Stress $= -31300$ lbs.

The net area required is $\frac{31300}{16000} = 1.96$ square inches. The same general form of section is used for tension members as for compression members. In the compression members the rivets were assumed to fill the holes and transmit the stresses from one side of the holes to the other. In tension members this assumption cannot be made, for the fibers are cut off by the rivet-hole, and consequently cannot transmit any tensile stress across the rivet-holes. This being the case, the two angles employed for tension members must have an area over and above the net area required equal to the area cut out or injured by the rivet-holes. The diameter of rivet-holes is increased by $\frac{1}{8}''$ in calculating the area of the fibers destroyed by punching

the hole.   For a $\frac{3}{4}''$ rivet the diameter of the hole is taken as $\frac{7}{8}''$.   See Table IV.

For this truss let all rivets be $\frac{3}{4}''$.   For a trial let the piece in hand (L_0L_2) be made up of two $3'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angles having an area of 2.62 square inches.   As shown by the arrangement of rivets on Plate III, but one rivet-hole in one leg of each angle must be deducted in getting the net area. One $\frac{3}{4}''$ rivet-hole reduces the area of two angles $2(\frac{7}{8} \times \frac{1}{4}) =$ 0.44 square inch, and hence the net area of two $3'' \times 2'' \times \frac{1}{4}''$ angles is $2.62 - 0.44 = 2.18$ square inches, which is a little greater than that required, and consequently can be safely used.

<div style="text-align:center">Piece L_2U_3.   Stress $= -17000$ lbs.</div>

$\frac{17000}{16000} = 1.06$ square inches net section required.

Two $2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$ angles $= 2.12$ square inches.

$2.12 - 0.44 = 1.68$ square inches net section.   As this is greater than the area required, and also the smallest standard angle with $\frac{1}{4}''$ metal which can be conveniently used with $\frac{3}{4}''$ rivets, it will be employed.

<div style="text-align:center">Piece L_3U_3.   Stress $= -16,300$ lbs.</div>

Use two $2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$ angles having a gross area of 2.12 square inches and a net area of 1.68 square inches.

82. Design of Joint L_0, Plate III.—The piece L_0U_1 must transfer a stress of 34500 lbs. to the gusset through a number of $\frac{3}{4}''$ rivets.   These rivets may fail in two ways. They may shear off or crush.   If they shear off, two surfaces must be sheared, and hence they are said to be in double shear.   From Art. 32, a $\frac{3}{4}''$ rivet in double shear will safely carry 8836, and hence in this case $\frac{34500}{8836} = 4$ is the number of rivets required.

The smallest bearing against the rivets is the $\frac{3}{8}''$ gusset-plate. From Art. 24, the safe bearing value in a $\frac{3}{8}''$ plate is 5625 lbs., showing that seven rivets must be employed to make the connection safe in bearing.

It is seen that as long as the angles are at least $\frac{1}{4}''$ thick, the gussets $\frac{3}{8}''$ thick, and the rivets $\frac{3}{4}''$ in diameter the required number of rivets in any member equals the stress divided by the bearing value of a $\frac{3}{4}''$ rivet in a $\frac{3}{8}''$ plate, or 5625.

The piece $L_0 L_2$ requires $\frac{31800}{5625} = 6$ rivets.

The rivets are assumed to be free from bending, as the rivet-heads clamp the pieces together firmly.

The location of the rivet lines depends almost entirely upon practical considerations. The customary locations are given in Table III.

83. Design of Joint U_1.—The number of rivets required in $L_2 U_1$ is $\frac{9100}{5625} = 2$ rivets. The best practice uses at least *three* rivets, but the use of *two* is common. As the top chord is continuous, evidently the same number is required in it.

Joint U_2 will require the same treatment.

84. Design of Joint L_2.

$L_0 L_2$ requires 6 rivets as in Art. 82.
$L_2 U_1$ requires 2 rivets as in Art. 83.
$L_2 U_2$ requires 2 rivets as in Art. 83.
$L_2 U_3$ requires $\frac{17000}{5625} = 4$ rivets.
$L_2 L_2'$ requires $\frac{16300}{5625} = 3$ rivets,

but this connection will probably be made in the field, that is, will not be made in the shop but at the building, so the number of rivets should be increased 25 per cent. Therefore 4 rivets will be provided for.

85. Design of Joint U_3.

U_2U_3 requires 7 rivets as in Art. 82.
L_2U_3 requires 4 rivets as in Art. 83.

If field-rivets are used, these numbers become 9 and 5 respectively.

86. Splices.—The members L_2L_2' and L_1L_2 are connected by means of the gusset-plate in designing joint L_2. It is better practice to make a full splice, that is, connect both legs of the angles in one member with the corresponding legs of the other by means of plates. The gusset-plate will answer for the vertical legs, and a plate equal in thickness to the thickness of the angle legs for the other. The width of this plate should equal that of the member L_1L_2.

87. End Supports.—In designing joint L_0 only enough rivets were placed in the bottom chord to transmit its stress to the gusset-plate. Usually a plate not less than $\frac{1}{2}''$ thick is riveted under the bottom-chord angles to act as a bearing plate upon the support. The entire reaction must pass through this plate and be transmitted to the gusset-plate by means of the bottom-chord angles, unless the gusset has a good bearing upon the plate. This is not the usual condition and is not economical. The reaction is about 24000 lbs. (Art. 65). $\frac{24000}{5625} = 5 =$ the number of $\frac{3}{4}''$ rivets required for this purpose alone. The total number of rivets in the bottom angles is $5+6 = 11$ rivets.

The bearing plate should be large enough to distribute the load over the material upon which it bears, and to admit two anchor-bolts outside the horizontal legs of the bottom angles.

88. Expansion.—Expansion of trusses having spans less than 75 feet may be provided for by letting the bearing plate slide upon a similar plate anchored to the supports, the anchor-bolts extending through the upper plates in slotted holes. See Plate III.

Trusses having spans greater than 75 feet should be provided with rollers at one end.

In steel buildings the trusses are usually riveted to the tops of columns and no special provision made for expansion.

89. Frame Lines and Rivet Lines.—Strictly, the rivet lines and the frame lines used in determining the stresses should coincide with the line connecting the centers of gravity of the cross-sections of the members. This is not practicable, so the rivet lines and frame lines are made to coincide.

90. Drawings.—Plate III has been designed to show various details and methods of connecting the several parts of the truss and the roof members. A great many other forms of connections, purlins, roof coverings, etc., are in use, but all can be designed by the methods given above. Plate III contains all data necessary for the making of an estimate of cost, and is quite complete enough for the contractor to make dimensioned *shop drawings* from. These drawings are best made by the parties who build the truss, as their draughtsmen are familiar with the machinery and templets which will be used.
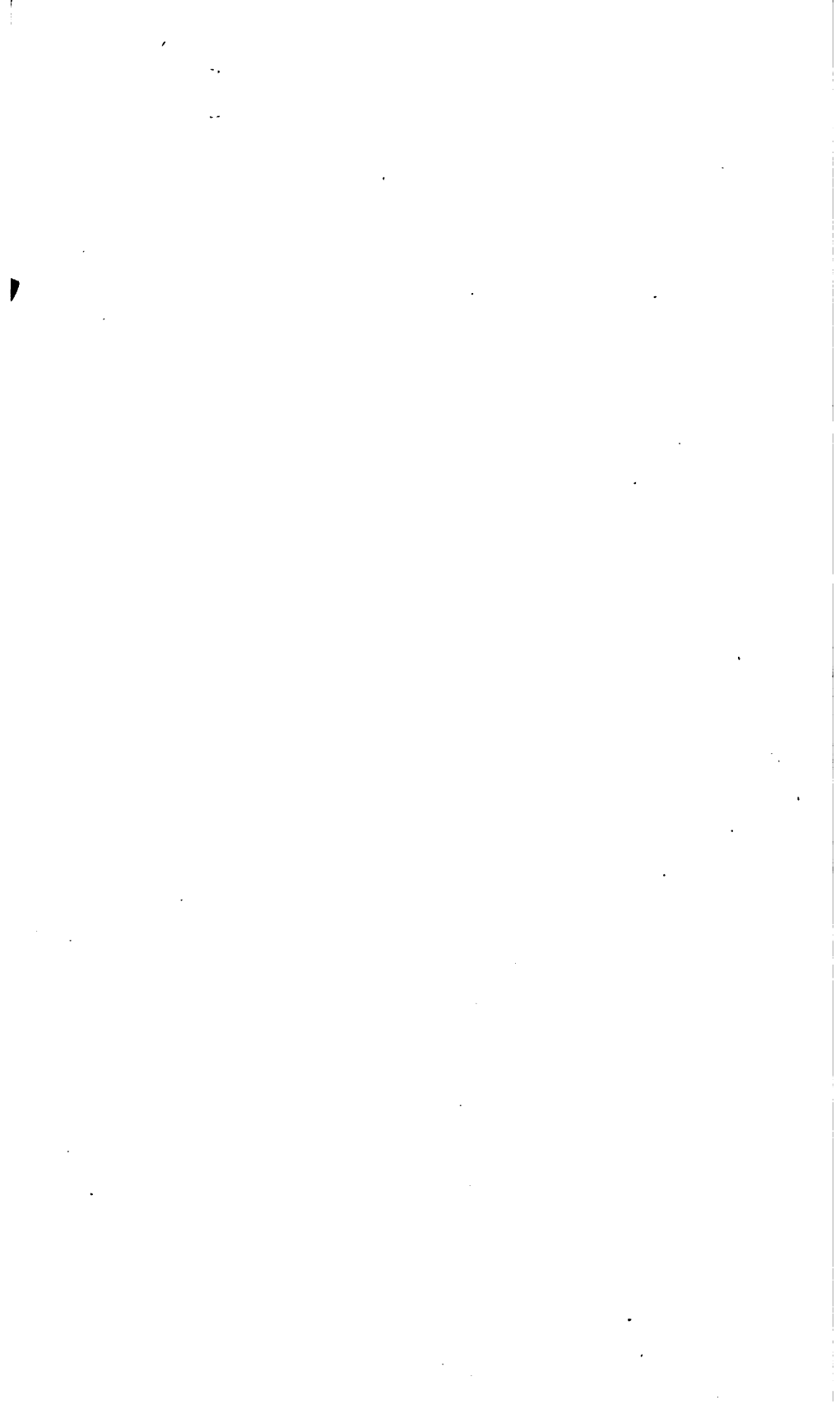
TABLES.

Table I.

WEIGHTS OF VARIOUS SUBSTANCES.

WOODS (SEASONED).

Name.	Weight in Lbs. per Cu. Foot.	Weight in Lbs. per Square Foot, Board Measure.
Ash, American, white	38	3.17
Cherry................................	42	3.50
Chestnut.............................	41	3.42
Elm	35	2.96
Hemlock..............................	25	2.08
Hickory	53	4.42
Mahogany, Spanish	53	4.42
" Honduras	35	2.96
Maple................................	49	4.08
Oak, live	59	4.92
" white............................	52	4.33
Pine, white...........................	25	2.08
" yellow, northern...................	34	2.83
" " southern...................	45	3.75
Spruce	25	2.08
Sycamore.............................	37	3.08
Walnut, black.	38	3.17

Green timbers usually weigh from one-fifth to one-half more than dry.

MASONRY.

Name.	Weight in Lbs. per Cubic Foot.
Brick-work, pressed brick.	140
" ordinary.	112
Granite or limestone, well dressed...	165
" " mortar rubble...................	154
" " dry...........................	138
Sandstone, well dressed.	144

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BRICK AND STONE.

Name.	Weight in Lbs. per Cubic Foot.
Brick, best pressed.	150
" common, hard	125
" soft, inferior	100
Cement, hydraulic loose, Rosendale	56
" " " Louisville	50
" " " English Portland	90
Granite	170
Limestones and marbles	168
" " " in small pieces	96
Quartz, common	165
Sandstones, building	151
Shales, red or black	162
Slate.	175

METALS.

Name.	Weight in Lbs. per Cubic Ft.	Weight in Lbs. per Square Ft., 1″ thick.
Brass (copper and zinc), cast	504	42.00
" rolled	524	43.66
Copper, cast	542	45.17
" rolled.	548	45.66
Iron, cast.	450	37.50
" wrought, purest	485	40.42
" " average.	480	40.00
Lead.	711	59.27
Steel.	490	40.83
Tin, cast	459	38.23
Zinc	437	36.42

TABLE II.

WEIGHTS OF ROOF COVERINGS.

CORRUGATED IRON (BLACK).

Weight of *corrugated iron* required for one square of roof, allowing six inches lap in length and two and one-half inches in width of sheet.

(Keystone.)

Thickness in Inches.	Weight in Lbs. per Sq. Ft., flat.	Weight in Lbs. per Sq. Ft., corrugated.	Weight in Pounds of One Square of the following Lengths.					
			5′	6′	7′	8′	9′	10′
0.065	2.61	3.28	365	358	353	350	348	346
0.049	1.97	2.48	275	270	267	264	262	261
0.035	1.40	1.76	196	192	190	188	186	185
0.028	1.12	1.41	156	154	152	150	149	148
0.022	0.88	1.11	123	121	119	118	117	117
0.018	0.72	0.91	101	99	97	97	96	95

The above table is calculated for sheets 30½ inches wide before corrugating. Purlins should not be placed over 6′ apart.

(Phœnix.)

Thickness in Inches.	BLACK IRON.			GALVANIZED IRON.		
	Weight in Pounds per Square Foot, flat.	Weight in Pounds per Square Foot, on Roof.	Weight in Pounds per Square Foot, on Roof.	Weight in Pounds per Square Foot, flat.	Weight in Pounds per Square Foot, on Roof.	Weight in Pounds per Square Foot, on Roof.
0.065	2.61	3.03	3.37	3.00	3.50	3.88
0.049	1.97	2.29	2.54	2.37	2.76	3.07
0.035	1.40	1.63	1.82	1.75	2.03	2.26
0.028	1.12	1.31	1.45	1.31	1.53	1.71
0.022	0.88	1.03	1.14	1.06	1.24	1.37
0.018	0.72	0.84	0.93	0.94	1.09	1.21
	Flat.	Corrugated.			Flat.	Corrugated.

The above table is calculated for the ordinary size of sheet, which is from 2 to 2½ feet wide and from 6 to 8 feet long, allowing 4 inches lap in length and 2½ inches in width of sheet.

The galvanizing of sheet iron adds about one-third of a pound to its weight per square foot.

TABLE II—*Continued.*

PINE SHINGLES.

The number and weight of pine shingles required to cover one square of roof.

Number of Inches exposed to Weather.	Number of Shingles per Square of Roof.	Weight in Pounds of Shingles on one Square of Roofs.	Remarks.
4	900	216	The number of shingles per square is for common
4½	800	192	gable-roofs. For hip-roofs add five per cent. to these
5	720	173	figures.
5½	655	157	The weights per square are based on the number per
6	600	144	square.

SKYLIGHT GLASS.

The weights of various sizes and thicknesses of fluted or rough plate-glass required for one square of roof.

Dimensions in Inches.	Thickness in Inches.	Area in Square Feet.	Weight in Pounds per Square of Roof.
12 × 48	1$\frac{3}{16}$	3.997	250
15 × 60	$\frac{1}{4}$	6.246	350
20 × 100	$\frac{3}{8}$	13.880	500
94 × 156	$\frac{1}{2}$	101.768	700

In the above table no allowance is made for lap.

If ordinary window-glass is used, single-thick glass (about $\frac{1}{16}''$) will weigh about 82 pounds per square, and double-thick glass (about $\frac{1}{8}''$) will weigh about 164 pounds per square, *no allowance being made for lap.*

TABLE II—*Continued.*

SLATE.

The number and superficial area of slate required for one square of roof.

Dimensions in Inches.	Number per Square.	Superficial Area in Square Feet.	Dimensions in Inches.	Number per Square.	Superficial Area in Square Feet.
6 × 12	533	267	12 × 18	160	240
7 × 12	457	....	10 × 20	169	235
8 × 12	400	....	11 × 20	154	
9 × 12	355	....	12 × 20	141	
7 × 14	374	254	14 × 20	121	
8 × 14	327	....	16 × 20	137	
9 × 14	291	....	12 × 22	126	231
10 × 14	261	....	14 × 22	108	
8 × 16	277	246	12 × 24	114	228
9 × 16	246	....	14 × 24	98	
10 × 16	221	....	16 × 24	86	
9 × 18	213	240	14 × 26	89	225
10 × 18	192	....	16 × 26	78	

As slate is usually laid, the number of square feet of roof covered by one slate can be obtained from the following formula:

$$\frac{\text{Width} \times (\text{length} - 3 \text{ inches})}{288} = \text{the number of square feet of roof covered.}$$

The weight of slate of various lengths and thicknesses required for one square of roof.

Length in Inches.	Weight in pounds, per square, for the thickness.							
	$\frac{1}{8}''$	$\frac{3}{16}''$	$\frac{1}{4}''$	$\frac{3}{8}''$	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$1''$
12	483	724	967	1450	1936	2419	2902	3872
14	460	688	920	1379	1842	2301	2760	3683
16	445	667	890	1336	1784	2229	2670	3567
18	434	650	869	1303	1740	2174	2607	3480
20	425	637	851	1276	1704	2129	2553	3408
22	418	626	836	1254	1675	2093	2508	3350
24	412	617	825	1238	1653	2066	2478	3306
26	407	610	815	1222	1631	2039	2445	3263

The weights given above are based on the number of slate required for one square of roof, taking the weight of a cubic foot of slate at 175 pounds.

TABLE II—*Continued.*

Terra-cotta.

Porous terra-cotta roofing 3″ thick weighs 16 pounds per square foot and 2″ thick, 12 pounds per square foot.

Ceiling made of the same material 2″ thick weighs 11 pounds per square foot.

Tiles.

Flat tiles $6\frac{1}{4}''\times10\frac{1}{2}''\times\frac{5}{8}''$ weigh from 1480 to 1850 pounds per square of roof, the lap being one-half the length of the tile.

Tiles with grooves and fillets weigh from 740 to 925 pounds per square of roof.

Pan-tiles $14\frac{1}{2}''\times10\frac{1}{2}''$ laid 10″ to the weather weigh 850 pounds per square of roof.

Tin.

The usual sizes for roofing tin are 14″×20″ and 20″×28″. Without allowing anything for lap or waste, tin roofing weighs from 50 to 62 pounds per square.

Tin on the roof weighs from 62 to 75 pounds per square.

For preliminary estimates the weights of various roof coverings may be taken as tabulated below:

Name.	Weight in Lbs. per Square of Roof.
Cast-iron plates ($\frac{3}{8}''$ thick)	1500
Copper	80 –125
Felt and asphalt	100
Felt and gravel	800–1000
Iron, corrugated	100– 375
Iron, galvanized flat	100– 350
Lath and plaster	900–1000
Sheathing, pine 1″ thick yellow, northern	300
" " " " southern	400
Spruce 1″ thick	200
Sheathing, chestnut or maple, 1″ thick	400
" ash, hickory or oak, 1″ thick	500
Sheet iron ($\frac{1}{16}''$ thick)	300
" " " and laths	500
Shingles, pine	200
Slates ($\frac{1}{4}''$ thick)	900
Skylights (glass $\frac{3}{16}''$ to $\frac{1}{2}''$ thick)	250– 700
Sheet lead	500– 800
Thatch	650
Tin	70– 125
Tiles, flat	1500–2000
" (grooves and fillets)	700–1000
" pan	1000
" with mortar	2000–3000
Zinc	100– 200

TABLE III.
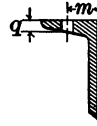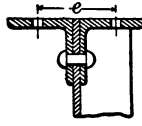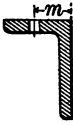
STANDARD SPACING OF RIVET AND BOLT HOLES IN ANGLES AND IN FLANGES AND CONNECTION ANGLES OF CHANNELS.

Angles.		Standard Channels.									
Depth of Leg, Inches.	m in Inches	Depth of Channel, Inches	Weight per Foot, Pounds.	m in Inches	e in Inches	g in Inche	Depth of Channel, Inches	Weight per Foot, Pounds	m in Inches	e in Inches	g in Inches
$\frac{3}{4}$	$\frac{7}{16}$	3	4.0	$1\frac{5}{16}$	$4\frac{1}{16}$	$\frac{1}{4}$	8	18.75	$1\frac{1}{2}$	$4\frac{1}{2}$	$1\frac{1}{32}$
		3	5.0	"	$4\frac{9}{32}$	$\frac{1}{4}$	8	21.25	$1\frac{1}{2}$	$4\frac{9}{32}$	$1\frac{1}{32}$
1	$\frac{9}{16}$	3	6.0	"	$4\frac{3}{8}$	$\frac{9}{32}$					
$1\frac{1}{8}$	$\frac{11}{16}$						9	13.25	$1\frac{3}{8}$	$4\frac{1}{2}$	$1\frac{9}{32}$
$1\frac{5}{16}$	$\frac{3}{4}$	4	5.25	1	$4\frac{7}{8}$	$\frac{9}{32}$	9	15.00	$1\frac{3}{8}$	$4\frac{1}{8}$	$1\frac{13}{32}$
$1\frac{3}{8}$	$\frac{7}{8}$	4	6.25	1	$4\frac{9}{32}$	$\frac{9}{32}$	9	20.00	$1\frac{1}{2}$	$4\frac{9}{32}$	$1\frac{9}{32}$
$1\frac{1}{2}$	$\frac{15}{16}$	4	7.25	1	$4\frac{11}{32}$	$\frac{5}{16}$	9	25.00	$1\frac{3}{4}$	$4\frac{3}{8}$	$1\frac{3}{8}$
$1\frac{3}{4}$	$\frac{15}{16}$	5	6.5	1	$4\frac{7}{32}$	$\frac{5}{16}$	10	15.0	$1\frac{1}{2}$	$4\frac{1}{4}$	$1\frac{7}{16}$
2	$1\frac{1}{8}$	5	9.0	$1\frac{1}{4}$	$4\frac{11}{32}$	$\frac{9}{32}$	10	20.0	$1\frac{1}{2}$	$4\frac{13}{32}$	$1\frac{7}{16}$
$2\frac{1}{4}$	$1\frac{1}{4}$	5	11.5	$1\frac{1}{4}$	$4\frac{1}{2}$	$\frac{5}{16}$	10	25.0	2	$4\frac{17}{32}$	$1\frac{15}{32}$
$2\frac{5}{8}$	$1\frac{1}{2}$						10	30.0	2	$4\frac{11}{32}$	$1\frac{15}{32}$
$2\frac{1}{2}$	$1\frac{3}{8}$	6	8.0	$1\frac{1}{8}$	$4\frac{7}{32}$	$\frac{11}{32}$	10	35.0	2	$4\frac{23}{32}$	$1\frac{1}{16}$
$2\frac{3}{4}$	1	6	10.5	$1\frac{1}{4}$	$4\frac{11}{32}$	$\frac{11}{32}$					
		6	13.0	$1\frac{3}{8}$	$4\frac{15}{32}$	$\frac{11}{32}$	12	20.5	$1\frac{3}{4}$	$4\frac{15}{32}$	$1\frac{15}{32}$
3	$1\frac{3}{4}$	6	15.5	$1\frac{3}{8}$	$4\frac{9}{16}$	$\frac{11}{32}$	12	25.0	$1\frac{3}{4}$	$4\frac{29}{32}$	$\frac{1}{2}$
$3\frac{1}{2}$	2						12	30.0	2	$5\frac{5}{32}$	$1\frac{15}{32}$
4	$2\frac{1}{4}$	7	9.75	$1\frac{1}{4}$	$4\frac{7}{32}$	$\frac{11}{32}$	12	35.0	2	$5\frac{5}{32}$	$\frac{1}{2}$
$4\frac{1}{2}$	$2\frac{1}{2}$	7	12.25	$1\frac{1}{4}$	$4\frac{11}{32}$	$\frac{3}{8}$	12	40.0	2	$5\frac{13}{32}$	$1\frac{7}{32}$
		7	14.75	$1\frac{1}{4}$	$4\frac{7}{16}$	$\frac{3}{8}$					
		7	17.25	$1\frac{1}{2}$	$4\frac{15}{32}$	$\frac{3}{8}$	15	33.0	$1\frac{7}{8}$	$4\frac{29}{32}$	$\frac{31}{32}$
5	$2\frac{3}{4}$	7	19.75	$1\frac{1}{2}$	$4\frac{19}{32}$	$\frac{3}{8}$	15	35.0	$1\frac{7}{8}$	$4\frac{15}{16}$	$\frac{31}{32}$
$5\frac{5}{8}$	$3\frac{1}{4}$						15	40.0	$1\frac{7}{8}$	$5\frac{31}{32}$	$1\frac{1}{32}$
		8	11.25	$1\frac{1}{4}$	$4\frac{1}{4}$	$\frac{3}{8}$	15	45.0	$2\frac{1}{4}$	$5\frac{1}{8}$	$\frac{31}{32}$
6	$3\frac{1}{2}$	8	13.75	$1\frac{1}{4}$	$4\frac{13}{16}$	$1\frac{13}{32}$	15	50.0	$2\frac{1}{4}$	$5\frac{1}{4}$	$\frac{31}{32}$
		8	16.25	$1\frac{1}{2}$	$4\frac{23}{32}$	$\frac{3}{8}$	15	55.0	$2\frac{1}{4}$	$5\frac{11}{32}$	$\frac{31}{32}$

Table III—Continued.

MAXIMUM SIZE OF RIVETS IN BEAMS, CHANNELS, AND ANGLES.

I Beams.						Channels.			Angles.			
Depth of Beam, Inches.	Weight per Foot, Pounds.	Size of Rivet, Inches.	Depth of Beam, Inches.	Weight per Foot, Pounds.	Size of Rivet, Inches.	Depth of Channel, Inches.	Weight per Foot, Pounds.	Size of Rivet, Inches.	Length of Leg, Inches.	Size of Rivet, Inches.	Length of Leg, Inches.	Size of Rivet, Inches.
3	5.5	$\frac{3}{8}$	15	42.0	$\frac{3}{4}$	3	4.0	$\frac{3}{8}$	$\frac{3}{4}$	$\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{8}$
4	7.5	$\frac{1}{2}$	15	60.0	$\frac{3}{4}$	4	5.25	$\frac{1}{2}$	1	$\frac{1}{2}$	$2\frac{3}{4}$	$\frac{3}{4}$
5	9.75	$\frac{1}{2}$	15	80.0	$\frac{3}{4}$	5	6.50	$\frac{1}{2}$	$1\frac{1}{4}$	$\frac{5}{8}$	3	$\frac{3}{4}$
6	12.25	$\frac{5}{8}$	18	55.0	$\frac{3}{4}$	6	8.0	$\frac{5}{8}$	$1\frac{5}{16}$	$\frac{5}{8}$	$3\frac{1}{2}$	1
7	15.0	$\frac{3}{4}$	20	65.0	1	7	9.75	$\frac{5}{8}$	$1\frac{5}{8}$	$\frac{3}{4}$	4	1
8	17.75	$\frac{3}{4}$	20	80.0	1	8	11.25	$\frac{3}{4}$	$1\frac{1}{2}$	$\frac{3}{4}$	$4\frac{1}{2}$	1
9	21.0	$\frac{3}{4}$	24	80.0	1	9	13.25	$\frac{3}{4}$	$1\frac{3}{4}$	$\frac{3}{4}$	5	1
10	25.0	$\frac{3}{4}$				10	15.0	$\frac{3}{4}$	2	$\frac{3}{4}$	$5\frac{5}{8}$	1
12	31.5	$\frac{3}{4}$				12	20.50	$\frac{3}{4}$	$2\frac{1}{4}$	$\frac{3}{4}$	6	1
12	40.0	$\frac{3}{4}$				15	33.0	$\frac{3}{4}$	$2\frac{5}{16}$			

RIVET SPACING.

All dimensions in inches.

Size of Rivets.	Minimum Pitch.	Maximum Pitch at Ends of Compression Members.	Minimum Pitch in Flanges of Chords and Girders.	Distance from Edge of Piece to Centre of Rivet Hole.	
				Minimum.	Usual.
$\frac{1}{2}$	$\frac{3}{4}$				
$\frac{5}{8}$	$1\frac{1}{8}$				
$\frac{3}{4}$	$1\frac{1}{2}$				
$\frac{7}{8}$	$1\frac{7}{8}$	$2\frac{1}{2}$	4	$1\frac{1}{8}$	$1\frac{1}{4}$
$\frac{3}{4}$	$2\frac{1}{4}$	3	4	$1\frac{5}{16}$	$1\frac{1}{2}$
$\frac{7}{8}$	$2\frac{5}{8}$	$3\frac{1}{2}$	4	$1\frac{1}{2}$	$1\frac{3}{4}$
1	3	4	4		2

TABLE IV.

RIVETS.

Tables of Areas in Square Inches, to be deducted from Riveted Plates or Shapes to Obtain Net Areas.

Thickness Plates in Inches.	Size of Hole, in Inches.*													
	¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞	15/16	1	1 1/16
¼	.06	.08	.09	.11	.13	.14	.16	.17	.19	.20	.22	.23	.25	.27
5/16	.08	.10	.12	.14	.15	.18	.20	.21	.23	.25	.27	.29	.31	.33
⅜	.09	.12	.14	.16	.19	.21	.23	.26	.28	.30	.33	.35	.38	.40
7/16	.11	.14	.16	.19	.22	.25	.27	.30	.33	.36	.38	.41	.44	.46
½	.13	.16	.19	.22	.25	.28	.31	.34	.38	.41	.44	.47	.50	.53
9/16	.14	.18	.21	.25	.28	.32	.35	.39	.42	.46	.49	.53	.56	.60
⅝	.16	.20	.23	.27	.31	.35	.39	.43	.47	.51	.55	.59	.63	.66
11/16	.17	.21	.26	.30	.34	.39	.43	.47	.52	.56	.60	.64	.69	.73
¾	.19	.23	.28	.33	.38	.42	.47	.52	.56	.61	.66	.70	.75	.80
13/16	.20	.25	.30	.36	.41	.46	.51	.56	.61	.66	.71	.76	.81	.86
⅞	.22	.27	.33	.38	.44	.49	.55	.60	.66	.71	.77	.82	.88	.93
15/16	.23	.29	.35	.41	.47	.53	.59	.64	.70	.76	.82	.88	.94	1.00
1	.25	.31	.38	.44	.50	.56	.63	.69	.75	.81	.88	.94	1.00	1.06
1 1/16	.27	.33	.40	.46	.53	.60	.66	.73	.80	.86	.93	1.00	1.06	1.13
1⅛	.28	.35	.42	.49	.56	.63	.70	.77	.84	.91	.98	1.05	1.13	1.20
1 3/16	.30	.37	.45	.52	.59	.67	.74	.82	.89	.96	1.04	1.11	1.19	1.26
1¼	.31	.39	.47	.55	.63	.70	.78	.86	.94	1.02	1.09	1.17	1.25	1.33
1 5/16	.33	.41	.49	.57	.66	.74	.81	.90	.98	1.07	1.15	1.23	1.31	1.39
1⅜	.34	.43	.52	.60	.69	.77	.86	.95	1.03	1.12	1.20	1.29	1.38	1.46
1 7/16	.36	.45	.54	.63	.72	.81	.90	.99	1.08	1.17	1.26	1.35	1.44	1.53
1½	.38	.47	.56	.66	.75	.84	.94	1.03	1.13	1.22	1.31	1.41	1.50	1.59
1 9/16	.39	.49	.59	.68	.78	.88	.98	1.07	1.17	1.27	1.37	1.46	1.56	1.66
1⅝	.41	.51	.61	.71	.81	.91	1.02	1.12	1.22	1.32	1.42	1.52	1.63	1.73
1 11/16	.42	.53	.63	.74	.84	.95	1.05	1.16	1.27	1.37	1.47	1.58	1.69	1.79
1¾	.44	.55	.66	.77	.88	.98	1.09	1.20	1.31	1.42	1.53	1.64	1.75	1.86
1 13/16	.45	.57	.68	.79	.91	1.02	1.13	1.25	1.36	1.47	1.59	1.70	1.81	1.93
1⅞	.47	.59	.70	.82	.94	1.05	1.17	1.29	1.41	1.52	1.64	1.76	1.88	1.99
1 15/16	.48	.61	.73	.85	.97	1.09	1.21	1.33	1.45	1.57	1.70	1.82	1.94	2.06
2	.50	.63	.75	.88	1.00	1.13	1.25	1.38	1.50	1.63	1.75	1.88	2.00	2.13

* Size of hole = diameter of rivet + ⅛″.

TABLE V.

WEIGHTS OF ROUND–HEADED RIVETS AND ROUND-HEADED BOLTS WITHOUT NUTS.

Wrought Iron.

Basis: 1 cubic foot iron =480 pounds. For steel add 2%.

Length under Head to Point. Inches.	Diameter of Rivet in Inches.						
	⅜	½	⅝	¾	⅞	1	1⅛
1	4.7	9.3	16.0	25.2	37.2	52.6	71.3
1¼	5.5	10.7	18.1	28.3	41.3	58.0	78.2
1½	6.2	12.1	20.2	31.3	45.5	63.5	85.1
1¾	7.0	13.4	22.4	34.4	49.7	68.9	92.0
2	7.8	14.8	24.5	37.5	53.9	74.4	98.9
2¼	8.5	16.2	26.6	40.5	58.0	79.8	105.8
2½	9.3	17.5	28.8	43.6	62.2	85.3	112.7
2¾	10.1	18.9	30.9	46.7	66.4	90.7	119.6
3	10.8	20.3	33.0	49.8	70.6	96.2	126.5
3¼	11.6	21.6	35.1	52.8	74.7	102.6	133.4
3½	12.4	23.0	37.3	55.9	78.9	107.1	140.3
3¾	13.1	24.3	39.4	59.0	83.1	112.6	147.2
4	13.9	25.7	41.5	62.0	87.3	118.0	154.1
4¼	14.7	27.1	43.7	65.1	91.4	123.5	161.0
4½	15.4	28.4	45.8	68.2	95.6	128.9	167.9
4¾	16.2	29.8	47.9	71.2	99.8	134.4	174.8
5	17.0	31.2	50.1	74.3	104.0	139.8	181.7
5¼	17.7	32.5	52.2	77.4	108.2	145.3	188.6
5½	18.5	33.9	54.3	80.4	112.3	150.7	195.6
5¾	19.3	35.3	56.4	83.5	116.5	156.2	202.5
6	20.0	36.6	58.6	86.6	120.7	161.6	209.4
6¼	20.8	38.0	60.7	89.6	124.8	167.1	216.3
6½	21.6	39.3	62.8	92.7	129.0	172.5	223.2
6¾	22.3	40.7	65.0	95.8	133.2	178.0	230.1
7	23.1	42.1	67.1	98.8	137.4	183.5	237.0
7¼	23.9	43.4	69.2	101.9	141.6	188.9	243.9
7½	24.6	44.8	71.4	105.0	145.7	194.4	250.8
7¾	25.4	46.2	73.5	108.0	149.9	199.8	257.7
8	26.2	47.5	75.6	111.1	154.1	205.3	264.6
8½	27.7·	50.2	79.9	117.2	162.2	216.2	278.4
9	29.2	53.0	84.1	123.4	170.8	227.1	292.2
9½	30.8	55.7	88.4	129.5	179.1	238.0	306.0
10	32.3	58.4	92.7	135.6	187.5	248.8	319.8
10½	33.8	61.2	96.9	141.8	195.8	259.8	333.6
11	35.4	63.9	101.2	147.9	204.2	270.7	347.4
11½	36.9	66.6	105.4	154.1	212.5	281.6	361.2
12	38.	69.3	109.7	160.2	220.9	292.5	375.0
One inch in length of 100 Rivets	3.07	5.45	8.52	12.27	16.70	21.82	27.61
Weight of 100 Rivet Heads.....	1.78	4.82	9.95	16.12	24.29	34.77	47.67

Height of rivet head = ⁶⁄₁₀ diameter of rivet.

TABLE VI.

WEIGHTS AND DIMENSIONS OF BOLT HEADS.

Manufacturers' Standard Sizes

Basis: Hoopes & Townsend's List.

Diameter of Bolt.	SQUARE.				HEXAGON.			
	Short Diameter	Long Diameter	Thickness.	Weight per 100.	Short Diameter	Long Diameter	Thickness.	Weight per 100.
Inches.	Inches.	Inches.	Inches.	Pounds.	Inches.	Inches.	Inches.	Pounds.
¼	7/16	.619	3/16	1.0	7/16	.505		.9
5/16	½	.707	¼	1.7	½	.578	¼	1.5
⅜	19/32	.840	9/32	2.8	19/32	.686		2.4
7/16	11/16	.972	⅜	4.9	11/16	.794	⅜	4.3
½	¾	1.061	7/16	6.8	¾	.866		5.9
9/16	27/32	1.193	½	9.9	27/32	.974	½	8.6
⅝	15/16	1.326	17/32	13.0	15/16	1.083		11.2
¾	1⅛	1.591	⅝	22.0	1⅛	1.299	⅝	19.0
⅞	1 5/16	1.856	¾	34.8	1 5/16	1.516	¾	33.1
1	1½	2.122	⅞	54.7	1½	1.733	⅞	47.4
1⅛	1⅝	2.298	1	73.3	1⅝	1.877	1	63.5
1¼	1¾	2.475	1⅛	95.7	1¾	2.021	1⅛	82.9
1⅜	2⅛	3.006	1¼	156.8	2	2.309	1⅜	132.3
1½	2⅜	3.359	1⅜	215.4	2⅜	2.743	1½	203.5
1⅝	2½	3.536	1½	260.3	2½	2.888	1⅝	244.4
1¾	2¾	3.889	1⅝	341.3	2¾	3.176	1¾	318.4
1⅞	3	4.243	1¾	437.4	3	3.464	1⅞	408.2
2	3⅛	4.420	1⅞	508.5	3⅛	3.610	2	469.9

Approximate rules for dimensions of *finished* nuts and heads for bolts (square and hexagon)·

Short diameter of nut = 1½ diameter of bolt;

Thickness of nut = 1 diameter of bolt;

Short diameter of head = 1½ diameter of bolt;

Thickness of head = 1 diameter of bolt;

Long diameter of square nut or head = 2.12 diameter of bolt;

    "   "   " hexagon nut or head = 1.73 diameter of of bolt.

TABLE VII.

UPSET SCREW ENDS.

Round Bars.
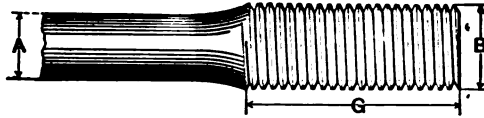
DIMENSIONS OF UPSET END.				DIMENSIONS AND PROPORTIONS OF BODY OF BAR.									
Diameter of Screw.	Length of Upset.	Area at Root of Thread.	Number of Threads per Inch.	Diameter of Bar.	Area at Body of Bar.	Weight per Foot of Bar.	Add for Upset.	Excess of Area of Thread over that of Body of Bar.	Diameter of Bar.	Area of Body of Bar.	Weight per Foot of Bar.	Add for Upset.	Excess of Area at Root of Thread over that of Body of Bar.
B	G			A					A				
In.	In.	Sq. In		In.	Sq. In.	Lbs.	In.	PrCt.	In.	Sq. In.	Lbs.	In.	Pr.Ct.
¾	4¼	.302	10	½	.196	1.668	6½	54	1 9/16	.249	.845	4¼	21
⅞	4½	.420	9	9/16	.307	1.043	5½	37					
1	4½	.550	8	11/16	.371	1.262	6¼	48	¾	.442	1.502	4½	25
1⅛	4¾	.694	7	13/16	.519	1.763	5½	34					
1¼	4¾	.893	7	⅞	.601	2.044	6¼	49	1⅛	.690	2.347	4½	29
1⅜	5	1.057	6	1	.785	2.67	4½	35	1 1/16	.887	3.01	4¼	19
1½	5	1.295	6	1⅛	.994	3.38	4½	30	1 1/16	1.108	3.77	3½	17
1⅝	5¼	1.515	5½	1¼	1.227	4.17	4½	23					
1¾	5¼	1.744	5	1 5/16	1.353	4.60	5	29	1⅜	1.485	5.05	4	18
1⅞	5½	2.048	5	1 7/16	1.623	5.52	4¾	26					
2	5½	2.302	4½	1½	1.767	6.01	5¼	30	1 9/16	1.918	6.52	4½	20
2¼	5½	2.650	4½	1⅝	2.074	7.05	5	28	1 11/16	2.237	7.60	4¼	18
2¼	5½	3.023	4½	1¾	2.405	8.18	4½	26	1 11/16	2.580	8.77	4	17
2⅜	6	3.419	4½	1⅞	2.761	9.39	4½	24					
2¼	6	3.715	4	1 15/16	2.948	10.02	5	26	2	3.142	10.68	3½	18
2⅝	6¼	4.155	4	2 1/16	3.341	11.36	4½	24	2⅛	3.547	12.06	4	17
2¼	6¼	4.619	4	2 1/16	3.758	12.78	4½	23					
2⅞	6¼	5.108	4	2¼	3.976	13.52	5¼	28	2 1/16	4.200	14.28	4½	22
3	6½	5.428	3½	2⅜	4.430	15.07	4½	23					
3⅛	6¾	5.957	3½	2 7/16	4.666	15.86	5½	28	2¼	4.909	16.69	4¾	21
3¼	6¾	6.510	3½	2 1/16	5.157	17.53	5¼	26	2⅝	5.412	18.40	4½	20
3⅜	7	7.087	3½	2 11/16	5.673	19.29	5	25	2¾	5.940	20.20	4½	19
3½	7	7.548	3½	2 11/16	6.231	21.12	4½	22					
3⅝	7¼	8.171	3½	2⅞	6.492	22.07	5¼	26	2 11/16	6.777	23.04	4¾	21
3¾	7¼	8.641	3	3	7.069	24.03	6¼	22					
3⅞	7½	9.305	3	3⅛	7.670	26.08	5¼	21					
4	7½	9.993	3	3¼	8.296	28.20	4½	20					

TABLE VIII.

RIGHT AND LEFT NUTS.
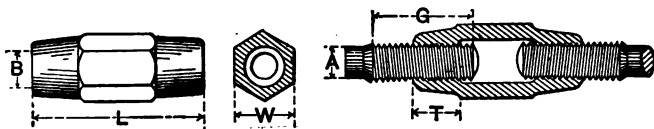
Dimensions of Nuts from Edge Moor Bridge Works' Standard.

Diameter of Screw.	Length of Upset.	Diameter of Bar.	Side of Square Bar.	Length of Nut.	Length of Thread.	Diameter of Hex.	Weight of One Nut.	Weight of One Nut and Two Screw Ends.
B	**G**	**A**	**A**	**L**	**T**	**W**		
Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Pounds.	Lbs.
$\frac{7}{8}$	$4\frac{1}{2}$	$\frac{5}{8}$	$\frac{9}{16}$	6	$1\frac{7}{16}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$4\frac{1}{4}$
1	$4\frac{1}{2}$	$\frac{11}{16}$ and $\frac{3}{4}$	$\frac{5}{8}$ and $\frac{11}{16}$	6	$1\frac{7}{16}$	$1\frac{5}{8}$	$1\frac{1}{4}$	$4\frac{1}{4}$
$1\frac{1}{8}$	$4\frac{5}{8}$	$\frac{13}{16}$		$\frac{3}{4}$	$6\frac{1}{4}$	$1\frac{5}{8}$	2	3	$7\frac{1}{4}$
$1\frac{1}{4}$	$4\frac{3}{4}$	$\frac{7}{8}$	" $\frac{15}{16}$	$1\frac{1}{8}$	$6\frac{1}{4}$	$1\frac{5}{8}$	2	3	$7\frac{1}{2}$
$1\frac{3}{8}$	5	1	" $1\frac{1}{16}$	$\frac{7}{8}$	" $1\frac{3}{16}$	7	$1\frac{7}{8}$	$2\frac{3}{8}$	$4\frac{3}{4}$	$11\frac{1}{4}$
$1\frac{1}{2}$	5	$1\frac{1}{8}$	" $1\frac{3}{16}$	1		7	$1\frac{7}{8}$	$2\frac{3}{8}$	$4\frac{1}{4}$	$11\frac{1}{4}$
$1\frac{5}{8}$	$5\frac{1}{4}$	$1\frac{1}{4}$		$1\frac{1}{16}$ " $1\frac{1}{8}$	$7\frac{1}{4}$	$2\frac{1}{16}$	$2\frac{1}{4}$	$6\frac{1}{4}$	$16\frac{3}{4}$
$1\frac{3}{4}$	$5\frac{1}{4}$	$1\frac{5}{16}$		$1\frac{3}{16}$	$7\frac{1}{2}$	$2\frac{1}{16}$	$2\frac{1}{4}$	$6\frac{1}{2}$	$16\frac{3}{4}$
$1\frac{7}{8}$	$5\frac{1}{2}$	$1\frac{7}{16}$ " $1\frac{5}{8}$	$1\frac{1}{4}$ " $1\frac{5}{16}$	8	$2\frac{1}{16}$	$3\frac{1}{8}$	$9\frac{1}{4}$	$23\frac{1}{4}$
2	$5\frac{1}{2}$	$1\frac{1}{2}$ " $1\frac{9}{16}$	$1\frac{3}{8}$	8	$2\frac{5}{16}$	$3\frac{1}{8}$	$9\frac{1}{4}$	$23\frac{1}{4}$
$2\frac{1}{8}$	$5\frac{3}{4}$	$1\frac{5}{8}$ " $1\frac{11}{16}$	$1\frac{7}{16}$ " $1\frac{1}{2}$	$8\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{1}{4}$	$12\frac{1}{2}$	$31\frac{1}{2}$
$2\frac{1}{4}$	$5\frac{3}{4}$	$1\frac{3}{4}$ " $1\frac{13}{16}$	$1\frac{9}{16}$	$8\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{1}{4}$	$12\frac{1}{2}$	$31\frac{1}{2}$
$2\frac{3}{8}$	6	$1\frac{7}{8}$	$1\frac{5}{8}$ " $1\frac{11}{16}$	9	$2\frac{3}{4}$	$3\frac{3}{8}$	$16\frac{1}{4}$	$41\frac{3}{4}$
$2\frac{1}{2}$	6	$1\frac{15}{16}$ " 2	$1\frac{3}{4}$	9	$2\frac{3}{4}$	$3\frac{3}{8}$	$16\frac{1}{4}$	$41\frac{3}{4}$
$2\frac{5}{8}$	$6\frac{1}{4}$	$2\frac{1}{16}$ " $2\frac{1}{8}$	$1\frac{13}{16}$ " $1\frac{7}{8}$	$9\frac{1}{2}$	$2\frac{15}{16}$	$4\frac{1}{4}$	$21\frac{1}{2}$	$53\frac{1}{4}$
$2\frac{3}{4}$	$6\frac{1}{4}$	$2\frac{3}{16}$	$1\frac{15}{16}$	$9\frac{1}{2}$	$2\frac{15}{16}$	$4\frac{1}{4}$	$21\frac{1}{2}$	$53\frac{1}{4}$
$2\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{4}$ " $2\frac{5}{16}$	2 " $2\frac{1}{16}$	10	$3\frac{1}{16}$	$4\frac{5}{8}$	$26\frac{1}{2}$	$66\frac{1}{4}$
3	$6\frac{1}{2}$	$2\frac{3}{8}$	$2\frac{1}{8}$	10	$3\frac{1}{16}$	$4\frac{5}{8}$	$26\frac{1}{2}$	$66\frac{1}{4}$
$3\frac{1}{4}$	$6\frac{3}{4}$	$2\frac{9}{16}$ " $2\frac{5}{8}$	$2\frac{5}{16}$	$10\frac{1}{2}$	$3\frac{3}{8}$	5	32	81
$3\frac{1}{2}$	7	$2\frac{11}{16}$	$2\frac{7}{16}$	11	$3\frac{3}{8}$	$5\frac{3}{8}$	$38\frac{1}{2}$	$97\frac{1}{2}$
$3\frac{3}{4}$	$7\frac{1}{4}$	3	$2\frac{11}{16}$	$11\frac{1}{2}$	$3\frac{11}{16}$	$5\frac{3}{4}$	45	116
4	$7\frac{1}{2}$	$3\frac{1}{4}$	$2\frac{7}{8}$	12	$4\frac{1}{16}$	$6\frac{1}{4}$	$53\frac{1}{2}$	138

TABLE IX.

PROPERTIES OF STANDARD I BEAMS.

1	2	3	4	5	6	7	8	9	10	11
Section Number.	Depth of Beam.	Weight per Foot.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia Axis 1–1.	Section Modulus Axis 1–1.	Radius of Gyration Axis 1–1.	Moment of Inertia Axis 2–2.	Radius of Gyration Axis 2–2.
	d		A	t	b	I	S	r	I'	r'
	ches.	Pounds.	Sq. Inches.	Inches.	Inches.	Inches.⁴	Inches.³	Inches.	Inches.⁴	Inches.
B 5	3	5.5	1.63	.17	2.33	2.5	1.7	1.23	.46	.53
B 5	3	6.5	1.91	.26	2.42	2.7	1.8	1.19	.53	.52
B 5	3	7.5	2.21	.36	2.52	2.9	1.9	1.15	.60	.52
B 9	4	7.5	2.21	.19	2.66	6.0	3.0	1.64	.77	.59
B 9	4	8.5	2.50	.26	2.73	6.4	3.2	1.59	.85	.58
B 9	4	9.5	2.79	.34	2.81	6.7	3.4	1.54	.93	.58
B 9	4	10.5	3.09	.41	2.88	7.1	3.6	1.52	1.01	.57
B 13	5	9.75	2.87	.21	3.00	12.1	4.8	2.05	1.23	.65
B 13	5	12.25	3.60	.36	3.15	13.6	5.4	1.94	1.45	.63
B 13	5	14.75	4.34	.50	3.29	15.1	6.1	1.87	1.70	.63
B 17	6	12.25	3.61	.23	3.33	21.8	7.3	2.46	1.85	.72
B 17	6	14.75	4.34	.35	3.45	24.0	8.0	2.35	2.09	.69
B 17	6	17.25	5.07	.47	3.57	26.2	8.7	2.27	2.36	.68
B 21	7	15.0	4.42	.25	3.66	36.2	10.4	2.86	2.67	.78
B 21	7	17.5	5.15	.35	3.76	39.2	11.2	2.76	2.94	.76
B 21	7	20.0	5.88	.46	3.87	42.2	12.1	2.68	3.24	.74
B 25	8	17.75	5.33	.27	4.00	56.9	14.2	3.27	3.78	.84
B 25	8	20.25	5.96	.35	4.08	60.2	15.0	3.18	4.04	.82
B 25	8	22.75	6.69	.44	4.17	64.1	16.0	3.10	4.36	.81
B 25	8	25.25	7.43	.53	4.26	68.0	17.0	3.03	4.71	.80

TABLE IX—*Continued.*

PROPERTIES OF STANDARD I BEAMS.

1	2	3	4	5	6	7	8	9	10	11
Section Number.	Depth of Beam.	Weight per Foot.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia Axis 1–1.	Section Modulus Axis 1–1.	Radius of Gyration Axis 1–1.	Moment of Inertia Axis 2–2.	Radius of Gyration Axis 2–2.
	d		A	t	b	I	S	r	I′	r′
Inches.	Inches.	Pounds.	Sq. Inches.	Inches.	Inches.	Inches.⁴	Inches.³	Inches.	Inches.⁴	Inches.
B 29	9	21.0	6.31	.29	4.33	84.9	18.9	3.67	5.16	.90
B 29	9	25.0	7.35	.41	4.45	91.9	20.4	3.54	5.65	.88
B 29	9	30.0	8.82	.57	4.61	101.9	22.6	3.40	6.42	.85
B 29	9	35.0	10.29	.73	4.77	111.8	24.8	3.30	7.31	.84
B 33	10	25.0	7.37	.31	4.66	122.1	24.4	4.07	6.89	.97
B 33	10	30.0	8.82	.45	4.80	134.2	26.8	3.90	7.65	.93
B 33	10	35.0	10.29	.60	4.95	146.4	29.3	3.77	8.52	.91
B 33	10	40.0	11.76	.75	5.10	158.7	31.7	3.67	9.50	.90
B 41	12	31.5	9.26	.35	5.00	215.8	36.0	4.83	9.50	1.01
B 41	12	35.0	10.29	.44	5.09	228.3	38.0	4.71	10.07	.99
B 41	12	40.0	11.76	.56	5.21	245.9	41.0	4.57	10.95	.96
B 53	15	42.0	12.48	.41	5.50	441.8	58.9	5.95	14.62	1.08
B 53	15	45.0	13.24	.46	5.55	455.8	60.8	5.87	15.09	1.07
B 53	15	50.0	14.71	.56	5.65	483.4	64.5	5.73	16.04	1.04
B 53	15	55.0	16.18	.66	5.75	511.0	68.1	5.62	17.06	1.03
B 53	15	60.0	17.65	.75	5.84	538.6	71.8	5.52	18.17	1.01
B 65	18	55.0	15.93	.46	6.00	795.6	88.4	7.07	21.19	1.15
B 65	18	60.0	17.65	.56	6.10	841.8	93.5	6.91	22.38	1.13
B 65	18	65.0	19.12	.64	6.18	881.5	97.9	6.79	23.47	1.11
B 65	18	70.0	20.59	.72	6.26	921.2	102.4	6.69	24.62	1.09
B 73	20	65.0	19.08	.50	6.25	1169.5	117.0	7.83	27.86	1.21
B 73	20	70.0	20.59	.58	6.33	1219.8	122.0	7.70	29.04	1.19
B 73	20	75.0	22.06	.65	6.40	1268.8	126.9	7.58	30.25	1.17
B 89	24	80.0	23.32	.50	7.00	2087.2	173.9	9.46	42.86	1.36
B 89	24	85.0	25.00	.57	7.07	2167.8	180.7	9.31	44.35	1.33
B 89	24	90.0	26.47	.63	7.13	2238.4	186.5	9.20	45.70	1.31
B 89	24	95.0	27.94	.69	7.19	2309.0	192.4	9.09	77.10	1.30
B 89	24	100.0	29.41	.75	7.25	2379.6	198.3	8.99	48.55	1.28

TABLE X.

PROPERTIES OF STANDARD CHANNELS.

1	2	3	4	5	6	7	8	9	10	11	12	13
Section Number.	Depth of Channel.	Weight per Foot.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia Axis 1-1.	Section Modulus Axis 1-1.	Radius of Gyration Axis 1-1.	Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Radius of Gyration Axis 2-2.	Distance of Centre of Gravity from Outside of Web.
	d		A	t	b	I	S	r	I'	S'	r'	x
Ins.		Lbs.	Sq. In.	Inches	Inches	Ins.⁴	Ins.³	Inches	Ins.⁴	Ins.³	Inches	Inches
C 5	3	4.00	1.19	.17	1.41	1.6	1.1	1.17	.20	.21	.41	.44
C 5	3	5.00	1.47	.26	1.50	1.8	1.2	1.12	.25	.24	.41	.44
C 5	3	6.00	1.76	.36	1.60	2.1	1.4	1.08	.31	.27	.42	.46
C 9	4	5.25	1.55	.18	1.58	3.8	1.9	1.56	.32	.29	.45	.46
C 9	4	6.25	1.84	.25	1.65	4.2	2.1	1.51	.38	.32	.45	.46
C 9	4	7.25	2.13	.33	1.73	4.6	2.3	1.46	.44	.35	.46	.46
C 13	5	6.50	1.95	.19	1.75	7.4	3.0	1.95	.48	.38	.50	.49
C 13	5	9.00	2.65	.33	1.89	8.9	3.5	1.83	.64	.45	.49	.48
C 13	5	11.50	3.38	.48	2.04	10.4	4.2	1.75	.82	.54	.49	.51
C 17	6	8.00	2.38	.20	1.92	13.0	4.3	2.34	.70	.50	.54	.52
C 17	6	10.50	3.09	.32	2.04	15.1	5.0	2.21	.88	.57	.53	.50
C 17	6	13.00	3.82	.44	2.16	17.3	5.8	2.13	1.07	.65	.53	.52
C 17	6	15.50	4.56	.56	2.28	19.5	6.5	2.07	1.28	.74	.53	.55
C 21	7	9.75	2.85	.21	2.09	21.1	6.0	2.72	.98	.63	.59	.55
C 21	7	12.25	3.60	.32	2.20	24.2	6.9	2.59	1.19	.71	.57	.53
C 21	7	14.75	4.34	.42	2.30	27.2	7.8	2.50	1.40	.79	.57	.53
C 21	7	17.25	5.07	.53	2.41	30.2	8.6	2.44	1.62	.87	.56	.55
C 21	7	19.75	5.81	.63	2.51	33.2	9.5	2.39	1.85	.96	.56	.58
C 25	8	11.25	3.35	.22	2.26	32.3	8.1	3.10	1.33	.79	.63	.58
C 25	8	13.75	4.04	.31	2.35	36.0	9.0	2.98	1.55	.87	.62	.56
C 25	8	16.25	4.78	.40	2.44	39.9	10.0	2.89	1.78	.95	.61	.56
C 25	8	18.75	5.51	.49	2.53	43.8	11.0	2.82	2.01	1.02	.60	.57
C 25	8	21.25	6.25	.58	2.62	47.8	11.9	2.76	2.25	1.11	.60	.59

TABLE X—*Continued.*
PROPERTIES OF STANDARD CHANNELS.

1	2	3	4	5	6	7	8	9	10	11	12	13
Section Number.	Depth of Channel.	Weight per Foot.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia Axis 1-1.	Section Modulus Axis 1-1.	Radius of Gyration Axis 1-1.	Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Radius of Gyration Axis 2-2.	Distance of Centre of Gravity from Outside of Web.
	d		A	t	b	I	S	r	I'	S'	r'	x
	Ins.	Lbs.	Sq. In.	Inches	Inches	Ins.^{4}	Ins.^{3}	Inches	Ins.^{4}	Ins.^{3}	Inches	Inches
C 29	9	13.25	3.89	.23	2.43	47.3	10.5	3.49	1.77	.97	.67	.61
C 29	9	15.00	4.41	.29	2.49	50.9	11.3	3.40	1.95	1.03	.66	.59
C 29	9	20.00	5.88	.45	2.65	60.8	13.5	3.21	2.45	1.19	.65	.58
C 29	9	25.00	7.35	.61	2.81	70.7	15.7	3.10	2.98	1.36	.64	.62
C 33	10	15.00	4.46	.24	2.60	66.9	13.4	3.87	2.30	1.17	.72	.64
C 33	10	20.00	5.88	.38	2.74	78.7	15.7	3.66	2.85	1.34	.70	.61
C 33	10	25.00	7.35	.53	2.89	91.0	18.2	3.52	3.40	1.50	.68	.62
C 33	10	30.00	8.82	.68	3.04	103.2	20.6	3.42	3.99	1.67	.67	.65
C 33	10	35.00	10.29	.82	3.18	115.5	23.1	3.35	4.66	1.87	.67	.69
C 41	12	20.50	6.03	.28	2.94	128.1	21.4	4.61	3.91	1.75	.81	.70
C 41	12	25.00	7.35	.39	3.05	144.0	24.0	4.43	4.53	1.91	.78	.68
C 41	12	30.00	8.82	.51	3.17	161.6	26.9	4.28	5.21	2.09	.77	.68
C 41	12	35.00	10.29	.64	3.30	179.3	29.9	4.17	5.90	2.27	.76	.69
C 41	12	40.00	11.76	.76	3.42	196.9	32.8	4.09	6.63	2.46	.75	.72
C 53	15	33.00	9.90	.40	3.40	312.6	41.7	5.62	8.23	3.16	.91	.79
C 53	15	35.00	10.29	.43	3.43	319.9	42.7	5.57	8.48	3.22	.91	.79
C 53	15	40.00	11.76	.52	3.52	347.5	46.3	5.44	9.39	3.43	.89	.78
C 53	15	45.00	13.24	.62	3.62	375.1	50.0	5.32	10.29	3.63	.88	.79
C 53	15	50.00	14.71	.72	3.72	402.7	53.7	5.23	11.22	3.85	.87	.80
C 53	15	55.00	16.18	.82	3.82	430.2	57.4	5.16	12.19	4.07	.87	.82

TABLES.

Table XI.

PROPERTIES OF STANDARD ANGLES.

1	2	3	4	5	6	7	8	9	10	11	12	13
Section Number.	Dimensions.	Thickness.	Weight per Foot.	Area of Section.	Distance of Centre of Gravity from Back of Flange.	Moment of Inertia Axis 1-1.	Section Modulus Axis 1-1.	Radius of Gyration Axis 1-1.	Distance of Cen. of Grav. from Ext. Apex on Line Inclined at 45° to Flange.	Least Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Least Radius of Gyration Axis 2-2.
	a×a	t	A		x	I	S	r	x″	I″	S″	r″
	Inches.	Ins.	Lbs.	Sq.In	Inches	Ins.⁴	Ins.³	Inches	Inches	Ins.⁴	Ins.³	Ins.
A 5	¾×¾	⅛	.58	.17	.23	.009	.017	.22	.33	.004	.011	.14
A 5	¾×	3/16	.84	.25	.25	.012	.024	.22	.36	.005	.014	.14
A 7	1 ×1	⅛	.80	.23	.30	.022	.031	.30	.42	.009	.021	.19
A 7	1 ×1	3/16	1.16	.34	.32	.030	.044	.30	.45	.013	.028	.19
A 7	1 ×1	¼	1.49	.44	.34	.037	.056	.29	.48	.016	.034	.19
A 9	1¼×1¼	⅛	1.02	.30	.36	.044	.049	.38	.51	.018	.035	.24
A 9	1¼×1¼	3/16	1.47	.43	.38	.061	.071	.38	.54	.025	.047	.24
A 9	1¼×1¼	¼	1.91	.56	.40	.077	.091	.37	.57	.033	.057	.24
A 9	1¼×1¼	5/16	2.32	.68	.42	.090	.109	.36	.60	.040	.066	.24
A 11	1½×1½	3/16	1.79	.53	.44	.11	.104	.46	.63	.045	.072	.29
A 11	1½×1½	¼	2.34	.69	.47	.14	.134	.45	.66	.058	.088	.29
A 11	1½×1½	5/16	2.86	.84	.49	.16	.162	.44	.69	.070	.101	.29
A 11	1½×1½	⅜	3.35	.98	.51	.19	.188	.44	.72	.082	.114	.29
A 13	1¾×1¾	3/16	2.11	.62	.51	.18	.14	.54	.72	.073	.10	.34
A 13	1¾×1¾	¼	2.77	.81	.53	.23	.19	.53	.75	.094	.13	.34
A 13	1¾×1¾	5/16	3.39	1.00	.55	.27	.23	.52	.78	.113	.15	.34
A 13	1¾×1¾	⅜	3.98	1.17	.57	.31	.26	.51	.81	.133	.16	.34
A 13	1¾×1¾	7/16	4.56	1.34	.59	.35	.30	.51	.84	.152	.18	.34
A 15	2 × 2	3/16	2.43	.71	.57	.27	.19	.62	.80	.11	.14	.39
A 15	2 ×2	¼	3.19	.94	.59	.35	.25	.61	.84	.14	.17	.39
A 15	2 ×2	5/16	3.92	1.15	.61	.42	.30	.60	.87	.17	.20	.39
A 15	2 ×2	⅜	4.62	1.36	.64	.48	.35	.59	.90	.20	.22	.39
A 15	2 ×2	7/16	5.30	1.56	.66	.54	.40	.59	.93	.23	.25	.38
A 17	2½×2½	¼	4.0	1.19	.72	.70	.39	.77	1.01	.29	.28	.49
A 17	2½×2½	5/16	5.0	1.46	.74	.85	.48	.76	1.05	.35	.33	.49
A 17	2½×2½	⅜	5.9	1.73	.76	.98	.57	.75	1.08	.41	.38	.48
A 17	2½×2½	7/16	6.8	2.00	.78	1.11	.65	.75	1.11	.46	.42	.48
A 17	2½×2½	½	7.7	2.25	.81	1.23	.72	.74	1.14	.52	.46	.48

TABLE XI—*Continued.*
PROPERTIES OF STANDARD ANGLES.

1	2	3	4	5	6	7	8	9	10	11	12	13
Section Number.	Dimensions.	Thickness.	Weight per Foot.	Area of Section.	Distance of Centre of Gravity from Back of Flange.	Moment of Inertia Axis 1-1.	Section Modulus Axis 1-1.	Radius of Gyration Axis 1-1.	Distance of Cen. of Grav. from Ext. Apex on Line Inclined at 45° to Flange.	Least Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Least Radius of Gyration Axis 2-2.
	a×a	t		A	x	I	S	r	x″	I″	S″	r″
	Inches	Ins.	Lbs.	Sq.In	Inches	Ins.⁴	Ins.³	Inches	Inches	Ins.⁴	Ins.³	Inches
A 19	3 ×3	¼	4.9	1.44	.84	1.24	.58	.93	1.19	.50	.42	.59
A 19	3 ×3	5/16	6.0	1.78	.87	1.51	.71	.92	1.22	.61	.50	.59
A 19	3 ×3	3/8	7.2	2.11	.89	1.76	.83	.91	1.26	.72	.57	.58
A 19	3 ×3	7/16	8.3	2.43	.91	1.99	.95	.91	1.29	.82	.64	.58
A 19	3 ×3	½	9.4	2.75	.93	2.22	1.07	.90	1.32	.92	.70	.58
A 19	3 ×3	9/16	10.4	3.06	.95	2.43	1.19	.89	1.35	1.02	.76	.58
A 19	3 ×3	5/8	11.4	3.36	.98	2.62	1.30	.88	1.38	1.12	.81	.58
A 21	3½×3½	3/8	8.4	2.48	1.01	2.87	1.15	1.07	1.43	1.16	.81	.68
A 21	3½×3½	7/16	9.8	2.87	1.04	3.26	1.32	1.07	1.46	1.33	.91	.68
A 21	3½×3½	½	11.1	3.25	1.06	3.64	1.49	1.06	1.50	1.50	1.00	.68
A 21	3½×3½	9/16	12.3	3.62	1.08	3.99	1.65	1.05	1.53	1.66	1.09	.68
A 21	3½×3½	5/8	13.5	3.98	1.10	4.33	1.81	1.04	1.56	1.82	1.17	.68
A 21	3½×3½	11/16	14.8	4.34	1.12	4.65	1.96	1.04	1.59	1.97	1.24	.67
A 21	3½×3½	¾	15.9	4.69	1.15	4.96	2.11	1.03	1.62	2.13	1.31	.67
A 21	3½×3½	13/16	17.1	5.03	1.17	5.25	2.25	1.02	1.65	2.28	1.38	.67
A 23	4 ×4	5/16	8.2	2.40	1.12	3.71	1.29	1.24	1.58	1.50	.95	.79
A 23	4 ×4	3/8	9.7	2.86	1.14	4.36	1.52	1.23	1.61	1.77	1.10	.79
A 23	4 ×4	7/16	11.2	3.31	1.16	4.97	1.75	1.23	1.64	2.02	1.23	.78
A 23	4 ×4	½	12.8	3.75	1.18	5.56	1.97	1.22	1.67	2.28	1.36	.78
A 23	4 ×4	9/16	14.2	4.18	1.21	6.12	2.19	1.21	1.71	2.52	1.48	.78
A 23	4 ×4	5/8	15.7	4.61	1.23	6.66	2.40	1.20	1.74	2.76	1.59	.77
A 23	4 ×4	11/16	17.1	5.03	1.25	7.17	2.61	1.19	1.77	3.00	1.70	.77
A 23	4 ×4	¾	18.5	5.44	1.27	7.66	2.81	1.19	1.80	3.23	1.80	.77
A 23	4 ×4	13/16	19.9	5.84	1.29	8.14	3.01	1.18	1.83	3.46	1.89	.77
A 27	6 ×6	7/16	17.2	5.06	1.66	17.68	4.07	1.87	2.34	7.13	3.04	1.19
A 27	6 ×6	½	19.6	5.75	1.68	19.91	4.61	1.86	2.38	8.04	3.37	1.18
A 27	6 ×6	9/16	21.9	6.43	1.71	22.07	5.14	1.85	2.41	8.94	3.70	1.18
A 27	6 ×6	5/8	24.2	7.11	1.73	24.16	5.66	1.84	2.45	9.81	4.01	1.17
A 27	6 ×6	11/16	26.4	7.78	1.75	26.19	6.17	1.83	2.48	10.67	4.31	1.17
A 27	6 ×6	¾	28.7	8.44	1.78	28.15	6.66	1.83	2.51	11.52	4.59	1.17
A 27	6 ×6	13/16	30.9	9.09	1.80	30.06	7.15	1.82	2.54	12.35	4.86	1.17
A 27	6 ×6	7/8	33.1	9.73	1.82	31.92	7.63	1.81	2.57	13.17	5.12	1.16

Column 9 contains the least radii of gyration for two angles back to back for all thicknesses of gusset plates.

Table XII.

PROPERTIES OF STANDARD ANGLES
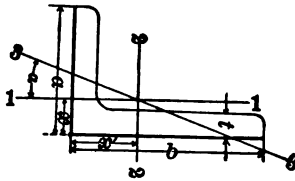

1	2	3	4	5	6	7	8
Section Number.	Dimensions.	Thickness.	Weight per Foot.	Area of Section.	Distance of Centre of Gravity from Back of Longer Flange.	Moment of Inertia Axis 1–1.	Section Modulus. Axis 1–1.
	b x a	t		A	x	I	S
	Inches.	Inches.	Pounds.	Sq. In	Inches.	Inches.⁴	Inches³.
A 91	2½×2	$\frac{3}{16}$	2.8	.81	.51	.29	.20
A 91	2½×2	$\frac{1}{4}$	3.6	1.06	.54	.37	.25
A 91	2½×2	$\frac{5}{16}$	4.5	1.31	.56	.45	.31
A 91	2½×2	$\frac{3}{8}$	5.3	1.55	.58	.51	.36
A 91	2½×2	$\frac{7}{16}$	6.0	1.78	.60	.58	.41
A 91	2½×2	$\frac{1}{2}$	6.8	2.00	.63	.64	.46
A 93	3 ×2½	$\frac{1}{4}$	4.5	1.31	.66	.74	.40
A 93	3 ×2½	$\frac{5}{16}$	5.5	1.62	.68	.90	.49
A 93	3 ×2½	$\frac{3}{8}$	6.5	1.92	.71	1.04	.58
A 93	3 ×2½	$\frac{7}{16}$	7.5	2.21	.73	1.18	.66
A 93	3 ×2½	$\frac{1}{2}$	8.5	2.50	.75	1.30	.74
A 93	3 ×2½	$\frac{9}{16}$	9.4	2.78	.77	1.42	.82
A 95	3½×2½	$\frac{1}{4}$	4.9	1.44	.61	.78	.41
A 95	3½×2½	$\frac{5}{16}$	6.0	1.78	.64	.94	.50
A 95	3½×2½	$\frac{3}{8}$	7.2	2.11	.66	1.09	.59
A 95	3½×2½	$\frac{7}{16}$	8.3	2.43	.68	1.23	.68
A 95	3½×2½	$\frac{1}{2}$	9.4	2.75	.70	1.36	.76
A 95	3½×2½	$\frac{9}{16}$	10.4	3.06	.73	1.49	.84
A 95	3½×2½	$\frac{5}{8}$	11.4	3.36	.75	1.61	.92
A 95	3½×2½	$\frac{11}{16}$	12.4	3.65	.77	1.72	.99
A 97	3½×3	$\frac{5}{16}$	6.6	1.93	.81	1.58	.72
A 97	3½×3	$\frac{3}{8}$	7.8	2.30	.83	1.85	.85
A 97	3½×3	$\frac{7}{16}$	9.0	2.65	.85	2.09	.98
A 97	3½×3	$\frac{1}{2}$	10.2	3.00	.88	2.33	1.10
A 97	3½×3	$\frac{9}{16}$	11.4	3.34	.90	2.55	1.21

TABLE XII—Continued.

PROPERTIES OF STANDARD ANGLES.

9	10	11	12	13	14	15	1
Radius of Gyration Axis 1-1.	Distance of Centre of Gravity from Back of Shorter Flange.	Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Radius of Gyration Axis 2-2.	Tangent of Angle ∝	Least Radius of Gyration Axis 3-3.	Section Number.
r	x'	I'	S'	r'		r''	
Inches.	Inches.	Inches.⁴	Inches.³	Inches.		Inches.	
.60	.76	.51	.29	.79	.632	.43	A 91
.59	.79	.65	.38	.78	.626	.42	A 91
.58	.81	.79	.47	.78	.620	.42	A 91
.58	.83	.91	.55	.77	.614	.42	A 91
.57	.85	1.03	.62	.76	.607	.42	A 91
.56	.88	1.14	.70	.75	.600	.42	A 91
.75	.91	1.17	.56	.95	.684	.53	A 93
.74	.93	1.42	.69	.94	.680	.53	A 93
.74	.96	1.66	.81	.93	.676	.52	A 93
.73	.98	1.88	.93	.92	.672	.52	A 93
.72	1.00	2.08	1.04	.91	.666	.52	A 93
.72	1.02	2.28	1.15	.91	.661	.52	A 93
.74	1.11	1.80	.75	1.12	.506	.54	A 95
.73	1.14	2.19	.93	1.11	.501	.54	A 95
.72	1.16	2.56	1.09	1.10	.496	.54	A 95
.71	1.18	2.91	1.26	1.09	.491	.54	A 95
.70	1.20	3.24	1.41	1.09	.486	.53	A 95
.70	1.23	3.55	1.56	1.08	.480	.53	A 95
.69	1.25	3.85	1.71	1.07	.472	.53	A 95
.69	1.27	4.13	1.85	1.06	.468	.53	A 95
.90	1.06	2.33	.95	1.10	.724	.63	A 97
.90	1.08	2.72	1.13	1.09	.721	.62	A 97
.89	1.10	3.10	1.29	1.08	.718	.62	A 97
.88	1.13	3.45	1.45	1.07	.714	.62	A 97
.87	1.15	3.79	1.61	1.07	.711	.62	A 97

Column 9 contains the least radii of gyration for two angles with short legs, back to back for all thicknesses of gusset plates.

TABLE XII—*Continued.*
PROPERTIES OF STANDARD ANGLES.

1	2	3	4	5	6	7	8
Section Number.	Dimensions.	Thickness.	Weight per Foot.	Area of Section.	Distance of Centre of Gravity from Back of Longer Flange.	Moment of Inertia Axis 1-1.	Section Modulus. Axis 1-1.
	b x a	t		A	x	I	S
	Inches.	Inches.	Pounds.	Sq. In.	Inches.	Inches.⁴	Inches.³
A 97	3½×3		12.5	3.67	.92	2.76	1.33
A 97	3½×3		13.6	4.00	.94	2.96	1.44
A 97	3½×3		14.7	4.31	.96	3.15	1.54
A 97	3½×3		15.7	4.62	.98	3.33	1.65
A 99	4 ×3		7.1	2.09	.76	1.65	.73
A 99	4 ×3		8.5	2.48	.78	1.92	.87
A 99	4 ×3		9.8	2.87	.80	2.18	.99
A 99	4 ×3		11.1	3.25	.83	2.42	1.12
A 99	4 ×3		12.3	3.62	.85	2.66	1.23
A 99	4 ×3		13.6	3.98	.87	2.87	1.35
A 99	4 ×3		14.8	4.34	.89	3.08	1.46
A 99	4 ×3		15.9	4.69	.92	3.28	1.57
A 99	4 ×3		17.1	5.03	.94	3.47	1.68
A 101	5 ×3		8.2	2.40	.68	1.75	.75
A 101	5 ×3		9.7	2.86	.70	2.04	.89
A 101	5 ×3		11.3	3.31	.73	2.32	1.02
A 101	5 ×3		12.8	3.75	.75	2.58	1.15
A 101	5 ×3		14.2	4.18	.77	2.83	1.27
A 101	5 ×3		15.7	4.61	.80	3.06	1.39
A 101	5 ×3		17.1	5.03	.82	3.29	1.51
A 101	5 ×3		18.5	5.44	.84	3.51	1.62
A 101	5 ×3		19.9	5.84	.86	3.71	1.74
A 103	5 ×3½		10.4	3.05	.86	3.18	1.21
A 103	5 ×3½		12.0	3.53	.88	3.63	1.39
A 103	5 ×3½		13.6	4.00	.91	4.05	1.56
A 103	5 ×3½		15.2	4.46	.93	4.45	1.73
A 103	5 ×3½		16.7	4.92	.95	4.83	1.90
A 103	5 ×3½		18.3	5.37	.97	5.20	2.06
A 103	5 +3½		19.8	5.81	1.00	5.55	2.22
A 103	5 ×3½		21.2	6.25	1.02	5.89	2.37
A 103	5 ×3½		22.7	6.67	1.04	6.21	2.52

TABLE XII—*Continued.*
PROPERTIES OF STANDARD ANGLES.

9	10	11	12	13	14	15	1
Radius of Gyration Axis 1-1.	Distance of Centre of Gravity from Back of Shorter Flange.	Moment of Inertia. Axis 2-2.	Section Modulus Axis 2-2.	Radius of Gyration Axis 2-2.	Tangent of Angle $\propto$	Least Radius of Gyration Axis 3-3.	Section Number.
r	x′	I′	S′	r′		r″	
Inches.	Inches.	Inches.⁴	Inches.³	Inches.		Inches.	
.87	1.17	4.11	1.76	1.06	.707	.62	A 97
.86	1.19	4.41	1.91	1.05	.703	.62	A 97
.85	1.21	4.70	2.05	1.04	.698	.62	A 97
.85	1.23	4.98	2.20	1.04	.694	.62	A 97
.89	1.26	3.38	1.23	1.27	.554	.65	A 99
.88	1.28	3.96	1.46	1.26	.551	.64	A 99
.87	1.30	4.52	1.68	1.25	.547	.64	A 99
.86	1.33	5.05	1.89	1.25	.543	.64	A 99
.86	1.35	5.55	2.09	1.24	.538	.64	A 99
.85	1.37	6.03	2.30	1.23	.534	.64	A 99
.84	1.39	6.49	2.49	1.22	.529	.64	A 99
.84	1.42	6.93	2.68	1.22	.524	.64	A 99
.83	1.44	7.35	2.87	1.21	.518	.64	A 99
.85	1.68	6.26	1.89	1.61	.368	.66	A 101
.84	1.70	7.37	2.24	1.61	.364	.65	A 101
.84	1.73	8.43	2.58	1.60	.361	.65	A 101
.83	1.75	9.45	2.91	1.59	.357	.65	A 101
.82	1.77	10.43	3.23	1.58	.353	.65	A 101
.82	1.80	11.37	3.55	1.57	.349	.64	A 101
.81	1.82	12.28	3.86	1.56	.345	.64	A 101
.80	1.84	13.15	4.16	1.55	.340	.64	A 101
.80	1.86	13.98	4.46	1.55	.336	.64	A 101
1.02	1.61	7.78	2.29	1.60	.485	.76	A 103
1.01	1.63	8.90	2.64	1.59	.482	.76	A 103
1.01	1.66	9.99	2.99	1.58	.479	.75	A 103
1.00	1.68	11.03	3.32	1.57	.476	.75	A 103
.99	1.70	12.03	3.65	1.56	.472	.75	A 103
.98	1.72	12.99	3.97	1.56	.468	.75	A 103
.98	1.75	13.92	4.28	1.55	.464	.75	A 103
.97	1.77	14.81	4.58	1.54	.460	.75	A 103
.96	1.79	15.67	4.88	1.53	.455	.75	A 103

TABLE XII—*Continued.*

PROPERTIES OF STANDARD ANGLES.

1	2	3	4	5	6	7	8
Section Number.	Dimensions.	Thickness.	Weight per Foot.	Area of Section.	Distance of Centre of Gravity from Back of Longer Flange.	Moment of Inertia Axis 1–1.	Section Modulus Axis 1–1.
	b x a	t		A	x	I	S
	Inches.	Inches.	Pounds.	Sq. In.	Inches.	Inches.⁴	Inches.³
A 105	6 ×3½	⅜	11.6	3.42	.79	3.34	1.23
A 105	6 ×3½	⁷⁄₁₆	13.5	3.96	.81	3.81	1.41
A 105	6 ×3½	½	15.3	4.50	.83	4.25	1.59
A 105	6 ×3½	⁹⁄₁₆	17.1	5.03	.86	4.67	1.77
A 105	6 ×3½	⅝	18.9	5.55	.88	5.08	1.94
A 105	6 ×3½	11⁄16	20.6	6.06	.90	5.47	2.11
A 105	6 ×3½	¾	22.3	6.56	.93	5.84	2.27
A 105	6 ×3½	13⁄16	24.0	7.06	.95	6.20	2.43
A 105	6 ×3½	⅞	25.7	7.55	.97	6.55	2.59
A 107	6 ×4	⅜	12.3	3.61	.94	4.90	1.60
A 107	6 ×4	⁷⁄₁₆	14.2	4.18	.96	5.60	1.85
A 107	6 ×4	½	16.2	4.75	.99	6.27	2.08
A 107	6 ×4	⁹⁄₁₆	18.1	5.31	1.01	6.91	2.31
A 107	6 ×4	⅝	19.9	5.86	1.03	7.52	2.54
A 107	6 ×4	11⁄16	21.8	6.40	1.06	8.11	2.76
A 107	6 ×4	¾	23.6	6.94	1.08	8.68	2.97
A 107	6 ×4	13⁄16	25.4	7.46	1.10	9.23	3.18
A 107	6 ×4	⅞	27.2	7.98	1.12	9.75	3.39

TABLE XII—*Continued.*

PROPERTIES OF STANDARD ANGLES.

9	10	11	12	13	14	15	1
Radius of Gyration Axis 1-1.	Distance of Centre of Gravity from Back of Shorter Flange.	Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Radius of Gyration Axis 2-2.	Tangent of Angle ∝	Least Radius of Gyration. Axis 3-3.	Section Number.
r	x'	I'	S'	r'		r''	
Inches.	Inches.	Inches.⁴	Inches.³	Inches.		Inches.	
.99	2.04	12.86	3.24	1.94	.350	.77	A 105
.98	2.06	14.76	3.75	1.93	.347	.76	A 105
.97	2.08	16.59	4.24	1.92	.344	.76	A 105
.96	2.11	18.37	4.72	1.91	.341	.75	A 105
.96	2.13	20.08	5.19	1.90	.338	.75	A 105
.95	2.15	21.74	5.65	1.89	.334	.75	A 105
.94	2.18	23.34	6.10	1.89	.331	.75	A 105
.94	2.20	24.89	6.55	1.88	.327	.75	A 105
.93	2.22	26.39	6.98	1.87	.323	.75	A 105
1.17	1.94	13.47	3.32	1.93	.446	.88	A 107
1.16	1.96	15.46	3.83	1.92	.443	.87	A 107
1.15	1.99	17.40	4.33	1.91	.440	.87	A 107
1.14	2.01	19.26	4.83	1.90	.438	.87	A 107
1.13	2.03	21.07	5.31	1.90	.434	.86	A 107
1.13	2.06	22.82	5.78	1.89	.431	.86	A 107
1.12	2.08	24.51	6.25	1.88	.428	.86	A 107
1.11	2.10	26.15	6.75	1.87	.425	.86	A 107
1.11	2.12	27.73	7.15	1.86	.421	.86	A 107

TABLE XIII.

LEAST RADII OF GYRATION FOR TWO ANGLES WITH UNEQUAL LEGS, LONG LEGS BACK TO BACK.
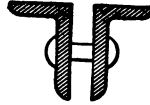
Dimensions, Inches.	Thickness, Inches.	Area of Two Angles, Square Inches.	Least Radii of Gyration for Distances Back to Back.			Least Radius of Gyration for one Angle.
			0 Inch.	⅜ Inch.	¾ Inch.	
2½×2	⁵⁄₁₆	1.62	0.79	0.79	0.79	0.43
2½×2	⅜	3.09	0.77	0.77	0.77	0.42
2½×2	½	4.00	0.75	0.75	0.75	0.42
3 ×2½	¼	2.63	0.95	0.95	0.95	0.53
3 ×2½	⅜	3.84	0.93	0.93	0.93	0.52
3 ×2½	⁹⁄₁₆	5.55	0.91	0.91	0.91	0.52
3½×2½	¼	2.88	0.96	1.09	1.12	0.54
3½×2½	½	5.50	1.00	1.09	1.09	0.53
3½×2½	¹¹⁄₁₆	7.30	1.03	1.06	1.06	0.53
3½×3	⁵⁄₁₆	3.87	1.10	1.10	1.10	0.63
3½×3	⁹⁄₁₆	6.68	1.07	1.07	1.07	0.62
3½×3	¹³⁄₁₆	9.24	1.04	1.04	1.04	0.62
4 ×3	⁵⁄₁₆	4.18	1.17	1.27	1.27	0.65
4 ×3	⁹⁄₁₆	7.24	1.21	1.24	1.24	0.64
4 ×3	¹³⁄₁₆	10.05	1.21	1.21	1.21	0.64
5 ×3	⁵⁄₁₆	4.80	1.09	1.22	1.36	0.66
5 ×3	⁹⁄₁₆	8.37	1.13	1.26	1.41	0.65
5 ×3	¹³⁄₁₆	11.68	1.17	1.32	1.47	0.64
5 ×3½	⅜	6.09	1.34	1.46	1.60	0.76
5 ×3½	⅝	9.84	1.37	1.51	1.56	0.75
5 ×3½	⅞	13.34	1.42	1.53	1.53	0.75
6 ×3½	⅜	6.84	1.26	1.39	1.53	0.77
6 ×3½	⅝	11.09	1.30	1.43	1.58	0.75
6 ×3½	⅞	15.09	1.34	1.49	1.64	0.75
6 ×4	⅜	7.22	1.50	1.62	1.76	0.88
6 ×4	⅝	11.72	1.53	1.67	1.81	0.86
6 ×4	⅞	15.97	1.58	1.68	1.86	0.86

TABLE XIV.
PROPERTIES OF T BARS.
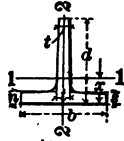
Equal Legs.

1	2	3	4	5	6	7	8
	DIMENSIONS.						Dist. Cent. of Gravity from Outside of Flange.
Section Number.	Width of Flange.	Depth of Bar.	Thickness of Flange.	Thickness of Stem.	Weight per Foot.	Area of Section.	
	b	d	s to n′	t to t_L		A	x
	Inches.	Inches.	Inches.	Inches.	Pounds.	Sq. Ins.	Inches.
T 5	1	1	$\frac{1}{8}$ to $\frac{5}{32}$	$\frac{1}{8}$ to $\frac{5}{32}$	.89	.26	.29
T 181	$1\frac{1}{8}$	$1\frac{1}{8}$	$\frac{3}{16}$ " $\frac{7}{32}$	$\frac{5}{32}$ " $\frac{7}{32}$	1.39	.41	.33
T 183	$1\frac{3}{16}$	$1\frac{3}{16}$	$\frac{3}{16}$ " $\frac{1}{4}$	$\frac{5}{32}$ " $\frac{7}{32}$	1.53	.45	.34
T 187	$1\frac{1}{2}$	$1\frac{1}{2}$	$\frac{3}{16}$ " $\frac{1}{4}$	$\frac{5}{32}$ " $\frac{1}{4}$	1.61	.47	.36
T 189	$1\frac{5}{8}$	$1\frac{5}{8}$	$\frac{3}{16}$ " $\frac{1}{4}$	$\frac{5}{32}$ " $\frac{1}{4}$	1.85	.54	.39
T 37	2	2	$\frac{1}{4}$ " $\frac{5}{16}$	$\frac{1}{4}$ " $\frac{5}{16}$	3.7	1.05	.59
T 39	2	2	$\frac{5}{16}$ " $\frac{3}{8}$	$\frac{5}{16}$ " $\frac{3}{8}$	4.3	1.26	.61
T 41	$2\frac{1}{4}$	$2\frac{1}{4}$	$\frac{1}{4}$ " $\frac{5}{16}$	$\frac{1}{4}$ " $\frac{5}{16}$	4.1	1.19	.68
T 69	3	3	$\frac{3}{8}$ " $\frac{7}{16}$	$\frac{3}{8}$ " $\frac{7}{16}$	7.8	2.27	.88
T 97	$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{3}{8}$ " $\frac{7}{16}$	$\frac{3}{8}$ " $\frac{7}{16}$	9.3	2.74	.99

Unequal Legs.

1	2	3	4	5	6	7	8
T 185	$1\frac{1}{4}$	$1\frac{1}{16}$	$\frac{3}{8}$ " $\frac{1}{2}$	$\frac{5}{16}$ " $\frac{7}{32}$	1.49	.44	.29
T 65	3	$2\frac{1}{2}$	$\frac{1}{4}$ " $\frac{7}{16}$	$\frac{3}{8}$ " $\frac{7}{16}$	7.2	2.07	.71
T 101	$3\frac{1}{2}$	4	$\frac{3}{8}$ " $\frac{7}{16}$	$\frac{3}{8}$ " $\frac{7}{16}$	9.9	2.91	1.20

TABLE XIV—*Continued.*

PROPERTIES OF T BARS.

Equal Legs—(Continued).

1	9	10	11	12	13	14
Section Number.	Moment of Inertia Axis 1-1.	Section Modulus Axis 1-1.	Radius of Gyration Axis 1-1.	Moment of Inertia Axis 2-2.	Section Modulus Axis 2-2.	Radius of Gyration Axis 2-2.
	I	S	r	I′	S′	r′
	Inches⁴.	Inches³.	Inches.	Inches⁴.	Inches³.	Inches.
T 5	.02	.03	.30	.01	.02	.21
T 181	.04	.05	.32	.02	.04	.25
T 183	.05	.06	.33	.03	.05	.26
T 187	.06	.07	.35	.03	.05	.27
T 189	.08	.08	.39	.05	.07	.29
T 37	.37	.26	.59	.18	.18	.42
T 39	.43	.31	.59	.23	.23	.42
T 41	.51	.32	.65	.24	.22	.45
T 69	1.82	.86	.90	.92	.61	.64
T 97	3.1	1.23	1.08	1.42	.81	.73

Unequal Legs—(Continued).

T 185	.04	.05	.29	.03	.01	.28
T 65	1.08	.60	.64	.90	.60	.28
T 101	4.3	1.54	1.23	1.42	.81	.70

TABLE XV.

COMMERCIAL SIZES OF YELLOW PINE LUMBER WITH RELA-
TIVE PRICES BASED UPON $1 PER THOUSAND FEET
BOARD MEASURE.

Common Boards SIS.

Nominal Size in Inches.	Length in Feet.						Actual Sizes.
	10	12	14	16	18	20	
1×8 No. 1	1.03	1.03	1.00	1.00	1.03	1.03	*SIS* or 2*S*
1×10 "	1.06	1.06	1.03	1.03	1.06	1.06	⅓⅝ thick.
1×12 "	1.20	1.20	1.12	1.12	1.12	1.12	

For rough boards add $1.00 per M.

Fencing SIS.

Nominal Size in Inches.	Length in Feet.						Actual Sizes.
	10	12	14	16	18	20	
1×4 No. 1	0.97	0.97	0.97	1.00	0.97	0.97	*SIS* or 2*S*
1×6 "	1.00	1.00	1.00	1.03	1.00	1.00	⅓⅞ thick
1×4 " 2	0.91	0.91	0.91	0.94	0.91	0.91	
1×6 "	0.91	0.91	0.91	0.94	0.91	0.91	

Dimension SISIE.

Nominal Size in Inches.	Length in Feet.							Actual Sizes. Inches.
	10	12	14	16	18	20	22–24	
2×6 No. 1	0.97	0.94	0.94	0.94	.097	0.97	1.10	1⅝×5⅝
2×8 "	0.97	0.94	0.94	0.94	.097	0.97	1.10	1⅝×7⅝
2×4 "	0.97	0.94	0.94	0.94	0.97	1.97	1.10	1⅝×3⅝
2×10 "	1.00	0.97	0.97	0.97	1.00	1.00	1.13	1⅝×9½
2×12 "	1.03	1.00	1.00	1.00	1.03	1.03	1.16	1⅝×11½

When dressed on 4 sides take ¼ inch off each side.
Rough lumber costs $1.00 more per M.
For dimensions sized to 1¾ inch add 75 cents per M.
For No. 2, when in stock deduct $1.50 per M.
For every 2 feet over 24 feet up to 32 feet add $1.00 per M.

TABLE XV—*Continued.*

COMMERCIAL SIZES OF YELLOW PINE LUMBER WITH RELA-
TIVE PRICES BASED UPON $1 PER THOUSAND FEET
BOARD MEASURE.

Heavy Joists, SISIE.

Nominal Size in Inches.	Length in Feet.							Actual Size in Inches.
	10	12	14	16	18	20	22–24	
3×6 & 3×8	1.19	1.16	1.16	1.16	1.19	1.19	1.28	
3×10 & 3×12	1.25	1.19	1.19	1.19	1.25	1.25	1.34	
2×14	1.28	1.22	1.22	1.22	1.28	1.28	1.38	
2½×14 to 3×14	1.28	1.22	1.22	1.22	1.28	1.28	1.38	

For rough lumber add $1.00 per M.
For 16 inch joists add $1.00 per M.   Add $2.00 per M for each 2 inches
over 16 inches.

Timbers.

Nominal Sizes in Inches.	Length in Feet.							Actual Size in Inches.
	10	12	14	16	18	20	22–24	
4×4 and 4×6 S and E....	1.16	1.13	1.13	1.13	1.16	1.16	1.22	¾" off S & E
4×8 to 8×8 rough.....	1.22	1.19	1.19	1.19	1.22	1.22	1.28	
4×10 to 12×12 rough....	1.28	1.25	1.25	1.25	1.28	1.28	1.34	

For every 2 feet over 24 feet up to 32 feet add $1.00 per M.

Note.

SIS   = surface upon one side.
SISIE =   "   "   "   " and one edge.
S4S   =   "   "   four sides.

TABLE XVI.

AVERAGE SAFE ALLOWABLE WORKING UNIT STRESSES, IN POUNDS, PER SQUARE INCH.

Recommended by the Committee on "Strength of Bridge and Trestle Timbers," Association of Railway Superintendents of Bridges and Buildings, Fifth Annual Convention, New Orleans, October, 1895.

KIND OF TIMBER.	TENSION.		COMPRESSION.			TRANSVERSE.		SHEARING.	
	With Grain.	Across Grain.	With Grain.		Across Grain.	Extr'me Fibre Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
			End Bearing.	Col'mns under 15 Diam					
Factor of Safety.	Ten.	Ten.	Five.	Five.	Four.	Six.	Two.	Four.	Four.
White Oak	1000	200	1400	900	500	10000	550000	200	1000
White Pine	700	50	1100	700	200	700	500000	100	500
Southern Long-leaf or Georgia Yellow Pine	1200	60	1600	1000	350	1200	850000	150	1250
Douglas, Oregon, and Yellow Fir	1200	......	1600	1200	300	1100	700000	150	......
Washington Fir or Pine (Red Fir)	1000	......	......	......	......	800	......	......	......
Northern or Short-leaf Yellow Pine	900	50	1200	800	250	1000	600000	100	1000
Red Pine	900	50	1200	800	200	800	600000	......	......
Norway Pine	800	......	1200	800	200	700	600000	......	......
Canadian (Ottawa) White Pine	1000	......	......	1000	......	......	......	100	......
Canadian (Ontario) Red Pine	1000	......	......	1000	......	800	700000	100	......
Spruce and Eastern Fir	800	50	1200	800	200	700	600000	100	750
Hemlock	600	......	......	800	150	600	450000	100	600
Cypress	800	......	1200	800	2000	800	450000	......	......
Cedar	800	......	1200	800	200	800	350000	......	400
Chestnut	900	......	......	1000	250	800	500000	150	400
California Redwood	700	......	......	800	200	750	350000	100	......
California Spruce	......	......	......	800	......	800	600000	......	......

TABLE XVII.
CAST-IRON WASHERS.
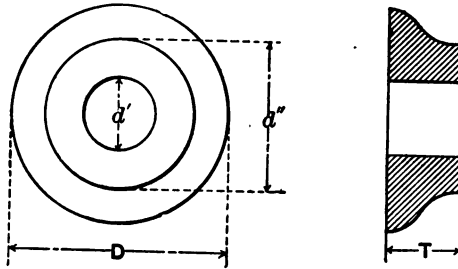
Diam.of bolt d. Inches.	D Inches.	d'' Inches.	d' Inches.	T Inches.	Weight. Lbs.	Bearing Area. Sq. In.
$\frac{1}{2}$	$2\frac{5}{8}$	$1\frac{3}{4}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{1}{2}$	5.16
$\frac{5}{8}$	3	$1\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{3}{4}$	6.69
$\frac{3}{4}$	$3\frac{1}{2}$	$2\frac{1}{8}$	$\frac{13}{16}$	$\frac{7}{8}$	$1\frac{1}{4}$	7.78
$\frac{7}{8}$	$3\frac{3}{4}$	$2\frac{7}{8}$	$\frac{15}{16}$	$\frac{7}{8}$	$1\frac{1}{2}$	10.35
1	4	$2\frac{1}{4}$	$1\frac{1}{16}$	$1\frac{1}{8}$	$2\frac{1}{2}$	11.68
$1\frac{1}{8}$	$4\frac{3}{4}$	$2\frac{3}{4}$	$1\frac{3}{16}$	$1\frac{3}{8}$	3	16.61
$1\frac{1}{4}$	6	3	$1\frac{5}{16}$	$1\frac{3}{8}$	$5\frac{3}{4}$	26.92
$1\frac{1}{2}$	$6\frac{1}{4}$	$3\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{1}{2}$	6	28.61
$1\frac{3}{4}$	$7\frac{1}{4}$	$3\frac{3}{4}$	$1\frac{7}{8}$	$1\frac{3}{4}$	$9\frac{1}{2}$	38.52
2	$8\frac{1}{4}$	$4\frac{1}{4}$	$2\frac{1}{8}$	2	$17\frac{1}{4}$	49.91
$2\frac{1}{4}$	$9\frac{1}{4}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{1}{4}$	20	62.77
$2\frac{1}{2}$	$10\frac{1}{4}$	$5\frac{1}{4}$	$2\frac{5}{8}$	$2\frac{1}{2}$	$27\frac{1}{2}$	77.11
$2\frac{3}{4}$	$11\frac{1}{4}$	$5\frac{3}{4}$	$2\frac{7}{8}$	$2\frac{3}{4}$	36	92.91
3	$12\frac{1}{4}$	$6\frac{1}{4}$	$3\frac{1}{8}$	3	46	110.19

For sizes not given $D = 4d + \frac{1}{4}''$; $d'' = 2d + \frac{1}{4}$.
$d' = d + \frac{1}{8}$; $T = d$.

11' 8 9/16"

12' 1' c.

10 1/2" long

14' 8 3/4"

16' 8" c.—c.

3/4" bol

1" Pl. 6" x 14"

¾" bolts 16½" long

12' or 1¼ x 12 on each side

18' 1" c.-c.

8"

3"

8"

12"

18' 4" c.-c.

1¼ Rod 22' 7½" long

Upset 1⅜"

6 x 6

1½"

This bearing plate
if the fibers of the
are compressed sli
Bearing area requ

4" x 6" x 24" W.Oak

10' c.-c.

¾"

¾"

1½"

6"

ay riv
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ed in
angle

1630

Pl.
10'

PLATE III.

U₃

Angle purlin

2-2½" x 2" x ¼" L⁸

¾"

2-4½" c-c
17000 #

20'0" c-c
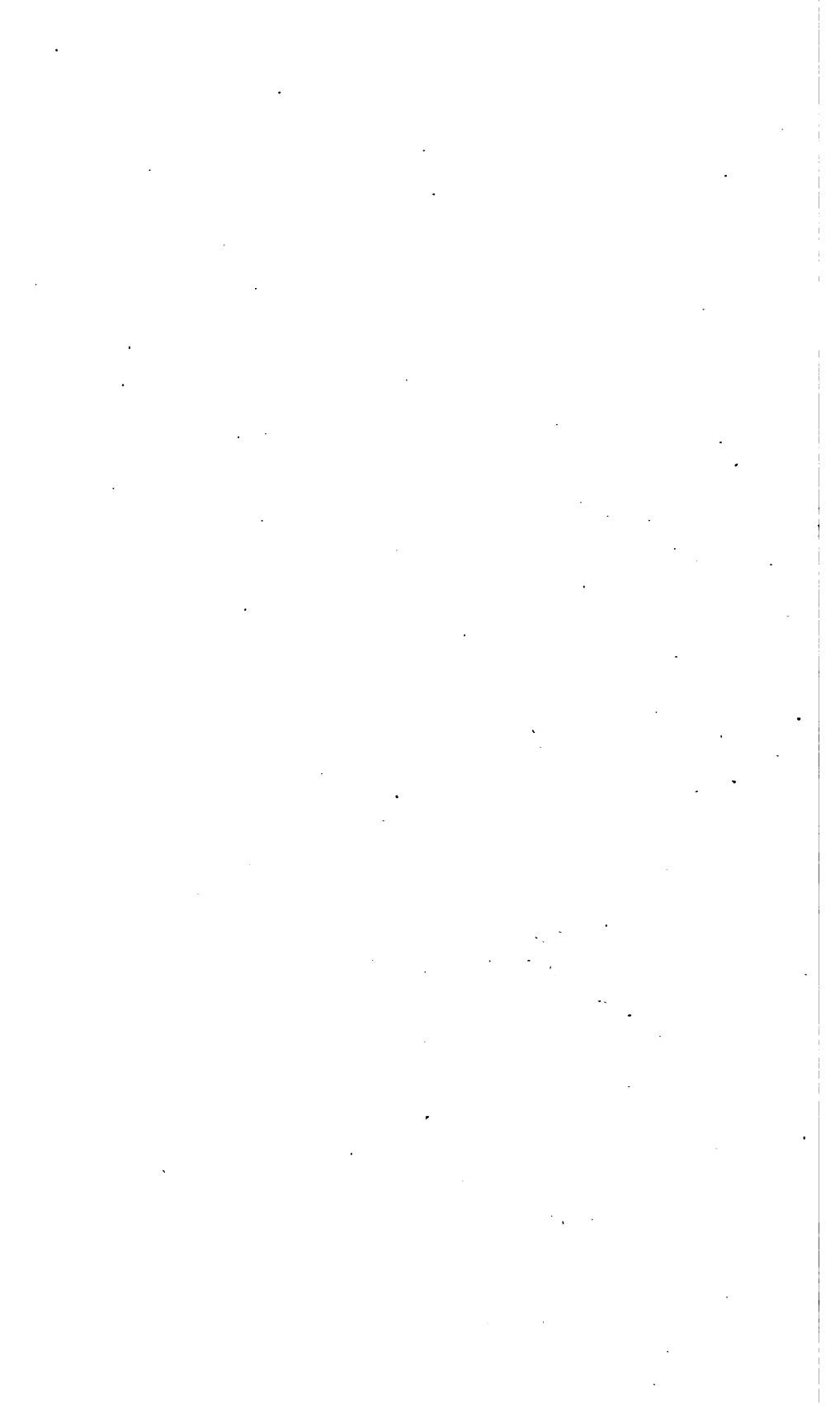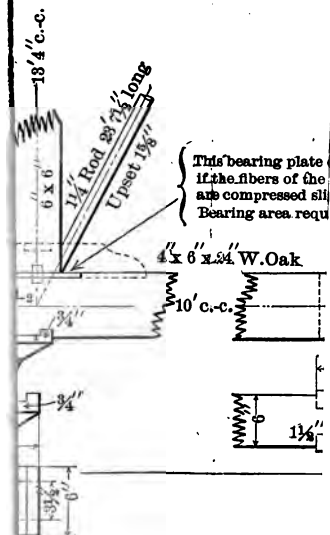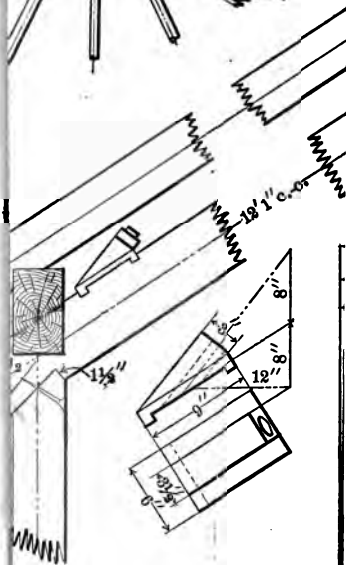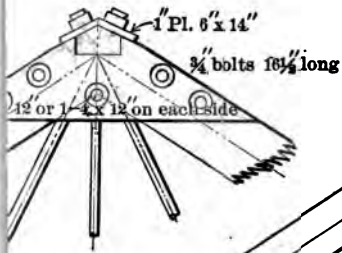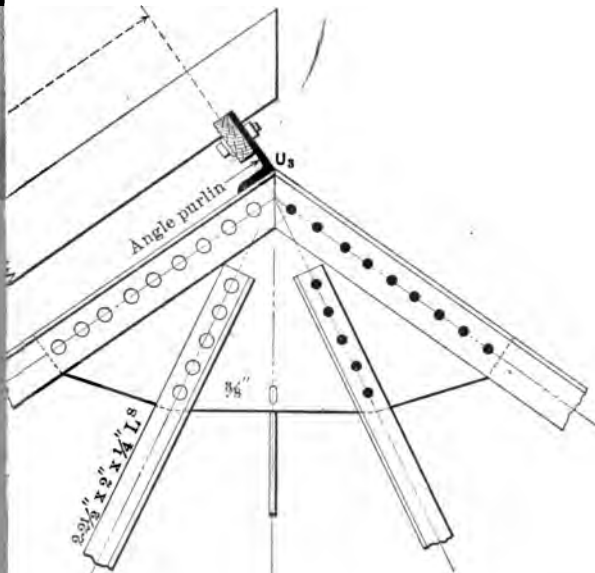
NOTE:- This drawing should be so complete
that a close estimate of the materials
required can be easily determined.
Details drawn to scale and all
general dimensions given, as well as
all rivets and their relative positions.

...ay rivets shall not be spaced
...ore than 3"c-c and shall be
...ed in all members composed
...angles.

¾" sag tie

16300 #

2-3" x 2½" x ¼" L⁸

...e Pl. 6" wide 21" long
10'0"

L₃
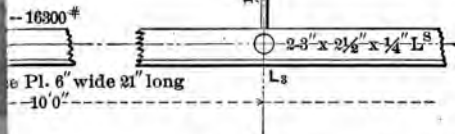
GENERAL DESIGN
FOR A
STEEL ROOF TRUSS

INDEX.

TABLES.

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