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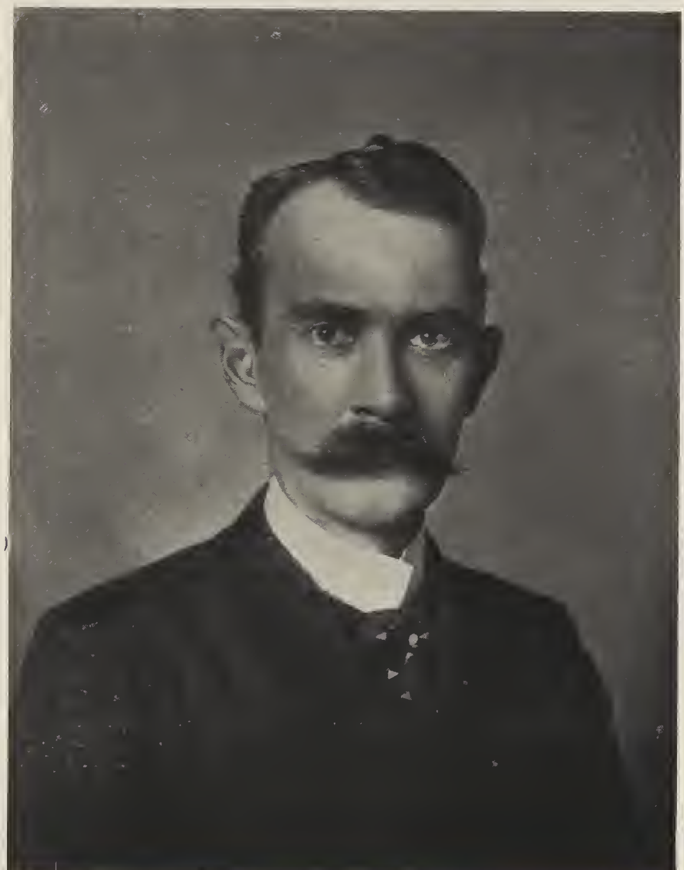
PART II.



F. E. Kidder

Building Construction

and

Superintendence

BY

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PART III.

Trussed Roofs and Roof Trusses

306 Illustrations

SECTION I.

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PREFACE.

Some years ago the author of this book contributed to "Architecture and Building" and the Architects' and Builders' Magazine a series of articles on "Trussed Roofs," which were so well received that he has been led to prepare a book which would treat the subject of trussed roofs and roof trusses in a more complete and comprehensive manner than has ever before been attempted.

Starting with the articles above mentioned as a nucleus, they have been revised and rearranged and a great amount of additional matter added.

The aim of the author has been to describe nearly every type of roof construction commonly met with in buildings such as architects have occasion to design, to point out the advantages of the different types of wooden and steel trusses for different spans and building requirements and to explain the process of computing the loads, drawing the stress diagram and proportioning the members and joints to the stresses. Special pains have been taken to make the mechanical principles involved as plain as possible and to describe the method of obtaining the stresses so that any intelligent person can apply them and that without violating any scientific principle. The author has had in mind the needs of architects, draughtsmen and builders rather than those of the engineer and hence more space has been given to the description of wooden trusses and the common types of steel trusses than to intricate engineering problems; the object being to make the book of practical value to the greatest number of persons.

The author desires to express his thanks to the publishers of the Engineering Record for the use of many illustrations from that excellent journal, and also to the several architects and manufacturers who have furnished him with working drawings from which many of the illustrations have been made.

The preparation of this book has involved an immense amount of labor, and the author hopes that its value to the persons for whom it is intended may be in some slight degree commensurate to the labor and expense involved.

F. E. KIDDER.

PUBLISHER'S NOTE.

When the author wrote the preface on the preceding page he had finished the first section of this work. The second section was blocked out, and it is the purpose of the publisher to put this in hands familiar with Mr. Kidder's methods and competent to carry out the work along the lines he has already laid down.

It was Mr. Kidder's plan to make this work a portion of his "Building Construction" Series. He had laid it out on broad lines, intending that it should be the most complete and elaborate work on Roof Trusses that had been presented to the American Architect or Builder.

This section, therefore, is presented as Part III of "Building Construction and Superintendence," which is in compliance with Mr. Kidder's last letter to the publisher, written just prior to his admission to the hospital, where he was to undergo the operation which unfortunately resulted in his death.

The second section will appear later as Part IV of the same series, and will be announced to all purchases of this Part as soon as it can be completed.

THE PUBLISHER.

TABLE OF CONTENTS.

FRANK EUGENE KIDDER.

IT seems appropriate in presenting this the last work from the pen of Mr. Kidder that a brief sketch of his life should be given. His work is so well known to architects and architectural students that we believe all will appreciate the portrait here presented and this brief outline of his life and work.

Mr. Kidder had only arrived at that time in his life when most men are doing their best work, but even in the few years allowed him he has given to his profession a series of books that will make his name memorable for all time, and this notwithstanding serious and continued illness.

Frank Eugene Kidder was born at Bangor, Me., Nov. 3, 1859.

As a boy he was always interested in building operations, and at the age of fifteen he determined to become an architect. With this aim in view, he took the course in Civil Engineering at the Maine State College, now the University of Maine, this coming the nearest to a course in Architecture that the College afforded. During his last year in college, he attended one term at Cornell University, taking the studies of the third year men in Architecture, and returned to Maine to graduate.

The Fall following his graduation he acted as instructor in drawing at the Maine College, and the following winter he entered the office of Ware & Van Brunt, in Boston, as a student. From this office he went to the office of H. J. Hardenbergh, in New York City, where he worked until the following summer.

At this time, Mr. D. W. Willard, now of the firm of Babb, Cook & Willard, was working for Mr. Hardenbergh, and on his recommendation Mr. Kidder determined to take a special course in Architecture at the Massachusetts Institute of Technology.

At the Institute, Mr. Kidder was associated with the classes of '81 and '82, taking the architectural studies of both the Junior and Senior years.

Leaving the Institute in 1881, he entered the office of A. H. Vinal of Boston, but his health had been undermined by too close application at the Institute, so that early in the Fall he was obliged to make a change. During this Fall he was employed to conduct a series of tests on fireproof materials, the most elaborate that had been made up to that time, for the Massachusetts Charitable Mechanics' Association. After completing these tests, he entered the employ of Norcross Bros., the well-known builders, whom he served as draughtsman, clerk and engineer, and where he obtained an insight into the practical details of building construction, which proved of the utmost value to him in after years.

In 1884 Mr. Vinal was appointed City Architect of Boston, and Mr. Kidder accepted the position of head draughtsman under him, and for about two years he held this important position, resigning to start in practice for himself.

He maintained an office at 54 Devonshire street, Boston, until the spring of 1888, when he suffered severe hemorrhages from the lungs, and was obliged to seek the healing climate of Colorado, where he was so fortunate as to regain his health.

As a practicing architect in Denver, Mr. Kidder has made a very creditable reputation, but it is as an author and structural engineer that he is best known to the profession at large.

Being of a mathematical and constructive turn of mind, he early took a special interest in the engineering problems of architecture, and his writings on these subjects date back to the year 1880. At College he learned to appreciate the great value of Trautwine's Pocket Book for Civil Engineers, and while a draughtsman, he determined to write a similar book for the use of architects. With this aim in view, at the age of 24, Mr. Kidder entered into a contract with Messrs. John Wiley & Sons for the publication of the "Architects and Builders Pocket-Book."

Encouraged by the success of this book, be began to devote his attention to writing, and in 1895 published "Churches and Chapels," now in its 3rd edition. The next year he brought out Part I of

"Building Construction and Superintendence" and two years later Part II of this same work. In 1901, Mr. Kidder, though hampered by ill health, undertook the stupendous task of thoroughly revising and almost entirely rewriting the "Architects and Builders Pocket-Book," then in its 13th edition. After three years of constant labor and study, he has given to the world in the latest edition of this book a work of inestimable value to all architects and builders.

In the summer of 1905 Mr. Kidder brought out his last book; a volume on the "Strength of Beams, Floors and Roofs." For the last two years Mr. Kidder has been engaged on the present work which he contemplated making the most complete and elaborate work on roof trusses ever placed before American architects and at the time of his death had just finished the first section which now appears as Part III of "Building Construction."

While engaged on this last work his illness assumed a serious turn and under the advice of his physician he underwent an operation in the hope of recovery but which resulted in his death which took place at Denver the twenty-seventh of October, 1905.

Mr. Kidder was well known as a "consulting architect," being, we believe, the first architect to assume that title in this country. In this capacity and through his later works and frequent papers in technical journals, Mr. Kidder came in touch with more of his professional brethren than usually falls to the lot of an architect.

Although he had accomplished so much, Mr. Kidder had barely reached his prime, when death cut short a career of great usefulness. No one can over-estimate the value of the services he rendered his profession, and in his death it has suffered an irreparable loss.

INTRODUCTION.

Although trusses have been used for supporting the roofs of large buildings for centuries and there are few modern buildings designed for public or semi-public purposes which do not require one or more trusses to support some part of the building, yet there is a great deal of ignorance on the part of draughtsmen and builders and also otherwise well-informed architects as to the manner in which the stresses act and the correct way of arranging the various members in different types of trusses to sustain the loads. In fact, the mechanical principles of any but the simplest form of truss and the manner of determining the stresses seems to be difficult of comprehension by persons who have not had the advantages of instruction at a technical school and it has been the experience of the author that it is somewhat difficult to make the subject perfectly plain without oral explanation.

In this work, therefore, a special effort has been made to make the explanations as clear as is possible in print, even at the risk of appearing rather elemental and common place, and the author hopes that he has so far succeeded that any person who will diligently follow the explanations and go to the trouble of drawing out the diagrams may become fairly proficient in designing ordinary trussed roofs.

The proper designing of a trussed roof requires not only a knowledge of the theory of trusses and the strength of materials but also a familiarity with the various types of trusses that are employed for the support of roofs and their adaptability to different forms of roofs; a practical knowledge of the economical spacing of the trusses and of roof construction in general and how to meet any special forms of construction in the most economical manner, or in short how to lay out the entire framework so as to meet the requirements of strength, the special requirements of the building and a wise economy. In the preparation of this book, therefore, the author has endeavored, first, to give practically all of the types of trusses that are used in buildings, with sufficient explanation as to their advantages and limitations as to enable the reader to select the type of truss best suited to his purpose; second, to show how various kinds and shapes of roofs may be best supported; third, to give the method of computing loads and determining stresses; and, finally, how to proportion the members and joints to the stresses.

To do this in the best manner it seemed advisable to divide the book into chapters and to describe certain kinds of roofs, such as domed roofs, church roofs, armory roofs, etc., in separate chapters. The method of paragraphing employed in Parts I. and II. of this series has also been retained, because of its convenience for cross-references, and also for convenience when used as a text-book.

One of the most valuable aids to the architect is a knowledge of what has been done in different lines of building construction, and for this reason a great many examples of existing trusses and of trussed roofs are illustrated. These examples have been selected as guides to the architect and draughtsman as to the shape of the truss, section of the members, detailing of the joints and the manner of bracing laterally and to the posts or walls. It is seldom that a truss can safely be copied outright, but after the stresses have been computed, such illustrations as have been given will be found of much assistance in deciding on the manner of building the truss, proportioning the joints and bracing the roof. This is particularly true in the matter of steel trusses.

In conclusion the author would advise those readers who are studying the subject for the first time and who wish to thoroughly understand the correct manner of designing a roof truss to first study carefully Chapters I., II. and III., which in a general way explain the mechanical principles of different kinds of trusses and then to study with great care Chapters VII –VIII. The only way in which one can learn to draw a stress diagram is by taking paper and pencil and drawing the diagram to a scale, line by line, in accordance with the instructions given. It can never be learned by simply reading the book. After a few of the examples given have been worked out, the student should apply the method to similar trusses with different proportions, when the general principle will become apparent and once this is understood it can readily be adapted to almost any kind of truss.

By persistent effort, any person of average intelligence should be able to master the principles of graphic statics, as applied to ordinary types of trusses, and once mastered the stresses can be very easily and quickly determined. Without being able to determine the stresses, however, it is impossible to economically proportion a truss to its load and span with any degree of accuracy, or with complete confidence in its safety.

Chapter I.

TYPES OF WOODEN TRUSSES AND THE MECHANICAL PRINCIPLES INVOLVED.

1. INTRODUCTION.—It is possible for one to correctly lay out or plan a trussed roof without being able to determine the stresses in the various parts, but to do so a knowledge of the mechanical principles involved is absolutely essential, unless one can find a similar case to follow, and even then he might copy a poor example.

Every architect, draughtsman and master builder should thoroughly understand the way in which the various members of the common types of trusses are made to support the weight of the roof, or other loads, why the braces run this way or that, and which members are in compression and which in tension. This much is necessary to enable one to make the preliminary drawings of the roof in an intelligent manner. If the general design is correctly laid out, a structural engineer may be engaged to determine the stresses and compute the size of the members, detail the joints, etc., while the general drawings are being finished by the architect. although, of course, it is desirable for the architect to be able to do all of this work himself.

In this chapter, the author has endeavored to explain the way in which the stresses act in the more common types of wooden trusses, and to give examples of nearly all the various forms used in building.

2. DEFINITIONS.—According to Professor Lanza the term "truss" may be applied to any framed structure intended to support a load.

In order, however, to distinguish a truss from a mere framework the author prefers the following definition:

"A truss is a triangular, polygonal or curved framework supported only at the ends (or in the case of a cantilever truss at the centre), and so designed that it cannot suffer distortion without crushing or pulling apart one of the pieces of which it is composed."

A true truss does not depend upon the rigidity of its joints for its stability, and imposes only a vertical pressure on the walls.

"A joint" of a truss is the intersection of two or more members of a truss. If a member is built up lengthways with two or more pieces of material, the place where the pieces join would ordinarily be spoken of as a joint, but such joints are not truss joints. In this work they will generally be designated as splices, in distinction from the joints proper.

"A member" of a truss is any straight or curved piece which connects two adjacent joints of the truss. Members are also often called pieces.

"Ties" are those members of a truss which are in tension only. They may be either of wood, iron or steel, and of any cross section.

"Struts" are those members of a truss which are in compression only. They may also be of either wood, iron or steel, and of any cross section, capable of resisting flexture.

"Tie-beams" are ties that are also subjected to a transverse strain. The main horizontal tie of wooden trusses is often called the "tie-beam," even when it has no transverse strain, but the term is not then strictly correct.

"Strut beams" are struts that are also subject to a transverse strain.

"Chords." In horizontal and bridge trusses, the top member and the main horizontal tie, are often called "chords," the upper one being the top chord and the lower one the bottom chord. (In queen rod trusses the top chord is sometimes termed the "straining beam.")

In the King rod or Queen rod trusses, the main slanting struts are often called "rafters" or "principal rafters," because they are usually parallel, or nearly so, with the rafters of the roof.

"Purlins" are horizontal beams, sometimes trussed, extending from truss to truss to support the rafters or ceiling joists.

"Stress." The term "stress" denotes an internal resistance which balances an exterior force, or if we imagine a piece of material, subject to an external force, cut in two at any point, the force with which one part of the piece acts upon the other at this section is called the "stress." In connection with trusses, the term is also very commonly used to denote the force which any given member is required to resist. In this country stress is commonly measured in pounds or tons.

"Unit Stress" is the stress on a unit of area, generally the square inch.

The "stress per square inch" is equal to the total stress divided by the number of square inches in the section on which it acts. Thus if a strut 6 inches square, is subject to a compressive stress of 18,000 lbs., the unit stress is 18,000, divided by 36 or 500 lbs.

"Strain." When a solid body is subjected to a stress of any kind, an alteration is produced in the volume or shape of the body, and this alteration is called the "strain." Strain is, therefore, the result of a stress or stresses. For safe stresses the strain produced in a strut or tie, is very minute; in the case of a beam, the strain is the elongation of the fibres on one side and the shortening of the fibres on the opposite side, due to the bending of the beam, the "pull" on the bottom fibres being commonly termed, the "fibre stress."

3. DEVELOPMENT OF THE SIMPLEST FORM OF A TRUSS.—The simplest method of supporting a weight between two supports (other than by a beam) is shown in Fig. 1.

Here we have a single load, W, to be supported about half way between the sides of a ravine. If the sides are sufficiently firm it is evident that the load may be supported by means of two inclined posts or "struts" meeting at their upper ends and supported at their lower ends by the ground.

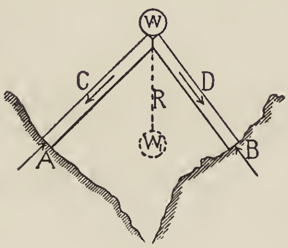


Fig. 1.

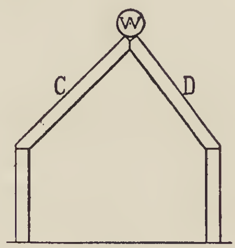


Fig. 2.

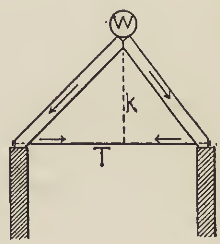


Fig. 3.

It is also evident that the weight W will produce compressive stresses in each of the struts, C and D, which will act in the direction indicated by the arrows.

To keep the struts in position the ground or ledge must offer a resistance in the opposite direction equal to the stresses in the struts, and the surface of the ledge should be exactly at right angles to the axis of the struts.

It may also be taken as evident that the effect on the struts and on their supports is the same whether the load is placed at W or suspended directly from below as at W_1, the only difference between the two cases being that in the latter an additional member, R, is required to transmit the weight to the upper ends of the struts.

The construction shown in Fig. 1, however, is not a true truss, as it exerts an outward thrust on its supports and depends upon them to keep the feet of the struts in position.

It is evident that if the struts C and D were supported only by two vertical posts, as in Fig. 2, the whole structure would immediately fall.

In a building the walls, as a rule, are only intended to support weights which act vertically, and the construction shown in Fig. 1 is not applicable to such conditions. To make it applicable it will be necessary to take up the horizontal components of the stresses in the struts by means of a third member, T, Fig. 3, leaving only the vertical components to be resisted by the walls or other supports.

4. COMPOSITION AND RESOLUTION OF FORCES.—The last paragraph brings us to the composition and resolution of forces, of which some knowledge will be required to understand the explanations which follow. For the benefit of those who have not studied mechanics, a brief explanation will be given of the laws relating to the action of forces when applied at a single point.

The "resultant" of two or more forces is that single force which would exactly replace them and have the same effect on the body acted upon as the given forces.

The "components" of a force are the two or more simple forces by which it may be replaced: hence any force may be considered as the resultant of two or more other forces by which it may be replaced.

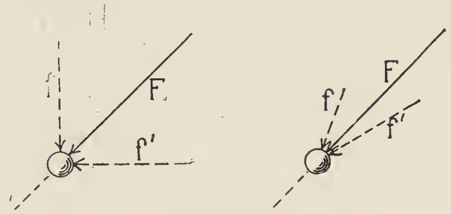


Fig. 4.

For example: if a force represented in direction and magnitude by the line F, Fig. 4, is applied to a ball resting on a plane horizontal surface it will cause the ball to move forward in the direction of the dotted line. The ball may be made to move in the same direction, however, by means of two forces, f and f^1, provided they are properly proportioned. The forces f and f^1 may therefore be considered as the components of the force F, as they fulfill the conditions of the above definition. The component forces may be at any angle with the given force less than 90°, but they must be applied on opposite sides of the given force. In trusses oblique forces are generally, although not always, resolved into horizontal and vertical components.

Again, if two forces, f and f^1, Fig. 5, act on a ball, the ball will move forward in the direction of the diagonal of a parallelogram, of which the forces form two sides, and *the resultant of any two forces is indicated in magnitude and direction by the diagonal of the parallelogram formed upon them.* Conversely, the magnitude of two compo-

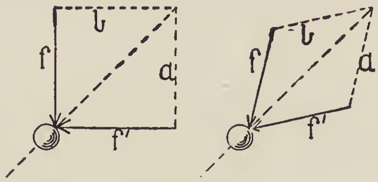


Fig. 5.

nents of any force is represented by the sides of the parallelogram having the given force as its diagonal.

Another application of this law is that any given force may be "balanced" by two other forces acting in the opposite direction to its components, or two forces may be balanced by a single force acting in the opposite direction to the resultant of the given forces. Thus, the force F in Fig. 6 (either diagram) acting on the ball may

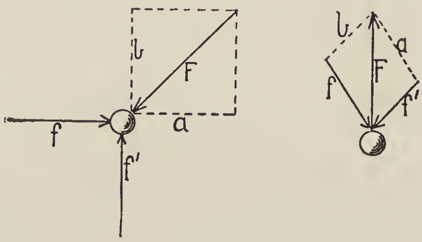


Fig. 6.

be exactly balanced by the two forces f and f^1 if equal in magnitude respectively to the sides a and b of the parallelogram, having the given force F for its diagonal. Conversely, the forces f and f^1 will be balanced by the force F.

A given force may also have any number of components, but as in trusses it is seldom that any given force is resisted by more than two other forces, it is not deemed necessary to consider more than two components.

Another principle that requires notice at this time is that whenever a strut or tie is subjected to compression or tension, that it shall be kept in position, *the resistance at each end must equal the stress in the piece*, and must act in opposite directions. Thus, if two boys are pulling on a rope so that one balances the other, and one boy is pulling with a force of 50 pounds, the other boy will be exerting exactly the same force, but the strain in the rope will be but 50 pounds. This is a truth not always comprehended. It may, perhaps, best be seen by considering a person pulling on a rope the other end of which is attached to the hook of a spring balance. In this case it will readily be seen that the gauge of the balance will indicate the force with which the person is pulling and also the stress on the rope, but it should also be remembered that the spring is pulling back with exactly the same force that the person is exerting.

These are very simple illustrations, but the author has found that the truth which they illustrate is not always comprehended, or, at least, is not considered.

To return to our truss, Fig. 3. It should now be readily understood that if the horizontal component of the thrust in the struts is resisted by the tie T only the vertical component will need to be resisted by the walls.

Again, the strain in the tie T will only be equal to the horizontal component of the stress in *one strut*, and the horizontal components of the strut stresses must equal each other. If the weight W is nearer one support than the other, so that the struts are not equally inclined, the thrust in the struts will not be alike, and the vertical component of the steeper strut will be greater than the vertical component of the other; but the horizontal components *must* be equal, otherwise the tie would be pulled along endways.

5. DEVELOPMENT AND ANALYSIS OF THE KING-ROD TRUSS.—The construction shown in Fig. 3 is a true truss and the simplest form in which a truss can be constructed. It also contains *all the pieces* required to support the weight W.

If a vertical member, k, were added, it must be evident that it would not affect the strain in the other pieces, except as it increases by its own weight the load at W; consequently, when only a single load is to be supported, and that is at the apex, a vertical rod is of no use, except to prevent the tie from sagging under its own weight.

The tie T may be of steel, iron or wood, and of any shape, provided that it has sufficient tensile strength and can be properly secured to the ends of the struts. In wooden trusses, however, it is

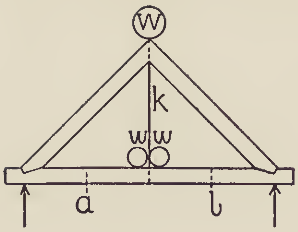


Fig. 7.

generally more convenient to make it of wood, as shown in Fig. 7, which shows the next development of this truss. We have here a load to be supported at the apex of the truss, and also a load (represented by w w) at the centre of the tie-beam. To prevent transverse strain in the tie-beam it is evident that we must use the tie k to suspend the load w w from the apex. The tie k will then be strained by the amount of the load w w, and the load at the apex will be increased by the same amount. This will consequently increase the compression in the struts and the tension in the tie-beam.

In practice the load w w is made up of the weight of the tie-beam from a to b and the loads supported by that portion of the beam.

Fig. 8 represents the next step in the development of this truss. Here we have three loads, which are supported in the following manner:

The load W_1 is supported by the struts a and c and the load W_3 is supported by b and d.

[Note.—Although the rafter a e is usually of one piece of material, it really forms two different members of the truss, being separated by a joint; hence in theory a and e are two different pieces.]

The horizontal components of the stresses in a and b are resisted by t, and those of the stresses in c and d neutralize each other. The vertical components of the stresses in c and d are taken up by the rod r and transmitted to the apex, where they are added to W_2, and the sum supported by the full length of the rafters. The thrusts produced by the stresses in the full rafters are again resisted by the full length of the tie. The above explanation may be illustrated by the diagram, Fig. 9, in which each stress is represented (in direction, but not necessarily in magnitude) by a line. Thus, the sides of the triangle A represent the stresses produced by the load W_1; B, the stresses produced by W_3; the vertical tie, the stress required to support the inner ends of triangles A and B, and the large triangle represents the stresses produced by W_2 and the tie r. It will thus be seen that the tie-beam and the lower portion of the rafters receive two stresses and the other parts but one.

If the load W_2 is half way between the supports, and W_1 and W_3

are half way between the supports and the centre, then the tension in r will equal $\frac{1}{2}$ W_1 + $\frac{1}{2}$ W_3, plus half the weight of the tie-beam. If the tie-beam supports a ceiling over its full length, then one half of the load will also be supported by r.

In practice the weight of the members is generally neglected in determining the stress in the rods.

Trusses of the form shown in Figs. 7 and 8 are commonly desig-

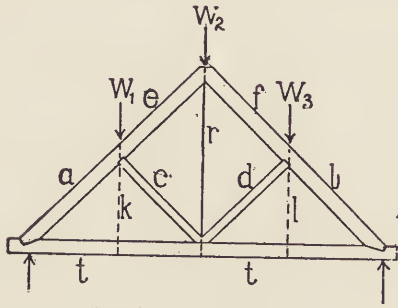


Fig. 8.—King Rod Truss.

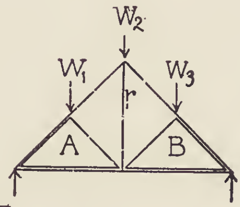


Fig. 9.

nated as "king-rod" or "king-post" trusses, they being the modern form of the old king-post truss, in which a wooden post was used for the centre tie. King-rod trusses are sometimes seen with rods at k and l. When the tie-beam supports a ceiling they may be used to advantage, but where there is no ceiling they are useless.

6. SIX-PANEL QUEEN TRUSS.—When the length of the rafter is greater than 24 feet it should be divided into three parts, as shown in Fig. 10, and purlins placed at each of the joints. The stresses in Fig. 10 are indicated by the lines of the diagram Fig. 11. By means of this diagram, and the explanation given for Fig. 9, the action of the various pieces in supporting the loads should be readily understood. For greater spans three pairs of struts may be used, as in Fig. 12, and this truss is also sometimes built with 10 panels, but it is not an economical type of truss for spans exceeding 65 feet.

The names given to Figs. 10 and 12 are those most commonly used to designate these types; they should not be confounded with the "queen-rod" truss, which has a horizontal top chord.

7. ANALYSIS OF THE QUEEN-ROD TRUSS.—If we have two equal loads, W_1, W_2, Fig. 13, to be supported at equal distances between the walls, we can support them in the manner shown in

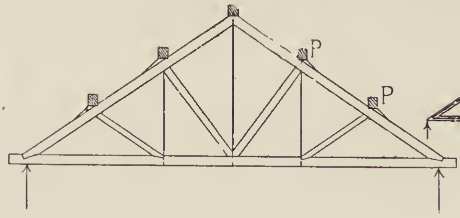


Fig. 10.—Queen Truss.

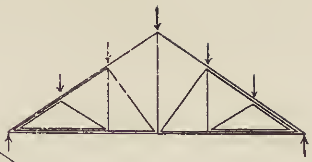


Fig. 11.

Fig. 14, and as long as the loads remain equal the frame will be stable; but should one load become greater than the other the heavier load would push the frame over, as shown by the dotted lines.

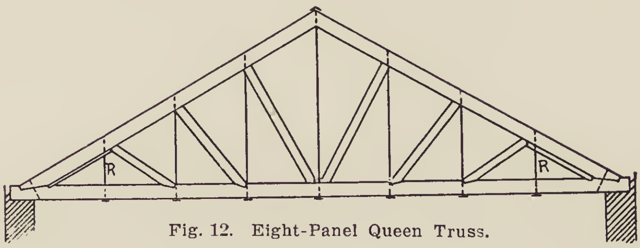


Fig. 12. Eight-Panel Queen Truss.

As in actual roof construction a part of the load, such as the snow and wind, is variable, one side of the roof often being loaded when the other is not, a truss of the shape shown in Fig. 14 cannot be used. To adapt this type of truss to the actual conditions of roof

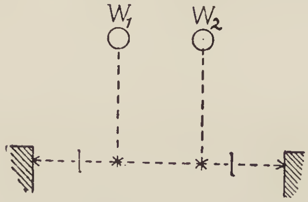


Fig. 13.

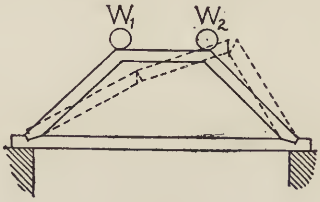


Fig. 14.

construction we must build it in the manner shown in Fig. 15. In this case we will assume that the point A is loaded and the point B unloaded. The load at A will then be supported by the rafter D and by the brace or strut C. The vertical component of the stress in C will be taken up by the rod E and transferred to the point B, where it will be resolved into a horizontal stress to be resisted by H and an oblique stress to be resisted by F, the horizontal components of the stresses in D, C and F being resisted by the tie-beam. The action of the pieces will be the same if both points are loaded provided the load at A is greater than that at B.

In practice the greater load may be at A at one time and at B at another time, so that it is necessary to use two sets of braces and two rods, as shown by the dotted lines.

The braces C and C^1 are called "counter braces," because their purpose is to counteract the effects of unequal loading; in roof

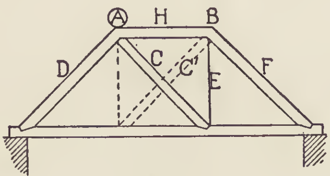


Fig. 15.

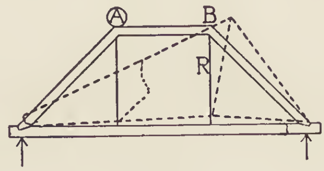


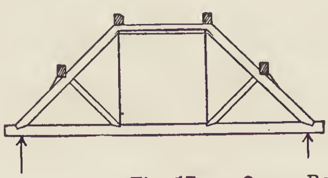
Fig. 16.

trusses they are generally brought into use only by the snow or wind.

Trusses of this type are often seen without counter braces, as in Fig. 16. When built in this way an unbalanced load at A would tend to distort the truss, as shown by the dotted lines.

When the dead loads, by which is meant those produced by the weight of the construction, are symmetrical and the tie-beam supports a ceiling, the weight on the rod R, the rigidity of the joints and the transverse strength of the tie-beam are generally sufficient to resist the tendency of the wind pressure to push the joint B out of position. When such a truss is placed longitudinally of the roof, and the truss and loads are symmetrical, then the loads at A and B will always be the same and the counter braces will not be required.

When such trusses are placed across the roof, and there is no ceiling to be supported, counter braces should always be used, as other-



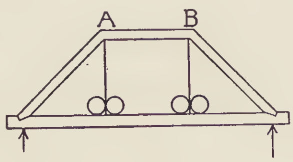


Fig. 17. Queen Rod Trusses. Fig. 18.

wise a severe wind pressure or an uneven distribution of snow may cause the truss to fail. The greater the inclination of the rafter the greater will also be the effect of the wind, and hence the greater need of counter braces.

Counter braces are also required in such trusses when they are subject to a moving load as in the case of bridges or when supporting floors.

When the length of the rafters exceeds 12 feet they should be braced as shown in Fig. 17. These braces will also assist considerably in preventing distortion under wind pressure, but in severe cases they are not sufficient.

In the trusses shown in Figs. 15 and 16 it obviously makes no difference with the strains in the rafters and tie-beam whether the loads are applied at A and B or are suspended directly underneath by means of rods, as shown in Fig. 18, the only difference in the two cases being in the strain in the rods.

Trusses of the shape shown in Figs. 17 and 18 are the modernized types of the old "Queen-post truss," in which all the members were of wood.

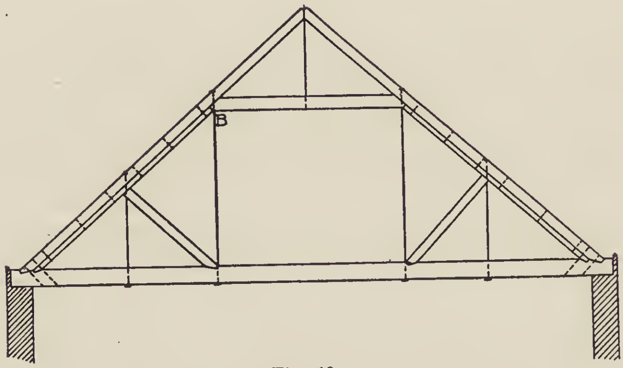


Fig. 19.

Fig. 19 shows a combination of a queen-rod and king-rod truss, sometimes used where it is desired to keep the centre of the attic free from obstructions.

In building this truss it will be more economical to form the lower portion of the rafters of two timbers as shown, than to make them of one size for the full length. This construction also allows of making a good joint at B. There is the same tendency to distortion under wind pressure in this truss as in the queen-rod truss, Fig. 16, but owing to the prolongation of the rafter to the ridge, the resistance to distortion is very much greater than in the queen-rod truss, so that for spans not exceeding 42 feet it will be perfectly safe to omit counter braces.

8. THE "HOWE" OR BRIDGE TRUSS.—For spans of from 34 to 44 feet, queen-rod trusses should be made of the form shown in Fig. 20, with a rod at the centre if the tie-beam is loaded, otherwise it is not needed, unless there are counter-braces.

When one of the points A or B is liable to be much more heavily loaded than the other, counter-braces should be used, as indicated by the dotted lines, and a rod placed at the centre to take up the vertical component of the stress in the counter-braces. If the members d and e, h and k are made and connected so as to resist both tension and compression, the counter-braces may be omitted, but it is generally more practicable to make d and e of rods, and use counter-braces. When the loads are nearly uniform and symmetrically disposed counter-braces are not required.

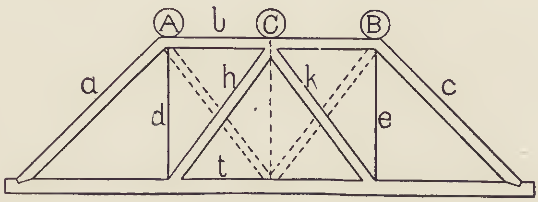


Fig. 20.

In this truss the loads at A and B, when uniform and symmetrical, are borne by the frame a b c. The load at C is supported by the braces h and k, and these again by the ties d and e, which transmit the vertical components of the stresses in h and k to the points A and B. The tie t resists the thrust of a and c through its entire length, and the middle portion, in addition, has to resist the horizontal components of the stresses in h and k. When the points A, B and C are symmetrically disposed with regard to the supports and there is no load on the tie-beam, the rods d and e will each be strained by an amount equal to one-half the load at C.

This type of truss may be extended by increasing the number of struts to almost any length, as shown by Figs. 21-24, although 100 feet is about the greatest practical span. The truss may also have either an even or uneven number of panels, although an even number is generally to be preferred.

Trusses of this type, with five or more panels are commonly called "Howe" trusses, although the original Howe truss was designed for bridges.

When adapted to the shape of the roof it is the most economical truss for wooden construction, for spans not exceeding 100 feet, and is a very easy truss to build. The horizontal members in these trusses are commonly called "chords," and the portion of the truss between two adjacent vertical members, a "panel."

9. MECHANICAL PRINCIPLE OF THE HOWE TRUSS. —The action of the pieces in supporting the loads is the same as in truss 20. When the rods and struts are symmetrically disposed

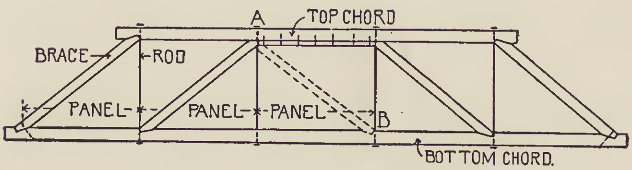


Fig. 21.—Five-Panel Howe Truss.

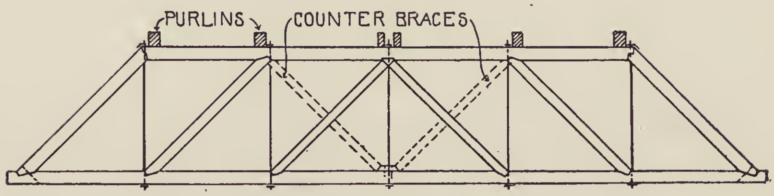


Fig. 22.—Six-Panel Howe Truss.

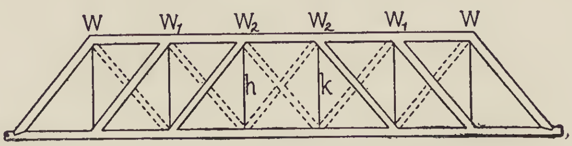


Fig. 23.—Seven-Panel Howe Truss.

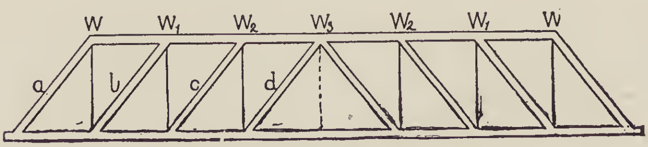


Fig. 24.—Eight-Panel Howe Truss.

each side of the centre, the strut d (Fig. 24) supports $\frac{1}{2}$ of W_3; strut c, $\frac{1}{2}$ $W_3 + W_2$; strut b, $\frac{1}{2}$ $W_3 + W_2 + W_1$; and the strut a, $\frac{1}{2}$ $W_3 + W_2 + W_1 + W$, although the stress will be increased in the proportion that the length of the struts bears to the vertical distance between centres of the chords. The stresses in the rods also increase in the same proportion toward the ends. On the other hand, the stress in the chords *is greatest at the centre*, the same as in a beam, the top chord always being in compression and the bottom chord in tension. If the tie-beam or bottom chord is loaded the loads are transmitted by the rods directly to the upper joints, and are added to the loads, W, W_1, etc.

When there is no load on the tie-beam, the centre rod of 6, 8 and 10 panel trusses, and the rods each side of the centre panel (as h and k, Fig. 23,) will have no stress from the roof load, and although they should be inserted to sustain the tie-beam, they need not be more than $\frac{3}{4}$ or $\frac{7}{8}$-ins. in diameter.

10. RULES TO BE OBSERVED WHEN DESIGNING HOWE TRUSSES.—*Height.* The height of the truss, always measured from "centre to centre" of the chords, should never be "less than one-ninth of the span," for spans up to 36 feet, or than one-tenth of the span for spans from 40 to 80 feet. As a general rule, a height of from one-seventh to one-sixth of the span will be most economical.

Number of Panels. As a rule, the number of panels should be such that the braces will have an inclination of from 36° to 60°, an inclination of about 45° being the most desirable.

[Note.—Howe trusses can be built with a height of one-twelfth of the span, when the latter exceeds 36 ft., or with the braces at an inclination of only 30°, but these proportions will greatly increase the stresses and necessitate larger timbers and rods.]

If the position of one or more of the purlins is fixed by some special requirement, then the panels should be so arranged that the upper end of a brace will come under the purlin, and that the inclination of none of the braces will be less than 36°.

Although it is generally better to have the truss symmetrical about the centre, it is not absolutely necessary, nor is it necessary that the panels be of uniform width.

When the truss is "not symmetrically loaded" however, it may be necessary to reverse the brace in one of the centre panels. This point is more fully considered in Chapter VIII.

Counter Braces. If there is any chance of the truss being more heavily loaded on one side of the centre than on the other, counter-braces—that is, braces in the opposite direction from the regular braces—should be inserted in the centre panels, as shown by the dotted lines in Figs. 22 and 23. When there is no likelihood of one side of the truss being more heavily loaded than the other, counter-braces will not be needed. Where and to what extent counter-braces should be used is explained in Chapter VIII.

Bearing on Wall or Post. The point where the centre lines of the end brace and of the tie-beam intersect should always come over the support, if possible, and generally at least 6 ins. beyond the inner face of the wall. This applies to all trusses.

11. TABLE OF DIMENSIONS FOR HOWE TRUSSES.—For symmetrical trusses having panels of uniform width, and uniformly loaded, the stresses in the different parts will be proportional to the span, number of panels, height of truss, spacing of trusses and the load per square foot.

It is therefore practicable to tabulate the strains in trusses of a given number of panels for different loads, spans, spacings and height. For the convenience of architects and builders the author has prepared the following table, which gives the required dimensions for wooden trusses having "six panels" or braces, and for heights of one-sixth and one-eighth of the span, and supporting a flat roof of either tin, sheet-iron or composition, with a plastered ceiling supported by the tie-beam.

These dimensions apply only "to trusses which have purlins placed at the upper joints" and where the height is at least equal to that given in the table; a greater height may be used with the same dimensions, but a less height materially increases the strains and will require an increase in the dimensions. It is also important that the panels be all of the same width (within a few inches) and always six in number.

If there is no ceiling to be supported the lower chord may be of the same size as the upper chord. Where the rafters rest directly on the top chord the dimensions of the latter *must be greatly increased*, and special calculations should be made therefor.

Six-panel trusses were chosen for the reason that this number of panels will generally be found the most economical of material up to spans of 60 feet. For greater spans seven or eight-panel trusses will probably be more economical where a ceiling is to be supported.

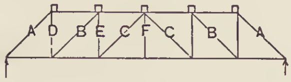
Wherever the conditions of load, span, height and spacing are not exactly as given in the table, or for localities where a greater allowance must be made for snow, "special calculations" should be made of the strains and corresponding dimensions, but even in such cases the table will serve somewhat as a check upon the calculations, and will be convenient for making preliminary drawings and estimates.

TABLE I.—DIMENSIONS FOR SIX-PANEL HOWE TRUSSES.

TO SUPPORT A FLAT (COMPOSITION) ROOF AND PLASTERED CEILING.

Purlins to be Placed at Upper Joints.

Computed for heights of one-sixth and one-eighth of the span and for a snow load of 16 pounds per square foot. Chords and braces, Norway pine; verticals, wrought iron rods.



Span.	Distance apart, C to C.	Total height.		Top chord.	Bottom chord.	Braces.			Rods (not upset).		
						A	B	C	D	E	F
Ft.	Ft.	Ft.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.
36	12	6	7	6x6	6x8	6x6	6x4	6x3	1 1/8	3/4	5/8
		5	2	6x8	6x8	6x6	6x6	6x4			
	15	6	8	6x8	6x8	6x6	6x4	6x3	1 1/4	7/8	5/8
		5	2	8x8	8x8	8x6	6x6	6x4			
	18	6	8	6x8	6x8	6x8	6x6	6x4	1 1/4	7/8	5/8
		5	2	8x8	8x8	8x8	6x6	6x4			
42	12	7	7	8x6	8x8	8x6	8x4	6x4	1 1/4	7/8	5/8
		5	11	8x8	8x8	8x6	8x5	8x4			
	15	7	8	8x8	8x8	8x6	8x5	6x4	1 3/8	7/8	3/4
		5	11	8x8	8x8	8x8	8x6	8x4			
	18	7	8	8x8	8x8	8x8	8x6	8x4	1 1/2	1	3/4
		6	1	8x10	8x10	8x8	8x6	8x4			
48	12	8	8	8x8	8x8	8x8	8x6	8x4	1 3/8	7/8	3/4
		6	8	8x8	8x8	8x8	8x6	8x4			
	15	8	8	8x8	8x8	8x8	8x6	8x4	1 3/8	1	3/4
		6	10	8x10	8x10	8x8	8x6	8x4			
	18	8	8	8x8	8x8	8x8	8x6	8x4	1 1/2	1	3/4
		6	10	8x10	8x10	8x10	8x6	8x4			
54	12	9	8	8x8	8x8	8x8	8x6	8x4	1 3/8	7/8	3/4
		7	6	8x8	8x10	8x8	8x6	8x4			
	15	9	8	8x8	8x8	8x8	8x6	8x4	1 1/2	1	3/4
		7	7	8x10	8x10	8x8	8x6	8x4			
	18	9	10	8x10	8x10	8x10	8x8	8x6	1 5/8	1 1/8	3/4
		7	7	10x10	10x10	10x8	8x8	8x4			
60	12	10	9	8x8	8x10	8x8	8x6	6x6	1 3/8	1	3/4
		8	4	8x10	8x10	8x10	8x6	8x4			
	15	10	10	8x10	8x10	8x10	8x6	6x6	1 1/2	1 1/8	3/4
		8	4	10x10	10x10	10x8	10x6	8x4			
	18	10	10	10x10	10x10	10x8	10x6	8x6	1 3/4	1 1/8	3/4
		8	4	10x10	10x10	10x10	10x6	8x6			
70	12	12	6	8x10	8x10	8x10	8x6	6x6	1 1/2	1	3/4
		9	7	10x10	10x10	10x8	10x6	8x6			
	15	12	6	10x10	10x10	10x8	10x6	8x6	1 3/4	1 1/8	3/4
		9	9	10x12	10x12	10x10	10x8	10x6			
	18	12	6	10x10	10x10	10x10	10x6	8x6	1 7/8	1 1/4	7/8
		9	9	10x12	10x12	10x12	10x8	10x6			
80	12	14	2	10x10	10x10	10x10	10x6	8x6	1 5/8	1 1/8	3/4
		10	10	10x10	10x10	10x10	10x6	8x6			
	15	14	2	10x10	10x10	10x10	10x8	8x6	1 7/8	1 1/4	7/8
		11	0	10x12	10x12	10x10	10x8	10x6			
	18	14	4	10x12	10x12	10x12	10x8	8x6	2	1 3/8	1
		11	1	10x12	10x14	10x12	10x8	10x6			

12. THE LATTICE TRUSS.—This is a form of truss designed by Ithiel Towne for bridges long before iron was used in this country for such work. Several railroad bridges were built on this principle and the truss has proved very efficient in supporting loads. The truss is well adapted to the support of flat roofs in localities where large timbers and iron rods are expensive or difficult to obtain.

The general shape of the truss, as used for supporting roofs is shown in Fig. 25. The truss is composed of top and bottom chords, united by a lattice of planks and by vertical pieces at the ends. The inclination of the braces or lattice should be the same in both directions and as near 45 degrees as an even division of the span will permit. In the original truss the planks forming the lattice were secured to each other at their crossings, and to the chords and end pieces by wooden pins called "treenails." In the modern truss iron bolts are used for this purpose, although dry oak pins might be used at the intersection of the braces.

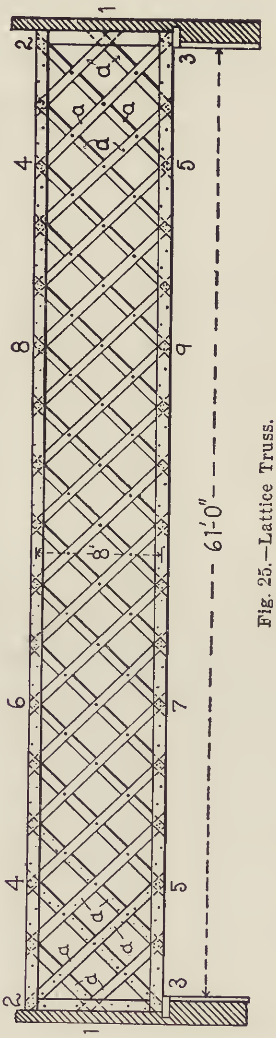


Fig. 25.—Lattice Truss.

The construction is very simple and can be made by any carpenter, and the materials are such as may be easily obtained in almost any village. There is no difficulty in making the truss strong enough to carry any roof load for spans up to 80 ft., but owing to the fact that it requires a large amount of lumber and cannot be tightened up, it is not as desirable a truss to use, where rods can be readily obtained, as the Howe truss.

Proportions and Construction. The height of a lattice truss, measured between the centre lines of the chords, should be from one-eighth to one-sixth of the span and the braces should be placed at an angle of about 45 degrees. When laying out a lattice truss, the first step should be to determine the height, and then the number of spaces between the joints in the top and bottom chords.

To find the number of spaces, multiply the span by two, and divide by the height, using the nearest whole number. Thus if the span is 60 ft. and the height 8 ft. there should be $\frac{2 \times 60}{8} = 15$ spaces. If the height is 10 ft. there should be 12 spaces. The truss shown in Fig. 25 has 16 spaces.

Having determined the height and number of spaces, fix the centre of the end joints, and divide the distance between into the number of spaces determined upon, thus fixing the position of the braces. The chords should be built of four thicknesses of plank, two on each side of the truss, and breaking joint opposite their centres, using as long planks for the tie-beam as can be obtained. At the ends, vertical planks should be cut between the chords, on each side of the bracing, to act as posts. The braces should be

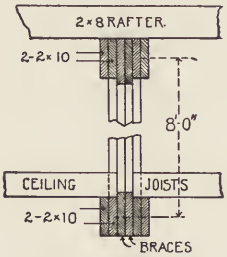


Fig. 26.—Vertical Section.

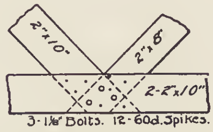


Fig. 27.—Detail of Joint 5, of Fig. 25.

bolted to the chords and end posts, and also to each other, where they cross. A goodly number of spikes should also be used in the joints, as indicated in Fig. 27.

The bottom chord should also be bolted every two feet between the joints, as this member is in tension. The top chord, being in compression, will be tied sufficiently by the bolts at the joints, and by a short bolt on each side of each butt joint. The strain on the joints near the ends of the truss will be much greater than on the centre joints.

The first three joints at each end, should have as many, and as large bolts, as is given in the last column of Table II. The bolts in the next three joints may be slightly reduced in size, and those in the centre joints still more.

When the span of the truss exceeds 40 feet, short pieces of plank should be spiked to the end braces, a, a, fitting tightly between the other set of braces, to give them additional strength.

It should be kept in mind that the strength of a lattice truss is usually measured by the strength of the joints.

13. STRESSES IN A LATTICE TRUSS.—A lattic truss acts in very much the same way as a beam in supporting a transverse load. The chords resist the bending moment and the bracing transmits the load to the supports, or, in technical language, resists the shearing stress. Half of the braces are in compression and half are in tension. Uprights at the ends are necessary to receive the shear at the top and at the middle, and transmit it to the support below.

The stress in the braces is greatest at the ends and decreases to nothing at the centre, and hence the braces near the centre of the truss may be made smaller than those at the ends.

The stress in the chords, on the contrary, is greatest at the centre and decreases toward the ends, hence the centre planks should be as long as can be obtained.

Rules for Computing the Stress in the Chords and Braces.

I. Under a uniformly distributed load, the maximum stress in the chords may be found by multiplying the total load by the span and dividing by eight times the height, both in feet.

II. The stress in each of the end braces, a, a, a, when the angle of inclination approximates 45 degrees, will be one-sixth of the total load, multiplied by 1.4.

The following table gives the dimensions for lattice trusses, built as shown in Fig. 25, for five different spans, and different spacings and heights, which will cover nearly all of the conditions under which these trusses should be used. In localities where a fall of snow 2 feet in depth is liable to occur these dimensions should be increased.

Referring to Fig. 25, it may be stated that each chord of the truss is built of four 2 × 10's in 10 and 20-foot lengths, the braces a, a, a, are 2″ × 10″, and the other braces 2″ × 8″. The joints at 1, 2, 3, 4, and 5 have three $1\frac{1}{8}$-inch bolts, the joints between 6 and 8 and 7 and 9 have two $\frac{7}{8}$-inch bolts, while the other joints have two 1-inch bolts. There should also be two $\frac{7}{8}$-inch bolts in the tie-beam, in each space between the joints, to assist in transmitting the tension from one plank to the other.

14. WOODEN TRUSSES WITH RAISED TIE-BEAMS.—All of the trusses thus far described have horizontal tie-beams,

TABLE II.—DIMENSIONS FOR LATTICE TRUSSES OF FIRST QUALITY WHITE PINE OR SPRUCE.

TO SUPPORT A GRAVEL ROOF AND PLASTERED CEILING, ALLOWING 20 POUNDS PER SQUARE FOOT FOR SNOW.

Span.	Spacing of trusses.	Height out to out.		No. of spaces.	Size of bottom chord.	Size of top chord.	End braces.	Inner braces.	Bolts in joints 1-5. See Fig. 25.
Ft.	Ft.	Ft.	Ins.						Inch.
40	12	5	6	16	4—2 x 6	4—2 x 6	2 x 6	2 x 6	2—1
		7	2	12	4—2 x 6	4—2 x 6			
	14	5	7	16	4—2 x 6	4—2 x 8	2 x 6	2 x 6	3—7/8
		7	3	12	4—2 x 6	4—2 x 8			
	16	5	8	16	4—2 x 8	4—2 x 8	2 x 8	2 x 6	3—1
		7	4	12	4—2 x 8	4—2 x 8			
50	12	6	8	16	4—2 x 8	4—2 x 8	2 x 10	2 x 8 and 2 x 6	3—1
		8	8	12	4—2 x 8	4—2 x 8			
	14	6	8	16	4—2 x 8	4—2 x 8	2 x 10	2 x 8 and 2 x 6	3—1 1/8
		8	8	12	4—2 x 8	4—2 x 8			
	16	6	9	16	4—2 x 8	4—2 x 10	2 x 10	2 x 8 and 2 x 6	3—1 1/8
		8	8	12	4—2 x 8	4—2 x 8			
60	12	8	4	16	4—2 x 10	4—2 x 10	2 x 10	2 x 8	3—1 1/8
		10	10	12	4—2 x 10	4—2 x 10			
	14	8	4	16	4—2 x 10	4—2 x 10	2 x 10	2 x 8	3—1 1/8
		10	10	12	4—2 x 10	4—2 x 10			
	16	8	4	16	4—2 x 10	4—2 x 10	2 x 10	2 x 8	3—1 1/4
		10	10	12	4—2 x 10	4—2 x 10			
70	14	9	5	16	4—2 x 10	4—2 x 12	2 x 10	2 x 8	1—2 1/4
		12	4	12	4—2 x 10	4—2 x 10			
	16	9	5	16	4—2 x 10	4—2 x 12	2 x 10	2 x 8	1—2 3/8
		12	4	12	4—2 x 10	4—2 x 10			
	18	9	6	16	4—2 x 10	4—2 x 12	2 x 10	2 x 8	1—2 1/2
		12	6	12	4—2 x 10	4—2 x 12			
80	14	11	0	16	4—2 x 12	4—2 x 12	2 x 10	2 x 8	1—2 3/4
		14	0	12	4—2 x 12	4—2 x 12			
	16	11	2	16	4—2 x 14	4—2 x 14	2 x 12	2 x 10 and 2 x 8	1—3
		14	0	12	4—2 x 12	4—2 x 12			
	18	11	2	16	4—2 x 14	4—2 x 14	2 x 12	2 x 10 and 2 x 8	1—3
		14	1	12	4—2 x 12	4—2 x 14			

Uprights at end same size as end braces.

which are the most desirable as well as the most economical whenever the conditions will permit.

In roofing churches, public halls, etc., a raised ceiling is often desired in order to give greater height to the room, without increasing the height of the walls.

SCISSORS TRUSSES. For such roofs, some form of the scissors truss (so named from its resemblance to a pair of scissors) is most often used. When correctly designed with members of the proper size, and with the joints carefully proportioned to the stresses, the scissors truss makes a very good truss for supporting the roof over halls and churches, up to a span of 48 feet, but above that, they should be used with much caution.

15. Mechanical Principle of the Scissors Truss.—If we take a simple king-rod truss, as shown in Fig. 28, and screw up on the centre rod, we will raise the tie in the centre and bring the feet of the rafters toward each other, as shown by the dotted lines; consequently by making the tie and centre rod of the proper length and strength we can have the form of truss shown in Fig. 29, and still keep the feet of the rafters a and b from spreading. To do so, however, the tie c c must offer a much greater resistance than when it is level, and it requires great strength in the centre tie r to keep the tie c c in position.

The pieces d d are simple struts used for bracing the rafters or to assist in supporting the purlins. In practice d and c are generally made in one piece, for convenience of construction, but it should

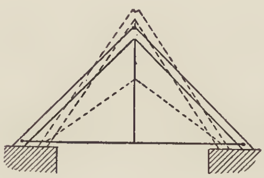


Fig. 28.

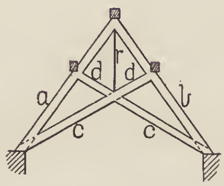


Fig. 29

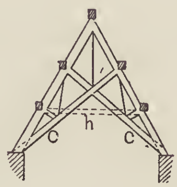


Fig. 30.

be remembered that the part c is in tension and the part d in compression. If there are two purlins on each side to be supported an additional brace and tie should be used, as shown in Fig. 30. This also increases the tension in the lower part of the tie c. Very often a "collar-beam" is placed across the truss, as shown by the dotted lines at h (Fig. 30) and the ends are made to take the place of the lower brace. If the collar-beam is made of two timbers bolted on each side to the ties c c, it will reduce the stress in the centre rod, but increases the stress in the other two rods. As there is no way of telling just how much stress the collar-beam will appropriate, it is generally better not to put in the collar-beam unless it is necessary to support the ceiling.

Trusses of the type shown in Figs. 29 and 30 are commonly known as "scissors trusses." When using such trusses the inclination of the tie-beams should be made as little as the conditions will permit, and the stresses should be accurately determined.

The lines a and c, Fig. 31, represent the stresses in the rafter and tie-beam, respectively, of the truss shown in Fig. 29, while the lines a^1 and c^1 represent the stresses in a truss of the same span and pitch, but with a horizontal tie-beam, the external load being the same in both cases. The line r, Fig. 31, represents the stress in the centre vertical tie, Fig. 29, due to the external load only, while in a king-rod truss, such as is shown in Fig. 8, the stress from the same load is represented by the line r^1.

As the size of the truss members is governed principally by the stresses, it is evident that for the same span and loads a truss of the shape shown in Figs. 29 and 30 will cost considerably more than one with a horizontal tie-beam.

Many examples of scissors trusses are shown in Chapter IV.

16. *Another type of truss*, much resembling the scissors truss,

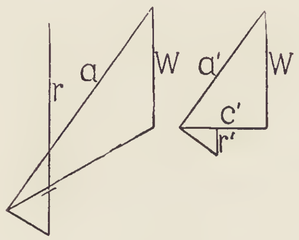


Fig. 31.

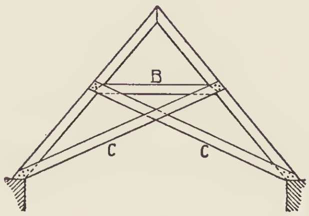


Fig. 32.

but being quite different in principle, is shown in Fig. 32. In this truss the pieces c c are in tension *their full length*, and should pass by each other. The piece B, in this truss is in compression, and must be of pretty large dimensions to resist the compressive stress. This type of truss should not be used for spans exceeding 35 ft. A further illustration of this truss is shown in Chapter III.

17. TRUSSES WITH TWO CENTRE RODS.—To return again to our elementary truss of two struts and a tie, it is evident that if the tie is jointed as at A and B, Fig. 33, and we connect these joints with the apex by means of the rods r r, we can, by screwing up on the rods from the top, bring the truss into the shape shown in Fig. 34, and if the parts have sufficient strength to resist the strain, it is evident that the truss will still be in equilibrium.

This gives us another type of truss with a raised tie-beam, and one that is much used both for wooden and steel construction.

The most common forms of this type of truss, for wooden construction, are shown in Figs. 35, 36 and 38. In these trusses only the rafters or principals a a and the struts s s are in compression, the other members being in tension.

The outward thrust of the rafters is resisted by the tie t, t^1, t, which is kept in position by the ties r r. Besides resisting the thrust

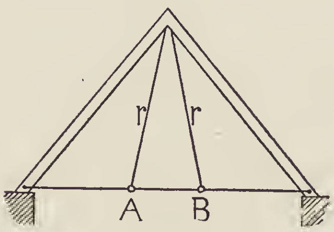


Fig. 33.

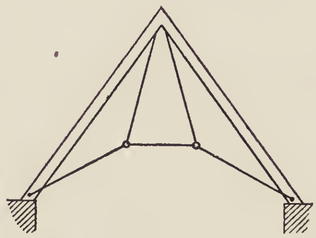


Fig. 34.

of the rafters, the portion of the tie t also acts as a belly-rod to sustain the thrust of the struts, which act in the same way as the post in a belly-rod truss. The purlins should always be placed opposite the braces s s.

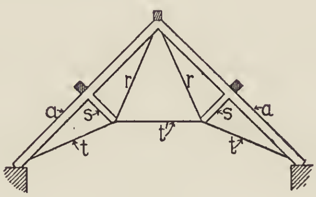


Fig. 35.

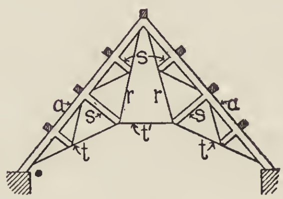


Fig. 36.

When built of wood this type of truss is most commonly classed with the scissors truss, but is also sometimes referred to as a "trussed rafter roof," as it really consists of two trussed rafters held in position by the ties t and r, as shown in Fig. 37, and the truss could be built in that way, but it would not be an economical shape, the figure being given merely to illustrate the principle of the truss and not as a model to follow.

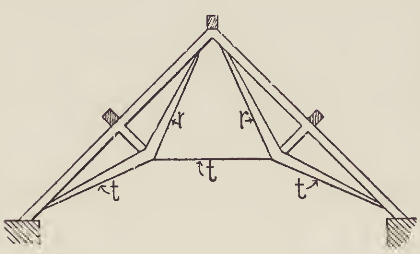


Fig. 37.

When built of steel, the type of truss shown by Figs. 35 and 36, is known as a "French" truss.

As in the trusses shown in Figs. 29 and 30, this type requires much greater resistance in the ties and rafters than a truss with a horizontal tie-beam, but the increase in the strains is not quite as great as in Figs. 29 and 30.

The trusses shown in Figs. 35 and 36, are not very well adapted to wood construction on account of the difficulty in making the joints, but for steel roof trusses in which a raised tie is desired, they are both practical and economical.

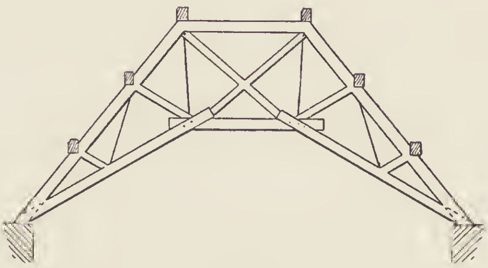


Fig. 38.

Fig. 38 shows the same type of truss, built of wood, but with the tops of the rafters cut off and a straining beam inserted between them.

Practical examples of this type of truss, as used in church roofs, are given in Chapter IV.

18. BRACED RAFTERS WITH WROUGHT IRON TIES.—When there is no ceiling to support, trusses of the type shown by Figs. 35 and 36, may be built with wooden rafters and wrought iron ties. Such trusses present a light appearance, offer a very practical

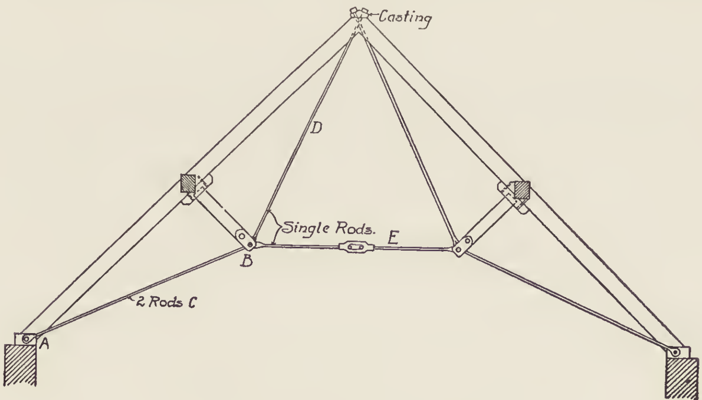


Fig. 39.—For Spans Up to 36 Feet.

form of construction, and are just about as desirable, for wooden roofs with moderate spans, as steel trusses while they are much cheaper.

Figs. 39 and 41, show examples of such trusses, suitable for many places. The dimensions given in Fig. 41, are for yellow pine or Oregon pine timbers and wrought iron rods and are ample for a slate roof, the trusses to be spaced from 12 to 14 ft. on centres.

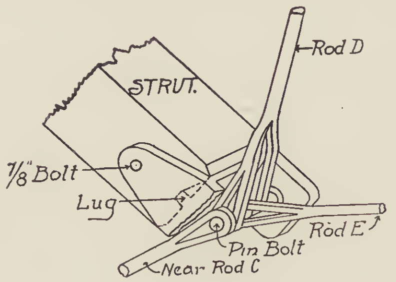


Fig. 40.—Detail of Joint B.

Trusses like Fig. 39 are sometimes seen with the rods C and D continuous. They should not be made in this way, however, as the stress in C is greater than that in D. The best way of making the connection at joint B is shown by Fig. 40, a cast-iron shoe being fitted to the end of the strut to receive the pin. For the truss shown by Fig. 41, a shoe made as shown in the detail drawing will make a better connection for the rods, two of the rods being placed outside of the brackets and three between the brackets.

For a truss with a single strut, a turnbuckle on the rod E. Fig 39,

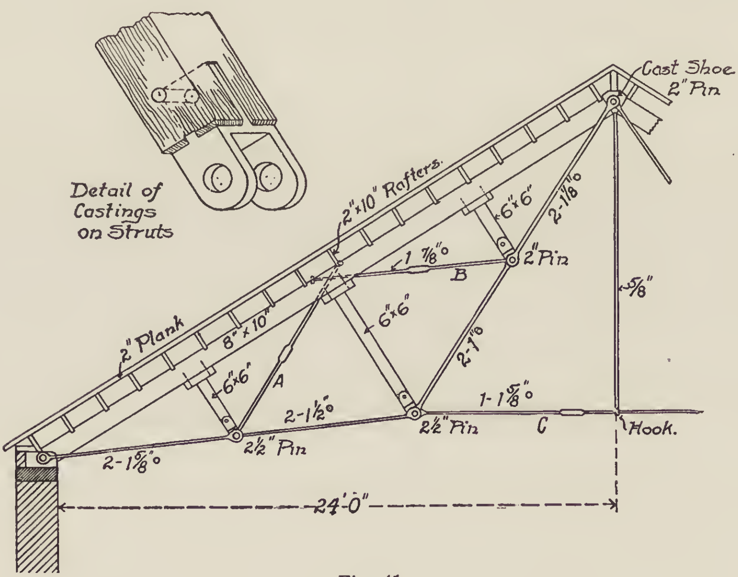


Fig. 41.

will be sufficient to tighten the rods. When there are three struts, there should be five turnbuckles, as in Fig. 41.

A cast-iron shoe should be made to receive the foot of the rafter, and the rods secured to a pin passed through the shoe and the rafter. At the apex of the truss, a cast-iron shoe and pin should also be used when the rods are in pairs, but when single rods are used, as in Fig. 39, they may be crossed and passed through a cast-iron washer, as shown. The pins which receive the ties should be computed for shearing, bearing and bending moment.

19. HOG CHAIN SCISSORS TRUSS.—Fig. 42 shows a type of truss that has been extensively used for church roofs, by Mr. D. S. Schureman, architect, of Rockford, Ill., and possibly by others. The half-tone illustration, Fig. 43, shows a truss of this type over the Second Congregational Church of Rockford, where the trusses

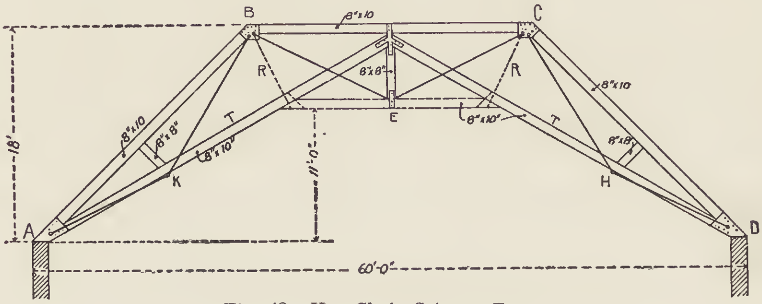


Fig. 42.—Hog-Chain Scissors Truss.

have a clear span of 80 feet, and are 51 feet apart. The space between the trusses is spanned by two Howe trusses. The roof is of slate. The timbers used in these trusses are 10″ × 12″, the king-post is 9 feet long, and the bottom of the king-post is 14 feet above the foot of the truss.

In the truss shown in Fig. 42 the horizontal beam at the foot of the king-post is put in merely to support the ceiling construction, and is not needed as a part of the truss. The rods R R, merely support the ends of the horizontal beam and a part of the ceiling, and would not be needed if there was no ceiling to support.

Although this truss somewhat resembles the scissors truss, the mechanical principle of the two trusses is entirely different.

In this truss the tie-beams T T are in tension, for their full length, and all of the other timbers are in compression. The truss is prevented from spreading by the ties T T, and the rods B, E, C (one on

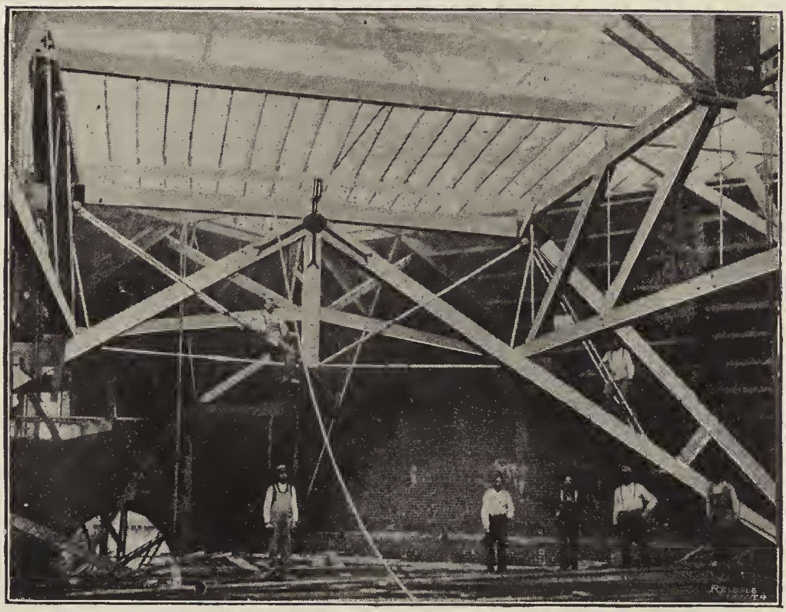


Fig. 43.—Trusses in Roof of Church at Rockford, Ill. D. S. Shureman, Architect.

each side of the truss). By tightening up on these rods, the strut beam is raised at the centre, and the feet of the truss drawn in. The rods A, K, B and C, H, D support the end of the short braces.

While this is a true truss, the author does not consider it as good a truss as that shown by Fig. 38, which gives about the same lines. The stresses in the truss, Fig. 42, are considerably greater than those in the truss, Fig. 38, and the connections are more difficult to make. The truss shown in Fig. 42, however, is but little affected by shrinkage of the timber.

20. ANOTHER TYPE OF TRUSS WITHOUT A HORIZONTAL TIE-BEAM.—Another type of truss, with a raised tie-beam often used in churches is illustrated by the truss shown in Fig. 44, although the manner of bracing the rafters often varies considerably.

This truss, however, is not a true truss, as it depends partly upon the resistance of the joints for its stability, and whenever used probably exerts more or less thrust on the walls.

As ordinarily constructed the truss shown in Fig. 44 may be considered as two compound rafters merely fastened securely together at the top and supported at the bottom by the walls, as illustrated by Fig. 45. It is evident that the tendency of the loads in Fig. 45 is to bend the rafters, as shown in Fig. 46, and also to tear them apart at the joint. The action of the truss, Fig. 44, therefore, con-

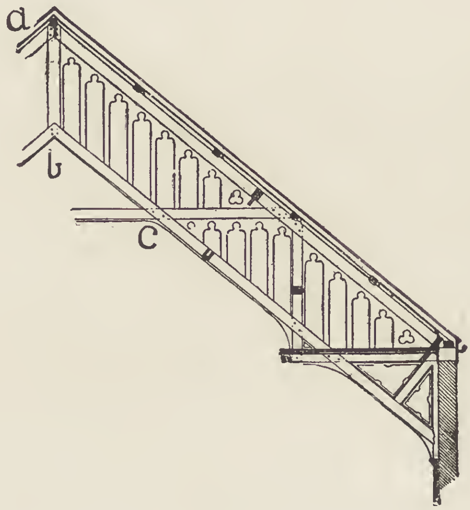


Fig. 44.

sists in resisting the tendency of the compound rafters to bend, and also the tendency to pull apart at joints a b c. Any bending of the rafters must produce an outward thrust on the walls, and any give in the joints would also have the same effect.

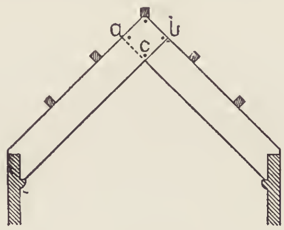


Fig. 45.

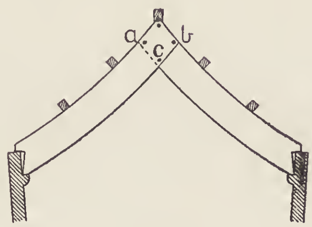


Fig. 46.

To return to Fig. 45, it will be seen that if the walls are sufficiently stable that they cannot be thrust outward, only a pin or butt joint will be required at the top, and only the lower purlins will tend to produce flexure.

It is also evident that the greater the depth of the compound rafters, and the more firmly they are fastened at their intersection, the greater will be the resistance to bending and tearing apart.

In practice it is probable that where trusses of this type are used, they derive their stability both from the resistance of the walls and buttresses and from the resistance of the framework.

The writer is doubtful if the exact resistance of the truss shown in Fig. 44 (which has actually been built) can be determined.

Whenever used the walls should be strengthened opposite the trusses by buttresses.

A truss of the form shown in Fig. 47 is a true truss and the strains in it can be accurately determined. When built in the proportion shown in the figure however, the tensile strain in the mem-

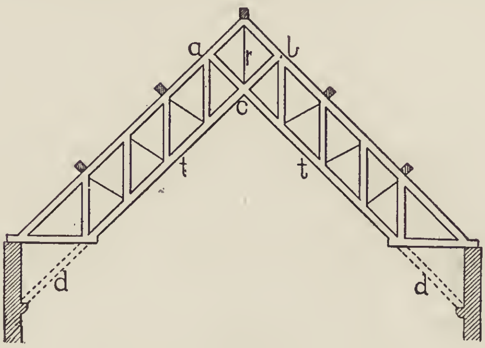


Fig. 47.

bers t, t and r is very great. In practice the strains in these pieces are somewhat lessened by placing braces at d d and by the resistance of the joints a, b and c.

21. THE HAMMER-BEAM TRUSS.—This is a type of truss much used for supporting open timbered roofs, especially in Gothic halls and churches. It is believed to have been first used in the great hall of Westminster Palace (Fig. 48), built in 1397, and different forms of it may be seen in many of the English buildings of the fifteenth century. The principal apartment of the palaces and educational establishments of that period was the hall, and most of these had beautifully framed roofs of timber, in which some form of the hammer-beam truss is almost invariably found.

The hammer-beam truss was also occasionally used in the parish churches, but the wooden roofs of the churches were generally flatter and less deeply framed than those of the halls, and as a rule less beautiful.

The truss derives its name from the horizontal beams at the foot of the principals, which were called "hammer beams."

The typical shape of the truss as found in modern buildings may be represented by Fig. 49, although it is difficult to find two trusses that are exactly alike in design. It is probable that when these trusses were first used it was expected that they would exert a small

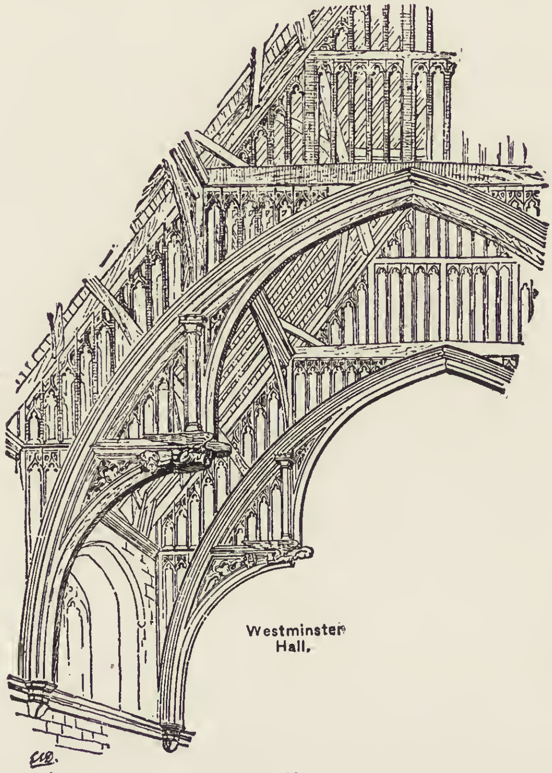


Fig. 48.

outward thrust on the walls, as the latter were usually very thick and generally reinforced by buttresses. By means of the curved brackets under the hammer beam, however, the thrust was applied at a considerable distance below the plate, and the direction of the resultant thrust was usually very steep.

The action of the stresses in a hammer-beam truss may perhaps be best explained by means of Fig. 50, which represents a truss in the act of falling, owing to the breaking of the curved rib b and the consequent pushing out of the walls. As soon as the wall commences to move the whole weight of the upper part of the truss is thrown on the lower portion of the principal, and tends to produce rupture at the points a and b.

It should be evident from this illustration that without the curved rib b the action of the truss is virtually the same as if constructed

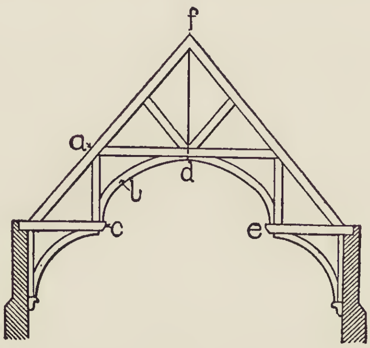


Fig. 49.

as shown in Fig. 51, as the pieces below the joint a, Fig. 49, may be supplanted by the single strut s, Fig. 51. Of course the truss in Fig. 49 would be more rigid, on account of the principals being continuous and from a certain amount of support offered by the top of the wall, but the thrust on the wall would be the same.

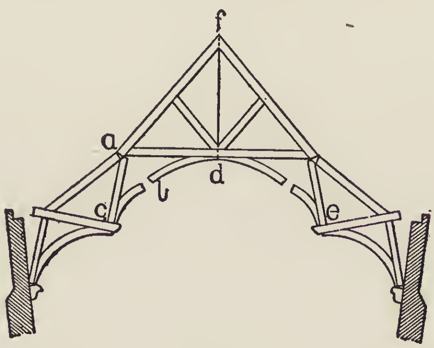


Fig. 50.

This thrust would act in the direction of the strut s, and would evidently be applied at the point where the strut rests on the wall or corbel.

To keep the truss, Fig. 51, in equilibrium it is evident that the wall must be capable of resisting the outward component of the thrust in the strut, and that the joint a must be sufficiently rigid to prevent the truss racking under a severe wind pressure.

If we do not wish the truss to exert an outward thrust we must connect the points c, d, e and f (Figs. 49 and 50) in such a way that they cannot change their relative position. This can best be done by means of iron or steel rods or bars, as shown in Fig. 52. It is evident that as long as these rods hold it will be impossible for the feet of the principals to spread or to exert a thrust on the walls. Furthermore, when these rods are used it is impossible for the joints c and e to drop, and hence the braces under the hammer-beam will not be required We thus see that a hammer-beam truss can be constructed in two ways—first, with a horizontal thrust which must be resisted by the walls and, second, without a horizontal thrust.

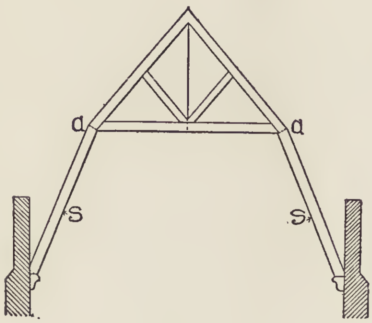


Fig. 51.

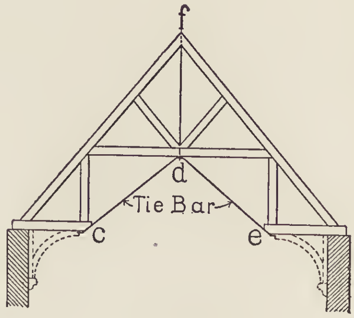


Fig. 52.

In the first case a brace beneath the hammer-beam is absolutely necessary; in the second it is not.

In practice the inclined tie-bars, shown in Fig. 52, are quite impossible, as they would spoil the desired effect, and it would be better to use a different type of truss. A straight wooden tie would also be objectionable for the same reason, so that if a tie is to be used, curved wooden ties must be employed to obtain the architectural effect. The object, therefore, of the bent pieces b, Fig. 49, and the corresponding pieces in Fig. 48, is to form a tie to take the place of the iron bars shown in Fig. 52. If the entire outward thrust of the principals is taken up by the inclined ties the strain becomes very great in proportion to the loads on the truss, so that in a truss of 36 feet span or more it is quite impracticable to put in curved pieces that will have the necessary resistance, and depend-

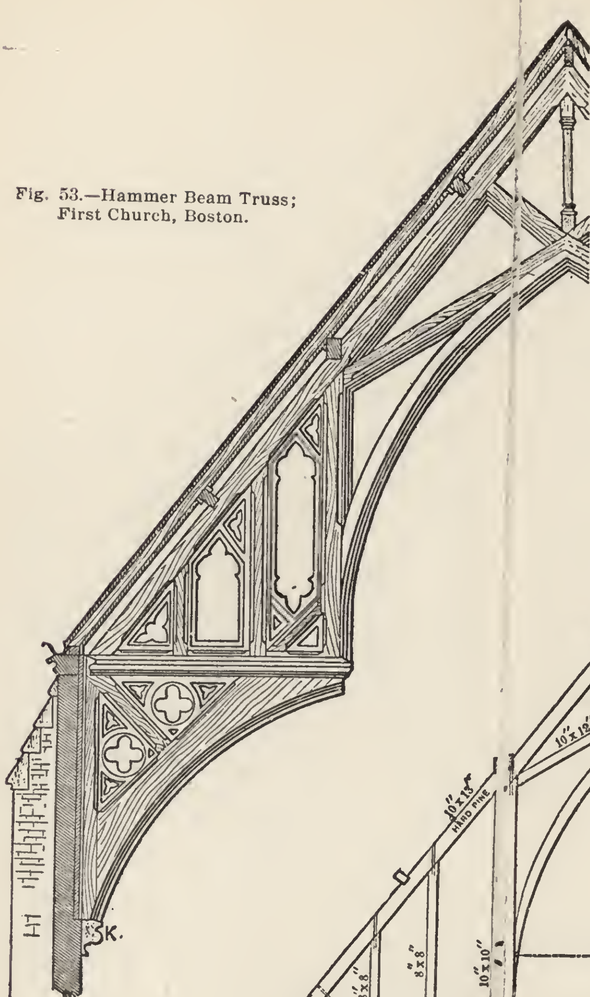


Fig. 53.—Hammer Beam Truss; First Church, Boston.

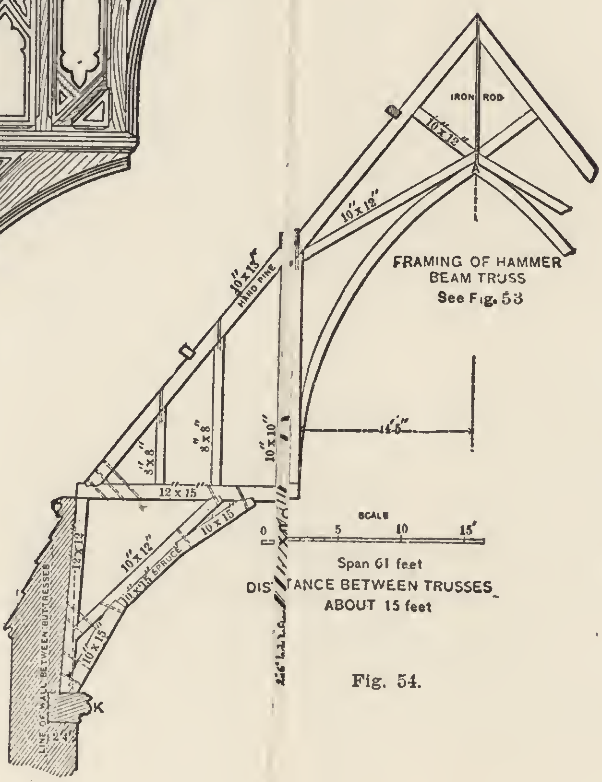


Fig. 54.

ence must be placed upon the walls and braces to assist the curved ties.

When using trusses of this type, therefore, the wall should be made very thick, and, if possible, reinforced by buttresses or cross walls, and the curved ties should be made so as to unite the joints c, d and e as efficiently as possible. They should not be bent, but should either be cut out of a single piece or built up to the desired shape with pieces well bolted together.

In all such trusses *a good-sized king-rod should be used*, as the curved ties will be useless without the king-rod to connect them with the peak. The king-rod can be concealed by the wooden king-post, which can be built around the rod, as in the truss shown by Figs 53 and 54.

All parts of the truss should be carefully computed, and no dependence should be placed upon the ornamental work and finished casings, although these add slightly to the resistance of the truss. All joints at the intersection of the pieces should be made as secure as possible, as this adds much to the resistance of the truss and reduces the thrust.

Figs. 53 and 54 show the construction of a modern hammer-beam truss designed in the office of Messrs. Ware & Van Brunt, architects, some twenty-four years ago. Fig. 53 shows one-half of the finished truss and Fig. 54 the framework. The truss is cased with black walnut and has a very handsome effect. It may be noticed that the upper part of this truss is a scissors truss, and there is no collar-beam. In this respect it differs from all the old trusses that the author has seen.

22. Fig. 55 shows another and quite different form of a hammer-beam truss. In this truss the only resistance to the outward thrust offered by the truss is the transverse strength of the principal and curved rib at the section A, A. It must be evident that if this truss should fail it would be by the breaking of the principals near this section, and that the least flexure in the principal will cause the lower brace to push out on the wall. If the walls, however, are sufficiently stable to resist the thrust, then the principals become merely struts and there is no transverse strain.

Very often in churches the trusses occur at an interior angle, as shown in Fig. 56, in which case the resistance of the wall is ample to take up the thrust and the curved ties are unnecessary except for architectural effect.

23. When it is desired to support a hammer-beam truss on a

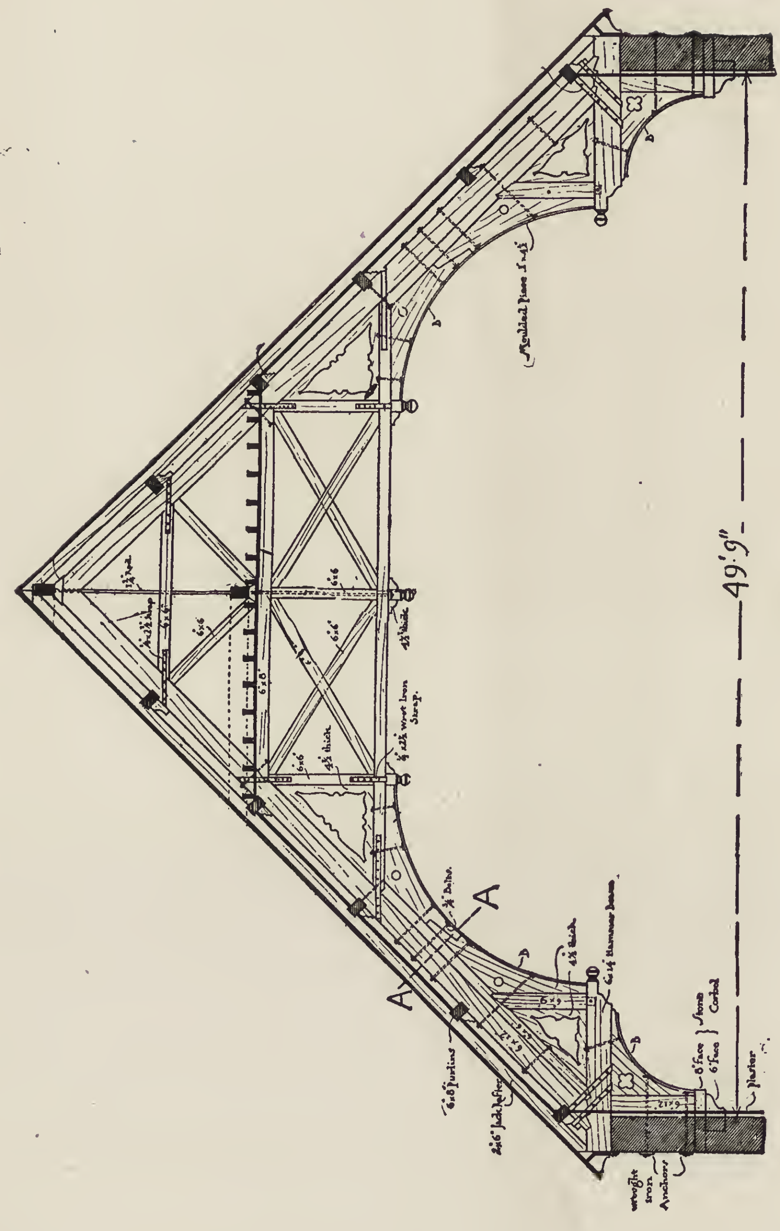


Fig. 55.

clerestory wall without making the wall very thick or bracing it from the outside, a form of truss like that shown in Fig. 57 may be used to advantage. This truss has the appearance of a hammer-beam truss, and when placed over a high nave the effect of the rods is not objectionable.

The tie-rods should extend through the hammer-beams to their outer end. For a truss of 32 feet span a $1\frac{1}{4}$-inch square bar will be ample, and it may be twisted to give a more pleasing effect.

The curved ribs a, a, in this truss are not in tension but in com-

Fig. 56.

pression, and the braces under the hammer beams are necessary to resist the vertical component of the thrust in the curved ribs. A truss similar to this was used in the new Grace Chapel, New York City. Several other examples of hammer-beam trusses are given in Chapter IV.

24. ARCHED TRUSSES.—SEGMENTAL ARCHED RIBS. —For open roofs, of wide span—100 feet or more—the segmental arched rib, with an iron tie, is probably the most economical wooden truss that can be used, as well as the most pleasing in ap-

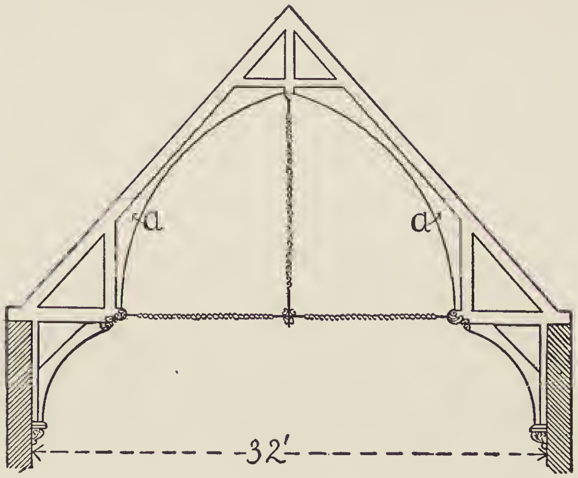


Fig. 57.

pearance. It has been quite extensively used in this country for supporting the roofs of exhibition buildings, armories, skating rinks, etc.

The truss shown in Fig. 58, which is over the large hall of the Mechanics' Charitable Association Building in Boston, is a good example of this type. In this truss the braced arch, possesses in itself sufficient strength and rigidity to transmit the roof load to the supports in the same way that a brick arch would, while the horizontal tie resists the thrust of the arch. The framework above the arch does not form a part of the truss, but is merely a series of braced posts to support the purlin.

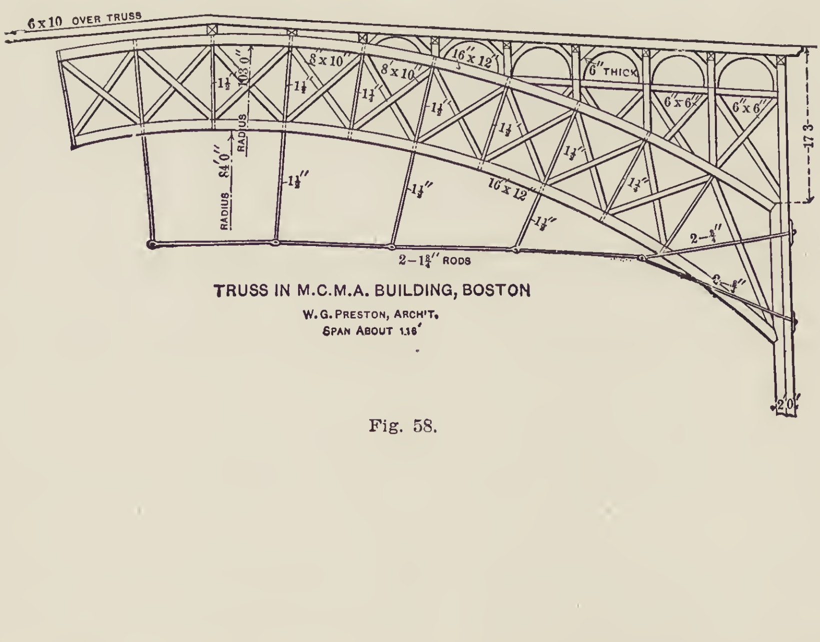


Fig. 58.

Fig. 59 shows one-half of a truss, which, with seventeen others, was designed for supporting the central bay of Sanger Hall, Philadelphia,* Messrs. Hazelhurst & Huckel, architects.

This building was erected, in 1897, for the use of the Eighteenth National Sangerfest, and was to be taken down and removed immediately after the four days' session. It was therefore but a temporary building, although built in accordance with the building ordinance, and the illustration is given as showing what is undoubtedly the cheapest method of supporting such a roof without the use

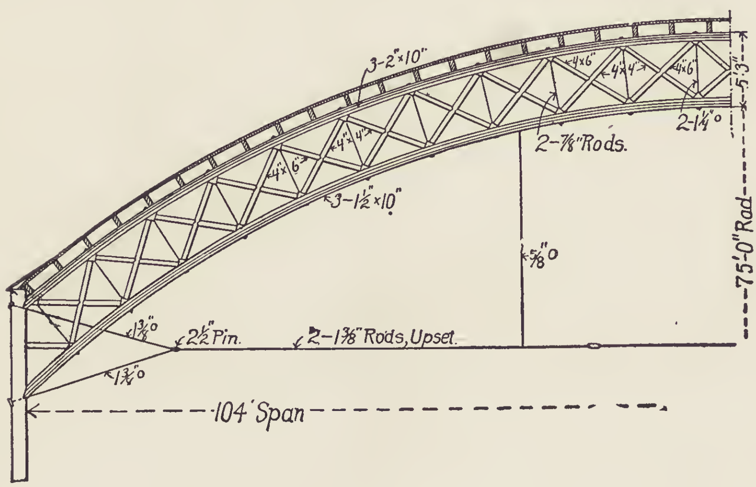


Fig. 59.

of objectionable columns. The trusses were spaced 20 feet from centres.

25. CRESCENT TRUSSES.—The crescent truss shown in Fig. 60 may be considered as a special type of the bowstring truss (see Section 38, Chapter II.). It is not as frequently used, and it is not as economical a type for large spans.

In this truss the upper chord and the diagonals are in compression, and the lower chord and the radials are in tension. Counter braces are required to resist wind pressure.

This truss can be made of wood, the chords being built up of planks bent to the curve and firmly bolted together. The radials should be of wrought iron or steel rods.

26. Fig. 61 shows another type of arched truss that may be used

*Described in the "Engineering Record," of January 9, 1897.

for wooden construction, when the span does not exceed 100 feet. This truss is built on the principle of the quadrangular truss explained in Section 36, Chapter II., the direction of the diagonals being reversed, so that they will be in compression and the radials in tension. The lower chord must, of course, be in tension, but it can

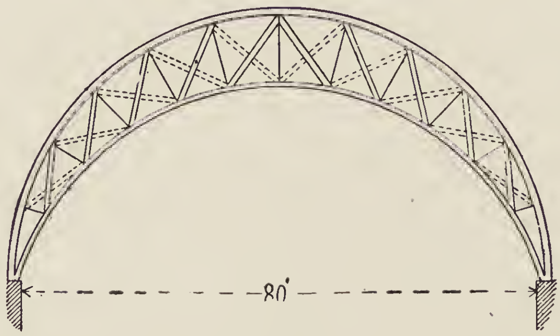


Fig. 60.

be built up of planks, bent to the curve and bolted together. It should be remembered that the strain in the lower chord terminates at the points X, X, and is transmitted through the rods to the ends of the rafters at A and B, the bracing below the points X, X being merely stay bracing. Counter braces should also be inserted as shown by the dotted lines.

For trusses with single spans of less than 100 feet this is the best type of truss to use when the building has a pitch roof and an arched effect is desired.

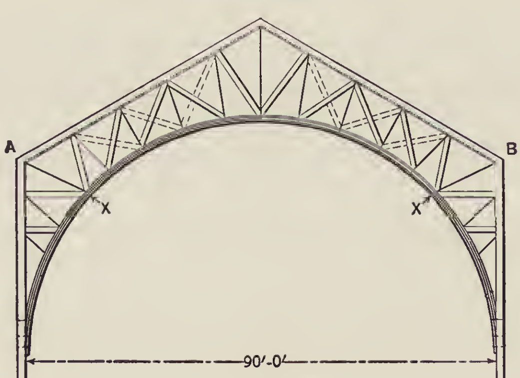


Fig. 61.

27. CANTILEVER TRUSSES.—Although cantilever trusses of wood are not often used, conditions sometimes exist, as in the case of a wide centre-span with shorter spans on each side, where

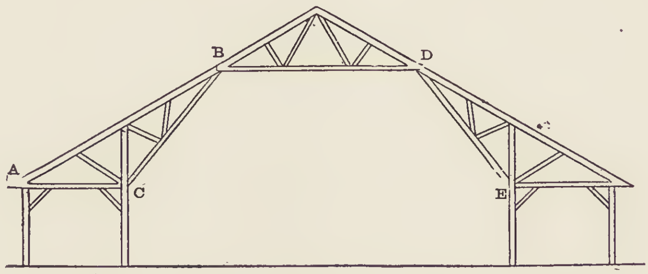


Fig. 62.

a cantilever truss will meet the requirements better than any other type.

Fig. 62 shows a simple cantilever trussed roof, suggested by Mr. John Beverly Robinson in an article advocating the use of the cantilever in building construction, published in the "Engineering

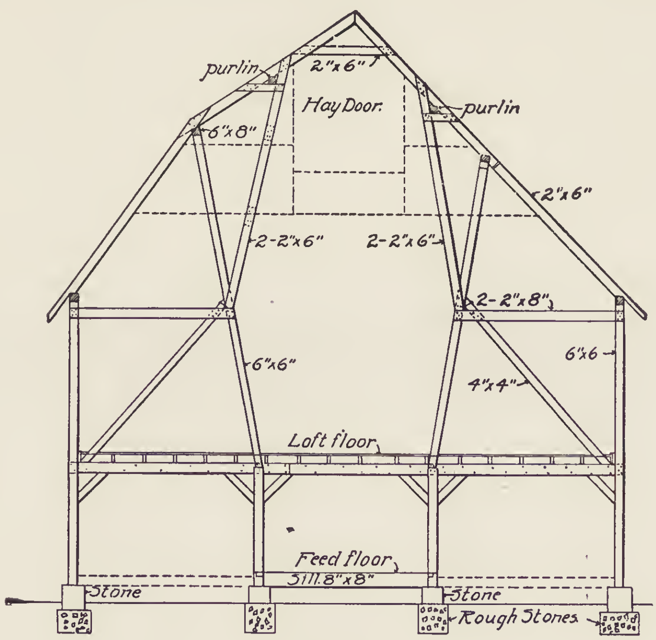


Fig. 63.

Magazine" for November, 1896, which is feasible for wooden construction.

Fig. 63* shows an "open center barn," in which the roof is supported on the cantilever principle, the posts and braces being from 8 to 10 feet, on centers.

Cantilever trusses when supporting a roof over a central span must be used in pairs, the truss on one side being independent of that on the other.

The principle of the cantilever truss is explained in Chapter II., Section 43.

*This cut was suggested by one contributed to "Carpentry and Building" by a carpenter who designed and built a barn in this manner. The sketch is intended to show the application to either a curb roof or one with straight pitch.

Chapter II.

TYPES OF STEEL TRUSSES.

28. Steel trusses are built on exactly the same principles as wooden trusses, and any truss that can be built of wood can also be built of steel, but owing to the different nature of the two materials, the types of trusses best adapted to wooden construction are not the most economical for steel.

Steel trusses are generally built of angles, channels, plates and eye bars, and as any of these shapes are better adapted to resisting a tensile stress rather than a compressive stress, economy requires that the form of the truss or arrangement of the members, shall be such that there shall be as few members in compression as practicable, and that the *shorter* web members shall be in *compression* and the *longer* ones in *tension.*

Almost any combination of triangles can be made to support a roof, when built of steel, but some combinations will require a much greater weight of metal, to support the given load than others.

Steel roof trusses having a span of less than 100 feet can generally be built more cheaply with riveted connections, or joints, and most of the arched trusses for wide spans, are also riveted together. For some types of trusses, however, the pin connection may be cheaper or more advisable construction.

Pin connected trusses may be more conveniently shipped, and where they are supported by brick walls, may sometimes be more economically erected.

Riveted trusses are almost always built of angles for the ties, and of a pair of angles or channels for the struts, the angles often being reinforced by a web plate. In pin-connected trusses eye bars are generally used for the ties, and a pair of channels, latticed or reinforced by plates for the struts.

29. NUMBER OF STRUTS OR PANELS.—The best form for a steel truss, and the most economical number of braces will depend in a great measure upon the inclination and construction of the roof, as well as upon the span. If purlins are used to support

the jack rafters then the distance between the struts may be as great as 12 feet, but if there are no jack rafters and the planking of the roof is nailed directly to the purlins, then the latter should not be placed more than 8 feet apart, and if the roof is covered with corrugated iron secured to the purlins, then the purlins can not be more than 5 feet on centres. Whenever the purlins are more than 4 feet apart, *they should come over the end of a strut* or brace, to avoid bending moments, consequently the spacing of the purlins will generally determine the number of struts in each half of the truss. For this reason the same form of truss may be required for a span of 40 feet as for a span of 80 feet, but, of course, the members will not be as heavy in the forty-foot truss as in the one with greater span. The trusses shown in this chapter are mostly drawn from actual cases and give a pretty good idea of the most economical division for different spans.

When the truss rafter is subject to a transverse strain, that is, when it is loaded between the joints, the distance between the joints

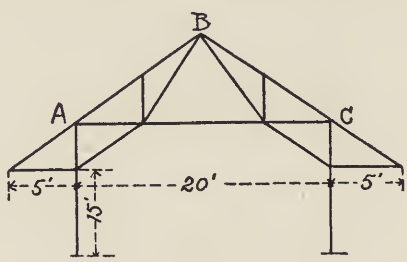


Fig. 64.

should not exceed 9 feet and preferably 7 or 8 feet, depending somewhat on the distance the trusses are apart.

Believing that a knowledge of the types of trusses best adapted to different conditions can be most readily obtained by means of practical examples, the author has prepared illustrations showing nearly every type of truss commonly used, which, with the explanations given, should enable the reader to select the one most economical for the support of any particular roof.

30. TRUSSES FOR PITCH ROOFS.—For ordinary conditions and for spans under 100 feet some one of the types shown by Figs. 64 to 75, will generally meet the requirements of strength and economy.

For a narrow shed or shop the shape of truss shown by Fig. 64 is the most economical, the truss proper being that portion enclosed within the points A, B, C. This truss is practically the same as that shown by Fig. 65.

For spans of from 24 to 48 feet, and with an inclination not exceeding 6″ to the foot, types 66 and 67 are the most suitable.

The truss type represented by these two figures has received the name of "Fan truss." The truss shown by Fig. 65 is known as

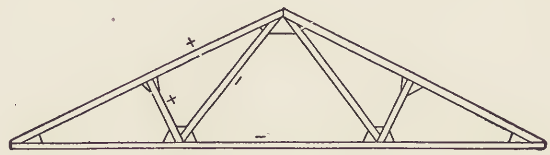


Fig. 65.—Simple Fink Truss; Span, 20 to 36 ft.

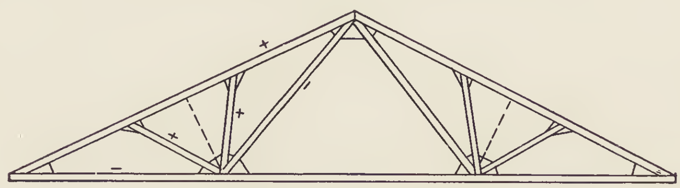


Fig. 66.—Simple Fan Truss.

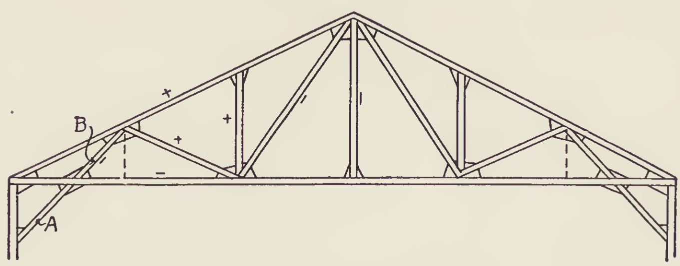


Fig. 67.—Fan Truss; Span, 40 to 50 ft.

a "simple Fink truss." The truss shown by Fig. 67 differs from that in 66, principally in the inclination of the braces. The braces A, B, in Fig. 67 being inserted to brace the truss from the column to prevent racking under wind pressure. Fig. 67 should be used when the truss is supported by columns, and Fig. 66 when the truss rests on brick walls. When the roof construction demands three purlins on each side of the truss, one of the forms shown by Figs. 68, 69, 70 or 71 should be used.

The names given to these trusses are often confounded by different writers; many engineers class the French and Fan trusses with the Fink truss. The term "French" appears to be generally given to those trusses in which the tie-beam is raised in the centre. The truss shown by Fig. 71, appears to have no generally recognized name. One writer refers to it as an "English" truss. This truss is not as economical as the Fink truss, except when the in-

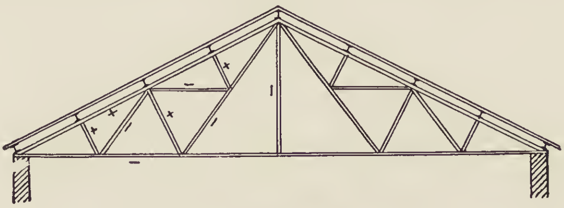


Fig. 68.—Compound Fink Truss.

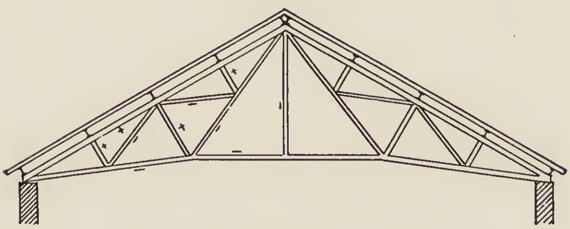


Fig. 69.—Eight-Panel French Truss.

clination of the rafter is less than one-fourth pitch, on account of the great length of the inner struts.

Although Fig. 71 somewhat resembles the Queen truss, Fig. 12, it will be seen that the diagonals run in the opposite direction, the diagonals in Fig. 71 being in tension, and the verticals in compression, the reverse of the Queen truss.

Much of the economy of Fink and Fan trusses lies in the fact that most of the members are in tension and the struts are short. Comparing Figs. 70 and 71, it will be noticed that the inner strut in the former is only $\frac{1}{3}$ as long as the strut in the latter. Another

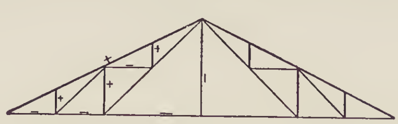


Fig. 70.—Fink Truss with Vertical Struts.

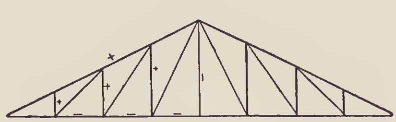


Fig. 71.

advantage of these trusses is that a partial load, as, for instance, a wind or snow load on one side of the truss never causes stresses in excess of those produced by a uniform load of the same intensity over the whole truss. As a general rule, the struts in Fink trusses are placed at right angles to the rafters, as in Figs. 68, 72 and 73, but if there are trussed purlins it is desirable to have vertical members to receive the ends of the purlins. Vertical struts are generally required in hip trusses.

31. DEPTH OF FINK AND FAN TRUSSES.—The depth of these trusses at the centre is usually determined by the roofing material that is to be used. Thus, slate should not be used on a

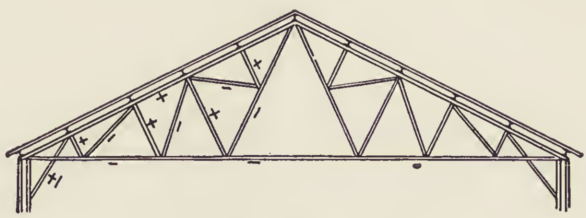


Fig. 72.—Ten-Panel Fink Truss.

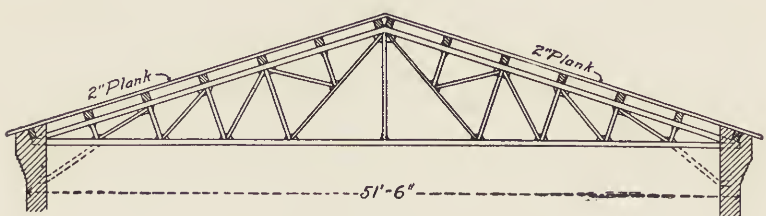


Fig. 73.—Twelve-Panel Fink Truss.

roof in which the rise is not equal to one-third of the span; the rise for wood shingles should not be less than one-fourth of the span, and for corrugated iron not less than one-fifth of the span. The minimum rise for other roofings is given in Chapter III.

Considering the construction of the roof and the weight of the trusses, the most economical pitch for a roof is about $\frac{1}{4}$ the span, or what is commonly called a quarter pitch, the rise of the rafters being 6″ in 12″ or 26 degrees and 34 minutes. When the rise is less than $^1/_6$ of the span some other type of truss will generally be required. When the inclination of the roof is determined almost entirely by the question of economy the rise is generally made from 6 to 7 inches in 12 inches.

32. CAMBERED TRUSSES.—With Fink or Fan trusses hav-

ing an inclination for the rafter not exceeding 30 degrees it is more economical to employ a horizontal chord or tie since it obviates bending of the laterals. Raising the bottom chord, also materially increases the strains in the truss members, hence it increases the cost. A truss whose bottom chord has a rise of two or three feet,

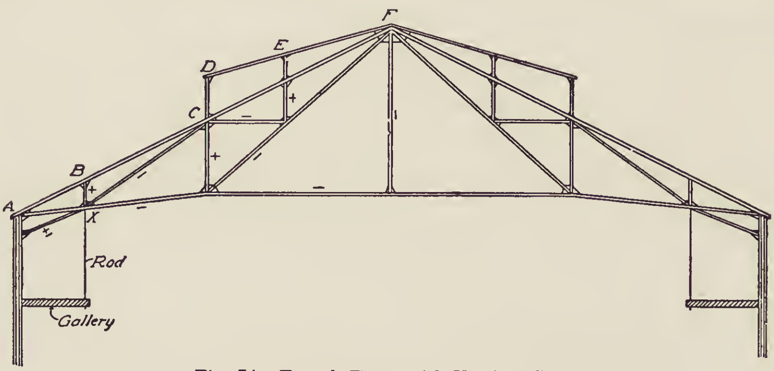


Fig. 74.—French Truss with Vertical Struts. Span, 80 ft.

as in Fig. 69, presents a better appearance, however, than one with a horizontal chord, and for steep roofs, it will generally be fully as economical to raise the bottom chord because of the shortening of the members. Trusses with raised ties are designated as "Cambered."

33. The diagram shown by Fig. 75 represents ½ of one of the steel trusses used in roofing a car barn for the North Jersey Rail-

Dimensions of Members in Fig. 75.

Rafters. .1—2	= 2 —	5″	× 3½″	× 7/16″	L's.
Rafters. .2—3	= 2 —	5″	× 3½″	× ⅜″	L's.
Main tie..1—4	= 2 —	5″	× 3½″	× ⅜″	L's.
Main tie..4—5	= 2 —	3½″	× 2½″	× 5/16″	L's.
a, a, a.......	= 2 —	2½″	× 2″	× ¼″	L's.
b, b, b.......	= 1 —	2½″	× 2″	× ¼″	L.
c.............	= 2 —	3″	× 2½″	× ¼″	L's.
d.............	= 2 —	3½″	× 2½″	× 5/16″	L's.

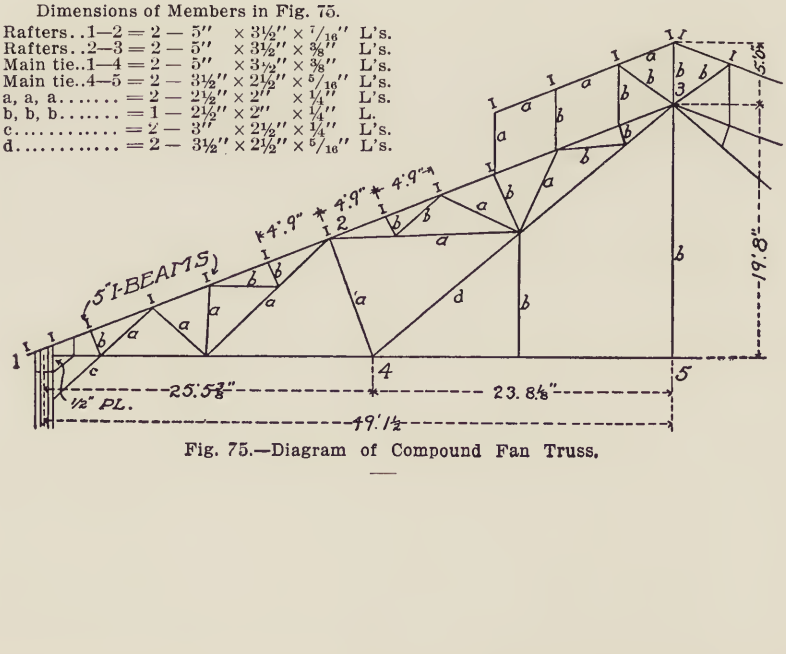


Fig. 75.—Diagram of Compound Fan Truss.

way Co., at Newark, N. J. There were 13 of these trusses, spaced 19′ 2¼″ on centres, each having a span of 98¼′ between the centres of the supporting columns to which the truss is riveted by splice plates engaging the end connection plate and the end web of the column. The dimensions of the principal members of these trusses are indicated in connection with the illustration. These trusses were shipped in four sections which were assembled in a horizontal plane and riveted up complete at the surface of the ground. The bottom chord was stiffened by lashing a rail on each side of it for its entire length and a sling being attached to the apex of the top chord, the truss was lifted and set on top of the columns by an 8″ × 8″ gin-pole 50 feet high. The roofing consists of corrugated iron supported by 5″ I-beam purlins weighing 10 lbs. to the foot, spanning from truss to truss and bolted to the rafters with two bolts at each end; the general spacing of the purlins being 4′ 9¾″. This may be con-

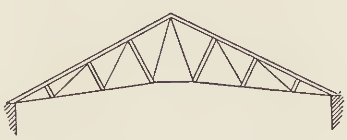


Fig. 76.—Warren Triangular Truss.

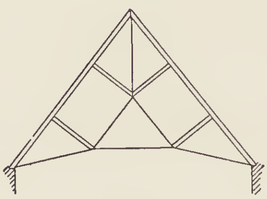


Fig. 77.

sidered as an example of an extremely light roof, the weight of each truss being only about 4,200 lbs., and the entire weight of the truss, purlins, bracing of the lower chord and corrugated roofing being only 8 lbs. for each horizontal foot of surface covered. The truss shown by Fig. 74 was designed for the roof of a drill hall having a span of 80′ and with a spacing, centre to centre, of 20′. The roof was to be constructed with 2 × 8 rafters supported by purlins at points A, B, C, D, E, and F. Sash were to be placed in the rise C D to light the interior of the building. The joint at X was located with reference to the position of the gallery rod; if there had been no gallery it would have been more economical to space the vertical struts uniformly as in Fig. 70.

For roofs having a span of 80 to 100 feet, and a rise of from one-fourth to one-third of the span, the Warren triangular truss shown by Fig. 76 is a good type. This truss is best adapted to pin connections.

For a steeproof, the type shown by Fig. 77 is about as economical as any. A complete example of this type of truss with the details of the computations is given in Part II., of Berg's "Safe Building."

The plus sign adjacent to a member, in all the trusses illustrated, denotes that the member is in compression, while the minus sign denotes tension. In Figs. 76 and 77, the members represented by single lines are in tension. The members above the main rafter as C D, D E, and E F, in Fig. 74, and a and b in Fig. 75 do not form a part of the truss proper but are merely a frame work to support the elevated roof and in drawing the stress diagram, they should be omitted.

34. FINK TRUSSES WITH PIN JOINTS.—Fig. 78 shows one-half of a Fink truss designed for pin connections. This truss has a span of 55′ 4″ between centres of end pins and the distance between the centres of trusses is 6′. The roof is covered with 12″ × 20″ slate, secured to $1\frac{1}{2} \times 2\frac{1}{4}''$ angle purlins weighing 3 lbs. to the foot and spaced $8\frac{1}{2}''$ on centres. The angles span from truss to truss and are bolted to the deck beam with $\frac{1}{2}''$ bolts. A $1\frac{1}{2}''$ by $2\frac{1}{4}''$ nailing strip is fastened to every third purlin for securing matched ceiling placed on the under side of the roof. Complete details of this truss were published in "Architecture and Building" for January 18, 1890. Fig. 79 shows details of the cast-iron struts. This truss, being put together entirely with bolts and pins, could easily be erected with unskilled labor.

35. TRUSSES FOR FLAT ROOFS.—For supporting flat roofs or roofs having a fall not exceeding 1″ to the foot, one of the types shown by Figs. 80 to 84 will generally be found economical. The choice of the particular type depending somewhat on the span and whether the truss is supported by columns or by brick or stone walls. For spans up to 50′ either of the forms shown by Figs.

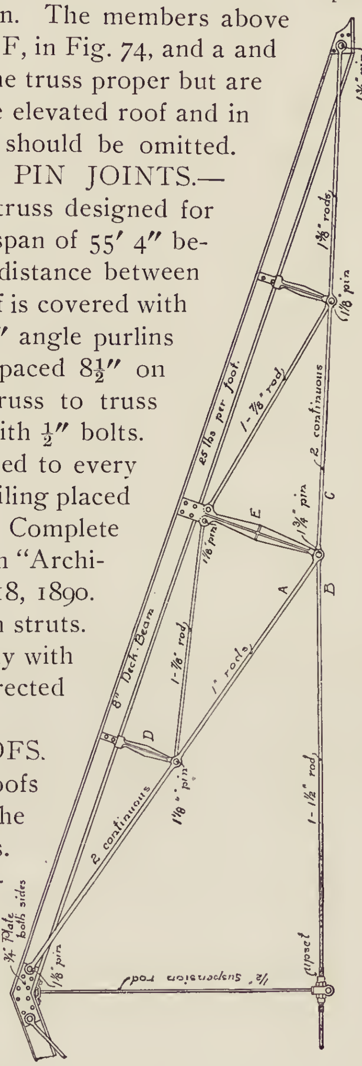


Fig. 78.—Fink Truss with Pin Connections.

80 or 81 will answer all practical requirements. The truss shown by Fig. 80 is intended to be used where the fall of the roof is at right-angles to the truss; this truss can be built, however, with an inclination to the top chord as in Fig. 81. The end brace in Fig. 80 is in tension while in Fig. 81 it is in compression. The portion of the

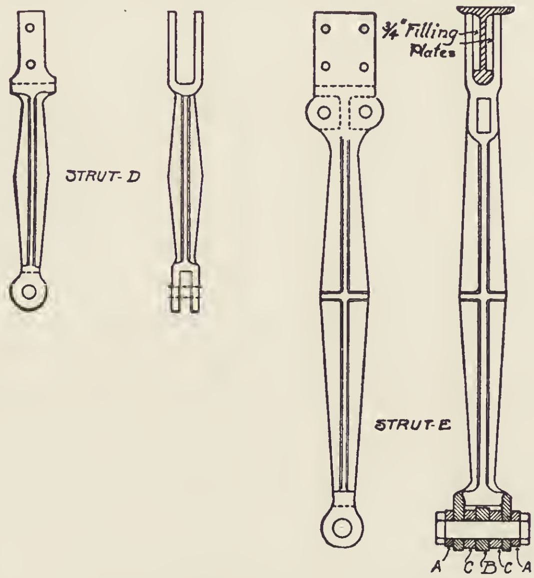


Fig. 79. Details of Struts in Fig. 78.

lower chord between the end joint and the wall, Fig. 80, has no stress from the roof load but is put in to brace the wall, and to stay the truss. In trusses supported by brick walls this type is preferable to that shown by Fig. 81, while the latter is more suitable when the roof is supported by columns. The vertical A, Fig. 81, is in-

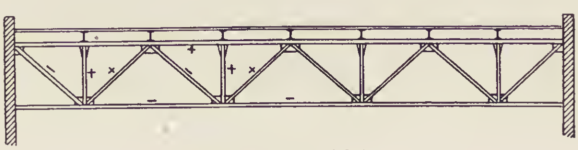


Fig. 80.—Span, 56 ft.

serted to receive the tension or compression from the brace B, and would have no stress from the roof load. The truss shown by Fig. 82, which represents an actual truss is known as a "Double Warren Truss" and is desirable where it is important to make the trusses as shallow as practicable; it can be built with light members and makes a very stiff roof, being especially suitable for roofs supported by steel columns. The strength of this truss under unsymmetrical loads, as, for example, when there is more snow on one side than on the other, would be materially increased by putting a vertical tie in the center as shown by the dotted line; without this member the braces AA, if subject to any stress whatever, would

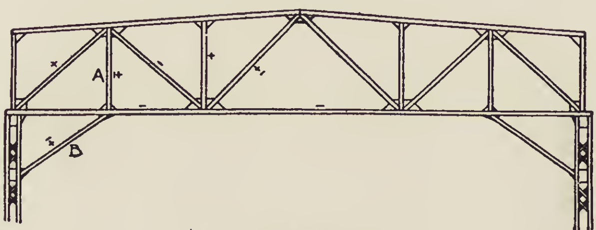


Fig. 81.—Span, 30 to 36 ft.

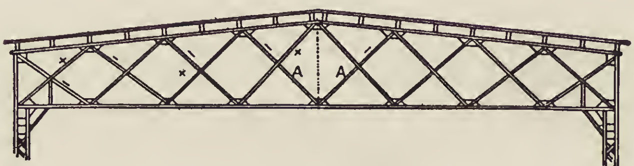


Fig. 82.—Span, 50 ft.

Fig. 83.—Span. 57 ft.

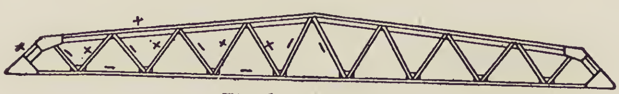


Fig. 84.—Span, 96 ft.

produce a bending in the bottom chord at the center. Fig. 83 represents an actual roof truss with a span of 57′ supported by steel columns. The entire load on the truss is transmitted to the columns by the braces B, B, which are in tension. Fig. 84 shows a Warren truss of 96′ span over a pier shed, New York City.

The plus and minus signs in these illustrations indicate compression and tension, respectively, under uniform dead load. The plus and minus sign together indicate that the member may be subject to either tension or compression, according to the direction of the wind or to an uneven distribution of snow. In most of these trusses an unsymmetrical load may change the stress in the diagonals near the centre of the truss. Trusses like those shown by Figs. 80-84 are almost invariably built with riveted connections and with angle or channel shapes for all members.

For Horizontal Steel Trusses Intended to Support Floor Loads, the Pratt truss shown by Figs. 85 and 86 is best adapted, the members indicated by double lines being in compression, and

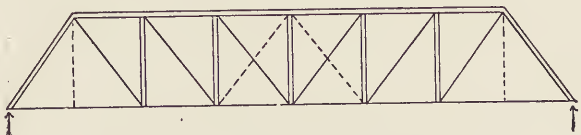


Fig. 85.—Pratt Truss; Span, 60 to 150 ft.

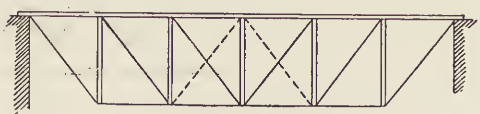


Fig. 86.—Suspended Pratt Truss; Span, 40 to 80 ft.

those indicated by single lines in tension. When supporting floors subject to moving loads, counter ties should be inserted as indicated by dotted lines. For this truss, pin connections are generally employed and are preferable to riveted connections. When properly proportioned this truss is capable of sustaining almost any load.

36. THE QUADRANGULAR TRUSS.—The truss shown by Fig. 83 is known as a quadrangular truss, although the more common shape for this truss is that shown by Fig. 87. This truss may be considered as two trussed rafters, held in place by the tie T at the centre. The portions of the truss which are in compression are indicated by double lines while the single lines represent the tension members. The dotted lines represent counter-braces or ties, which might be brought into action in case of a heavy snow load on one side only or during a severe gale.

The piece B has no strain when the truss is subjected only to a vertical load, although it is usually put in to brace the post P, which carries all of the load transmitted to the support. This truss is well adapted to steel construction up to spans of 180 feet. When the span exceeds 100 feet, and the truss rests on a brick wall one end of the truss should be supported on rollers to allow for expansion and contraction in the steel. The proportions of the truss shown in Fig. 87 are those of the roof trusses of the Jersey City station of the Central Railroad of New Jersey. The trusses in this building have a span of 142 feet 4 inches, centre to centre of bearings, with a depth in the centre of 24 feet 4 inches. The distance between trusses is 32 feet 6 inches. The joints are pin-connected,

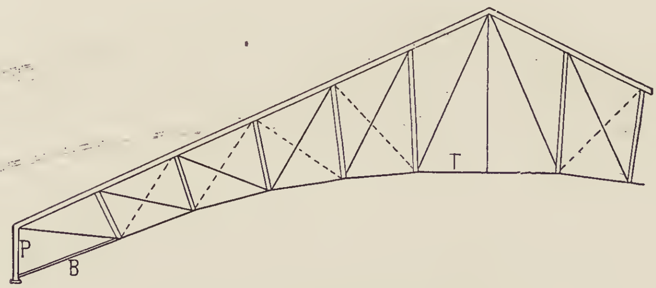


Fig. 87.—Typical Quadrangular Truss.

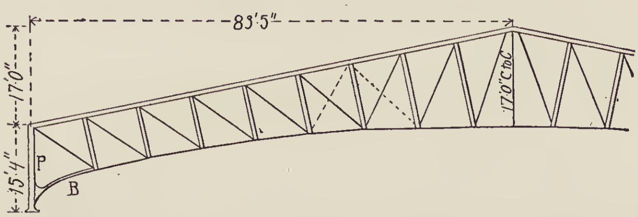


Fig. 88.—Truss over Amphitheatre; Madison Square Garden.

eye-bars being used for the tension members, and one end of the truss is on rollers. Fig. 88 gives the proportions of similar trusses over the amphitheatre of the Madison Square Garden, New York.*

In both of these trusses the posts and braces at the ends are made a part of the truss, although they cannot be represented in the strain diagram, and the post P receives the whole load from the truss at its upper end, the diagonal tie transmitting the entire web stress to the top of the post. The brace B should be so constructed as to resist both tension and compression. This truss, like the other, is pin-connected, with eye-bars for the tension members. The principal dimensions of the truss are given in the drawing.

*Details of these trusses were published in "Architecture and Building" for April 26, 1890.

Fig. 89 shows another example of this type of truss with intermediate supports for the purlins.

The truss shown by Fig. 90 also belongs to this type, but differs from the trusses shown by Figs. 87-89, in that the diagonals are all

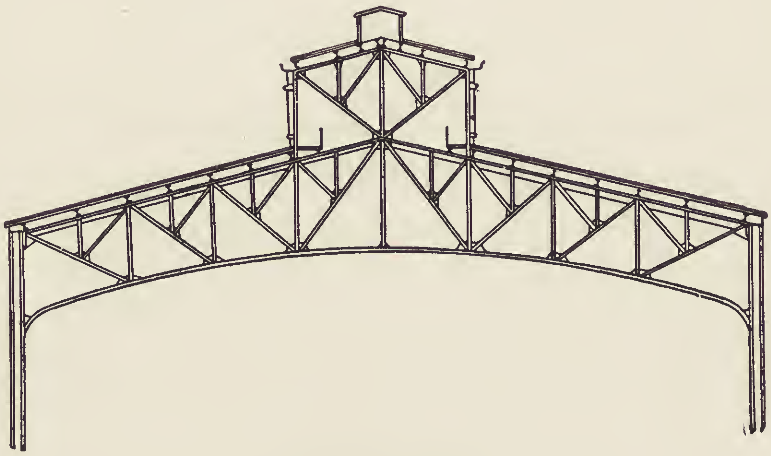


Fig. 89.—Span, 102 ft.

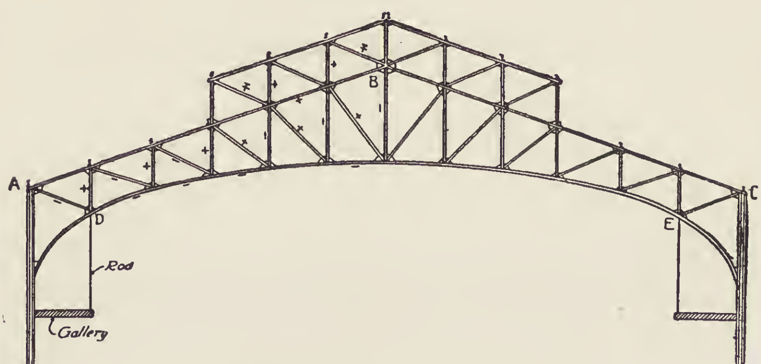


Fig. 90.—Quadrangular Truss; Span, 80 ft.

in the same direction. This causes a *reversal of the stresses in the web members,* as indicated by the plus and minus signs, the three diagonals at each side of the centre being in compression and the adjacent uprights in tension. In the trusses shown by Figs. 87-89, all of the diagonals are in tension and all the verticals in compression. The truss proper, is included within the figure A, B, C, E, D.

It may be noticed that the lower chord of these trusses is segmental in shape, giving a graceful outline, and also the most economical proportions for wide spans.

For shorter spans a full semicircle or semi-ellipse may be obtained by giving a greater curve to the lower chord and continuing it by means of braces. Fig. 91 shows how this was accomplished in the roof trusses over the wings of the Manufacturers' and Liberal Arts Building of the World's Columbian Exposition at Chicago.

The parts included between the joints A, B, C, D form a quadrangular truss, while the portions X, X are merely braces external to the truss.

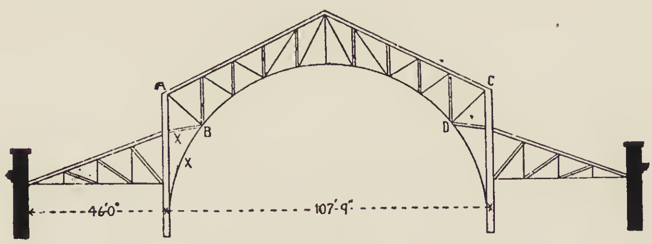


Fig. 91.

The trusses supporting the central roof of the Mining Building, represented in Fig. 111, are also of the same type, although in this building the lower chord and braces have the form of a semi-ellipse. In both of these figures the tension members are represented by single lines. All the members in this truss were made of angles, so as to take up either tension or compression, and the joints were riveted.

The stresses in quadrangular trusses due to wind and snow should be determined independently of the dead load, and the members computed for the maximum stress that may be produced by any possible combination of loading.

There are numerous examples in this country of quadrangular trusses of from 100 to 180 feet span.

The truss shown by Fig. 92, which is a diagram of one of the trusses over the Kansas City Auditorium, may be considered as a cross between the Warren truss and the Quadrangular truss, for if the end panels were omitted the truss would be almost identical in shape to that shown by Fig. 90. This truss has pin connections at the joints. The three diagonals each side of the centre are formed by lattice channels, and all of the ties are formed by eye-bars. The plus sign denotes compression and the minus sign tension. The diagram below the truss drawing shows the manner in which the trusses were braced laterally. All measurements are from centre of pins. A description of this truss and of the building may be found in the "Engineering Record" for July 22, 1899.

37. ARCHED TRUSSES.—For open roofs of wide span arched trusses are generally the most economical, and as a rule give the most pleasing appearance.

The economy of an arched truss lies in the fact that the principal compression members follow approximately the line of greatest

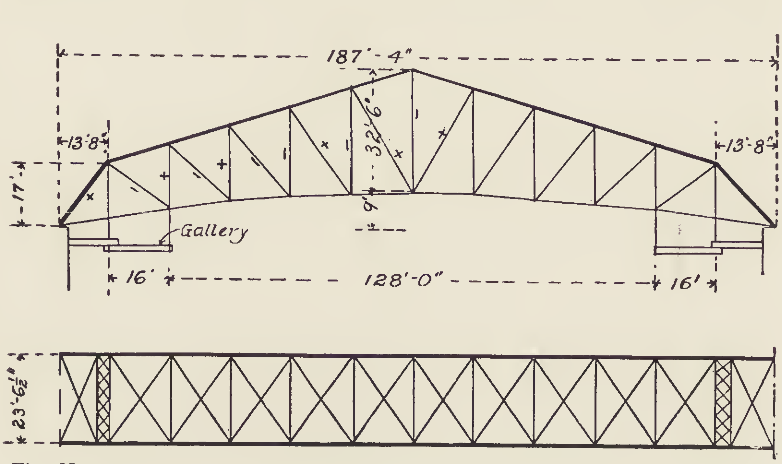


Fig. 92.—Elevation of Truss and Plan of Two Trusses Showing Lateral Bracing.

strain, so that the bracing can be made very light. Thus, if the frame shown in Fig. 93 is so built that the joints come in the line of a parabola with the lower ends secured by a horizontal tie, and all the joints are *uniformly loaded*, the frame will remain in equilibrium without bracing. In practice perfect equality of loads cannot be maintained, and the weight of the tie must also be supported from the upper chord, so that to make a practical truss it is necessary to introduce bracing between the arched rib and the tie, either as shown in Fig. 94 or Fig. 95.

The braces provide for the inequality of the loads, and the tension members take up the thrust in the braces. The effect of the inequality of the loads, however, is usually such that only very slight strains are brought upon the bracing, so that the amount of material required for a truss of this shape is considerably less than in a truss with a straight rafter. That the framework shown in Fig. 93 shall be in equilibrium, the joints must be in the curve of a parabola, but as bracing is always necessary, the upper chord may have the form of the arc of a circle without greatly increasing the strains in the bracing.

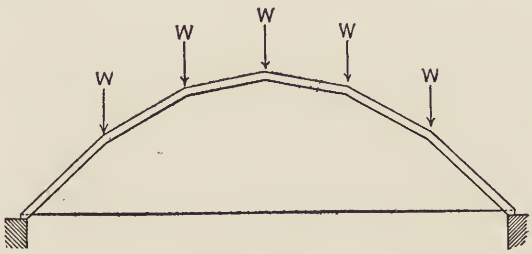


Fig. 93.

Trusses of the types shown in Figs. 94 and 95 have been more extensively used for bridges than for roofs, as they are not so well adapted to the usual conditions of roof construction. For a segmental or parabolic roof, however, this type makes a very economical truss.

In the truss shown by Fig. 94, the vertical pieces B, C and D are in compression when the load is applied at the top, and the vertical

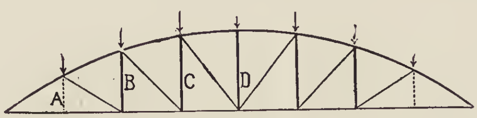


Fig. 94.

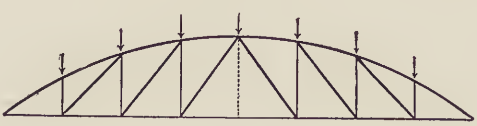


Fig. 95.

member A has no strain except the weight of the tie-rod. If the loads are applied at the bottom all the members of the truss except the curved rafter will be in tension.

With the bracing arranged as shown in Fig. 95, the diagonals are always in compression under a vertical load; the centre vertical has no strain except such as comes from the tie-beam and its load.

The arrangement of the bracing shown in Fig. 95 is the best for a wooden truss.

The arch form in roof trusses is used principally in the types known as:

A. Bowstring and Crescent Trusses.

B. Segmental Arched Ribs.

C. Braced Arches.

38. BOWSTRING TRUSSES.—Although the trusses shown by Figs. 94 and 95 are known as bowstring trusses, the usual type of the bowstring truss is that shown by Fig. 96.

In this truss all of the members except the upper chord are in tension, and the strains in the bracing are comparatively slight, thus

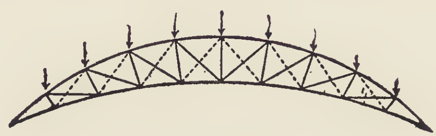


Fig. 96.

making it a very economical truss for steel construction. For trusses having a span of 75 feet or less the suspending pieces should be radials, but in trusses of larger spans they may be vertical, as in Fig. 97.

Bowstring trusses of the form shown in Fig. 97 have been much used in England and to a considerable extent in this country. The upper chord is usually bent to the arc of a circle, with a rise in the centre of from one-fifth to one-third of the span. The depth of the truss in the centre should be about one-half of the rise.

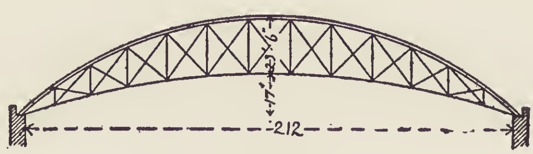


Fig. 97.

There are numerous examples of wrought iron bowstring roof trusses of from 120 to 211 feet span, principally in the roofs of train sheds. A truss of the shape shown in Fig. 97, with a span of 212 feet, a total rise of 40 feet 6 inches and a depth in the centre of 23 ft. 6 ins., was constructed with a 15-inch I-beam for the upper chord and a 4-inch round rod, in short lengths, for the lower chord.

The diagonals were made of flat bars and the vertical pieces of cross-shaped bars to give stiffness to the truss, although these pieces are subject to a tensile strain.

For a straight-pitch roof of moderate span there is no economy in using this type of truss, as the extra cost of the necessary framework above the truss for supporting the purlins will offset the saving in the truss itself.

Of late years the bowstring truss has been but little used in this country, engineers and architects seeming to prefer the braced arch instead.

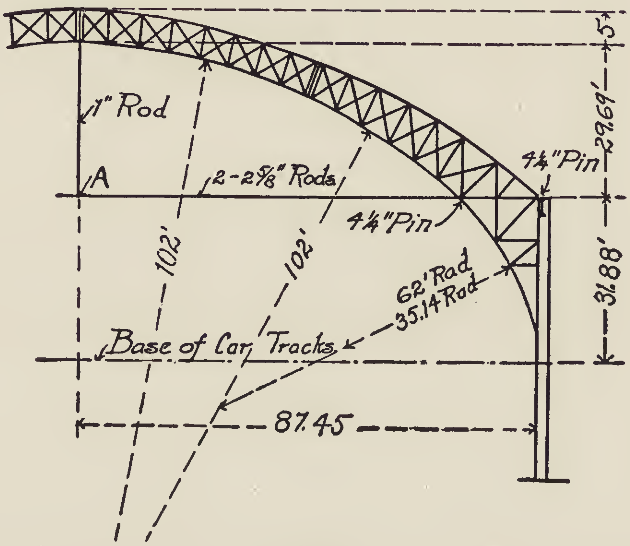


Fig. 98.

The Crescent truss has been described in Section 25, Chapter I. It is seldom used, except for the support of dome roofs, for which examples are given in Chapter V.

39. SEGMENTAL ARCHED RIBS.—Fig. 98, which is a diagram of one of three arches used in roofing the train shed of the Sullivan Square station of the Boston Elevated Railway, is a good example of this type. This construction is the same in principle as that of the wooden arch shown by Fig. 59; it can hardly be considered as a truss in the ordinary meaning of the word, although it is a perfectly legitimate form of construction.

The arches over the Sullivan Square station spring from steel columns and are provided with tension rods which take up the thrust.

The arch proper rests on two $4\frac{1}{4}''$ pins at each end as indicated in the diagram, the tie-rods being connected to these pins. The bracing below the pins is riveted to the column and the arch itself is built of angles and plates with riveted connections. Fig. 98A shows the joint at A, where the tie-rods are connected and are held up by a $1''$ suspension rod from the crown of the arch. A more complete description of this truss is given in the "Engineering Record" of June 15, 1901.

40. THREE-HINGED BRACED ARCHES.—This truss differs from all the other types of trusses that we have considered in that it consists essentially of two separate parts, each acting as a single piece and depending upon the opposing force of its mate to keep it in position. As usually built, each part is a semi-braced arch, the upper and lower members being so connected by bracing as to form a stiff frame or curved rafter.

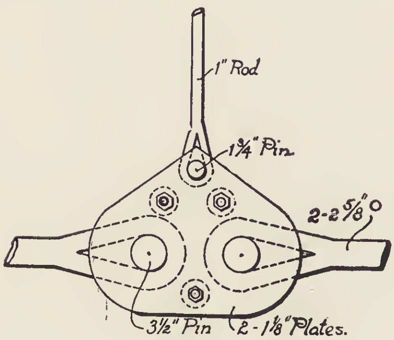


Fig. 98A.

The first use of the braced arch appears to have been in building railway bridges for French railroads, the earlier forms being rigidly connected at the top.

The first suggestion for hinging the ribs at the crown was made by M. Manton, a French engineer, who, in 1861, suggested the type of arched truss shown in Fig. 99.

It is evident that the fundamental principle of this truss is the same as that of the roof trusses shown in Figs. 100 to 102.

The first application of this principle to roof trusses, at least on a large scale, the author believes to have been in the train sheds of a Union Railway station at Frankfort-on-the Main, Germany, which was completed in the year 1888. These trusses have a span of about 184 feet. The large roof of Machinery Hall, of the Paris Exposition of 1889, was supported by this type of truss, the span in this case being 368 feet, exceeding anything hitherto attempted in a roof truss. The use of this truss in the Manufacturers' and Liberal Arts Building of the Columbian Exposition made the type familiar to most of the architects of this country, and it has since been extensively used for roofing exposition buildings, armories, train sheds, etc.

In all of these trusses the arched ribs are supported at the bottom on a turned steel pin, and with the exception of the Frankfort truss all abut against a pin at the top, thus giving the name—Three-Hinged Arch.

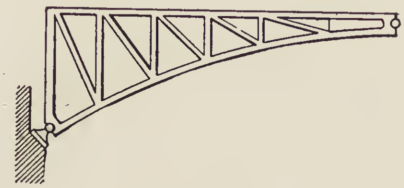


Fig. 99.

Usually the trusses are surmounted by a lantern, as shown in Figs. 101 and 102, which are bolted or riveted together at the centre.

With the shape of ribs generally used there is always a slight outward thrust at the bottom, which must be resisted either by the

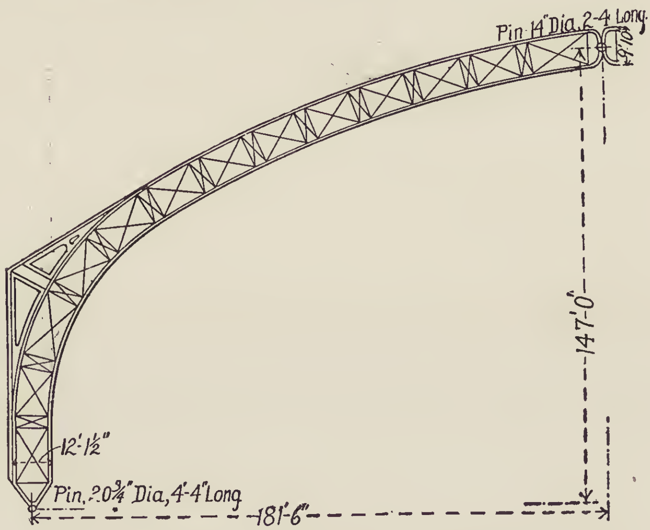


Fig. 100.—Half-Truss, Machinery Hall, Paris Exposition, 1889.

abutments, as in Fig. 99, or by tie-bars connecting the lower pins.

Most of the trusses erected in this country have tie-bars placed just beneath the floor, but the trusses in the Frankfort Depot and Machinery Hall, Paris, had no ties, and Messrs. D. H. Burnham & Co. used three-hinged arches of 160 feet span in the First Regiment

Armory, Chicago, which support two floors and a gallery, besides the roof, without ties.

The special advantages of this type of truss for the class of buildings above mentioned are economy, maximum clear space beneath the truss and provision for expansion and contraction. Much of the economy of the truss lies in the fact that it requires no columns to support it, and, the base of the truss being very near the ground level, it is well proportioned to resist wind pressure. A great advantage of this truss is the free movement allowed under temperature changes without strain to the structure, the centre rising or falling freely with a slight rotation of the semi-arches about the pivots. In the case of the trusses of the Paris Exposition it was es-

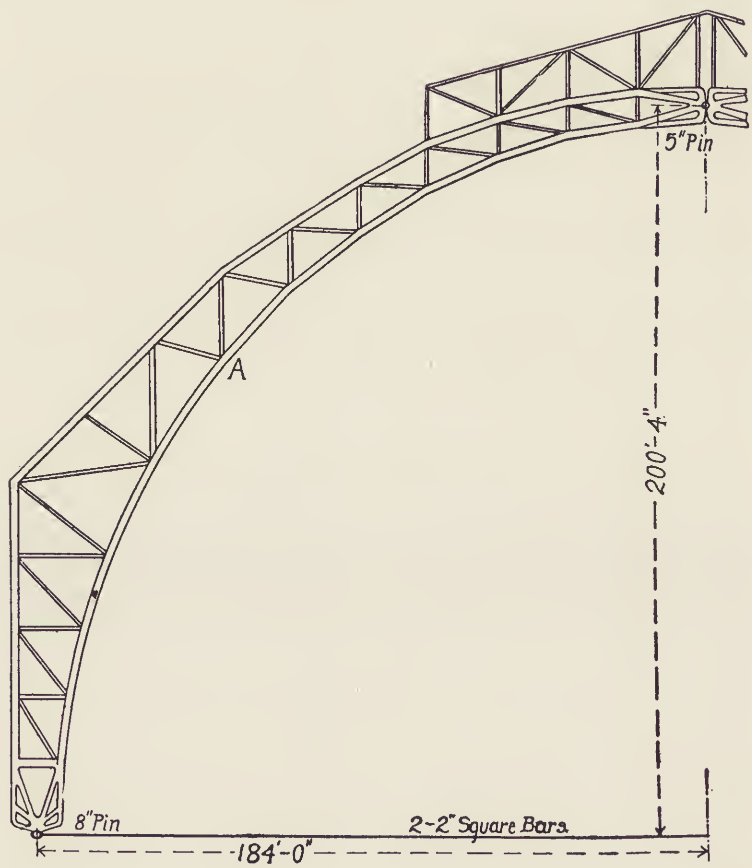


Fig. 101.—Half-Truss, Manufacturers' and Liberal Arts Building, Chicago, 1893.

timated that a range of temperature of 100° Fahr. would produce a change in level of $2\frac{1}{8}$ inches at the centre pivot.

The arched ribs are always built of plates and angles with riveted connections, and frequently with a solid plate web at the bottom.

The determining of the stresses and detailing of the members and joints will require the service of a competent structural engineer, but the illustrations given will enable the architect to decide on the general shape of the truss for the purpose of making preliminary drawings and the computations and detail drawings can be made later.

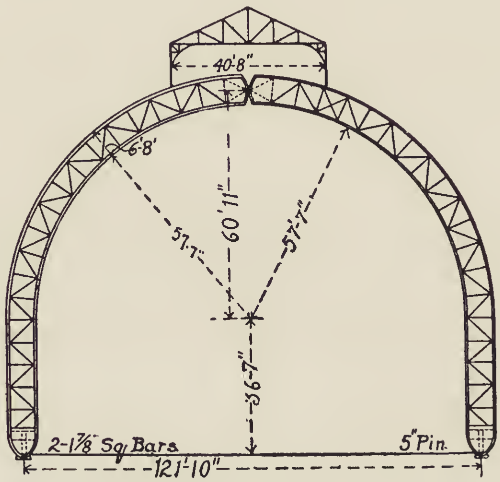


Fig. 102.—From Machinery Hall, Chicago, 1893.

41. The trusses in the Liberal Arts Building and in Machinery Hall at Chicago were spaced about 50 feet apart from centres.

In the Liberal Arts truss the inner rib is bent to a curve up to the joint A, and above that it is made up of straight pieces.*

Similar trusses are also used over the train sheds of the Pennsylvania Railroad station and the Reading Railroad station in Philadelphia.

The trusses of the former building have a span between pins of 300 feet 8 inches, a rise of 108.49 feet, with a depth at the springing of 5 feet 3 inches. The lower pins are $5\frac{1}{2}$ inches in diameter.† The trusses in the Reading Railroad Depot have a span of 260 feet.

*Complete drawings and details of the trusses of the Liberal Arts Building are contained in Vol. XXVI. of the "Engineering Record," and of the Machinery Hall trusses in Vol. XXVII.

†Details of these trusses were published in the "Engineering Record" of June 10. 1893.

The roof of the Central Armory, Cleveland, O., Messrs. Lehman & Schmitt, architects, is supported by six three-hinged plate-girder arches, that is, with a solid web instead of bracing. This is the only instance of a solid web in large arches with which the writer is acquainted, and it would appear as though it might be advantageously employed in many instances.

The trusses in the Cleveland Armory have a span of 120 feet and a rise of 52 feet 6 inches, centre to centre of pins. The two end arches also support a gallery as well as the roof. Complete details

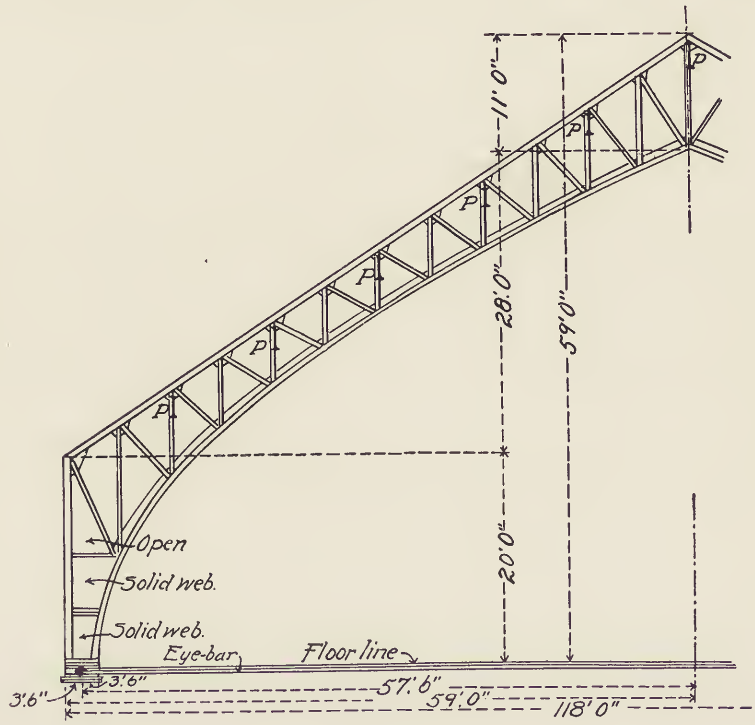


Fig. 103.

of these trusses are published in the "Engineering Record" of December 26, 1896.

Other examples of three-hinged arch trusses are given in Chapters VI.

42. BRACED ARCHES WITHOUT HINGED JOINTS.—For spans of from 80 to 120 feet this type is often built without the pin connections as in Fig. 103. The mechanical principle being essentially the same as in the three-hinged truss.

Table III., Chapter VI., gives the span and distance apart of a number of braced arches, of wide span.

43. CANTILEVER TRUSSES.—The term "cantilever" was originally used to designate a projecting beam which served as a bracket; in mechanics it is used to denote a beam or girder fixed at one end, either by being built into a wall, or, most commonly, by extending a sufficient distance beyond its support to form an anchorage for the cantilever. Thus, in Fig. 104, we have a beam resting on two supports; the portion B is a cantilever, while the part C forms the anchorage for it.

[In applying the term cantilever to trusses it is customary to interpret it as including both the projecting arm and the balancing

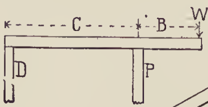


Fig. 104.

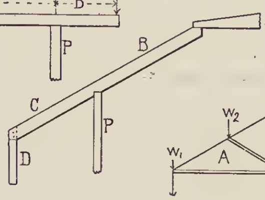


Fig. 105.

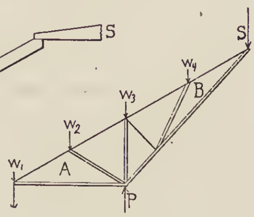


Fig. 106.

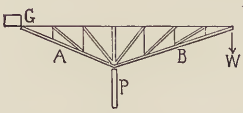


Fig. 107.

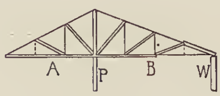


Fig. 108.

arm, as both portions form one piece of framework, and the term will be so used in this work.]

It is obvious that if the entire beam (Fig. 104) were uniformly loaded the post P would carry the greater part of the weight, and also that an additional load at W might produce an upward pull on the post D, in which case the stress on P would exceed the load on the beam.

Both conditions of loading occur in practice, although it probably most often happens that the outer end of the truss requires anchorage rather than a support.

As applied to roof construction some such arrangement as is shown in Fig. 105 is generally required to make this method of support practicable; that is, a wide centre span, with shorter spans or aisles on each side of it.

The projecting or inner arm of the cantilever is usually made from one-quarter to one-third of the centre span, and a simple truss, represented by S, is used to support the balance of the roof, the centre truss being supported by the arms of the cantilever. In all such cases, therefore, cantilever trusses must be used in pairs, one on each side of the building, and there must be rooms or passages outside of the principal span to permit of the outer or balancing arm. Such an arrangement is generally found in large halls, armories, exhibition buildings, etc., and it might sometimes be provided in other classes of buildings.

Of course, in a large building a simple beam such as is shown in Fig. 105 could not be used, but the principle of construction is the same, whether the cantilever be a simple beam or a large truss.

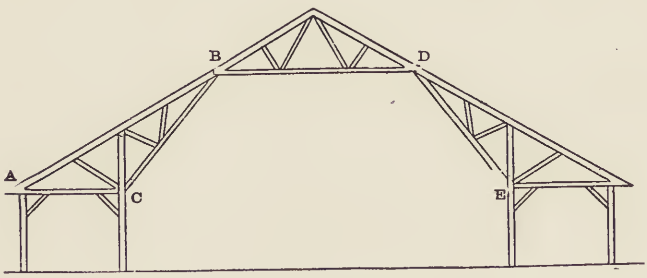


Fig. 109.—Suggestion for Wooden Cantilever Truss.

Fig. 106 shows the diagram for a truss to take the place of the beam C B, Fig. 105, the single lines representing the tension members and the double lines compression members, and Fig. 109 shows the complete arrangement of the trusses.*

The truss shown in these figures may be extended to almost any extent, and the form of the lower chord may be changed, but the general outline of the truss will be found best adapted for all cases where a wide central roof is to be supported by cantilevers.

For bridge trusses or floors the shape shown in Fig. 107 may be used, and for shed and platform roofs open on one side a truss of the shape shown in Fig. 108 is about the only practicable device. In this latter truss the proportions of the arms are such that a slight support is required at W, thereby bringing the lower portion of the rafter into compression.

*This latter illustration was offered by Mr. John Beverly Robinson as a suggestion for a simple trussed cantilever roof in an article advocating the use of the cantilever in building construction, published in the "Engineering Magazine" for November, 1896.

It will be seen from Figs. 106-108 that the strains in a cantilever truss are directly the reverse of those in trusses supported at both ends, the upper chord or rafter in the cantilever being in tension, while in all other trusses, except the hinged arch,* it is in compression.

44. ADVANTAGES AND DISADVANTAGES OF THE CANTILEVER TRUSS—The special advantages possessed by the cantilever truss are: a greater clear height in the centre than can be obtained with any other type (excepting the three-hinged arch), a light and graceful appearance, no horizontal thrust, and conse-

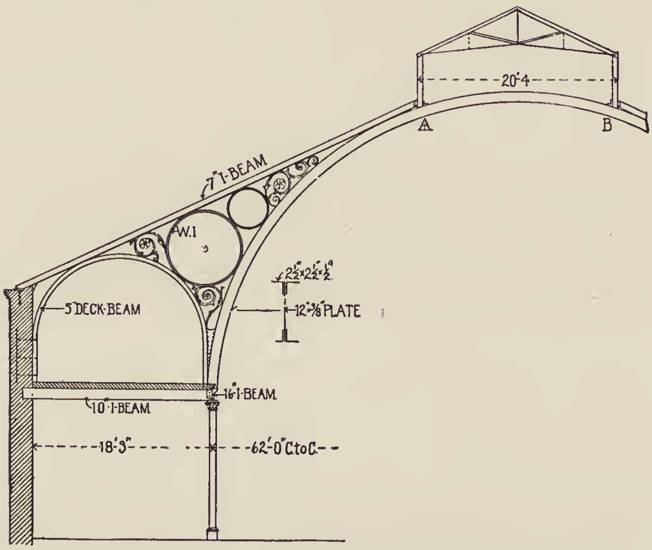


Fig. 110.—Cantilever Truss, Agricultural Hall, Norwich, England.

quently no tie-rods required. The particular advantage of this truss foi very great spans is that it can be erected without scaffolding under the centre, and in bridge work this is considered as its only advantage.

It is claimed by prominent engineers that the cantilever is not an economical type of truss, and not as desirable for spans of 150 feet or more as the three-hinged arch.

It also does not permit of as readily overcoming expansion and contraction as either the three-hinged arch, the bowstring truss or the quadrangular truss. For certain classes of buildings, however, and especially where the central span does not exceed 150 feet, it can perhaps be used with better architectural effect than is possible with other types, and with about the same economy. For roofing platforms, grand stands, etc., where an outer support is not desired, it is the only type of truss available.

45. EXAMPLES OF CANTILEVER ROOF CONSTRUCTION.—There are few examples of cantilever roof trusses of constructional importance, although there are many examples of cantilever bridge trusses.

Fig. 110 shows the proportions and some of the details of the roof trusses of the Agricultural Hall, Norwich, Eng. By using curved ribs for the compression members the truss is given a very light and graceful appearance, and it would appear to contain a minimum amount of material.

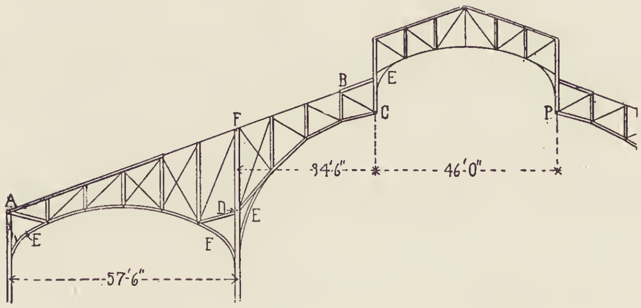


Fig. 111.—Cantilever Truss, Mining Building, Chicago, 1893.

The portion of the arch between the points A and B was probably put in to preserve the arch effect, as it could have been dispensed with.

Fig. 111 shows the proportions of the cantilever trusses used in the Mining Building at the Columbian Exposition; the double lines represent the compression members (with the exception of the arched chord of the outer span, which is in tension) and the single lines the tension members. The cantilever portion of the truss is included between the points F, B, C, D, and the anchorage truss between the points A, F, D. The curved ribs, E, E, E, do not enter the stress diagram, but merely serve as stay braces and in carrying out the arched effect.

In this roof the anchorage arm more than balances the cantilever and its load, and hence exerts a vertical load upon the outer support. The distance between trusses was 64 feet 5 $^5/_{16}$ inches.

Mr. E. C. Shankland, the engineer in charge of the constructional work of the Exposition buildings, says of this truss: "They are interesting as adding another type to the variety of trusses in the park, although they are not at all economical." The truss as built was noticeably heavy.

The lantern truss is a separate quadrangular truss supported by the cantilevers at the points C and P.

Provision for expansion and contraction in this roof was made by means of a $5\frac{1}{2}$-inch slotted hole at the joint P, thus permitting a lateral movement of the 3-inch pin of $2\frac{1}{2}$ inches.

The cantilever trusses have pin-connected joints and the quadrangular truss riveted joints.*

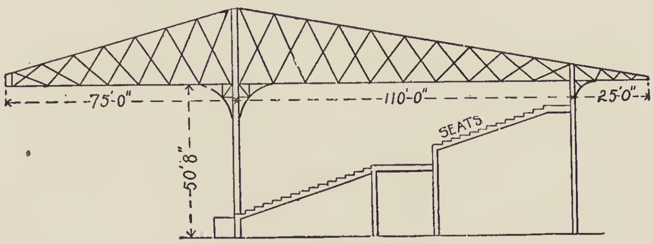


Fig. 112.

Fig. 112 is a diagram of one of the cantilever trusses supporting the roof of the grand stand at the Monmouth Park (N. J.) racing track, the details of which were published in "Architecture and Building" in February, 1890. This is an instance where the cantilever was the only type of truss that could be used, and the form adopted is both simple and economical.

As will be seen from the drawing, the main supporting post extends to the top of the truss, as is usually the case with cantilever trusses, and the truss is riveted to each side of it. The upper and lower chords were made of two angles and a web plate, the upper chords or rafters acting as a tie-beam between the bracing. The bracing consists of angle bars used in pairs and varying from $3 \times 2 \times \frac{1}{4}$ inches to $3 \times 3 \times \frac{5}{16}$ inches, the whole frame being connected by rivets.

*The details and stresses in these trusses were published in the "Engineering Record," Vol. XXIX., page 9.

Chapter III.

LAYOUT OF TRUSSED ROOFS—BRACING OF THE ROOF AND TRUSSES.

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46. The general arrangement or layout of the roof construction should be considered when making the preliminary studies, as in buildings with trussed roofs the manner in which the roofs are to be supported affects both the cost and the appearance of the building, and often the internal arrangement.

In deciding upon the "layout," the type of truss or trusses to be used, their approximate span and rise should be carefully considered and fixed, as upon these will often depend the shape of the ceiling, the arrangement of the supports and the height and pitch of the roof.

If these factors have been wisely determined, the consideration of the strength and details of the construction may be left until the general plans are completed, as within certain limits of span and rise any truss may be made of sufficient strength and rigidity, and the size of the truss members does not, as a rule, affect the rest of the building.

When designing a trussed roof it should be remembered that short spans and a level tie are in general the most economical, and if the span can be reduced by using posts and without interfering with the internal arrangements it is better to do so.

In roofing buildings having large rooms, with permanent walls between, advantage should always be taken of the walls by placing posts over them to support the purlins directly above.

The principal points to be fixed in laying out a trussed roof are: The type or types of trusses to be used, the pitch of the roof, the distance that the trusses shall be placed apart and the spacing of the purlins. As the settlement of these questions will be determined in a great measure by the character and style of the building and the purpose for which it is to be used, it seems best to take them up in connection with different classes of buildings, or forms of construction.

FLAT ROOF CONSTRUCTION.

47. By the term "flat roof" is here meant a roof in which the rise is not greater than $1\frac{1}{2}$ or 2 inches to the foot, the exact pitch being determined by the kind of roofing to be used.

PITCH OF FLAT ROOFS.—For pitch and gravel (composition) roofs the rise should not be less than $\frac{3}{8}$ inch nor greater than $\frac{5}{8}$ inch to the foot; for tin or copper, standing seam steel roofing and canvas roofing a pitch of $\frac{1}{2}$ inch to the foot is sufficient, but there is no objection to a steeper pitch, unless it be that the steeper pitch requires the greater amount of material, and hence weighs and costs more.

LAYOUT.—As a rule,where flat roofs are supported by trusses, the trusses are placed across the building in parallel lines, with their ends resting on or built into the side walls. In supporting the roof from the trusses either of two methods may be adopted. The more common method, probably, is to rest the ends of the rafters directly on the top chords of the trusses, as shown in Fig. 113, the trusses being made of different heights, so as to give the desired pitch to the rafters. This method answers very well for wooden roofs of

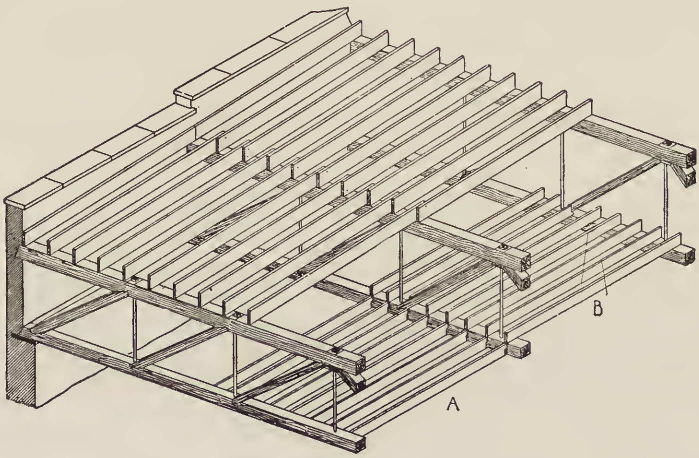

Fig. 113.

moderate span, but when the span is 60 feet or over it will be more economical to support the rafters on purlins, as shown in Fig. 114. The advantages of the latter method are, that the purlins being placed over the joints of the trusses, no transverse strain is produced in the top chord, and the roof is better tied to the walls in both directions. The use of purlins also permits of smaller sizes for the rafters and the placing of the trusses further apart. By bracing the purlins as shown at A, Fig. 114, the trusses may be spaced from 20 to 24 feet apart. The purlins should be spaced from 8 to 12 feet apart, according to the width of the panels of the trusses.

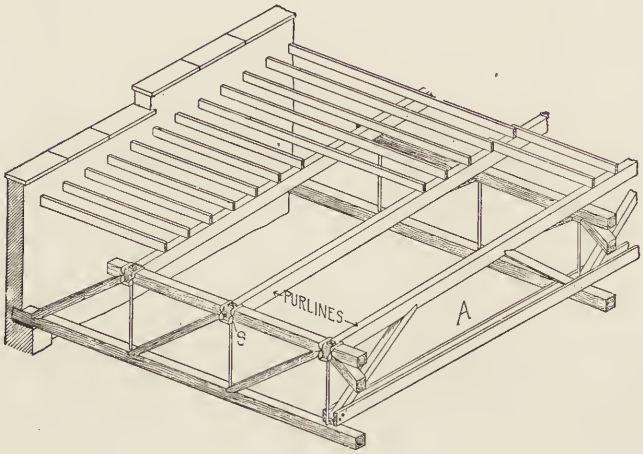

Fig. 114.

If it is necessary to use shallow trusses, the purlins may be placed over every other joint as in Fig. 115.

The forms of construction above described are also applicable to deck roofs, and the same principles apply to them as to flat roofs.

The ceiling joists will naturally extend from truss to truss, either resting on top of the tie-beams, as shown at B, Fig. 113, or framed

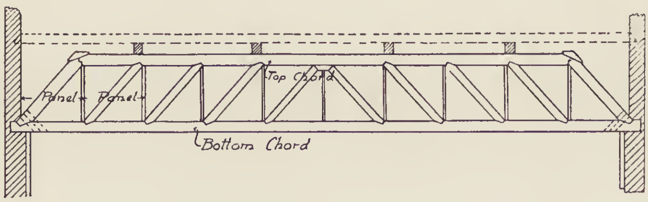

Fig. 115.—Ten-Panel Howe Truss.

between them, as at A. If framed between the tie beams the bottom of the joists should be kept $\frac{1}{2}$ inch below the bottom of the beams, to allow for furring the latter, if the laths are to be applied directly to the joists. If the ceiling is to be "strapped" or "cross furred" the joists may be flush with the tie-beams. If the spacing of the trusses exceeds 16 feet it will be more economical to support the ceiling joists on the tie-beam of the trussed purlins.

Types of Trusses for Flat Roofs.—If wooden trusses are to be used the Howe Truss will generally prove the most economical for spans up to 90 feet. For longer spans it may be cheaper to use a segmental arch truss, as shown in Figs. 58 and 59 of Chapter I.

If the roof is built in a timber country, the lattice truss, as described in Section 12, may be employed.

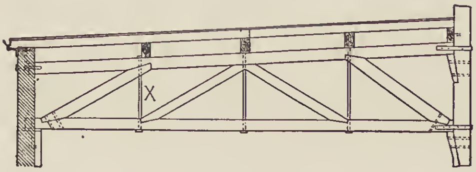

Fig. 116.

The rules which should govern the height and proportions of Howe and Lattice trusses are given in Sections 10 and 12, and Tables I and II will be found useful in making the preliminary drawings.

If the pitch of the roof is parallel with the trusses, as is frequently the case in buildings having a clerestory above the central portion, the top chord may be given the same inclination as the roof, as shown in Fig. 116, the minimum ratio of height to span being taken on the line of the rod X.

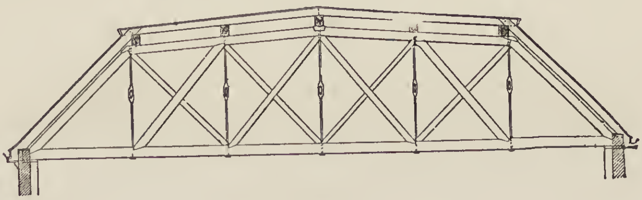

Fig. 117.

For deck roofs the trusses may be made of the shape shown by Fig. 117, and for such roofs, it will be well to put in counter-braces, to resist the wind pressure against the sides of the roof.

48. *If steel trusses are to be used*, one of the types shown by Figs. 80 to 85, Chapter II, will generally be most economical, the particular type to be selected depending largely upon the span, Figs. 80 and 81 being best adapted to spans under 50 feet and Figs. 82-85 to spans over 50 feet. For spans exceeding 100 feet, a truss such as

is shown in Fig. 88 will generally be as economical as any, when the truss is supported by brick walls.

With steel trusses it will nearly always be more economical to use purlins, supported at the joints of the trusses, for carrying the jack rafters, or sheathing. Where an absolutely fireproof roof is not required, an economical construction, for spans up to 60 feet will be to have heavy timber purlins, from 6 to 8 feet center to center, and on top of these place plank sheathing to receive the roofing. Such construction would resist fire fully as long as an unprotected steel truss.

For strictly fireproof construction, I-beam purlins or rafters will generally be required, spaced from 5 to 8 feet on centers, with tile or concrete filling between. For spans up to 50 feet, it will generally be cheaper to support the roof beams on the truss, the joints in the latter being spaced to suit the beams. For greater spans, it will be more economical to space the trusses twenty feet or more apart, and to support the roof beams on heavy I-beam or trussed purlins.

49. Spacing of Wooden Trusses.—For spans up to 75 feet, the most economical spacing will be from 12 to 16 feet. For greater spans, the spacing should be from 16 to 24 feet.

Spacing of Steel Trusses.—With steel trusses, the greatest economy of material is obtained, when the distance center to center of trusses is about one-fifth of the span, but as the cost of manufacture varies almost directly as the number of trusses, a spacing of from one-fourth to one-third of the span will generally be most economical, all things considered. Table IV. and the examples given in Chapter VI. will give a good idea of the usual spacing for different spans and types of steel trusses.

PITCH ROOFS.—WOOD CONSTRUCTION.

50. The most economical method of supporting a pitch roof of wide span depends upon many circumstances, such as the character of the building, the pitch of the roof and nature of the roofing, the width of the building, the shape of the ceiling below and whether the roof is to be of wood or iron.

The pitch of a roof, by which is meant the inclination of the rafters with a horizontal line, has a very close relation to the cost, and in buildings of a purely utilitarian character should be carefully considered.

In measuring or designating the pitch of a roof three different methods are in vogue. Architects often speak of the pitch in terms of the angle, measured in degrees, but builders more frequently designate the pitch in terms of the proportion of height to span. Thus, a roof having a span of 36 feet and a height at the centre of 9 feet, would be called a one-fourth pitch; if the centre height were 12 feet it would be a one-third pitch, and so on. The latter method, however, is apt to be confusing at times, and the former is not always easy to measure, so that the writer prefers to designate the pitch in terms of the rise in one horizontal foot, as shown in Fig. 118. This designation is both accurate and easy to lay out.

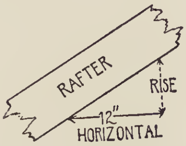

Fig. 118.

The most economical pitch for roofs supported by wooden trusses, with a horizontal tie-beam, is a rise of 6 inches to the foot, or approximately 26 degrees. This gives about the minimum amount of roofing material, with a sufficient depth to the truss and efficient resistance to the weather. For roofs supported by trusses of the scissor or hammer-beam types the pitch should be 45 degrees or over.

On dwellings and public buildings, where the roof is an important factor in the external appearance, the pitch of the roof is most frequently decided by architectural considerations, but in every case it should be such that the water from rain or snow will run off without being driven under the roofing.

The following table gives what is generally considered as the least desirable inclination for different kinds of roofing, and also the equivalent designations for the pitch:

TABLE III.

LEAST DESIRABLE PITCH FOR ROOFING MATERIALS.

	Rise in 12 inches.	Degrees.	Proportion of height to span
Corrugated iron	4½ inches.	20 dg., 33 min.	....
V-crimped steel	2 "	9 " 27 "	one-twelfth.
Shingles, wood	6 "	26 " 34 "	one-fourth.
" tin	6 "	26 " 34 "	"
Slates, large sizes	4 4-5 "	21 " 48 "	one-fifth.
" medium sizes	6 "	26 " 34 "	one-fourth.
" small sizes	8 "	33 " 40 "	one-third.
Tiles, clay	7* "	30 " 15 "	....
" metal	7 "	30 " 15 "	....

*While tiles may be used with this pitch, they do not show to advantage; a rise of 10 to 14 ins. is much more desirable.

So-called "ready roofings" consisting of an asphaltic composition on canvas or burlap may be laid on any pitch exceeding one inch in twelve. For factories, warehouses, etc., these roofing materials form a light, economical and fairly durable covering.

51. Simple Forms of Roof Trussing.—The simplest method of trussing a roof where the span is not over 35 feet is shown in Fig. 119, the truss being of the type shown by Fig. 32, with its principals set flush with the rafters. The trusses should be placed about 6 feet apart, so that the timbers need not be large, and to distribute the weight more evenly on the walls. Such a roof is best adapted to sheds, pavilions and unfinished lofts where no ceiling is required.

For pavilions, platform roofs, etc., of from 35 to 40 feet in width, where a row of posts down the center is not objectionable, the rafters may be economically supported in the manner shown by Fig.

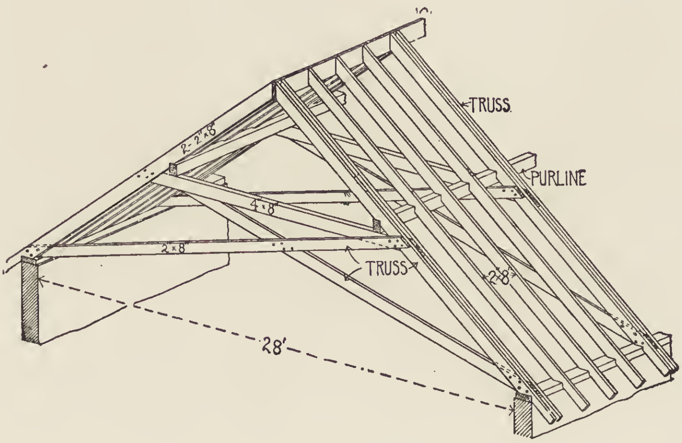

Fig. 119.

120, a purlin being let into the underside of the rafters about 1 inch and supported by braces placed above the posts, the tie-beam being bolted to the plate, which also holds the ends of the rafters.

When the span is greater than 35 feet, or where there is to be a ceiling, the common method of construction is to use parallel trusses, spaced from 12 to 18 feet apart, with their ends resting on the side walls, and with purlins spanning from truss to truss as shown in Fig. 121, the ceiling being supported by the tie-beam, either as shown at A or B.

For wooden roofs of this kind the types of trusses shown by Figs. 8, 10, 12 or 17 (Chapter I) are best adapted, the number of braces or panels being such that the principals, or truss rafters, will be supported at distances of from 8 to 10 feet, measured on the slant.

If it is desired to have a room in the attic, a truss like that

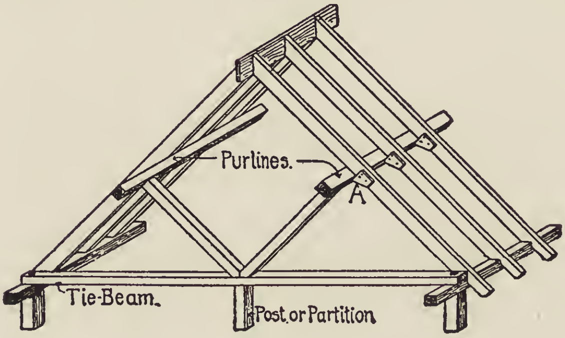

Fig. 120.

shown by Fig. 19 may be used. Where a ceiling is not desired, trusses similar to Figs. 39 and 41 present a neat appearance, while at the same time being economical and effective.

In laying out a roof like that shown by Fig. 121, it will generally be found most economical to divide the trusses so that the purlins (which should always be placed at or near a joint) will not be greater than 12 feet apart, and to space the trusses not more than 18 feet apart for shingle roofs, or 16 feet apart for slate or tile roofs; this will permit of using 2x6 rafters for shingle roofs and 2x8 rafters for slate roofs and a single beam for the purlins.

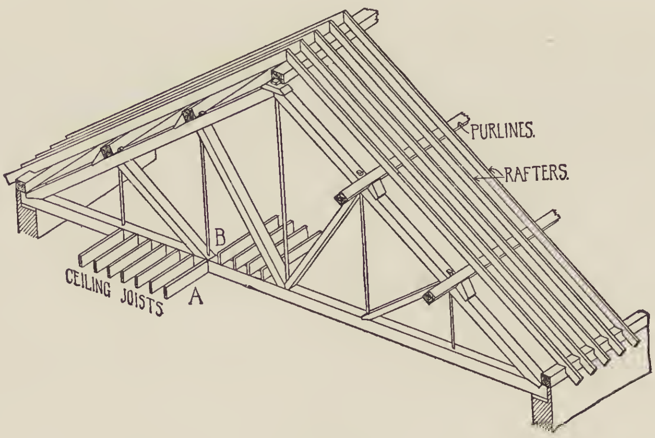

Fig. 121.

52. ROOFS WITH INTERMEDIATE SUPPORTS.—While a truss, particularly if it has a horizontal tie-beam, acts in general in the same way as a beam, yet it differs from a beam in that it cannot, as a rule, be strengthened by introducing a support between the end bearings.

Thus, if we should put a post under the truss shown in Fig. 122 at the point indicated, the only aid that it would afford the truss

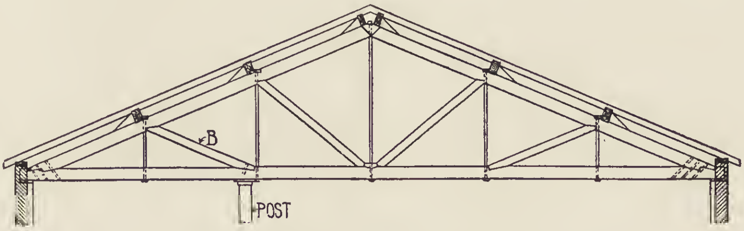

Fig. 122.—Example of Bad Construction.

would be that it would take up the vertical component of the force acting in B or about one-half of the load on the lower purlin. If the post was introduced between the two joints it would do no good at all, and might be a source of danger in case the end bearings settled so as to produce a transverse strain in the tie-beam.

If the post was placed under the centre of a truss of the type shown it would divide the truss into two trusses, each of one-half

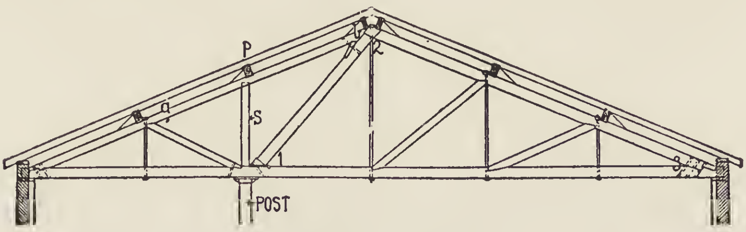

Fig. 123.—Proper Construction.

the span, and would reduce the strains accordingly. Wherever an intermediate support is introduced, however, *the space on each side should be treated as an independent span* and two trusses used instead of one.

Thus, if we had a roof to support of the shape shown in Figs. 122 and 123, and it was desired to introduce a post at about one-third of the span, the truss should be arranged as shown in Fig. 123, there being in reality two trusses, one on each side of the post, with their tops connected by the beam *a b*, the load on the purlin P being directly carried by the posts. Although the tie-beam is shown in one piece there would be no strain over the post, and it could be jointed there if necessary. The framework, 1, 2, 3, should be calculated as an independent truss. The strains in such a roof would be considerably less than if there were no post, but where the width is less than 60 feet the saving in the sizes of the truss members would be about offset by the cost of the post unless the post was placed at the centre. For greater widths an intermediate support will effect a considerable saving, and in any case would materially reduce the weights coming on the walls.

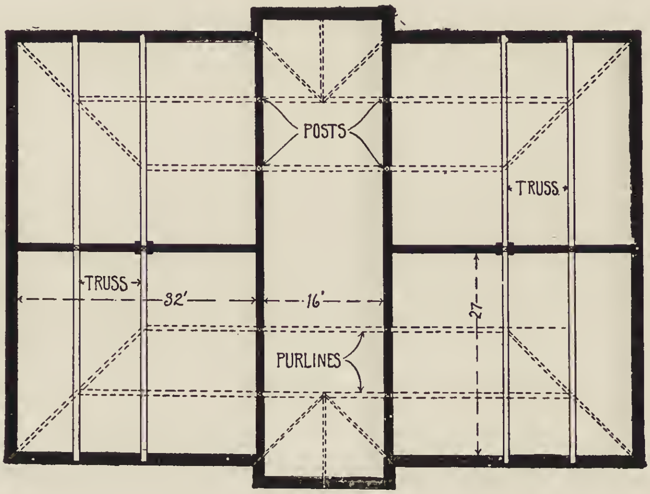

Fig. 124.

SCHOOL HOUSE ROOFS.

53. The construction of a pitch roof over a large school building generally requires an arrangement of trusses and posts that is peculiar to this class of buildings, and while not difficult or complicated is yet worthy of description.

Such buildings are generally covered with a hip roof, terminating in a deck, and the size of the rooms and the length of the rafters usually necessitate the use of purlins to support the rafters and trusses to support the purlins.

Fig. 124 may be considered as representing the general arrangement of the supporting walls of an eight-room school building, and the method of construction applicable to this plan can generally be applied to any building with large rooms and brick partitions.

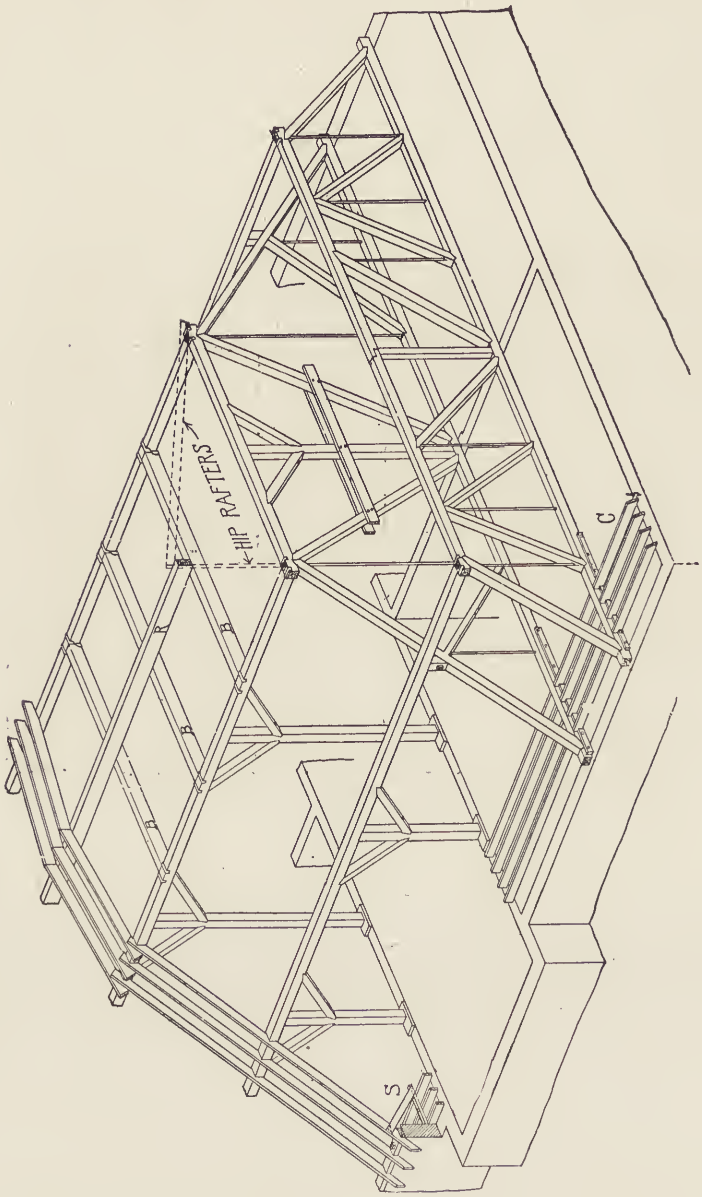

Fig. 125.

The roof of this building is supposed to be hipped, the pitch roof terminating in a deck 18 feet inside of the wall line. The pitch of the roof is $10\frac{1}{2}$ inches to the foot, and the wall plate is 3 feet 6 inches above the bottom of the ceiling joists.

An economical construction for such a roof requires the use of two lines of purlins, one under the edge of the deck and the other half way between it and the wall plate.

The most economical manner of supporting the purlins and the deck roof is shown in Fig. 125 the position of the purlins, trusses and posts being also shown on the plan Fig. 124.

To support the purlins over the school rooms two trusses are required over each room. For convenience of construction the top chord of the lower trusses may be made continuous or spliced over the post. Advantage should be taken of the partition walls to support the purlins at points directly above by means of posts, which should be braced as shown in the drawing. The top chords of the trusses serve as purlins to support the rafters, and the purlins parallel to the front and rear of the building are placed at the same height and supported from the trusses by stirrup irons or patent joist hangers. The rafters of the deck roof generally have a rise of from $\frac{3}{4}$ inch to 3 inches to the foot, and their inner ends should rest on a purlin, R, which can generally be supported by beams, B B, hung from the deck purlins. If the height of the purlin R does not give sufficient pitch to the roof it maybe blocked up from the beams B B. All parts of the roof should be well tied together, the posts tenoned into the purlins and the rafters well spiked to the purlins and wall plate. If the wall plate is raised above the ceiling joists it should be tied every few feet by tie-braces spiked to the rafters and to the ceiling joists, as shown at S.

The bottom chords of the trusses will naturally be used to support the ceiling joists over the school rooms, as shown in the figure. In some cases, it may be more economical to rest the tie beam on the top of the wall, and hang the ceiling from the trusses by means of rods and girders. If there are gables, the purlins can be extended to the gable walls, and the valley rafters will be supported by the purlins, as shown in Fig. 124.

54. Fig. 125A shows a method of supporting a school house roof, so as to utilize the attic space. This particular construction was used by the author to support the roof of a two-story school house 83 ft. 9 ins. wide, with school rooms on each side and a hallway about 25 ft. wide through the centre. As it was desired to obtain an assembly room in the attic, the author utilized the interior walls for supporting the roof, so that the only obstruction in the hall is the

two rows of posts, 15′10″ C to C longitudinally. In this way a large hall was obtained at very little additional expense over the ordinary roof construction. As may be seen, each pair of posts supports a cantilever truss on each side and a Howe truss in the centre. The top chord of the Howe truss extends over the posts to form the top of the cantilevers. The rafters of the side roof extend from the wall plate to the deck, and are supported by the purlins P, which rest on the bottom chords of the cantilevers. The sloping roof is covered with shingles and the deck roof with tin. The roof is braced longitudinally by trussing the rafters and ceiling joists. The truss timber is of Oregon pine.

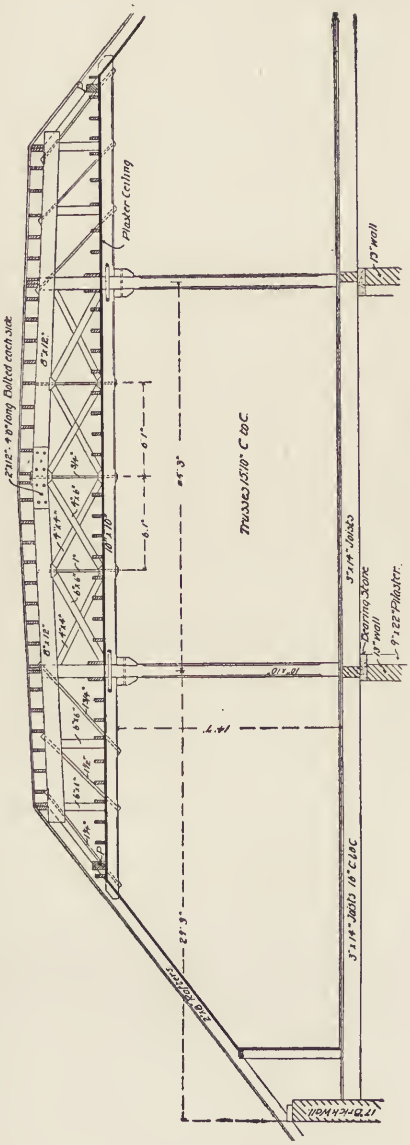

Fig. 125A.—Section of School House Roof.

55. ROOFS WITH LONGITUDINAL TRUSSES. — When the roof to be supported is not too long, or posts can be placed near the end walls, it is often cheaper to support the roof by longitudinal trusses, particularly where there is to be no ceiling, or where it is desired to have a greater clear height in the centre than at the sides. Fig. 126 shows the manner in which the roof of a large car house and stable at Avondale, Ohio (James W. McLaughlin, architect), is supported by longitudinal trusses. As will be seen, the top chord of the trusses forms a support for the rafters at a point a little above their

centre, and the upper lengths are supported entirely by the trusses, the rafters being mutually supported at the ridge, as in a small pitch roof. With such construction, however, the tops of the trusses should be connected frequently by horizontal tie-beams, to prevent

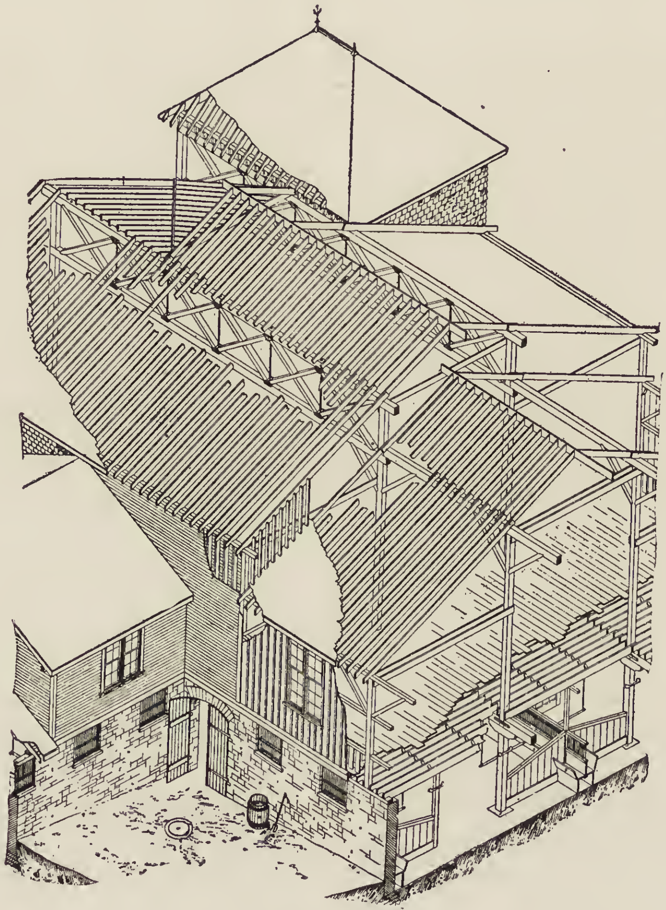

Fig. 126.

them from being pushed outward by the upper rafters. These tie-beams should also be of such a section that they may act as struts when under wind pressure. In this particular building one of the trusses forms the ridge of the front portion of the roof, while the other truss supports one side of a sort of tower. In·this instance it

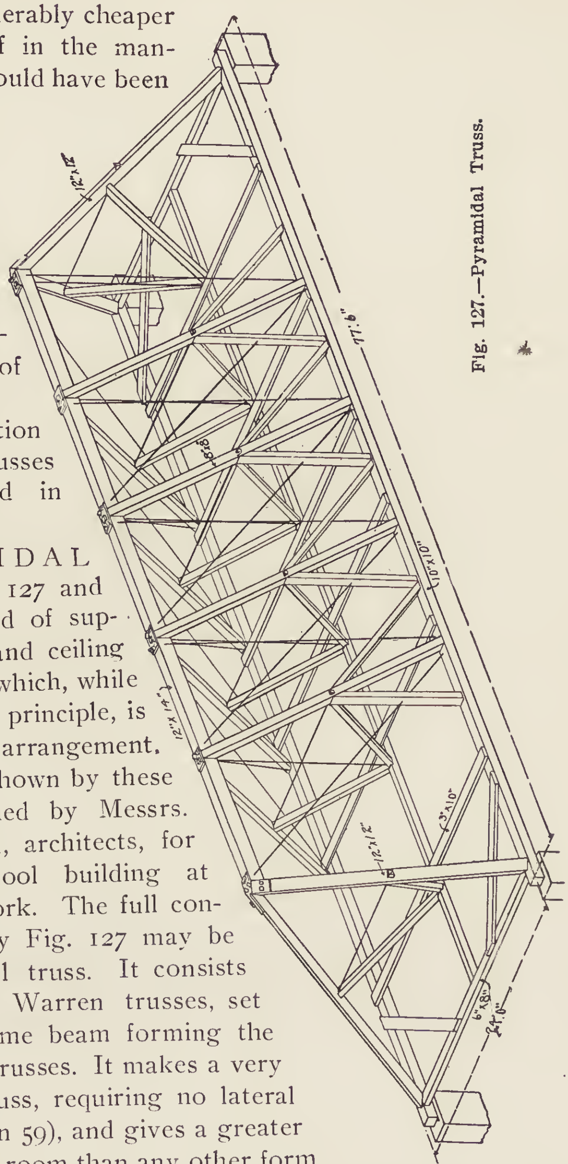

Fig. 127.—Pyramidal Truss.

was probably considerably cheaper to support the roof in the manner shown than it would have been by trusses extending across the building.

The construction shown at the right in Fig. 126 is that commonly employed for supporting the roof of large barns.

Further application of longitudinal trusses will be considered in Chapter IV.

56. PYRAMIDAL TRUSSES.—Figs. 127 and 128 show a method of supporting the roof and ceiling over a large room which, while embodying no new principle, is yet an uncommon arrangement. The construction shown by these figures was designed by Messrs. Boring and Tilton, architects, for supporting a school building at Newtown, New York. The full construction shown by Fig. 127 may be called a pyramidal truss. It consists essentially of two Warren trusses, set on a slant, the same beam forming the top chord of both trusses. It makes a very strong and stiff truss, requiring no lateral bracing (see Section 59), and gives a greater clear height for the room than any other form of construction. The same principle could be adopted in roofing a square area with supports at four corners only, using a pair of King rod or Queen trusses in the place of the Warren truss, Fig. 127. This latter construction was used by the same architects in

roofing two halls in the East Orange, N. J., Town Hall, drawings of which were published in the Engineering Record of Oct. 26, 1901. The truss shown in Fig. 127 has been patented by Mr. Boring, and should any one desire to avail himself of this idea, he should obtain permission from Mr. Boring, which we understand will not be a difficult matter.

SIMPLE STEEL ROOFS.

57. Simple steel roofs supported by brick or stone walls are built in very much the same way as wooden roofs with the exception that steel shapes are used in place of the timbers or joists. The connections between the pieces are usually made of bolts and rivets, rivets being

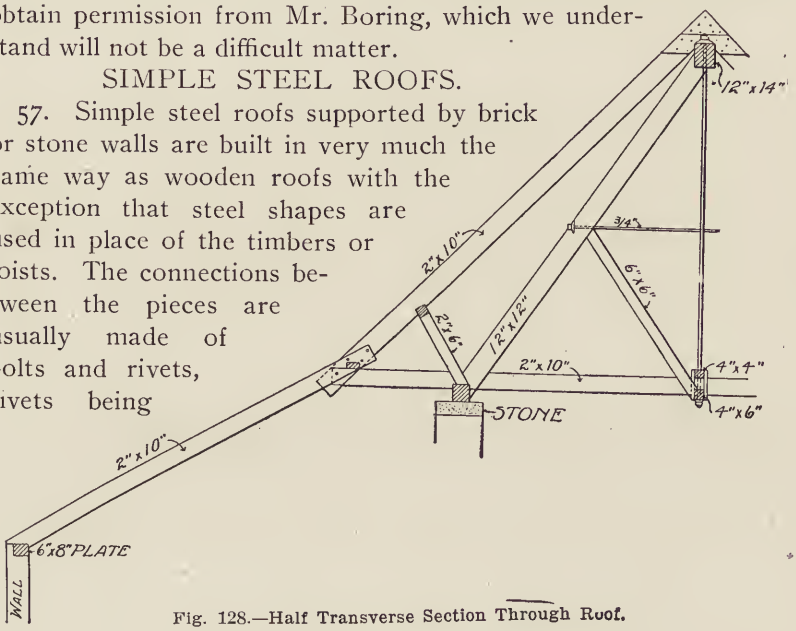

Fig. 128.—Half Transverse Section Through Roof.

used in all the points which can be assembled at the mill, while those joints that have to be made at the building are bolted. Where the span does not exceed 60 feet, angles are almost exclusively used for the members except in cases where the rafters or tie-beams are subject to a transverse strain, in which case a pair of channels or a pair of angles and a web plate are generally employed. The framing of the roof above the truss will depend in a great measure upon the kind of roofing that is to be used. If the roof is to be of corrugated iron it will be most economical to use 5, 6 or 7-inch I-beam purlins, spaced from 4 to 5 inches apart and bolted to the rafters of the truss, as in Fig. 75, the corrugated iron being secured to the I-beams by galvanized iron straps. The trusses may be spaced from 16 to 20 feet centre to centre. If the roof construction is to be of wood supported by steel trusses the best construction would be to use 10″x12″ purlins spaced about 7 feet centre to centre and on top of these spike 2½ or 3-inch planks to support the roofing: this makes a slow burning construction and for an open roof is comparatively cool, and any form of roof covering can be placed on top of it. If a cheaper form of construction is desired the purlins may be spaced from 8 to 10 feet, centre to centre, and 2″x6″ jack rafters set on top of them to receive ordinary $\frac{7}{8}$ sheathing. The only difference in the cost between this construction and that of the plank roof would be in the 2x6 rafters and $\frac{7}{8}$ sheathing, which would cost a little less than the $2\frac{1}{2}$ or 3-inch planks, but it would not offer nearly as good protection from fire or make as cool a roof. The purlins should be secured to the trusses by means of angles bolted to the trusses and lag screwed or bolted to the purlins. The joints in the trusses should always be arranged to suit the spacing of the purlins.

58. *If an incombustible roof* with slate or tile covering is desired the most economical construction would be that shown by

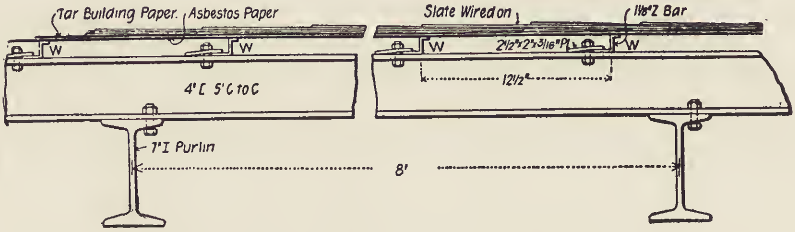

Fig. 129.—Detail of Slate Covering on Steel Roof.

Fig. 129, which is a detail of the roof construction of a power-house in St. Louis, designed by Mr. E. W. Stern, C. E., and described in the "Engineering Record" of Feb. 19, 1898.

The roof is supported on riveted trusses carrying horizontal I-beam purlins, which support channel bar jack rafters 5 feet apart. On these elements light uneven legged Z-bars are clamped about one foot apart, parallel with the purlins and so close together that the ordinary 28-inch slate may be wired directly to them without requiring intermediate sheathing boards. Before the slate was laid, however, a layer of asbestos paper was stretched tightly over the Z-bars. One thickness of tarred building paper was laid on that before the slate was set. The Z-bars are not punched, bolted, riveted, nor wired, but are securely held by clamps formed by bolting a thin flat strip of steel to the top flange of the channel, as shown in the detail, Fig. 130. The bolt is placed a little nearer one end than the other so as to increase the leverage and to spring it down and give a good grip when the nut is screwed up snug. The inclined upper end of the clip is driven down by a hammer from the original position indicated by dotted lines, to an oblique angle in contact with the Z-bar flange. The asbestos paper was used as a preventive of condensation. This construction is that generally employed where the roofing is of slate or clay tiles.

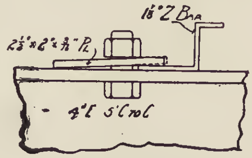

Fig. 130.–
Fastening of Z-Bars.

Occasionally the small bars to which the slates are fastened span directly from truss to truss, but this necessitates placing the trusses quite close together.

A description of a steel roof framed in this way is given in Section 34.

BRACING OF ROOFS AND TRUSSES.

59. A properly designed truss will be perfectly rigid in the direction of its length under any stress likely to come upon it, but of course it depends upon the roof framing, or upon diagonal or lateral braces to keep it from tipping sideways, and if the truss is supported by posts, some provision must be made to prevent the building racking, as shown by Fig. 131. (A *lateral brace* is one placed at right angles to the plane of the truss; as a rule it is made in the form of a strut, but capable of resisting tension also. *Sway braces* are the diagonal rods between the laterals).

In the case of open buildings such as train sheds, drill halls, exposition buildings, etc., the bracing of the roof and walls to resist the wind pressure against the side or end requires nearly as much engineering skill as for designing the trusses. Buildings with masonry walls are more rigid than those with frame walls, or in which the trusses are supported by posts, but even with brick buildings the effect of the wind blowing against the side or end of the building and roof should be carefully considered.

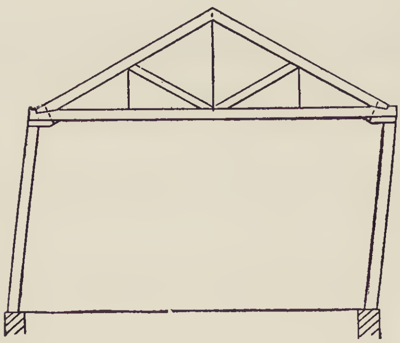

Fig. 131.

A simple truss roof, such as is shown by Fig. 121, supported by brick walls of moderate height will usually be sufficiently braced by the purlins and ceiling joists, especially if the sheathing is laid diagonally. If the roof is hipped, the hip trusses in connection with the ceiling beams, rafters and purlins will brace the roof endways so as to make a very rigid construction. If the roof terminates against gable walls, the roof should be braced in the end panels by diagonal braces extending from the purlins to the ceiling joists close to the gable. The ceiling joists and purlins should be well anchored to the gables.

60. When a roof, such as is shown by Fig. 121, is placed over a long room, there is danger of the building being sprung in the centre of the long sides by wind pressure. This may be provided for by building horizontal trusses on top of the tie-beams of the main

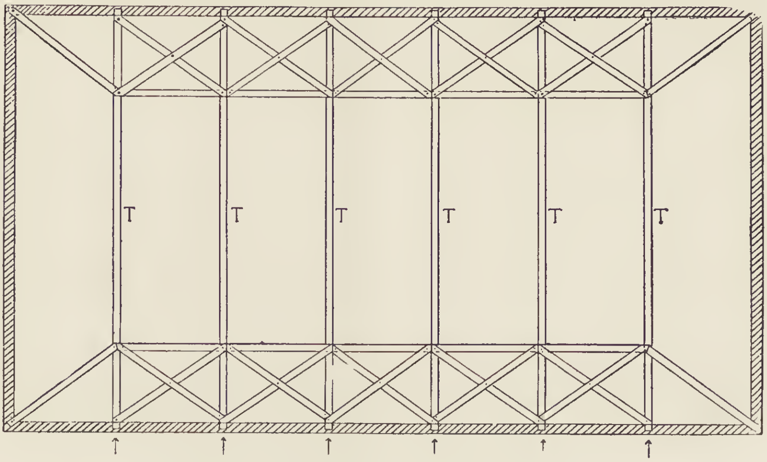

Fig. 132.—Plan Showing Horizontal Bracing.

transverse trusses, as shown by Fig. 132, utilizing the wall as one chord of the truss and the main tie-beams for the vertical members. With the wind from the direction indicated by the arrows the bracing on that side forms a Warren truss with the wall in compression and the diagonals in tension, while the bracing against the opposite wall forms a Howe truss with the braces in compression. The bracing should therefore be well bolted to the tie-beams, or secured with lag screws, so as to resist either tension or compression. The end braces should also be well bolted to the wall. By means of this bracing or trussing, the horizontal wind pressure will be transferred from the centre of the building to the ends.

The author recently roofed a building 70 feet wide and 125 feet long with walls 30 feet high, the interior being one large room, with Howe trusses braced in this way, and it has been found to answer very satisfactorily. The bracing of long and narrow churches is considered in Chapter IV.

When trusses are supported by posts there should be a rigid connection between the post and truss, Figs. 67, 72 and 81, show the usual method of bracing steel trusses from the posts and for wooden trusses and posts, examples of bracing are shown in Chapter VI.

61. The method of longitudinal bracing employed in the roof shown by Fig. 75 is partially shown by Fig. 133, the lines a, a and b, b representing angles fastened to the tie-beams of the trusses, T, and acting as both struts and ties. Corresponding braces were used between the rafters of the trusses.

The method of bracing the roof over the Kansas City Auditorium

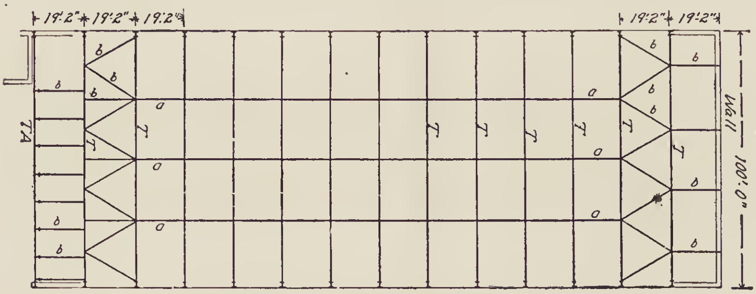

Fig. 133.—Plan of Roof Shown in Fig. 75.

is shown by the lower diagram of Fig. 92. This building is about 314x198 feet, and is virtually one story in height with an auditorium 72 feet in maximum height occupying most of the interior. The roof is of wood covered with tar and gravel and supported by 10 steel trusses, arranged in pairs, each pair of trusses being braced together by lateral struts and diagonal rods in the planes of the top chords and by 5 panels of vertical transverse bracing between each pair of trusses, with five lateral struts between each pair of tie-beams.

Details of these trusses with a plan and interior view of the building were published in the Engineering Record of July 22, 1899. The walls of this building are of brick and stone.

Examples of the bracing of large armory roofs are given in Chapter VI.

Chapter IV.

OPEN TIMBER ROOFS AND CHURCH ROOFS.

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62. As a rule the shape of the trusses used for church roofs is determined more by the external or internal appearance desired for the building than by economical or mechanical considerations, and hence such roofs present peculiarities of construction which place them in a distinct class.

OPEN TIMBER ROOFS.

Nearly all European churches have either vaulted ceilings or open timber roofs, in which the entire construction of the roof is exposed, and such roofs are very frequently used in this country.

Open timber roofs seem particularly appropriate to churches, and especially to those which are of the Gothic style of architecture. They are also frequently used over large halls and occasionally over a two-story hall in a private dwelling.

In this country wooden ceilings having the appearance of being the actual roof are frequently placed *beneath the roof*, but in such cases the actual supports of the roof are concealed, and such roofs will therefore be treated under another head, "open timber roofs" being intended to include only those in which the larger part, if not all, of the trusses is exposed to the audience.

Open timber construction is not well adapted to an irregular plan or to broken roofs, as the beauty of this work depends in a great measure upon symmetry and repetition of the forms. It is also desirable that exposed roof construction be as simple as possible in its contructive features, as in irregular or complicated construction it is difficult to make the connections so that they will have ample strength and at the same time a neat appearance. As a rule open timber work is only employed where the roof is a simple pitch roof, with perhaps another roof of the same kind intersecting it.

Open trusses are usually made either entirely of wood, or the few rods that are used are cased in imitation of timbers. Occasionally light tie-bars are used to connect the ends of the hammer beams, but in such cases they are made as inconspicuous as possible.

The construction of an open timber roof is essentially the same as that explained in Chapter III for supporting pitch roofs, the only difference being that the purlins have to be finished, and also the rafters, if they are exposed, and the trusses are made more ornamental.

In fact, the principal interest attaching to an open roof, from a constructive point of view, is in the character of the trusses and the manner in which they are built.

In churches of the cathedral type, that is, with a nave lighted through clerestory windows, the roof of the nave is generally made of rather a low pitch, and is frequently supported by trusses of the king post type, but in which the braces are supplanted by open panels, as shown in Fig. 134. The span of such trusses is usually not more than 30 feet, and by making the cross sections of the tie-beam and principals about one-fourth larger than would be used if

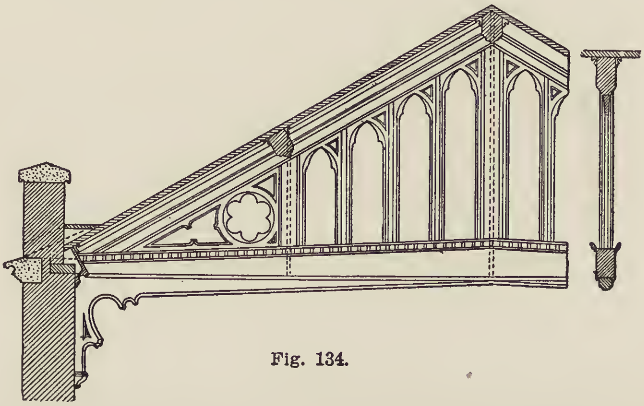

Fig. 134.

the truss was to be built in the ordinary way, they will have sufficient strength without the brace. For spans of more than 30 feet the braces should be put in, cutting through the panels, or a queen post truss with collar beam should be used.

In trusses built as shown in Fig. 134 the author would advise placing concealed rods in the centre of the truss and under the purlin, as these can be tightened after the panel work is in until the parts are made to fit very tightly together.

The brackets against the wall are not necessary for the support of the truss, but they materially stiffen the building when the wind is blowing against either side.

In nearly all other types of churches, especially when finished with an open roof, the side walls are usually comparatively low, so

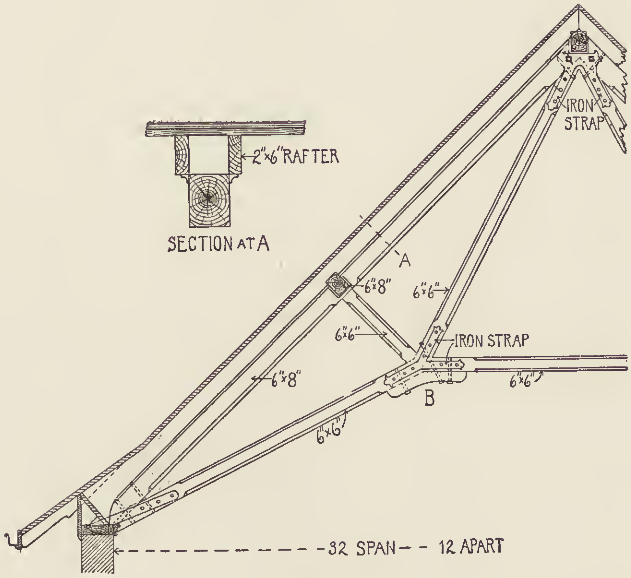

Fig. 135.

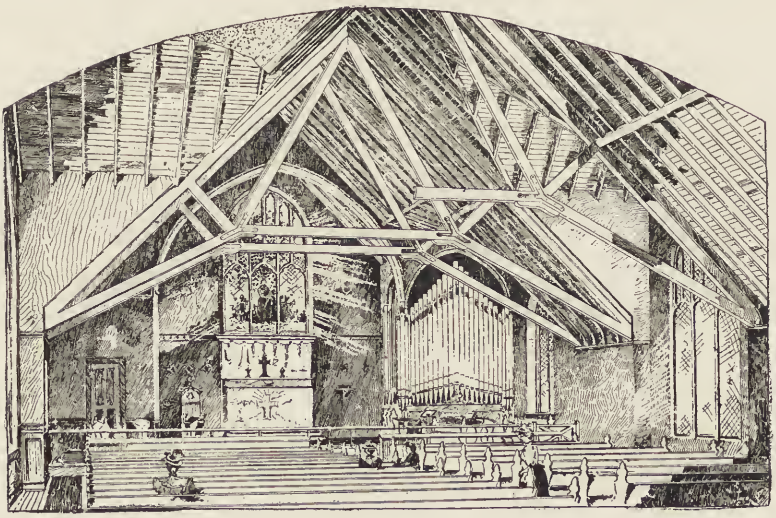

Fig. 136.—Interior View Showing Truss, Fig. 135.

that a horizontal tie-beam would be objectionable, and hence the trusses in such buildings are usually built in some form of the hammer beam or scissors types.*

63. Probably the simplest method of constructing an open timber roof, when the span is from 30 to 35 feet, is that shown in Figs. 135 and 136, which show the roof construction of a small Episcopal church in Denver. In this instance all of the rafters are exposed, and also the under side of the roof boards, which are $1\frac{1}{8}$ inches thick and dressed and beaded on the under side.

The principals are kept 6 inches below the roof boards and a rafter placed against each side of them, as is shown in Section A,

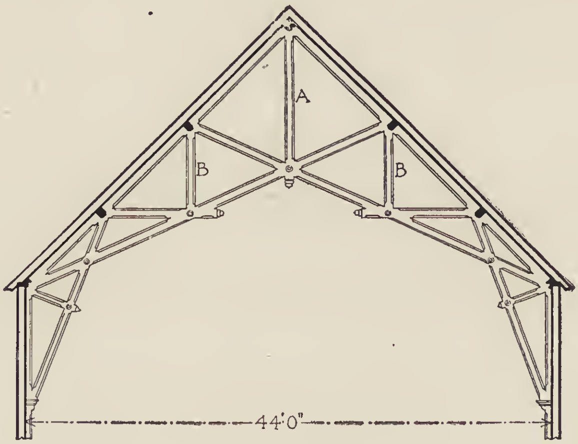

Fig 137.

Fig. 135. This was done to permit of keeping the purlin beneath the rafters. The trusses and purlins are made of solid hard pine, dressed and varnished. The best manner of supporting the ends of the purlins where all of the work is exposed is by means of a Duplex Hanger, as this gives ample strength and makes a neat joint. When the purlins are cased the hangers can be completely covered up

In trusses of this shape the most difficult joint to make is that at B, and the method here shown is the best that the writer has yet seen where the work is exposed.

There is one serious objection to this form of construction from a practical point of view, in that such roofs are very cold in winter and hot in summer, owing to there being but a thin covering be-

*For a description of these types of trusses, see Sections 14, 15 and 21, Chapter I.

tween the room and the outer air. When the roof boards are exposed they should be matched, unless laid in two thicknesses, and building paper or Cabot's "quilt" should be laid under the slate or shingles.

The writer strongly favors ceiling on the under side of the rafters, as shown in Fig. 137, and planting false ribs on the ceiling if a paneled effect is desired.

64. EXPOSED SCISSORS TRUSSES. When the width of the room exceeds 32 feet, and an open timbered roof is desired without

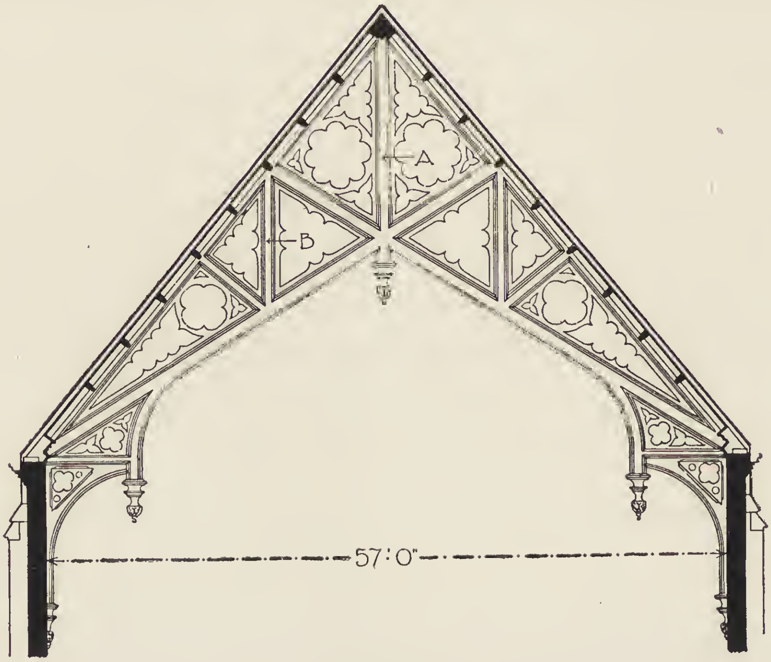

Fig. 138.

going to the expense of a hammer beam truss, an adaptation of the scissors truss may be adopted with a pleasing effect, and without sacrifice of strength. Fig. 137 shows a scissors truss in about its simplest form, and Fig. 138 shows what is practically the same truss more elaborately treated, the constructive members being nearly the same in each truss. Fig. 139* shows a little different treatment of the scissors truss, the main ties in this truss terminating at the foot of the king post. This is a true scissors truss, but it will require

*Interior of Boston Highlands M. E. Church, Walter J. Paine, Architect. From the American Architect and Building News of Nov. 10, 1900.

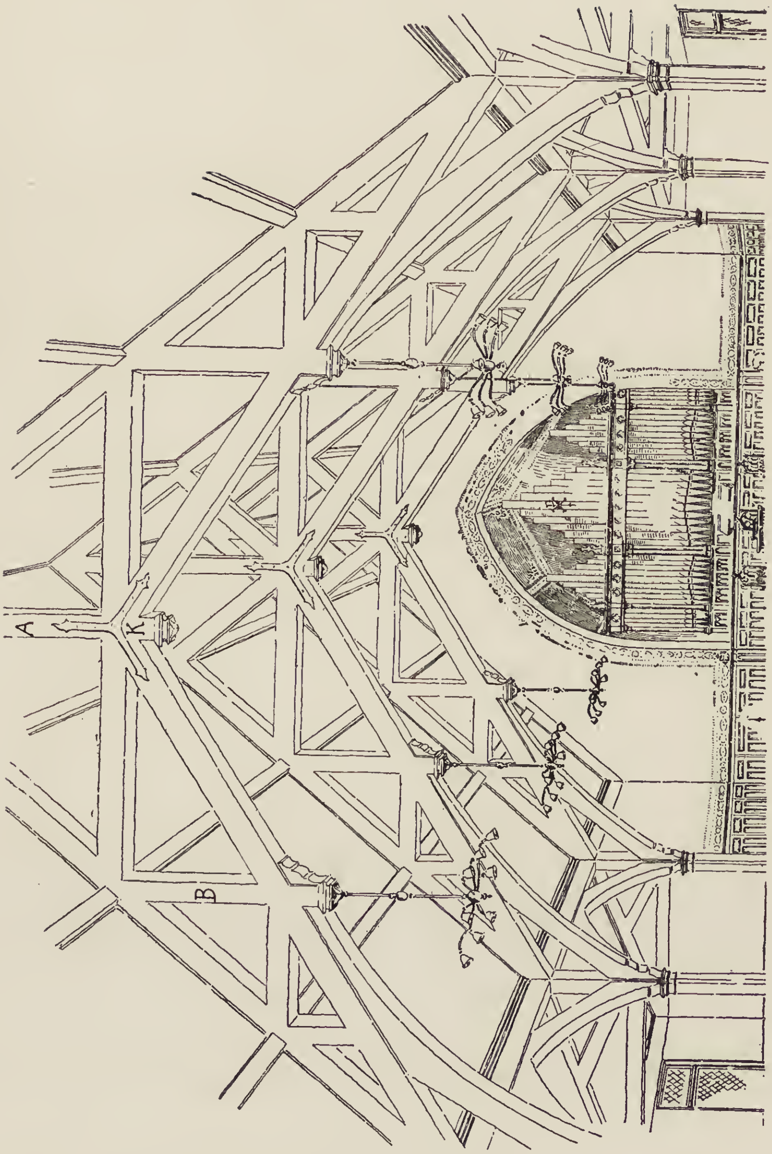

Fig. 139.—Church Interior.

Walter J. Paine, Architect.

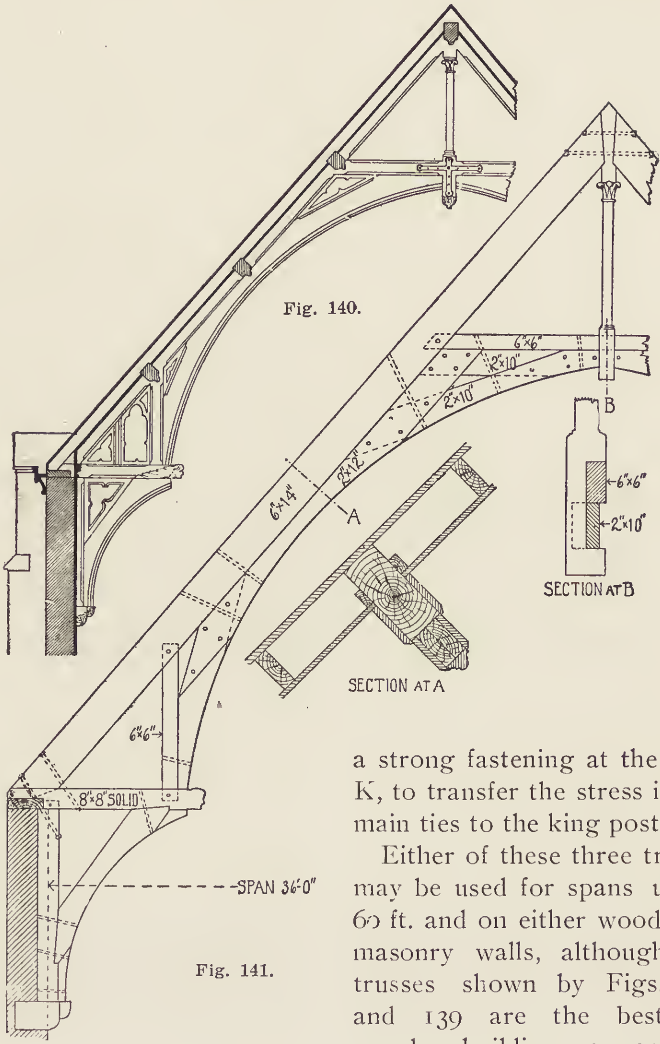

Fig. 140.

Fig. 141.

a strong fastening at the joint K, to transfer the stress in the main ties to the king post.

Either of these three trusses may be used for spans up to 60 ft. and on either wooden or masonry walls, although the trusses shown by Figs. 138 and 139 are the best for wooden buildings on account of the long braces below the truss proper. The pieces A and B should be hollow with a rod inside.

The same principles apply to the designing of these trusses as to the rough trusses, hereinafter described.

65. OPEN TIMBER ROOFS WITH HAMMER BEAM TRUSSES. Fig. 140 shows a half elevation of an ornamental truss

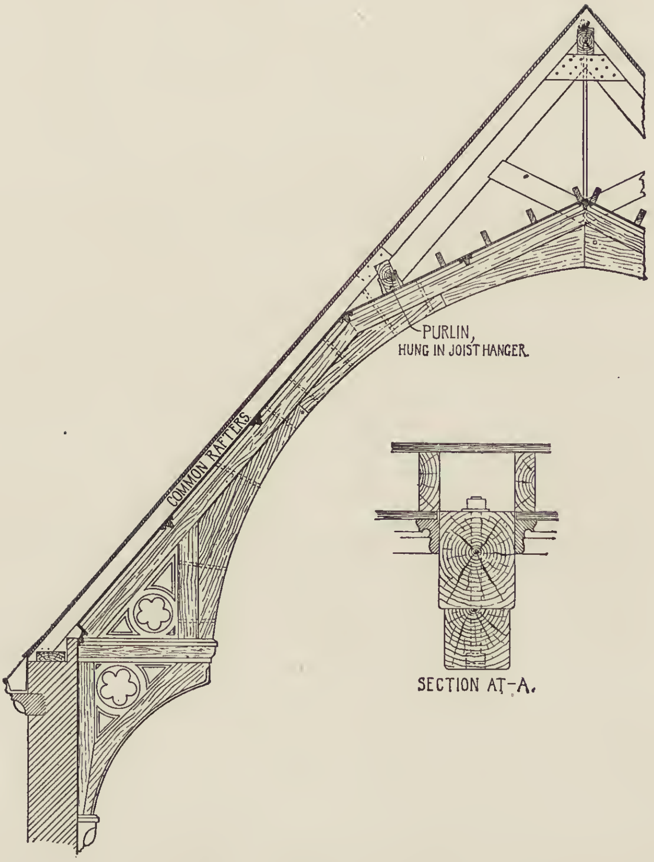

Fig. 142.

that is quite frequently used in churches of moderate width. The truss has the appearance of a hammer-beam truss, but is not a true truss, as it depends upon the transverse strength of the rafter and the resistance of the wall for its stability. Fig. 141 shows the contruction of the truss and the necessary size of the parts for a span of 36 feet and a spacing of 12 to 14 feet. The portion built up of 2-inch plank is made of two thicknesses, one plank being cut the full length shown, while the other is cut between the pieces that cross the long piece. All but the carved hammer beam and the king post are cased with $\frac{3}{4}$-inch finished lumber. The author would not recommend the use of a truss of this type for a greater span than 36 feet, and

Fig. 143.—Interior of Church.

even for such a span it is best to reinforce the wall by buttresses placed opposite the truss.

In the roof shown in Fig. 140 the purlins are placed quite near together, as this distributes the load more evenly on the truss, and also adds to the appearance of the ceiling. If desired, the ceiling can be divided into square panels by putting false ribs, in imitation of rafters, between the purlins.

Neither of the trusses shown in Figs. 135 and 140 are well adapted to a roof that is intersected by transepts of the same width of the nave. If transepts or side gables are desired they should be made narrow, so as to come between two trusses, as in Fig. 136, or a different type of truss should be used.

Another very simple truss for an open-timber roof is shown in Fig. 142, the appearance of the ceiling being shown by the half-tone illustration, Fig. 143, which is taken from a church designed by Mr. C. C. Haight.

This makes a very effective ceiling for a small church, and the enclosed space under the ridge makes the room easier to warm and more comfortable in the summer; it also leaves the room entirely free from timbers, which is desirable in a small church.

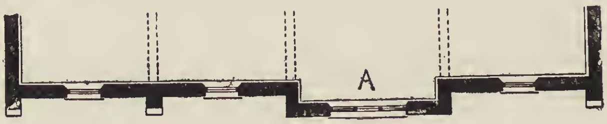

Fig. 144.

In this design the finished ceiling is nailed to the under side of the rafters; or, if preferred, to furring strips, and mouldings are planted on to divide the ceiling into panels, and in the opinion of the author this method gives the more comfortable room and the better accoustic properties.

The truss, however, depends in a great measure upon the trans-

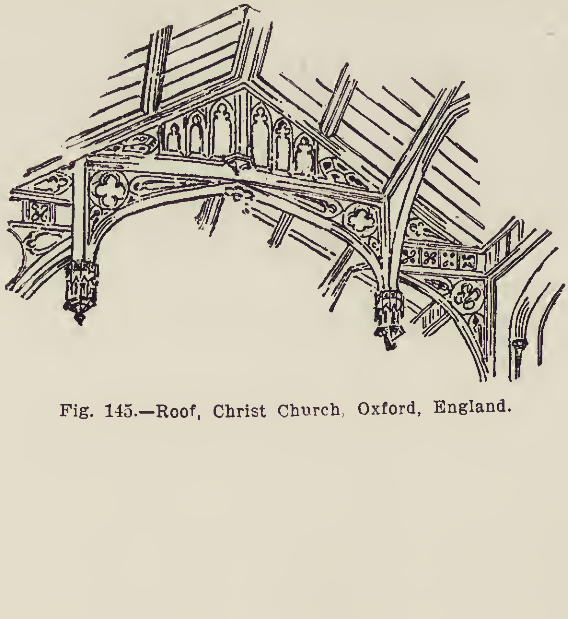

Fig. 145.—Roof, Christ Church, Oxford, England.

Fig. 146.—Roof of Eltham Palace Hall.

verse strength of the principals and the resistance of the walls to keep it from spreading, and is only suitable for spans of 35 feet and under.

When trusses like those shown in Figs. 140 and 142 are used, it is desirable that the walls be stiffened either by buttresses or by offsets.

Breaks in the walls like that shown at A, Fig. 144, add very greatly to the stability of the wall, and permit of using a truss with some horizontal thrust. When they are terminated by a gable, with

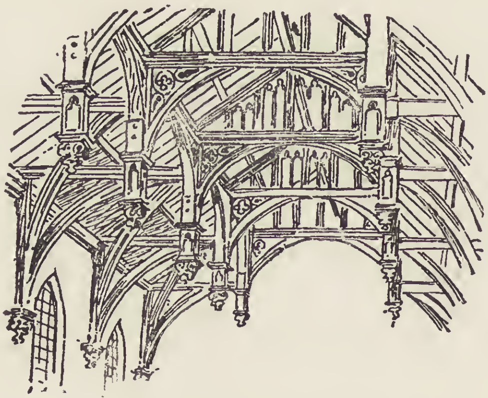

Fig. 147.—Lambeth Palace.

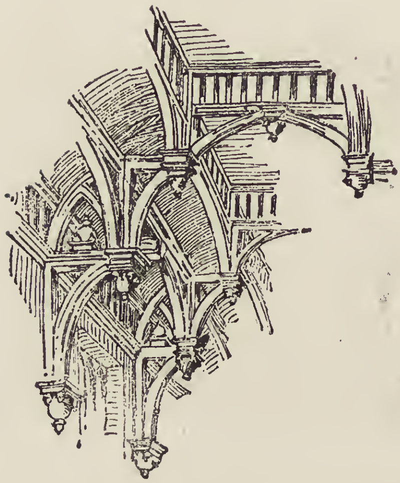

Fig. 148.—Middle Temple Hall.

Fig. 149.--Hampton Court Palace.

Fig. 150.—Church of the Redeemer.

tracery windows, they also make a very effective feature in small churches.

66. When the span of the roof exceeds 35 feet, some form of the hammer-beam truss is generally adopted, the best form being that used in Westminster Hall and shown in Fig. 48.

Elaborate open-timber roofs are quite common in England, the truss being generally of the hammer-beam type, with braced or trussed purlins between the trusses. Several examples of such roofs are shown in Figs. 145-149.

Fig. 150 shows a very handsome open-timber roof on a church

Fig. 151.—St. Stephen's Church, Lynn, Mass.
Messrs Ware & Van Brunt, Architects.

at Paterson, N. J., designed by Messrs. Cady, Berg & See, the truss being of the Westminster Hall type.

When the church has a transept of the same width as the nave, the roof over the square formed by the intersection of the two and designated the "crossing," must be roofed either by diagonal trusses, as shown in Fig. 150, or by trussed valley timbers, as shown in Fig. 151.

For open diagonal trusses the hammer-beam truss, or some truss with a centre rod, seems best adapted.

Very often, however, the valley timbers themselves may be made to support the roof, the walls at the angles being usually capable of resisting any thrust that may come upon them.

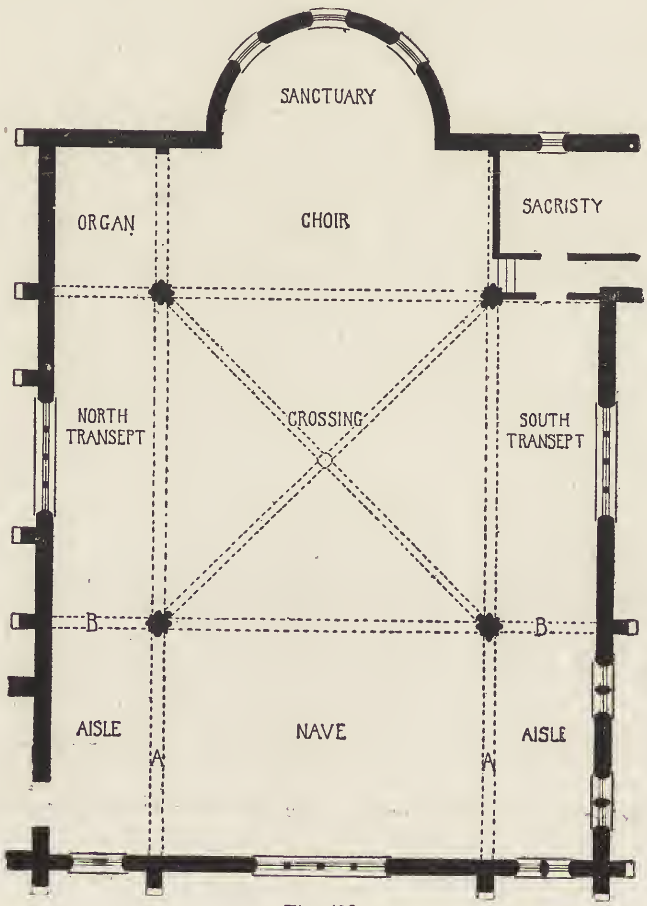

Fig. 152.

Figs. 151 and 152 show a very good and practicable method of roofing a large rectangular church by means of trusses of moderate span, and at the same time giving to the church the appearance of having a nave, transept and aisles. This plan is particularly adapted to Episcopal churches, and gives a pleasing and ecclesiastical interior at moderate cost.

By bolting the ends of the trusses together where they meet on the columns or piers, a sort of chain is formed to receive the thrust of the valley timbers, and the trusses A and B reinforced by the outside walls and buttresses offer additional resistance.

In the church shown in Fig. 151 the choir piers are connected with the main walls by stone arches. While these were built largely for the architectural effect, some rigid connection should always be made between the tops of the columns and the side and end walls.

The arrangement shown by the plan, Fig. 152, is also well adapted to plastered ceilings with concealed trusses.

CHURCH ROOFS WITH SUSPENDED CEILING.

67. Owing to the greater cost of the trusses and finish of open-timbered roofs, and the additional cost of heating the building, most modern churches have a suspended ceiling supported from the tie-beams of the trusses, which conceals all of the constructional features.

The ceiling, however, is generally raised in the centre, and is often domed or vaulted; it may be finished with wood, metal, or plaster, the last material being the most common. Occasionally a portion of the truss is exposed beneath the ceiling, and the latter is paneled to give the appearance of an open-timbered roof, as shown in perspective in Fig. 153 and in section in Fig. 154. Such construction is usually much cheaper than true open-timbered construction, and in small churches nearly as effective, and it has the advantage that the roof can be constructed so as to have no horizontal thrust, thus permitting of lighter walls; or, if necessary, of frame walls, besides making the room easier to heat and ventilate.

68. A very economical and quite pleasing roof construction for a small chapel is shown in Fig. 155. The truss is the regular scissors truss, with only one purlin on each side. Ceiling joists extend across the roof just above the purlins, and a false beam is built on the ceiling, as shown at A. The exposed portions of the truss are cased, and the lower portion of the centre rod is boxed in to give the appearance of a king post. The under side of the rafters and

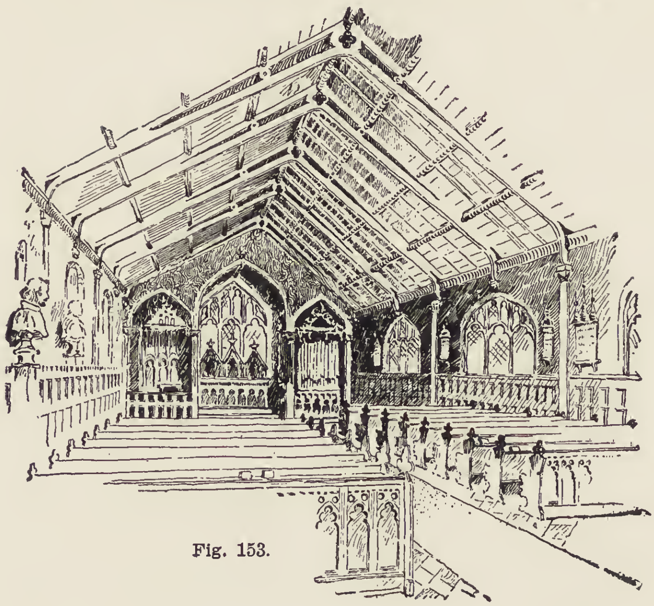

Fig. 153.

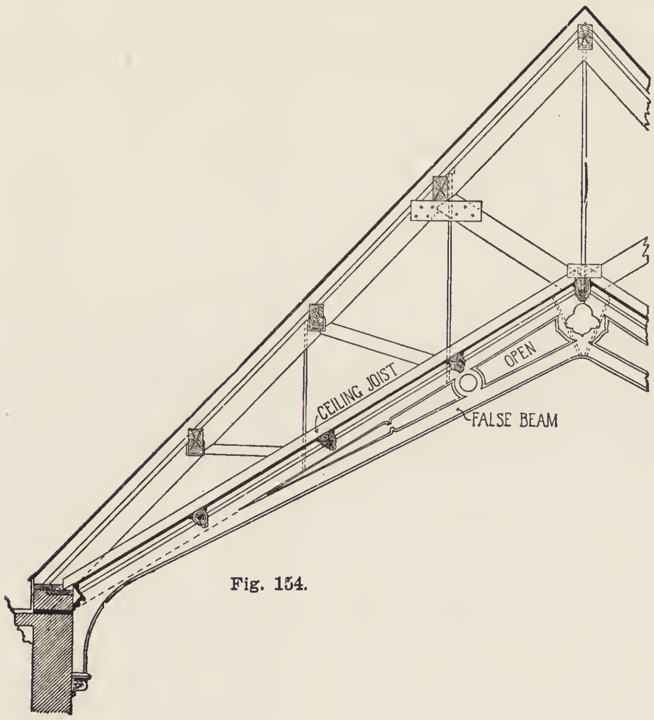

Fig. 154.

ceiling joists is plastered, but it can be finished with ceiling and paneled if desired.

The truss may be built up of four thicknesses of 1-inch boards spiked together. Two boards should extend the full length of the tie-beams laping over each other, and the other two boards cut between. At the intersection of the ties and principals the boards should lap over each other alternately, and be strongly spiked together and a bolt inserted, as shown by dotted lines, to give additional security.

The ends of the purlins are supported by rafters spiked to the side of the principals, by the casings of the principal and strut and

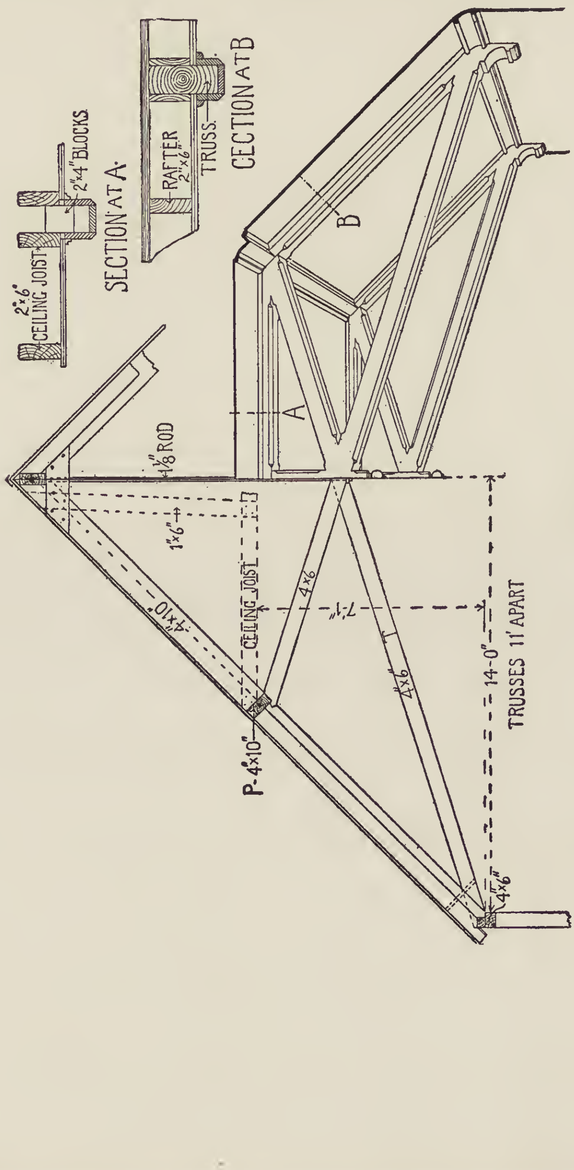

Fig. 155.

also by spikes toe-nailed into the principals. If the roof area to be supported is much greater than that shown, hangers should be used for supporting the purlins, and the size of the truss timbers and rods may need to be increased. The ceiling joists should be supported at their centre by narrow boards nailed to the joists and to the upper end of the rafters.

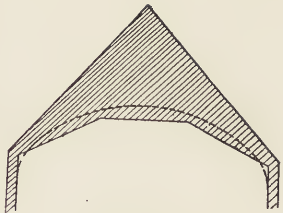

Fig. 156.

With trusses of this shape the building may have transepts of the same width as the nave, the roof over the crossing being supported by diagonal trusses, with a common tie-rod in the centre.

Details for the connection of diagonal trusses are described in section 77.

69. As a rule, where a church has a suspended ceiling and the width of the building is between 32 and 45 feet the shape most commonly adopted for the ceiling is that shown by Fig. 156, as such a

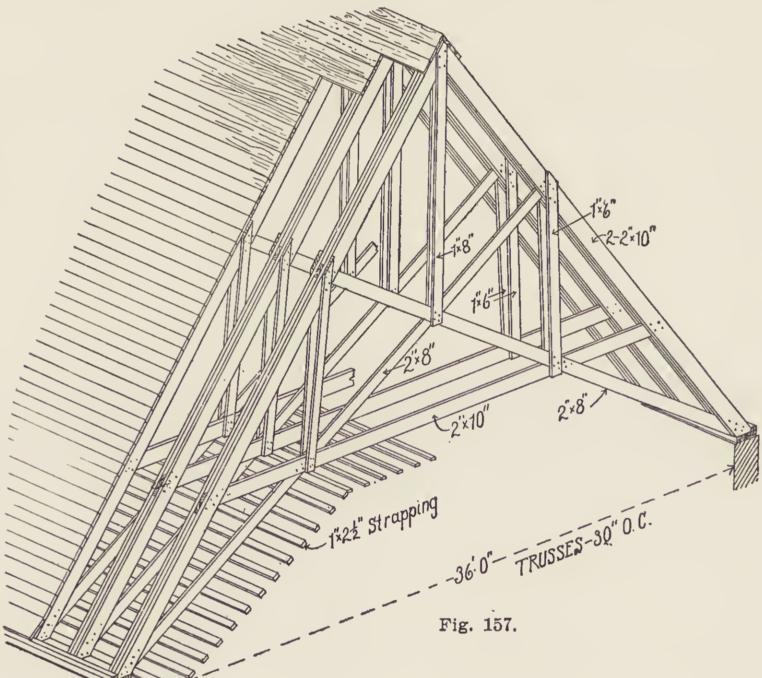

Fig. 157.

ceiling is about the cheapest to construct, and gives the desired height and a fairly good surface for decoration. Occasionally a curved outline is given to the ceiling as shown by the dotted line.

For such roofs the scissors truss is best adapted, as its outlines correspond with those of the roof section. When the span of such a roof does not exceed 35 feet, the roof and ceiling may be economically constructed by spacing the rafters from 28 inches to 32 inches on centres and trussing each pair of rafters as shown in Figs. 157 and 158. [The dimensions given in Fig. 158 are the smallest that should be used for a shingle roof with a span of 34 ft., the trusses being spaced 2 ft., C to C.] By this method no purlins are required and very little ironwork, and the amount of timber in the roof will not much exceed that of an ordinary roof without trusses.

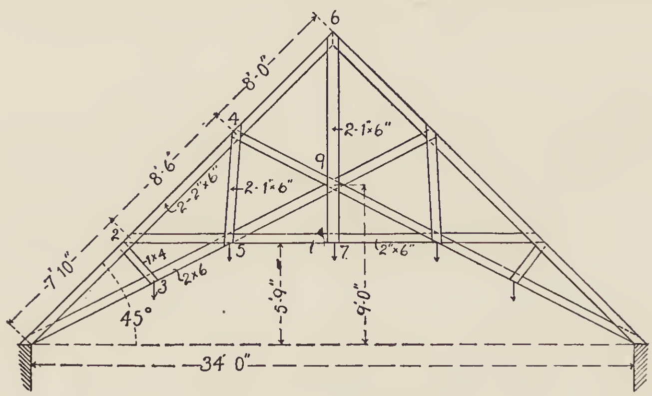

Fig. 158.—Trussed Rafters.

It also has the advantage that the weight is evenly distributed over the walls. If the ceiling is of wood it may be nailed directly to the under side of the trusses; if of plaster the laths are nailed to furring strips or strapping fastened to the under side of the tie and collar beams. The author has adopted this method of trussing in several churches with good results. It is not, however, a desirable method of construction when there are side gables or dormers and requires a rise to the roof of at least 10½ ins. in 12.

In constructing trusses of the scissors type the author has found it best, under ordinary conditions, and with spans of 36 feet and under, to build the tie-beams and rafters out of planks bolted together, giving the tie-beams a section about double that determined by calculation. By this method of construction it is easier to get good connections at the intersections and not as much ironwork is required. It is often difficult, too, to get large timbers that are well seasoned, and unless they are fairly dry the joints will shrink so as to let the truss spread a little.

In one church designed by the author the trusses, which were of the shape and dimensions shown in Fig. 159, were built up of 1-inch boards spiked together, each layer being spiked separately to the next. As the church was built in a country town, the saving over large timbers and iron rods was quite considerable. The only objection to the use of thin stuff is that in seasoning in the building the outer boards are apt to curl and separate from the inner ones, and for this reason it is desirable that a few bolts be used to hold them more securely in place.

Wooden ties, especially in the centre, are also not as desirable as

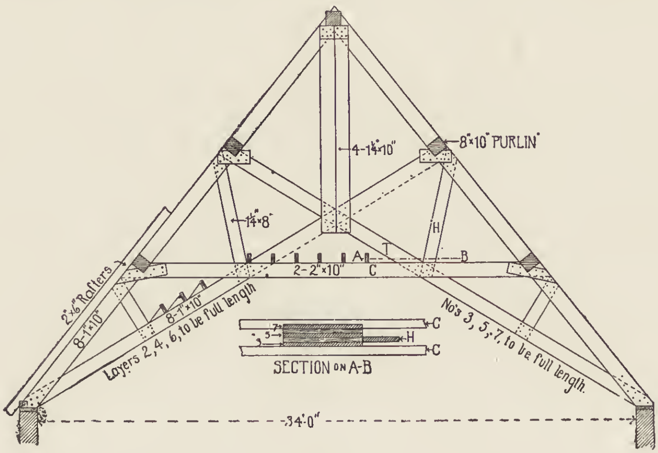

Fig. 159.

rods, as there is no way of tightening them; they are also dangerous in case of fire.

In building up the tie-beams pains should be taken to see that enough long pieces are used to carry the entire tensile stress.

70. MANNER OF SUPPORTING THE CEILING.—In simple roofs, with ceilings formed of plane surfaces, the ceiling joists are generally supported by resting at their ends on the tie-beams of the trusses, as shown in Fig. 159, the joists extending from truss to truss. With such construction the tie-beams usually project beneath the ceiling and are either cased or furred and plastered.

When the span is quite wide, however, so that the strains in the trusses become considerable, thereby necessitating large timbers for the tie-beams, it is more economical and better construction to support the ceiling joists by beams or girders, spanning from truss to truss and corresponding to the purlins in the roof. Such construction is shown in Fig. 160 and in plan in Fig. 162. The ends of the supporting beams *B* should come opposite or close to a joint of the truss (which should be supported by a rod), so that no bending moment will be produced in the tie-beam.

The advantages of this method are that the tie-beams, being relieved of all transverse strain (except that produced by their own weight) need only be made strong enough to resist tension, and the load on the truss is also lightened by one-half of the weight on the outer ceiling joists, as this is carried directly by the wall, while in the other method the entire weight of the ceiling comes on the trusses.

When the truss span is less than 36 feet there will be no saving by this method, as it is always necessary to make wooden tie-beams considerably larger than the calculated size, on account of the cutting for the rods at the joints, and frequently, also, on account of splicing.

The ceiling shown in Figs. 153 and 154 is supported by ceiling beams or purlins, principally to obtain the paneled effect and that the ceiling boards may run at right angles to the trusses.

By dropping the truss beams and purlins beneath the ceiling, as in Fig. 160, the ceiling is divided into rectangular panels, adapting it to a more effective ceiling decoration than is practicable with perfectly plane ceilings.

77. ROOFING THE CROSSING.—When there is a large gable on each side of the main roof, so that the ceiling has the shape of a cross, and the ceiling is suspended, it is usually necessary to support the roof and ceiling over the crossing by diagonal trusses. Fig. 162 shows the framing plan of the roof and ceiling of the church in which the truss shown in Fig. 160 is used. In this church one corner of the crossing is supported by the tower, the other three corners being supported by wooden posts, two of which come in a partition. The four gables are of the same width, the distance between the posts being 37 feet 10 inches. The beams *B B* (Fig. 162) are the ceiling purlins, of which two are shown in Fig. 160. The beams *P* are the roof purlins, which were supported as shown in the same figure, and the three trusses, *C*, *C*, *C*, are of the shape therein shown. All of the ceiling purlins are supported by double stirrups,

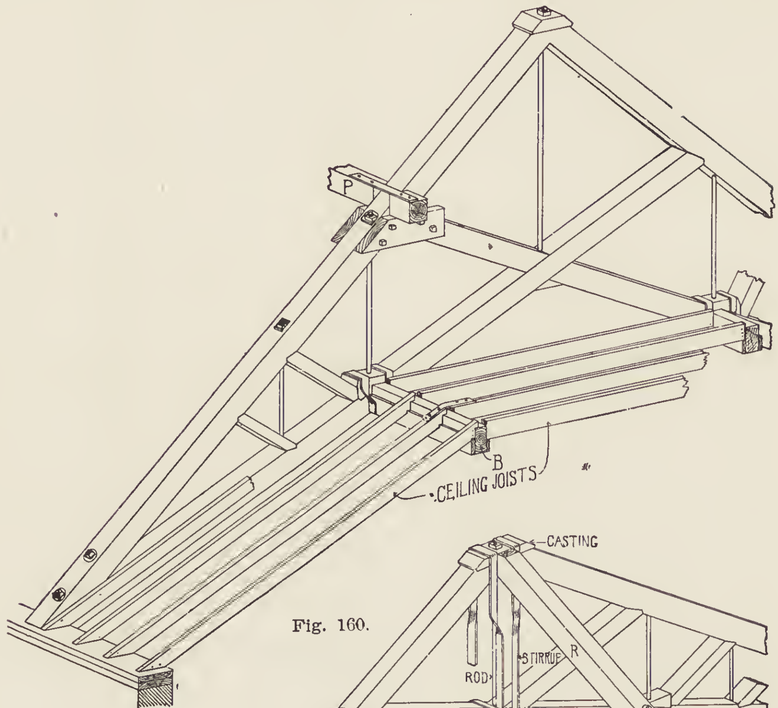

Fig. 160.

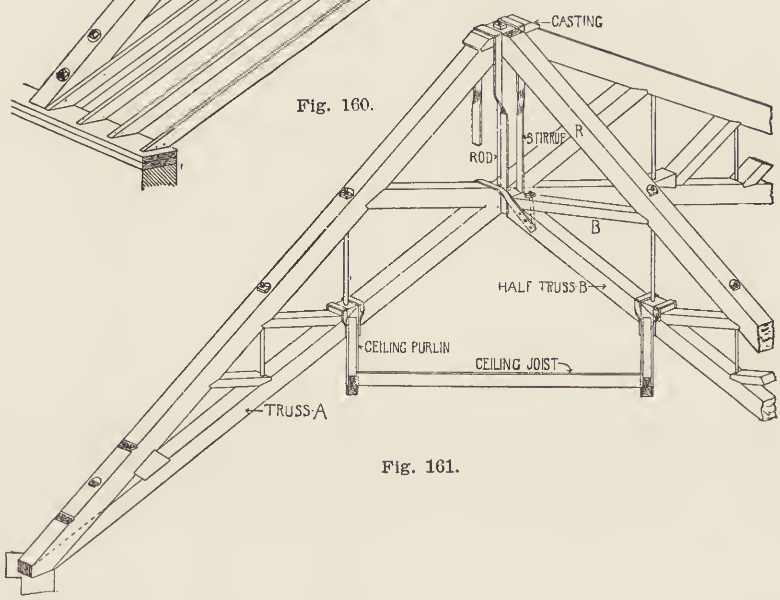

Fig. 161.

which straddle the rods. The truss *A* is a full truss, having the same rise as the trusses *C*, but, of course, a greater span.

The other diagonal is made of half trusses, supported from the centre of the truss *A* by large stirrups, as shown in Fig. 161. These diagonal trusses not only serve to support the inner ends of the ceiling and roof purlins, but they also support the valley ends of the common rafters. The ridges are also supported by purlins, marked *R P* on the plan. These purlins have a span of nearly 19 feet, and hence it was necessary to brace them from the lower joint of the diagonal trusses, as shown by the dotted lines. There is a practical difficulty in using two full diagonal trusses of this shape (with wooden tie-beams), in that it is a very difficult matter to make a sufficiently strong joint where the tie-beams intersect; the strain at this point being very great and it being quite impracticable to connect the parts by iron tie-plates.

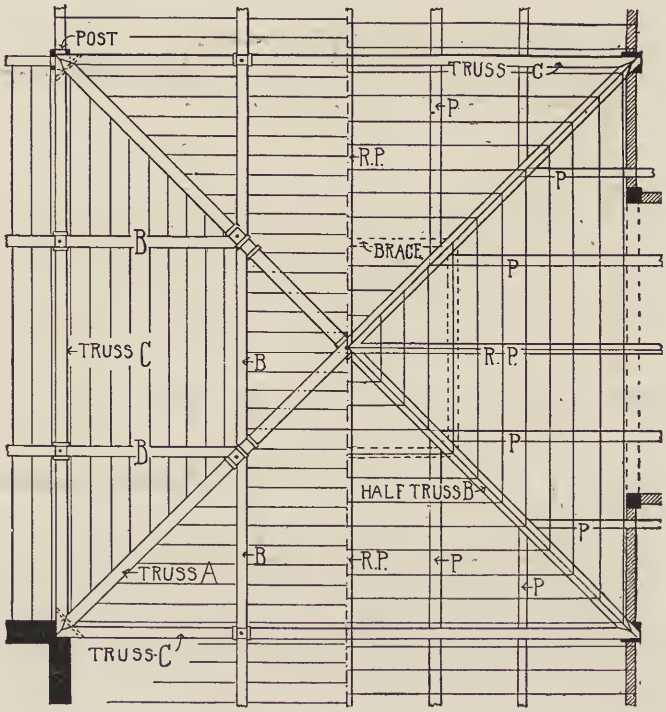

Fig. 162.

It was for this reason that the author adopted the method shown in Fig. 161, of one complete truss and two minor trusses. By extending the main ties of the truss *A* to the rafters a strong joint is made at the centre, and the weight of the half trusses being supported by the stirrup from the top of truss *A*, does not come upon the centre rod. The end of the half truss is prevented from slipping out of the stirrup by iron ties bolted on each side of the half truss. Only one half truss is shown in Fig. 161, but another is carried on the other side of truss *A*. As truss *A* supports the inner ends of the half trusses it must be calculated to support not only the portion of roof and ceiling that is carried directly by it, but also half of the loads supported by the two half trusses. The half trusses, having only half the span, may have much lighter timbers than truss *A*.

Although the trusses *B* have been referred to as "half trusses," each is in reality a full truss, the brace *B* forming one principal and the lower portion of the main rafter the other. The portion *R* of

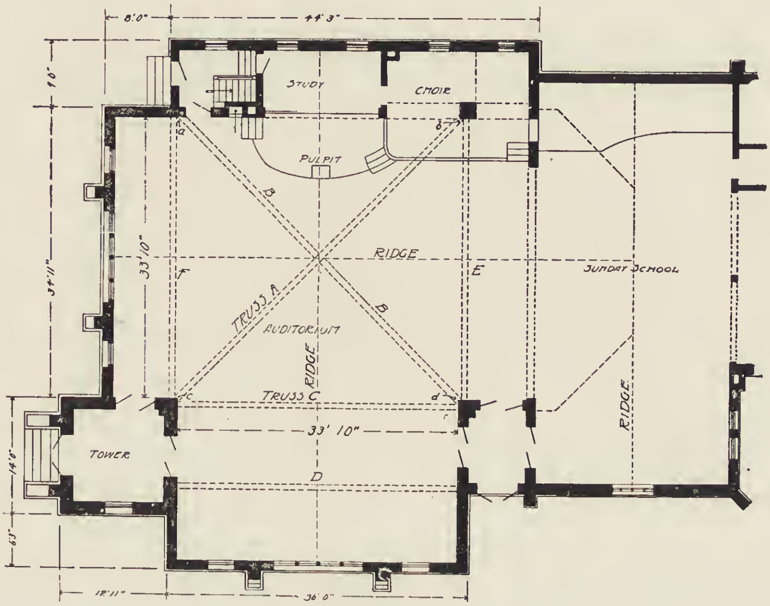

Fig. 163.

the rafter, above the joint, is not a part of the true truss, but merely a beam to support the upper purlin and to brace truss *A* sideways.

In roofing the church shown by the floor plan Fig. 163, the author used the same method of construction as shown by Figs. 160-162, except in the manner of supporting the half diagonals *B B*. The points a, b, c and d form a perfect square with a gable on each side. The ceiling is of lath and plaster, level in the centre and sloping up from the side walls. The span of this roof being a little less than that of the roof shown by Fig. 160, it was practicable to throw the weight of the half trusses on the centre rods of truss *A*.

Instead of using a single rod at the centre of the through truss, two rods were used, one on each side of the truss, and the washer at the bottom extended 4 ins. beyond the sides of the full truss, as shown in Fig. 164 to form brackets for supporting the tie-beams of the half trusses. The tie-beams of the half trusses were also tied together by two bent iron straps, s, spiked to each side of each tie-beam and passing over the tie-beams of the through truss.

The tops of the half trusses were secured to the top of the

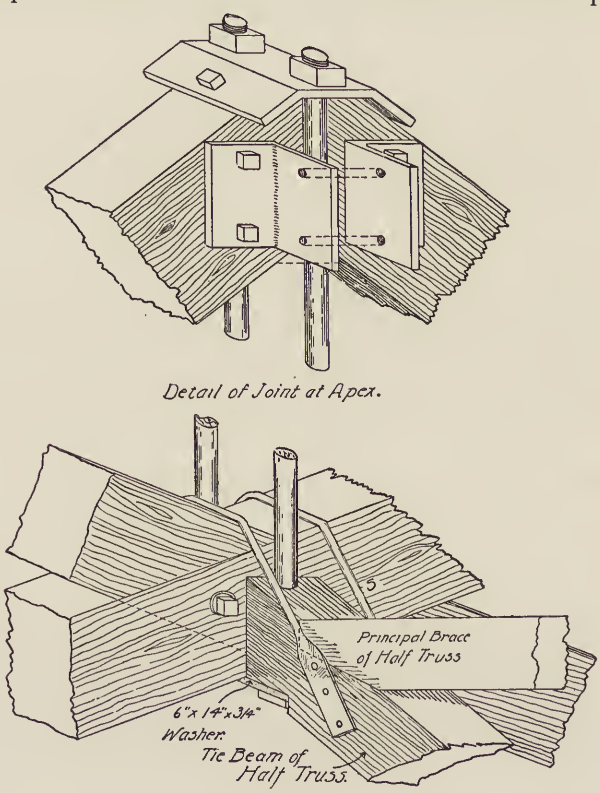

Fig. 164.

through truss by means of four 6″x6″ angles bolted to the principals, as shown in Fig. 164. This construction proved very satisfactory both for strength and facility in erecting. It is also an economical construction.

Fig. 165 shows the dimensions of the full diagonal truss A and Fig. 166 those of the trusses C, D, E, and F.* The framing of the

*Figs. 165 and 166 are reduced from the working drawings.

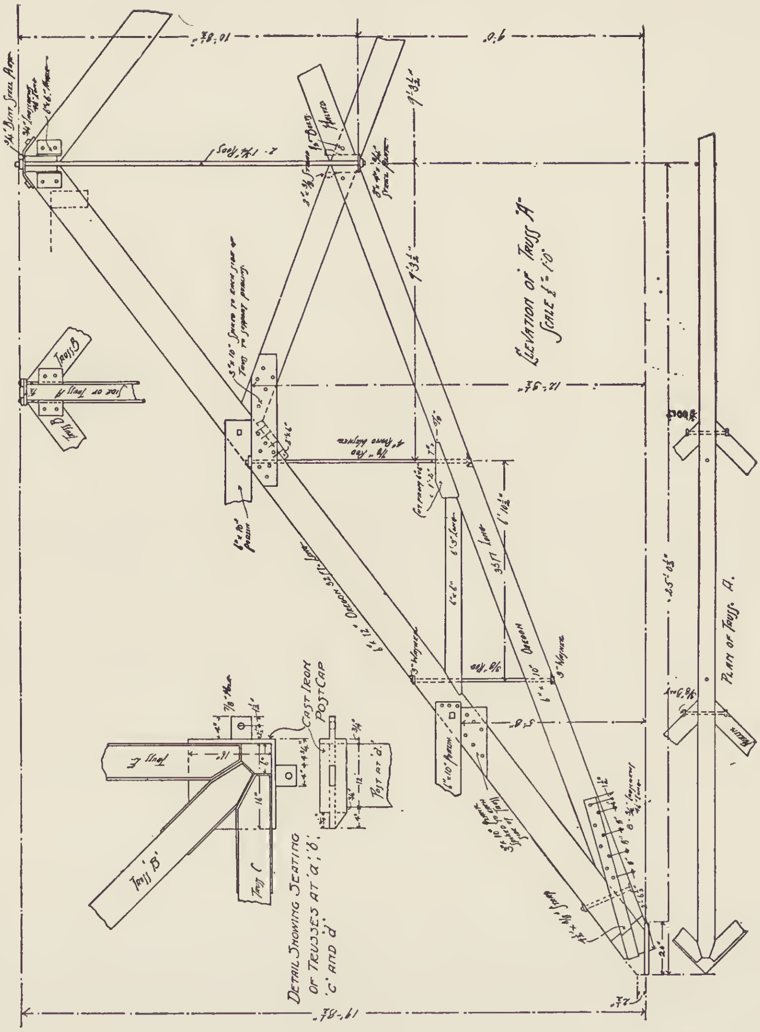

Fig. 165.—Dimensions of Diagonal Truss.

roof and ceiling was similar to that shown by the plan, Fig. 160.

When the four angles of the crossing are formed by solid walls 10 feet long or more from the angle the walls will have sufficient stability to resist any thrust that can come upon them from the diagonals, and in such cases four braced valley beams may be used, abutting against each other at the top without provision in the diagonals themselves for taking up the thrust; but when the roof is supported by posts, as in this case, the diagonals should give no

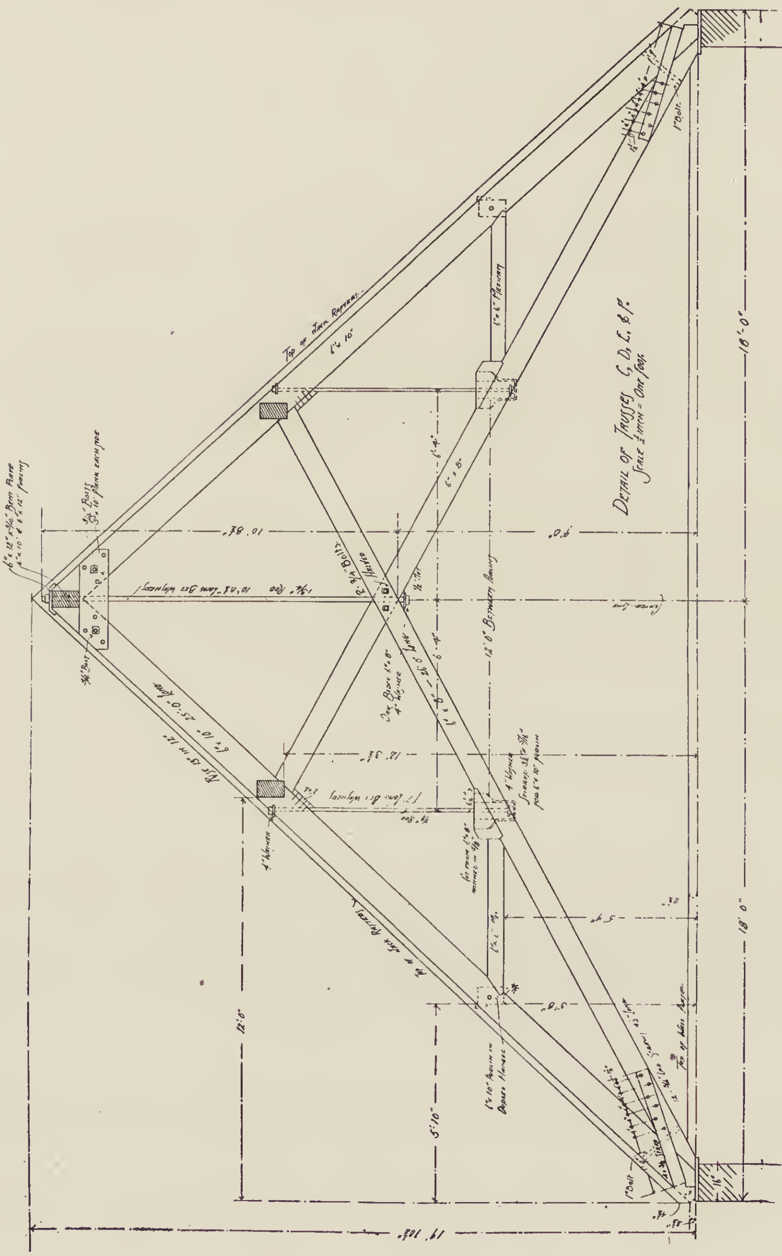

Fig. 1CC.—Dimensions of Scissors Truss of 34' Span.

Fig. 167.

horizontal thrust. A further precaution against spreading in the roof illustrated by Figs. 160-162 was taken by bolting the trusses *C* together over the supports, as shown by the dotted lines in the plan, the bolts also passing through the diagonals.

Where *steel trusses* are used both diagonals may be made exactly alike, as the intersection joint can easily be made in steel by means of plates and rivets. The centre member in such a case should, of course, have a sectional area equal to the sum of that required for the trusses acting separately.

72. LEVEL CEILINGS.—When there is a gallery in the church, thus necessitating high walls, a sufficiently high ceiling is usually afforded by using trusses with horizontal tie-beams, with the ceil-

ing joists resting on top of the beams and spanning from truss to truss. By placing a cove at the juncture of the walls and ceiling the apparent height of the room is increased and the appearance and also the acoustic properties improved.

It is usually very much cheaper to support the roof and ceiling in this way, because simple trusses may be used, generally without very large timbers or rods, and very little finishing lumber is required.

As illustrating the method of supporting a roof and level ceiling

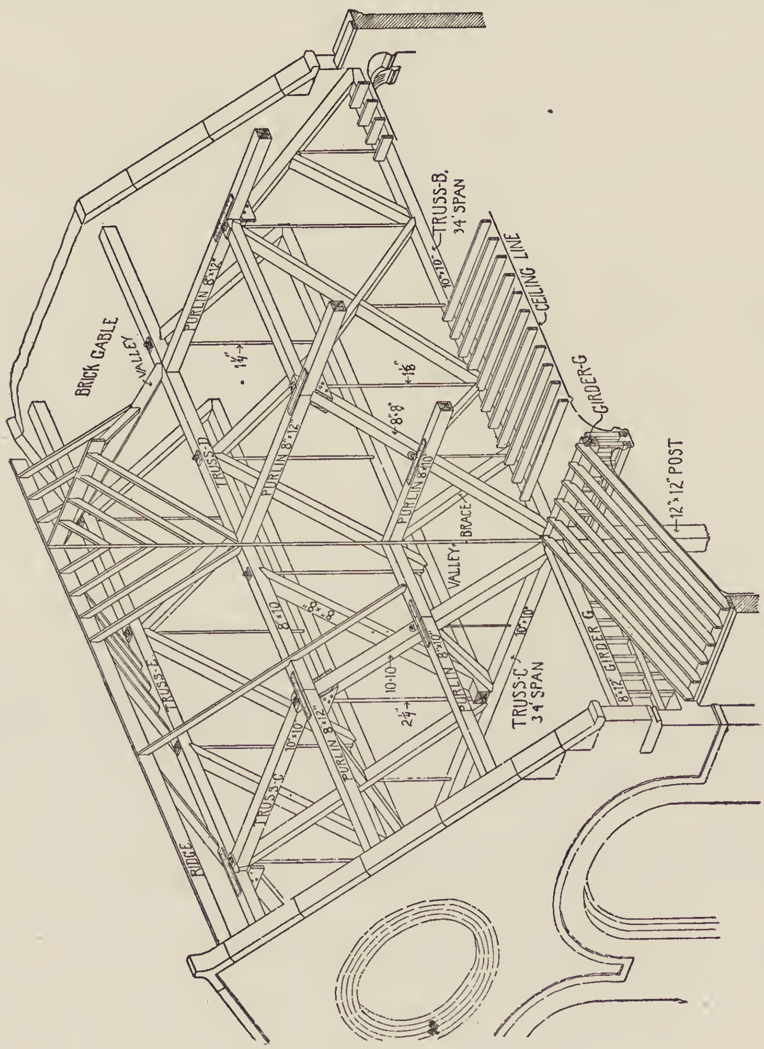

Fig. 168.

over a crossing with four gables, Fig. 168 may prove of interest. This and the plan, Fig. 169, also show how a cruciform roof and exterior may be placed over a rectanglar building or room, the plan being a portion of that of the church shown in Fig. 167, while Fig. 168 shows the actual construction of the central portion of the roof. The building being built on a narrow lot and it being desired to utilize the entire ground space (in width), it was necessary to plan the building in the shape of a simple rectangle, only very shallow recesses being permitted on the principal side and none on the other.

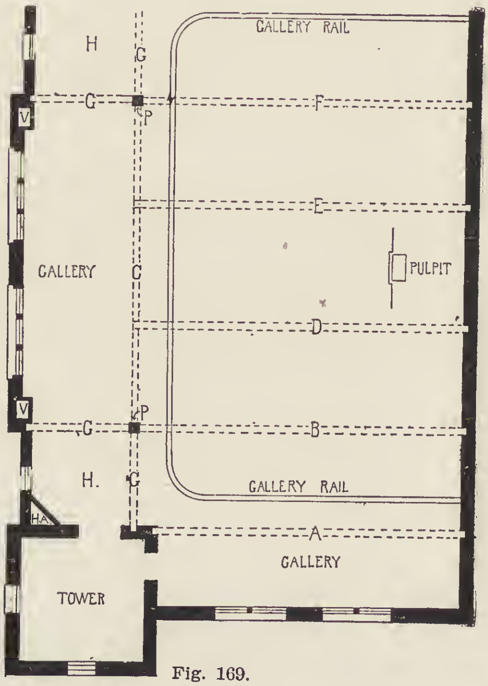

Fig. 169.

By means of two posts, which were also utilized in supporting the gallery, the width of the main roof was reduced to 36 feet, and by placing shed roofs with the cornice slightly depressed over the spaces *H*, *H*, the appearance shown in the half-tone illustration was obtained. The central portion of the roof is supported by three trusses, *B*, *C*, *F*, of exactly the same outline, and by two Howe trusses, *D* and *E*, which are supported at one end by the wall and at the other by truss *C*, the members of the latter truss being made larger than those of trusses *B* and *F* on account of the additional loads, although only 10x10-inch timbers were required. The tie-beams of trusses *E* and *D* were framed flush with the ceiling joists to form a square panel in the centre. The top of these trusses come on a level and in line with the upper purlins, so as to receive the common and valley rafters. The inner end of the purlins extending from trusses *B* and *F* to trusses *D* and *E* are supported from the tops of the latter by stirrups. The inner ends of the lower

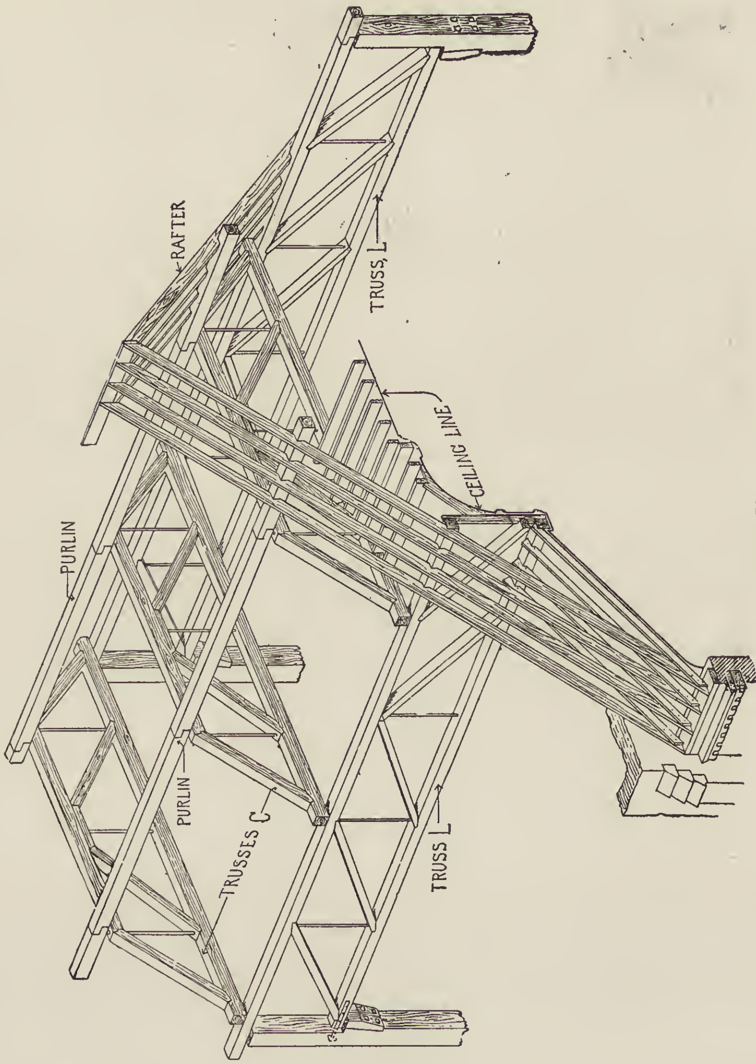

Fig. 170.

purlins, which support the valley, are supported by braces from the tie-beams of the trusses. The tops of the two posts have heavy cast-iron caps, with a flat top. In any roof construction the purlins should be well tied together endways and securely anchored to the walls.

In the church above illustrated the pulpit was placed where shown on the plan, and, although the room is rather long for the

Fig. 171.

width, the acoustic properties. both for speaking and singing, appear to be perfect.

73. ROOFING WITH LONGITUDINAL TRUSSES.—When the room to be covered is rectangular in plan and more than 40 feet wide, and the side gables, if any, are comparatively narrow, the roof and ceiling can often be best supported by means of two Howe trusses, placed longitudinally of the roof and supporting smaller transverse trusses if necessary. The author has used this method of construction several times, and with economical and satisfactory results.

To illustrate this method of construction, Fig. 170 has been drawn from the working drawings of the church shown in perspective and plan by Figs. 171 and 172.

The interior of the audience room is shown by Fig. 173. The ceiling is of lath and plaster, and is divided into panels by false beams, as shown in the drawing. The general shape of the ceiling is also shown by the heavy line in Fig. 170. The width of the audience room between walls is 56 ft. 8 ins. and the length from the inner angle of the tower to the post *P*, either side of the pulpit opening, 51 ft. 8 ins.; the width of the gable recesses at the sides is 17 feet.

As shown by Fig. 170, the roof and ceiling are supported primarily by two Howe trusses placed as indicated by the dotted lines on the

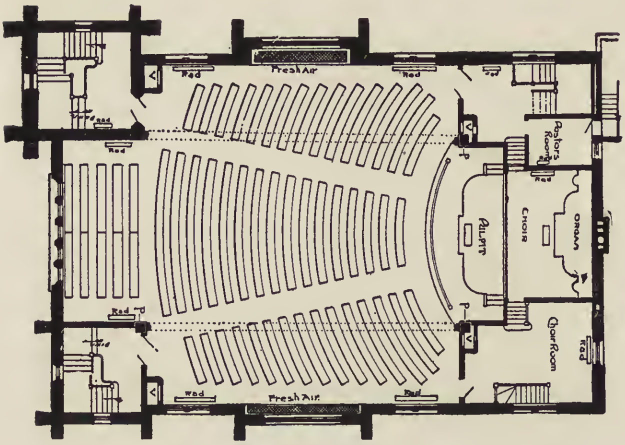

Fig. 172.

plan. The top chords of these trusses serve as one of the purlins on each side of the roof to support the rafters and the lower chords support the outer ceiling joists. To support the ceiling joists and rafters over the space between the Howe trusses, four smaller trusses, C, were placed across the space with their ends resting on the top chords of the longitudinal trusses. The tie-beams of these transverse trusses support the ceiling joists and the top chords support another set of purlins. The tie-beams of the transverse trusses drop below the ceiling joists, and are cased to correspond with the false beams used in dividing the ceiling. From an inspection of Fig. 170, it will be seen that about four-fifths of the entire roof and ceiling are supported by the two Howe trusses, L L, and these again are supported by three wooden posts (marked P on the plan), and by the inner angle of the tower, hence but very little weight comes on the walls.

The advantages of this system of roofing are: First, the system gives a greater clear height for the audience room than can be obtained with transverse trusses with the same height of walls.

[In the example illustrated the bottom of the ceiling joists of the centre ceiling is 16 ft. 4 ins. above the wall plate.]

This large space above the wall plate adds both to the apparent size of the audience room and to the comfort of the audience.

Second. The shape of ceiling naturally adapted to this system of trussing is appropriate and fitting to the plan.

Third. By this system *no thrust is exerted on the outside walls* and but little weight, while the rigidity and weight of the centre roof and

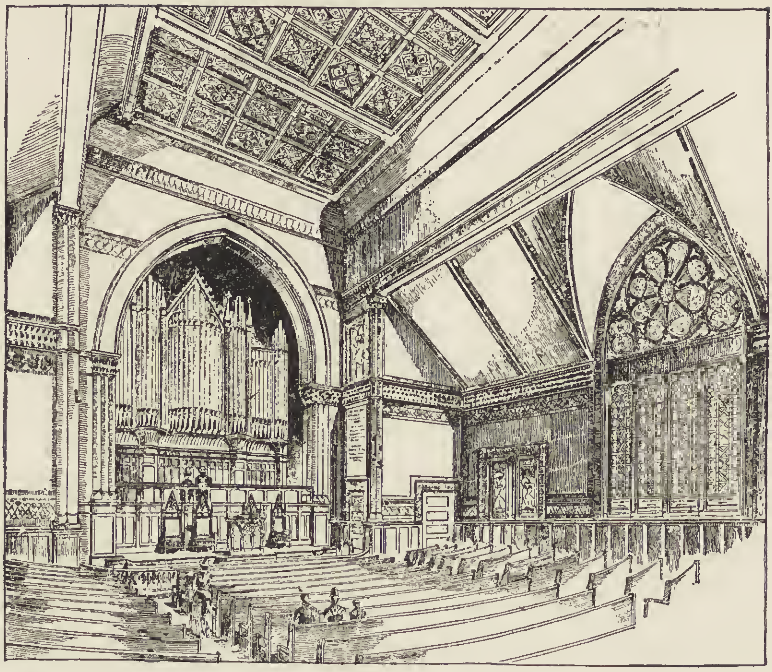

Fig. 173.

ceiling really tend to stiffen the walls, and to prevent the building from racking sideways.

Fourth, *economy*. Trusses with horizontal chords are the simplest and easiest to construct, and where they are adapted to the shape of the roof, require the least amount of material. And not only are the trusses themselves such as any carpenter can easily construct, but the general construction is easily erected and can be put up before the side walls are completed, thus advancing the completion of the building.

As soon as the transverse trusses are in place, the centre ceiling joists may be set, and thus a convenient permanent stage is provided for the use of workmen while completing the roof. This system is as well adapted to a wooden as to a brick church, and can be used over rooms up to 100 ft. in depth and 80 ft. in width, provided that the pitch of the roof is 45 degrees or more.

74. In laying out such a roof construction the points to be studied are to space the longitudinal trusses so that they may have sufficient *depth*, and that they will not give too long a span to the lower raft-

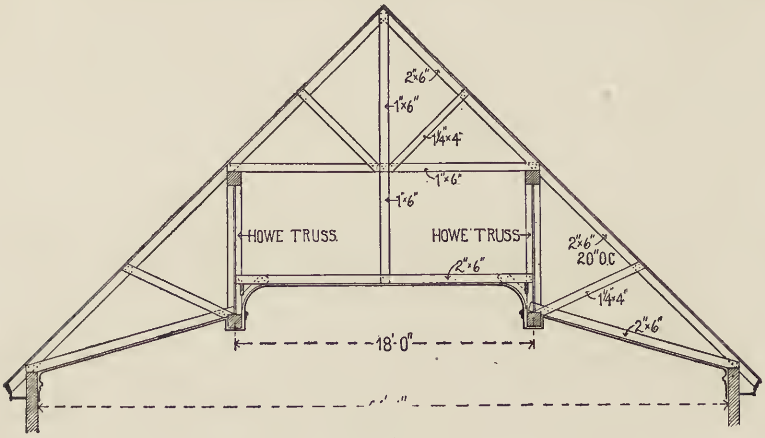

Fig. 174.

ers; also that the ceiling may be divided in pleasing proportions. As stated in Section 10, the height of a Howe truss, measured from the centres of the chords, should be one-sixth of the span when the space will permit, and never less than one-eighth for moderate spans. It is not necessary that the trusses shall be of the same span, but they should be of the same height.

The number of panels in the longitudinal trusses should be arranged so that the secondary trusses will come over the upper end of a strut. The centre panels of the Howe trusses should also be *counter braced*, to provide for any inequality in the loads. The posts which support the main trusses should extend to the basement floor, resting upon iron plates set on piers of masonry. If the basement has a concrete floor the iron plates should be an inch or more above the concrete, so that the ends of the posts will not rot. The posts should also be braced in at least two directions. Occasionally the walls of the church and tower may be utilized for supporting the trusses, but in a long building it is generally more economical to use posts, as was done in the building described, as in that way the span of the trusses can be materially shortened. From the posts to the end walls the roof and ceiling may be supported by beams, braced from the post and wall, or by short trusses.

In the construction illustrated, the longitudinal trusses were studded up on the inside to receive the lathing for the plaster frieze and the ends of the cove furring. Only one piece of this studding is shown. The lower lengths of rafters were also braced from the bottom chords of the longitudinal trusses by 1x6 boards, which are not shown in the drawing.

Fig. 174 shows the application of this system of trussing to a smaller church, where the transverse trusses are not required. A building 42 feet wide and 64 feet long can be roofed in this way with only two trusses.

When the ceiling beams are raised above the lower chord, as in Fig. 174, they should be supported on "ledger boards" or planks, bolted to the struts and to occasional uprights, so that they will have no tendency to push out the bottom of the trusses.

In one church designed by the author, but which, being built at a distance, he did not superintend, the contractor supported the centre ceiling beams by simply spiking them to the tops of the curved ribs, and without extending them to the trusses, as indicated by the drawing. The result was that the weight of the ceiling coming on the curved ribs caused them to spread the trusses and thus to push out the side walls. The nuts on the vertical rods in the Howe trusses should be at the top, where they can be got at, or better still, turn buckles may be placed near the centre of the rods, so that if the trusses "sag" from shrinkage, they may be raised by shortening the rods.

The application of longitudinal Howe trusses to the support of roofs with vaulted ceilings is described in Chapter V.

CHAPTER V.

VAULTED AND DOMED CEILINGS; OCTAGONAL AND DOMED ROOFS.

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75. VAULTED CEILINGS UNDER STEEP-PITCHED ROOFS.—It is often desirable to place a vaulted ceiling above the nave or auditorium of a church or hall, while maintaining a simple pitch roof on the outside. This may be accomplished in several ways; the most desirable construction for any particular building depending largely upon the width of the room, the character of the supports and whether the ceiling is to be a simple or groined vault.

When the width of the room to be roofed does not exceed 36 feet, and the walls are of masonry the roof and ceiling may be supported by trusses of either the hammer beam or scissors type.

Fig. 175 shows a section of the vaulted ceiling and the truss sup-

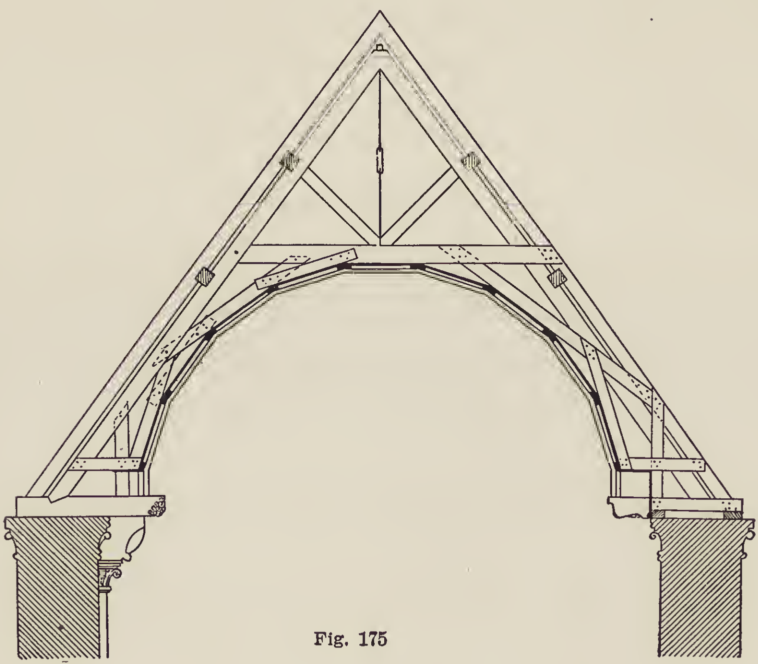


Fig. 175

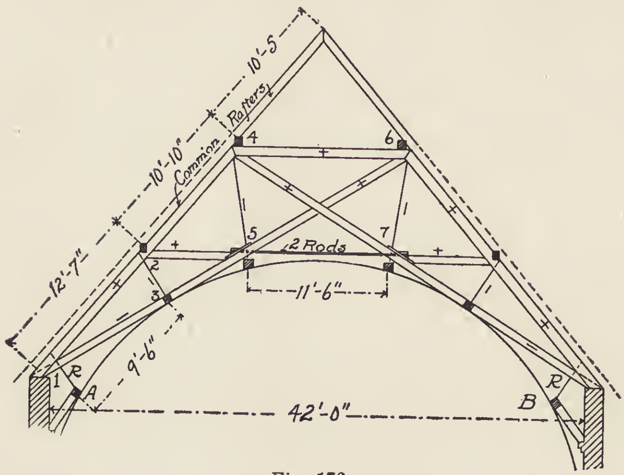


Fig. 176.

porting it as contemplated by Mr. H. H. Richardson in his design for the Cathedral of All Saints, Albany, N. Y. In this case the short hammer beam is supported by an engaged column and corbel instead of a bracket. The ceiling also terminates in a series of corbels and arches which throw it out from the wall line. The span of the truss is about 32 feet. Between the trusses the ceiling was to be supported by cradling spiked to collar beams and to the rafters, as shown in the right-hand side of the figure, the duty of the trusses being principally to support the rafters.

This form of construction was frequently used in mediæval buildings. It requires very heavy walls, and large timbers in the trusses to keep the roof from spreading.

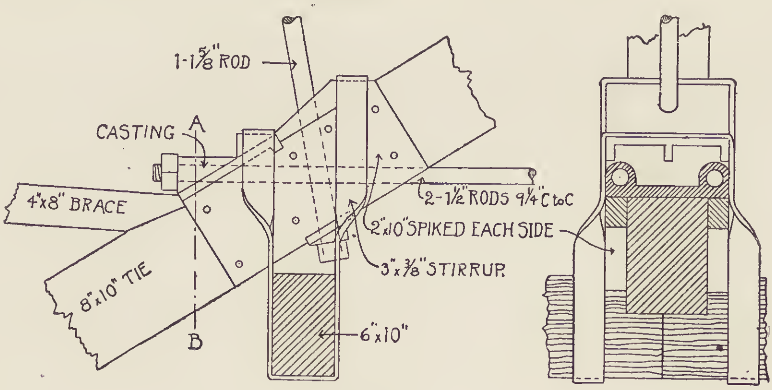


Fig. 177.

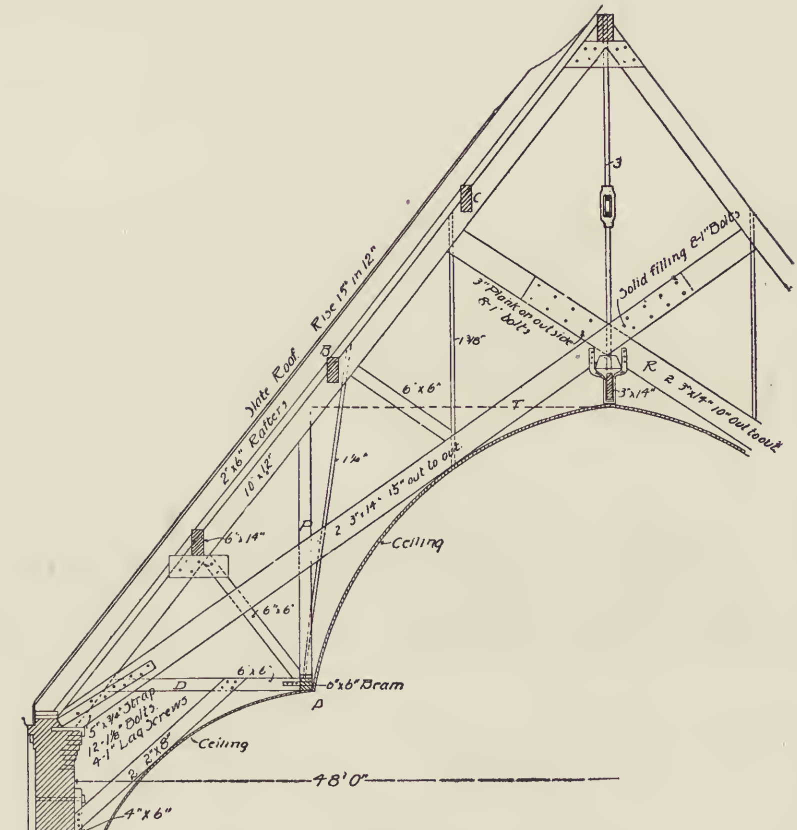


Fig. 178.

For light walls, it would be better and safer to use ornamental tie-beams in place of the hammer beams, and an ornamental king post, as in the roof of Trinity Church, Boston [see Fig. 182].

A flat vaulted ceiling, with a contour about as indicated by the dotted line in Fig. 156, can readily be suspended from scissors trusses, but such trusses cannot well be used for semi-circular vaults, unless the top of the wall is considerably above the springing of the vault.

76. Fig. 176 shows a scissors truss designed by the author for supporting a ceiling that was very nearly semi-circular in section, and Fig. 177 shows a detail of the joints at 5 and 7. The ceiling was formed by curved ribs cut from 2-inch plank and spiked to the

longitudinal beams shown by the etched sections in Fig. 176. The beams at *A* and *B* are held in place by the rods *R R*.

Fig. 178 shows the construction of a double vaulted ceiling over a Catholic church* that is quite interesting. The shape of the ceiling is roughly shown by Fig. 179.

Fig. 179.—Showing Vaulted Ceiling.

Although the ceiling is quite elaborate, the manner of supporting the roof and ceiling is really quite simple. Fig. 178 shows one of the supporting trusses, which are ordinary scissors trusses. The 6″x6″ beam at *A*, extends the full length of the church and is suspended from the top of the truss near the purlin *B*, by a $1\frac{1}{4}''$ rod. The beam is also rigidly held in place by means of the horizontal beam *D*, and by a 6x6 brace. The cross beam *D* is tied to the longitudinal beam by a bolt strap passing through the latter. The longitudinal beam at *A* supports the bottom of the diagonal ribs of the upper vault and the top of the ribs of the lower vault. The double lines at *P* indicate the studding which forms the end of the upper cross vaults, *P*, *P*, Fig. 179. This roof could probably have been more economically constructed, by using longitudinal Howe

*Designed by Andrew Roth, Architect.

trusses at P, letting the tie-beam answer for the longitudinal beam at A, and supporting the centre of the roof by means of king rod trusses, resting on the Howe trusses.

77. LONGITUDINAL TRUSSES FOR VAULTED CEILINGS.—When the

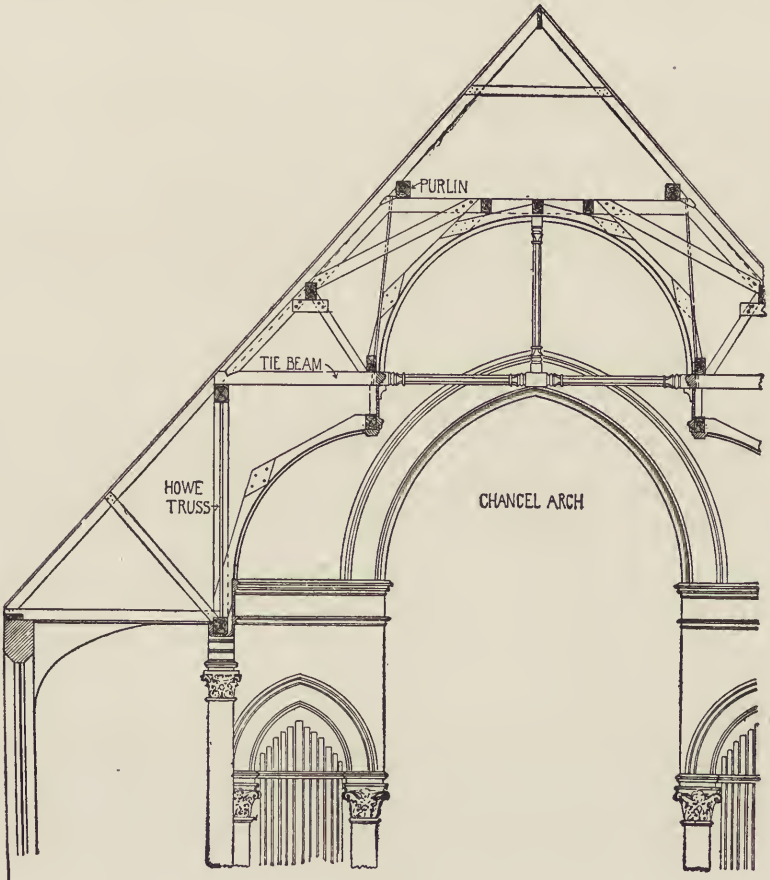


Fig. 180.

ceiling is in the shape of a barrel vault, the system of longitudinal trussing will often be the only one that can be used. Fig. 180 shows a transverse section through a church roof and ceiling, where this system is the only one that could have been employed. The section is approximately that of the roof and ceiling of the Emanuel Baptist Church, Brooklyn, N. Y., Francis H. Kimball, architect, which has a very handsome vaulted ceiling, with groined vaults over the side windows, the shape of the ceiling giving the appearance of nave and aisles, although there are no posts except one under each longitudinal truss near the front.

In this case portions of the transverse trusses show below the ceiling, being treated in an ornamental manner and giving an appearance of strength to the vaulting.

This arrangement could also be adapted to a vaulted ceiling, without the outer portions shown, the side walls taking the place of the Howe trusses, while the latter are placed at each side of the central vault.

Fig. 181 shows a section through the roof and ceiling of the Temple Emanuel, Denver, which are supported on the same principle as the roof shown in Fig. 180, the principal difference between the two being in the form of the ceiling and of the trusses between the longitudinal trusses. In Fig. 181 the blackened sections *C*, *C*, are the chords of two Howe trusses, each having a clear span of 64 ft., a total height of 11 ft. 6 ins. The distance between the trusses, centre to centre, is 35 ft. 6 ins. The central portion of the roof and ceiling is supported by scissors trusses, four feet on centres, which rest on the top chords of the Howe trusses.

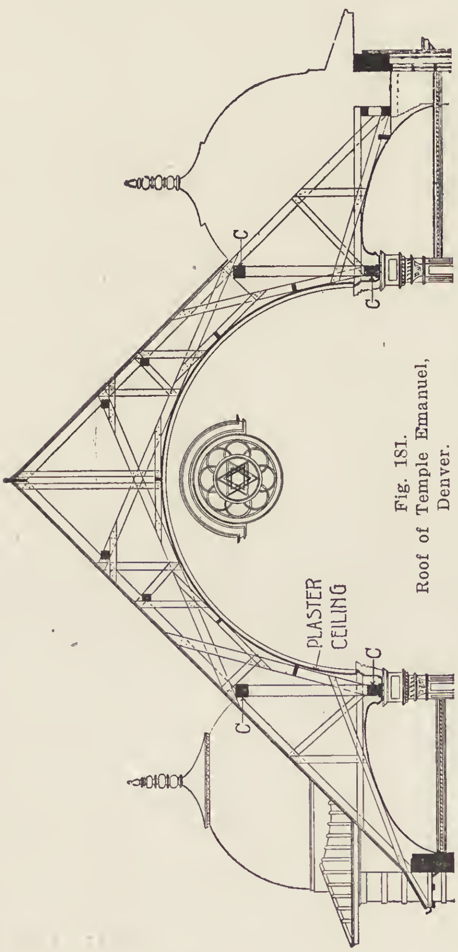


Fig. 181. Roof of Temple Emanuel, Denver.

The scissors trusses support the cradeling for the barrel vault which extends the full length of the audience room.

78. CEILINGS WITH CROSS OR GROINED VAULTS. — When the ceiling consists of intersecting vaults the longitudinal

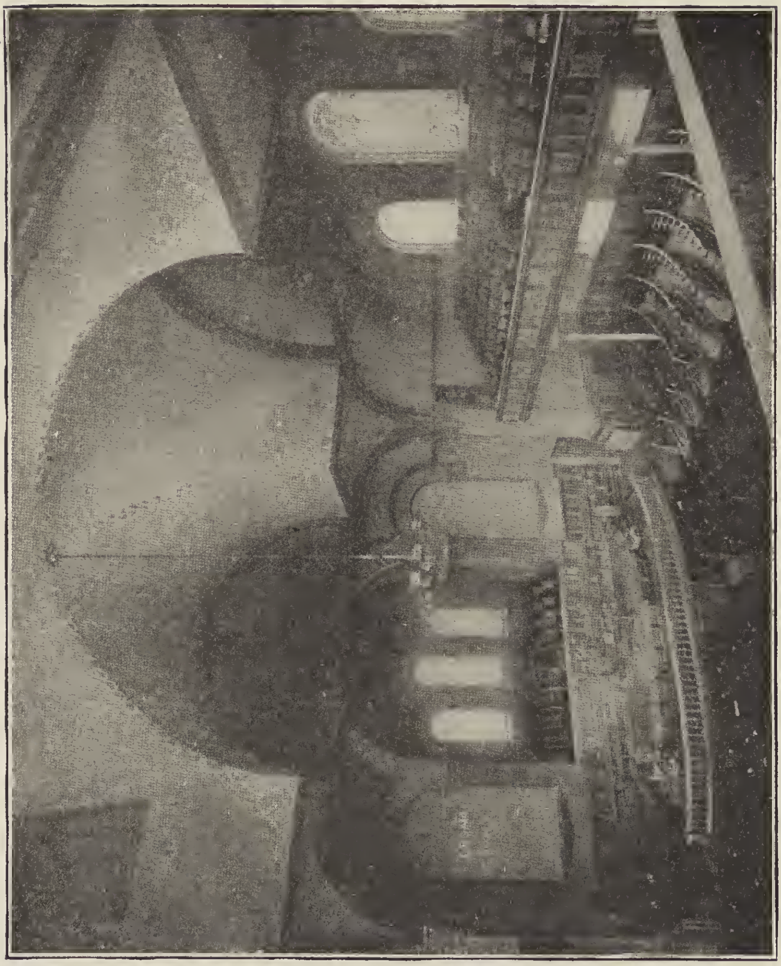


Fig. 182.—Interior, Asbury M. E. Church, Denver.

system of trussing cannot, of course, be used, as the trusses would cut through the side vaults.

A ceiling like that shown in Fig. 182 is occasionally used in Romanesque churches, and the manner in which the roof and ceiling of this church are supported may be of interest.

The audience room is formed on the plan of a Greek cross, the arms being of the same width and of nearly equal length. The two front corners of the crossing are formed by large wooden posts, furred and plastered, being located so that they do not obstruct the view from any of the seats.

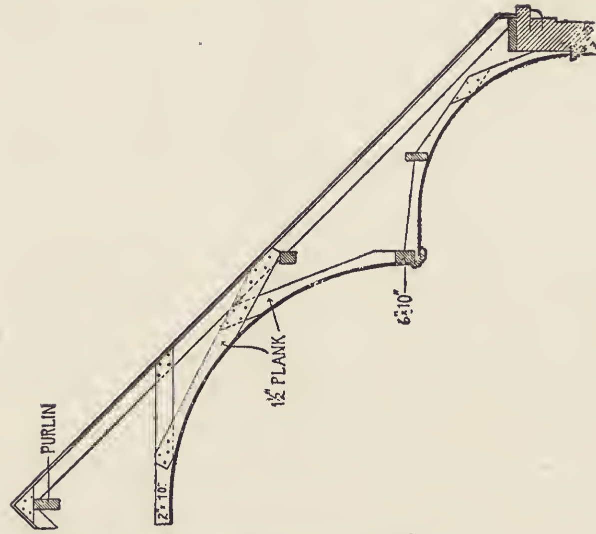


Fig. 183.

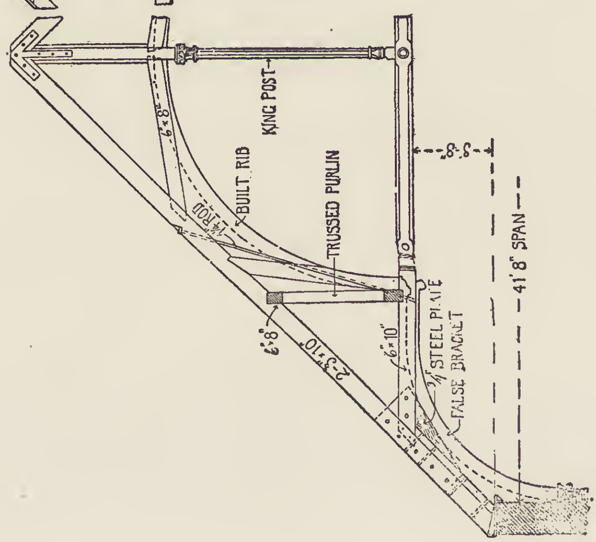


Fig. 184.

As the length of the arms is only about 20 feet, the roof and ceiling are supported entirely by four double trusses, one on each side of the crossing, the purlins from these trusses to the gable walls being trussed.

Fig. 183 shows a section through the roof and ceiling, and Fig. 184 a detail of one of the trusses. The truss is a combination of the king-post and queen rod truss, the king-post being used principally to give the idea of a truss, as all the other members, except a portion of the tie-beam, are concealed. Circular ribs were placed between the tie-beam and king-post, projecting about 6 inches below the ceiling, but these were used merely to further the appearance of a truss, although they add slightly to the lateral stiffness. But one purlin on each side of the roof was used, as the upper length of the rafters is braced by the collar beams, which were necessary to support the furring for the central vault.

An unusual feature of this truss, and as a rule an undesirable one, is

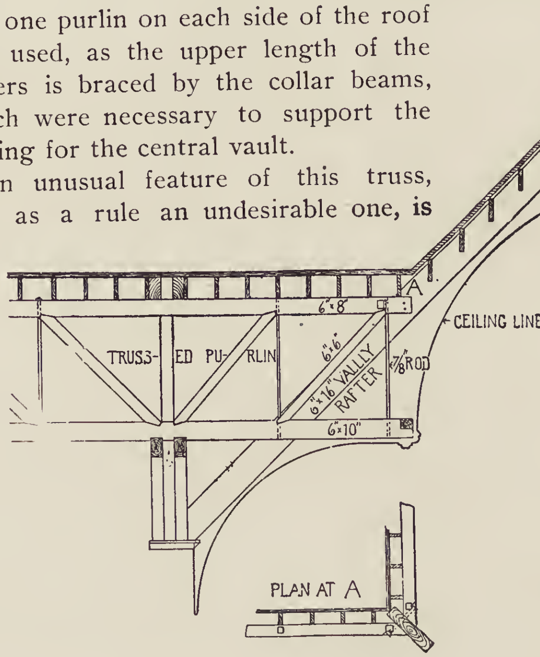


Fig. 185.

that the tie-beam is raised 3 feet 8 inches above the bottom of the principals, so that the portion of the principals below the tie-beam has to support the full weight of the truss. This brings a transverse strain at the juncture of the tie-beam and principal, and to resist it a steel plate was bolted in the centre of each truss as shown.

In this particular instance there was very little opportunity for the principals to spread at the bottom, as there is 20 feet of wall on each side which must be moved before the iron shoes can slide outwards, and as long as they cannot move no transverse strain can come upon the principals. The steel plates were inserted, however, as an extra precaution. Of course the walls could have been carried up to the tie-beam, but this would have added considerably

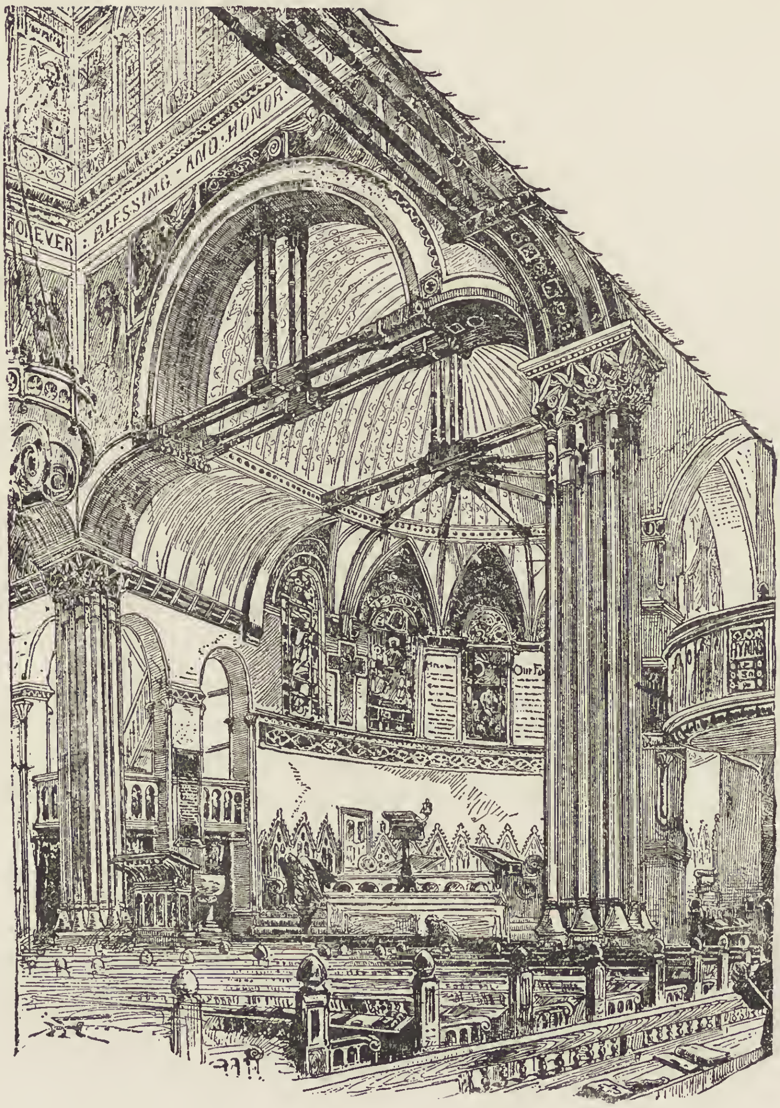


Fig. 186.—Interior of Trinity Church, Boston.

to the expense of the building, as they are of stone, and would also have detracted from the external appearance. Two trusses like that shown were placed side by side and 6 inches apart. The crossing is supported by heavy valley timbers, and by extending the trussed purlins, to take the pendentive angles of the vaulting.

Fig. 185 shows a detail of the inner end of the trussed purlins, one coming in from each side of the crossing, and each pair being bolted together and to the valley timber at the top.

The ribs of the vault, at the intersection, are supported at their lower end by a built up diagonal rib, corresponding to the hip rafter of a roof, and this is supported by the lower chords of the trussed purlins, which are hung by $\frac{7}{8}$-inch rods from the top, the purlin being trussed to act as a cantilever.

This method of supporting the crossing is often applicable when the ceiling has a different shape from that shown.

The shape of the ceiling shown in Fig. 182 is very similar to that in Trinity Church, Boston, but in Trinity Church the crossing is covered by a tower with a flat paneled ceiling.

The author does not know how the roof and ceiling of Trinity Church are supported, but from the appearance of the building he judges that the trussing must be similar to that shown in Fig. 184, except that the crossing is covered by the tower.

79. STEEL TRUSSES FOR VAULTED CEILINGS.—Within the past few years quite a number of churches have been roofed with steel trusses, and a few have been built with steel supports extending to the foundation.

Figs. 187, 188 and 189* show, in part, the steel construction of St. Jerome's Roman Catholic Church, New York, Messrs. Dehli & Howard, architects, Mr. Bernt Berger, structural engineer. This church is a brick and stone building 98x150 ft. with steel columns, framework and roof. "The nave, which is 36 ft. wide and 120 ft. long, terminates in an apse and has a vaulted ceiling 60 ft. above the floor.

On each side of the nave there is a 20-ft. aisle with side walls about 32 ft. high and nearly flat roofs, with a gallery floor 22 feet above the main floor. About 76 feet from the front wall of the church is the axis of the transept, which is about 39 ft. wide and has a vaulted ceiling corresponding to that in the nave, and a low dome on pendentives, over the intersection.

The clerestory walls, about 25 feet above the aisle roof, are supported on double longitudinal I-beam girders attached to the main columns, and the walls of the tower above the first story are also supported by the steel framework. The main roof has six riveted trusses of the quadrangular type spanning the nave roof and four spanning the transept. The trusses are all similar to that shown by

*From the "Engineering Record" of Dec. 14, 1901.

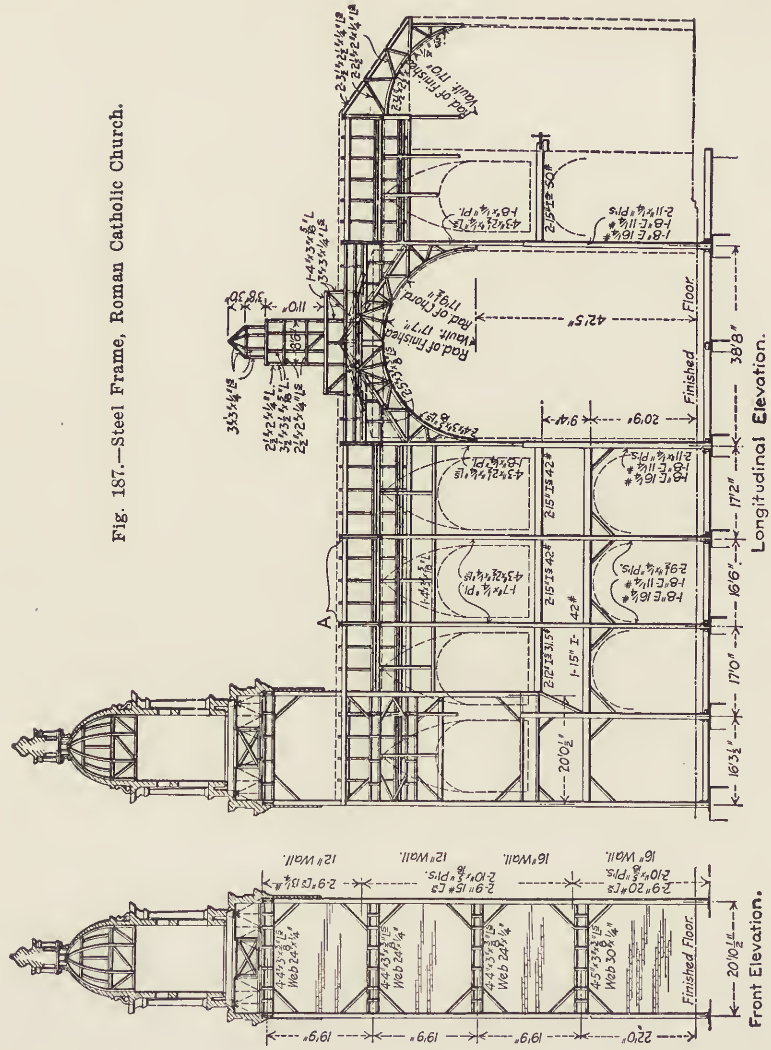


Fig. 187.—Steel Frame, Roman Catholic Church.

Fig. 188 and are of ordinary riveted construction, with T-shaped members made of pairs of angles riveted together, back to back, on both sides of $\frac{3}{8}''$ gusset connection plates at panel points. The vertical member in the centre of the truss is made double to allow central connections for the ridge purlin and the middle longitudinal ceiling girder. The trusses are connected longitudinally by three lines of I-beam purlins and two lines of eaves girders, which latter are made of a channel and angle riveted together. The purlins sup-

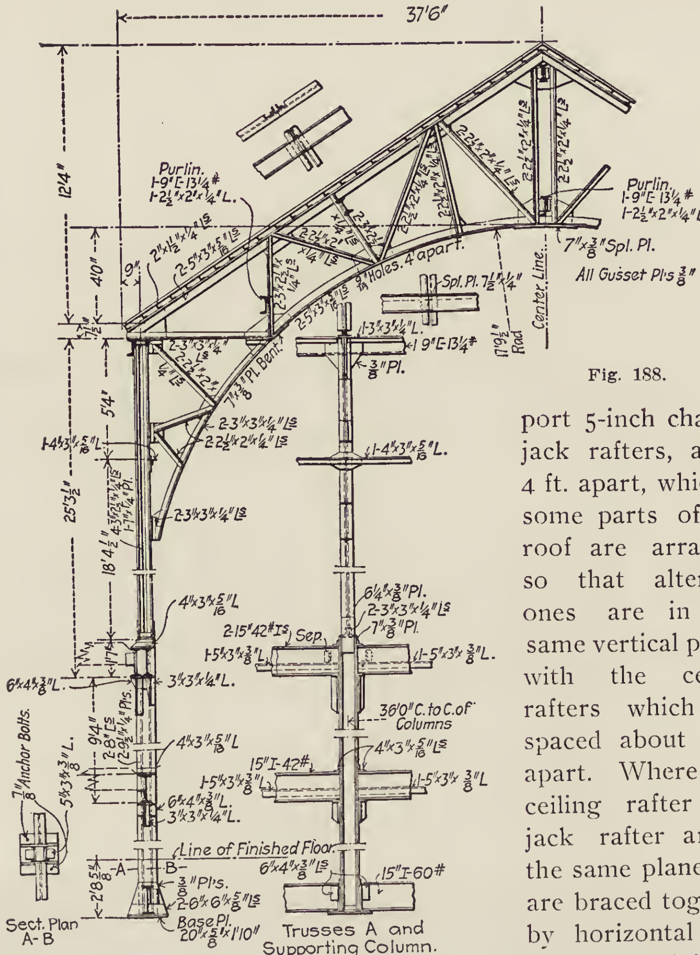


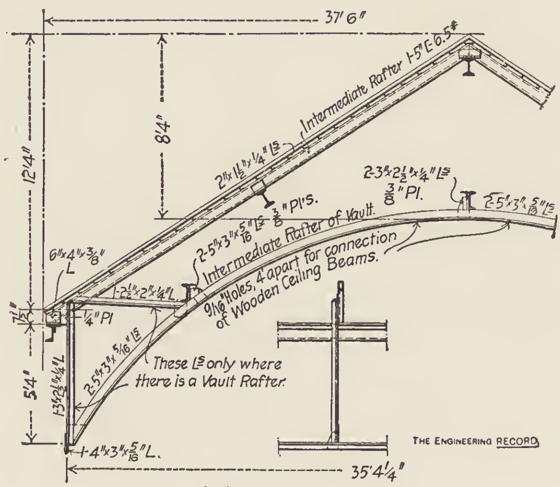
Fig. 188.

port 5-inch channel jack rafters, about 4 ft. apart, which in some parts of the roof are arranged so that alternate ones are in the same vertical planes with the ceiling rafters which are spaced about 8 ft. apart. Where the ceiling rafter and jack rafter are in the same plane they are braced together by horizontal and vertical field riveted angles at the eaves (Fig. 189). The ceiling rafters have gusset plates and angle clips connecting them to the longitudinal beams which are riveted to the vertical members in the main roof trusses (see Fig. 189). There is no diagonal bracing and no X-bracing in the roof panels, but the columns are knee-braced with pairs of 4x3-inch angles to the gallery floor girders.

The pitched roofs are covered with slate laid on 2x1½x¼-inch angles riveted to the top flanges of the rafters, roof trusses and jack rafters.

Wooden furring strips are bolted to the lower chord flanges of the roof trusses, to the ceiling rafters and to the column brackets, to carry the lathing for the vaulted ceilings. The total weight of structural steel in the building is about 370 tons." [For a more complete description of the steel framework of this building see the Engineering Record of Dec. 14, 1901.]

80. Fig. 190* shows one of four arched trusses which support the roof, and ceiling of the Roman Catholic Church at Tremont, New York City. These trusses were also designed by Mr. Berger for the architect, Mr. John E. Kerby. The principal dimensions of the trusses, which are spaced 16 ft. 4 ins. from centre to centre, are given in the illustration. The trusses are about 7 ft. deep at the



Intermediate Rafters

Fig. 189.

crown, $7\frac{1}{2}$ ft. deep at the eaves and $2\frac{1}{2}$ ft. deep midway between these points. The lower chords support the furring and metal lath for the plaster ceiling which forms a Gothic vault and conceals the trusses. All the trusses are alike except one end truss which supports part of the transept roof, and is a little heavier than the others, the dimensions given in Fig. 190 being for this truss, the chord angles of the other trusses being $^1/_{16}$-inch thinner. This truss carries on one side the ends of the longitudinal lattice girders, *E*, *E*, which are $31\frac{1}{2}$ ft. long, and have two $6 \times 3\frac{1}{2} \times ^7/_{16}$-inch angles in each flange. These girders support the ends of a special arch truss corresponding with the top of the main trusses and located on the cen-

*This cut and the accompanying description is taken by permission from the "Engineering Record" of Oct. 19, 1901.

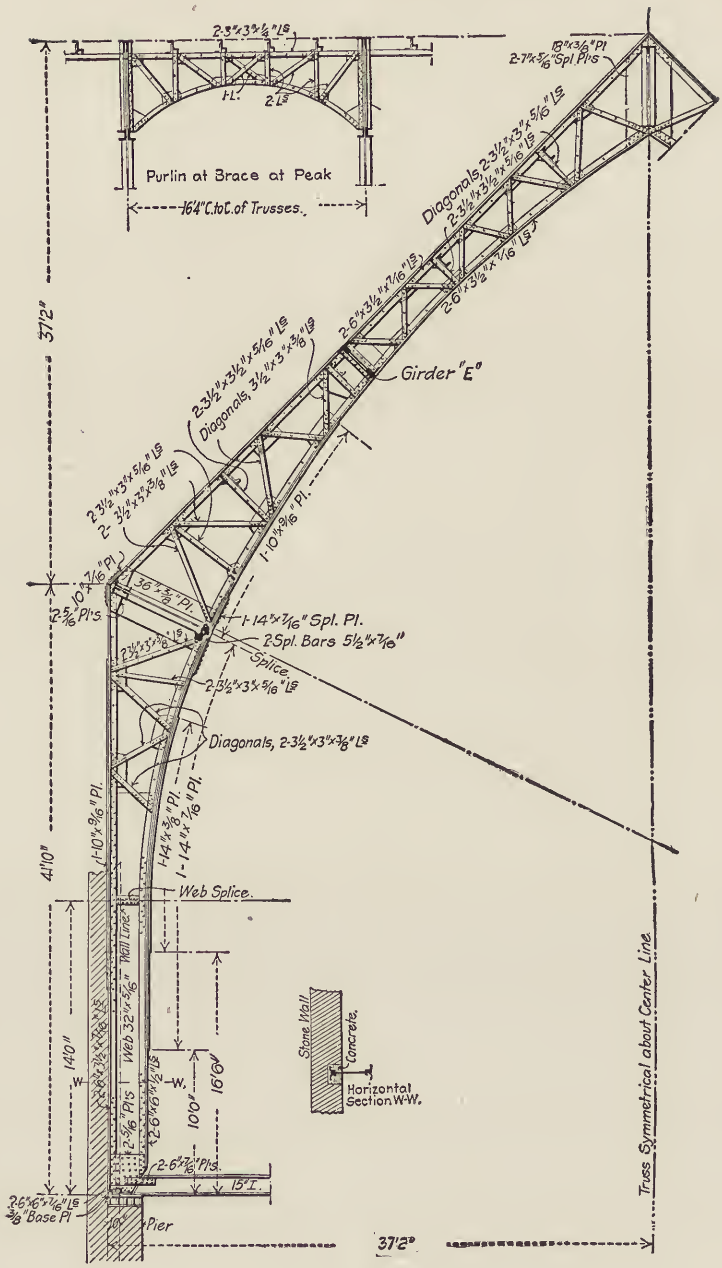

Purlin at Brace at Peak
16'4" C. to C. of Trusses.
Girder "E"
Web Splice.
Splice
Diagonals, 2-3½"x3"x⅜" Ls
1-14"x7/16" Spl. Pl.
2-Spl. Bars 5½"x7/16"
Stone Wall
Concrete.
Horizontal Section W-W.
Pier
37'2"
41'10"
14'0"
16'6"
10'0"
15" I.
3/8" Base Pl
Truss Symmetrical about Center Line

Fig. 190.

tre line of the transepts; they also support the ridge and valley rafters of the transept roof which intersects the main roof like a dormer. Between the other trusses a 12-inch channel purlin is used in place of the girder at *E*. The feet of the trusses are tied together across the building by the floor beams, shown in the figure. Beneath these beams is a finished basement.

Below the eaves, the trusses depend upon the outside wall for longitudinal bracing. Chases 12 ins. wide and 10 ins. deep were left in the masonry to receive the trusses, and were filled solid with concrete after the trusses were set. Above the eaves, the trusses are united longitudinally by purlins 10 ft. apart. At the peak and at the eaves these trusses are lattice girders with horizontal top chords and curved bottom chords. The intermediate purlins are 12-inch channels riveted to web angles of the trusses in planes normal to the top chords. The purlins support four 3x3x¼-inch Z-bar jack rafters in each panel between roof trusses. The top flanges of the jack rafters are in the planes of the flanges of the top chords of the arch trusses and the bottom flanges rest on top of the purlins. The jack rafters are riveted to the purlins by means of connection angles.

This roof was proportioned for a dead load of 60 lbs. (the covering being slate) and a live load of 50 lbs. per square foot, and to resist a horizontal wind pressure of 30 lbs. per square foot. The main arch trusses were received from the bridge shop in four sections each, with field splices at the peak and eaves. The vertical end sections were set up in place in the wall chases by boom derricks and lashed to the heavy stone walls. The middle sections were simultaneously raised in pairs by two gin poles which supported them until the splice joints were made and the purlins and longitudinal girders were connected to them.

81. OCTAGONAL ROOFS.—A plain octagonal roof would naturally be supported by intersecting trusses, spanning from opposite angles. If the ceiling below is level or unfinished, either Queen or Howe trusses may be used for wooden construction, while for steel construction a truss of the Fink type will usually be most economical, although almost any type of steel truss may be used, as with steel it is more easy to connect the half trusses at the centre.

If the ceiling is raised in the centre, as is generally the case in churches or auditoriums, trusses like those shown by Figs. 36, 42, or 176 should be used for wooden construction.

For steel construction the Quadrangular truss is usually best adapted, if the roof is to be supported on masonry walls. The three hinged arch truss is also well adapted to octagonal roofs.

If a wooden truss has a centre rod it will be easier to make the centre connection at the intersection of the ties if only two full trusses are used, with four half trusses suspended from them.

As has been previously stated, it is very difficult to join intersecting wooden tie-beams so as to obtain the necessary resistance, and especially so if the ties are inclined. For this reason, when framing a wooden octagonal roof of more than 40 feet span, the writer advocates the use of trusses of the type shown by Fig. 176.

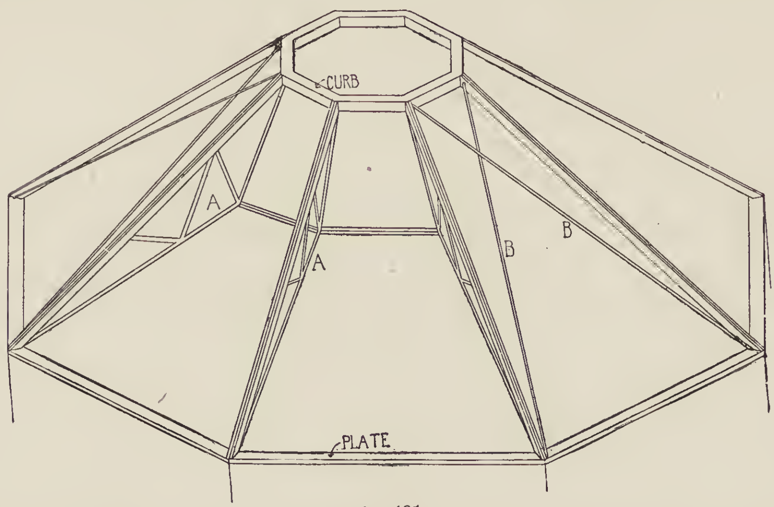


Fig. 191.

using rods for the horizontal tie, and either arranging the ties so that they can cross over each other or else connecting them to a central plate.

82. OCTAGONAL ROOFS WITH LANTERN.—When an octagonal roof has a raised ceiling, and is surmounted by a lantern, which must be entirely open and unobstructed, as is often the case with church roofs, the problem of supporting the roof, especially if wooden construction must be used, becomes more difficult.

The simplest method of supporting such a roof is that shown by Fig. 191, in which the roof is supported by eight hip rafters, abutting at the top against a heavy curb or plate, which also supports the lantern. The outward thrust of the hip rafters is resisted by the wall plate, which must be made a complete octagon, and connected at the angles so that the octagonal ring cannot be broken. As long as the plate and connections hold intact the rafters cannot spread.

The hips may be made of a single timber or they may be trussed, in the manner indicated at *A, A*. To prevent distortion of the frame from unequal distribution of snow or from wind, diagonal ties should be introduced in each panel, as shown at *B, B*, although if the roof were boarded diagonally these ties might be omitted. The curb plate should also be made as rigid as possible by proper connection at the angles. The best

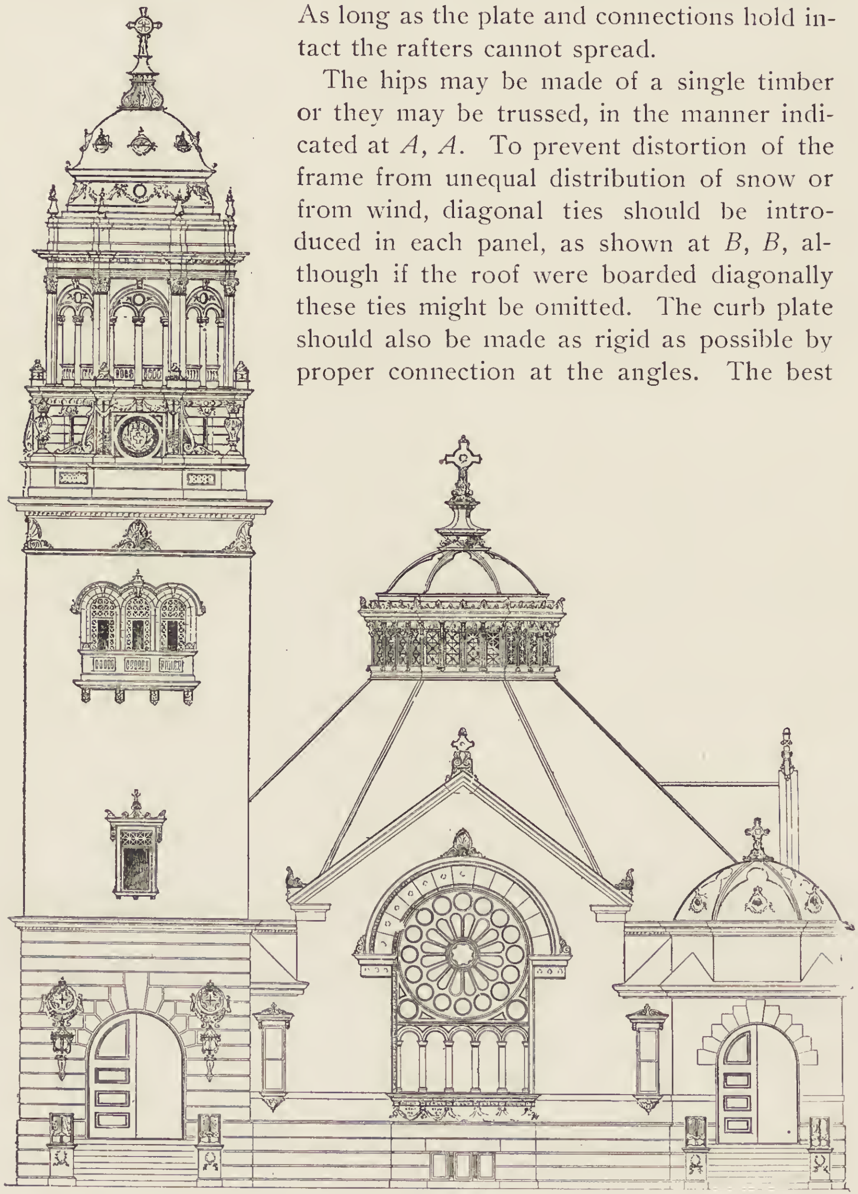


Fig. 192

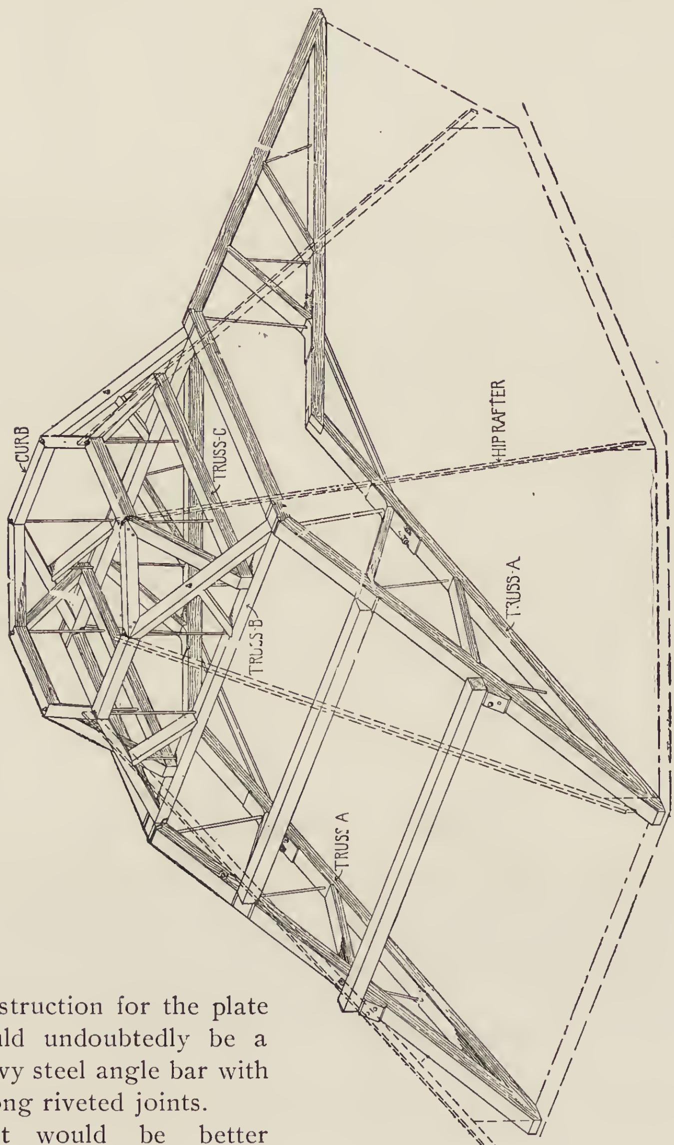


Fig. 193.

construction for the plate would undoubtedly be a heavy steel angle bar with strong riveted joints.

It would be better on such a roof either to run the common rafters horizontally from hip to hip, or else to have a number of purlins, so that the common rafters cannot push outward on the plate between the angles.

The principal objection to the adoption of this mode of construction, especially on churches, is that it is not always practicable to make the plate continuous around the roof on account of gables, chimneys, etc. With gables, however, the difficulty can be overcome by using a flat steel plate, which can be built in the brickwork, and, in fact, a steel plate is much to be preferred to a wooden plate, even on top of the wall, for the reason that the angle connections can be made more perfectly with a steel plate than with a wooden

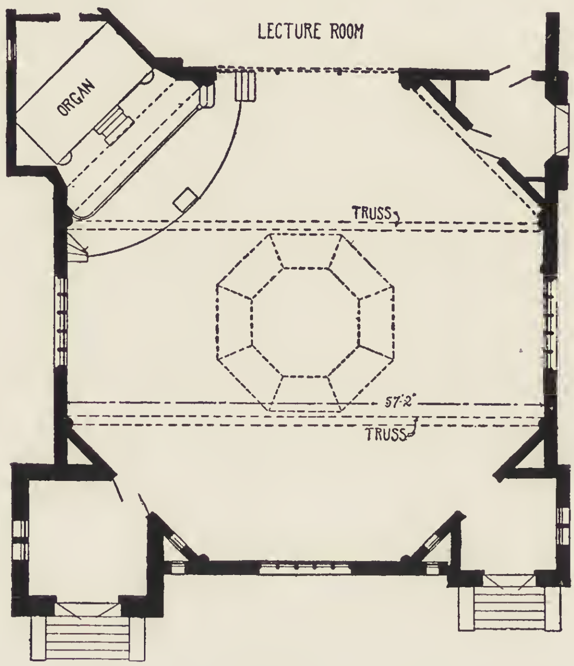
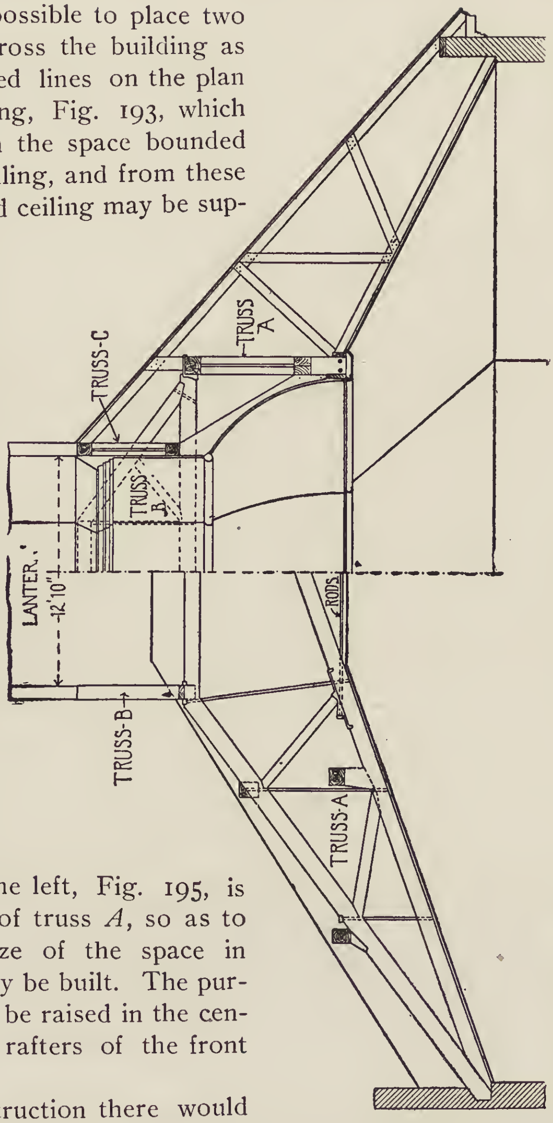


Fig. 194.

one, and it should be borne in mind that with this construction, if a single joint should fail, the stability of the whole roof would be endangered. With this construction the ceiling would naturally be hung from the roof framing, and its shape may be varied almost indefinitely.

Another method of supporting an octagonal roof with a raised ceiling and an open lantern is shown by Fig. 193. The building for which this construction is drawn is shown in elevation by Fig. 192 and in plan by Fig. 194. A section through the roof and ceiling, taken on a centre line, is shown at the right, Fig. 195. With a roof



Section Through Center.

Fig. 195.

Section on Line of Truss.

of this pitch it is possible to place two scissors trusses across the building as shown by the dotted lines on the plan and by the drawing, Fig. 193, which can be kept within the space bounded by the roof and ceiling, and from these trusses the roof and ceiling may be supported, leaving the space under the lantern entirely clear. The lantern is supported by two secondary trusses, *B*, and two still smaller trusses, *C*. A portion of the rafters are supported by purlins, while others are braced or trussed, as shown in the section, Fig. 195.

The position of the hip rafters is shown by the dotted lines, Fig. 193.

The section at the left, Fig. 195, is taken on the line of truss *A*, so as to show the exact size of the space in which the truss may be built. The purlins would have to be raised in the centre to receive the rafters of the front panel.

With this construction there would be no thrust on the walls from the trusses, and very little, if any, from the common rafters, so that it would not be necessary to carry the plate through the gables, although it would be a very wise safeguard to connect the ends of the wooden plates each side of the gables by an iron bar, say $\frac{1}{2}$x4 inches laid in the gable wall. When using scissors trusses, particularly when the angle between the

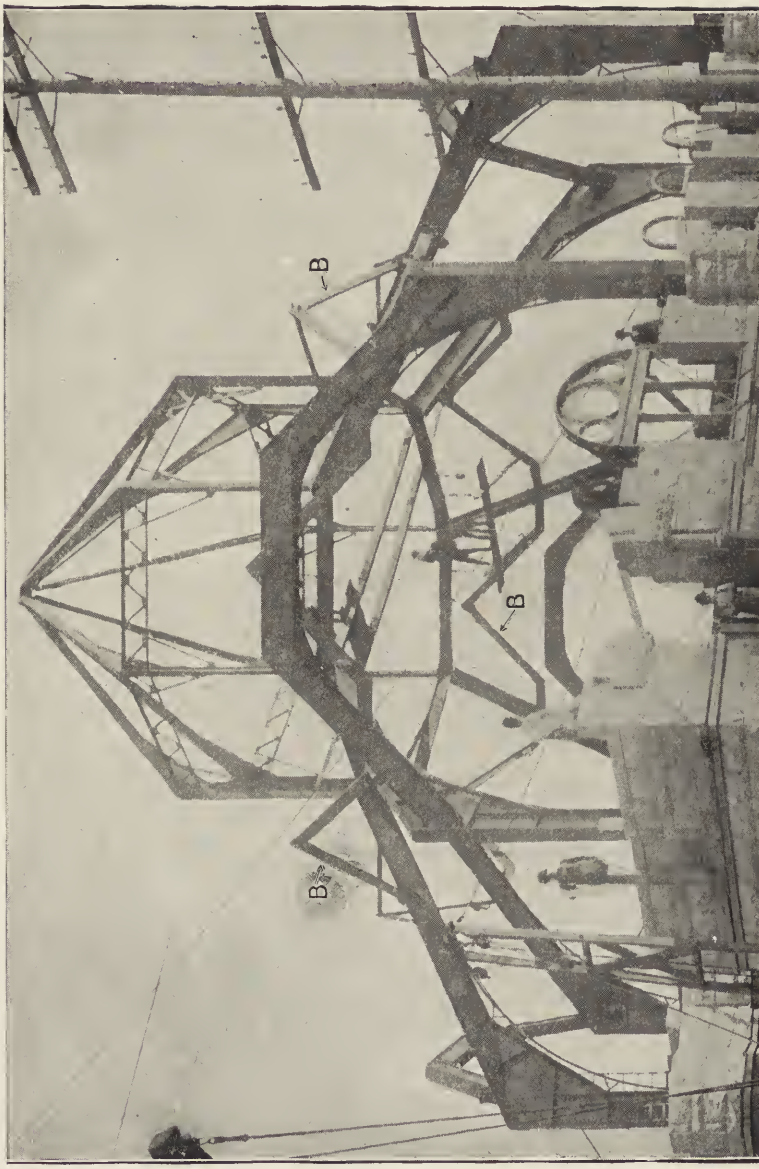


Fig. 196.—An Iron Synagogue Roof and Dome, St. Louis, Mo.
Messrs. Link & Rosenheim, St. Louis, Architects;
Mr. Julius Baier, St. Louis, Mo., Engineer.

rafters and tie is very small, it is always well to use whatever assistance may be obtained from the walls or other sources to prevent spreading.

Whether the construction shown in Fig. 191 or that shown in Fig. 193 will be the best to use on a given building will depend upon the particular conditions, and also upon whether the construction is to be of wood or steel. If the church shown was to have a steel roof, the writer would be inclined to use the construction indicated by Fig. 191, or else that described in Sections 83 and 84, while for wooden construction he would use that shown by Fig. 193.

83. STEEL OCTAGONAL ROOFS.—Several steel roofs have recently been constructed after the manner shown by Fig. 196. As will be seen from the description following,* this method of construction was used largely on account of the shape of the ceiling, which approaches so close to the roof surface that any other method of bracing was not permissible.

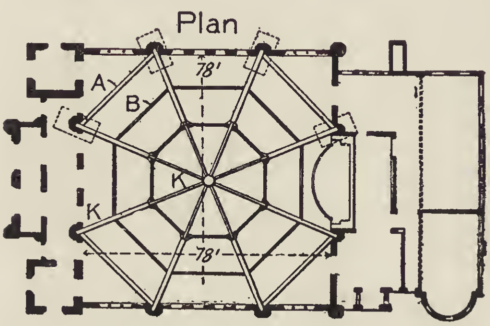


Fig. 197.—Plan, Iron Synagogue Roof, St. Louis, Mo.

This roof covers a square audience room 78 feet across, inside of the walls. The square is changed to an octagonal form by arches sprung across the four corners at a height of about 26 feet above the floor line. The general outline of the roof is that of an octagonal pyramid supporting a central lantern 33 feet in diameter, each side of the main roof being intersected by a gable rising as high as the base of the lantern. The interior is finished with an ornamental plastered ceiling formed by eight vaulted surfaces which spring from intersections on the lines of the main ribs supporting the lantern, and project up into the gable.

The necessity of providing a strong and stiff support for the roof and plastered ceiling within the limited available space between the roof line and the interior finish, led to the adoption of the steel framework shown. This framework consists of eight main ribs, or built beams abutting against and supporting an octagonal ring of 15-inch I's which forms the base of the lantern at the centre. The

*Taken from the "Engineering Record" of June 20, 1896.

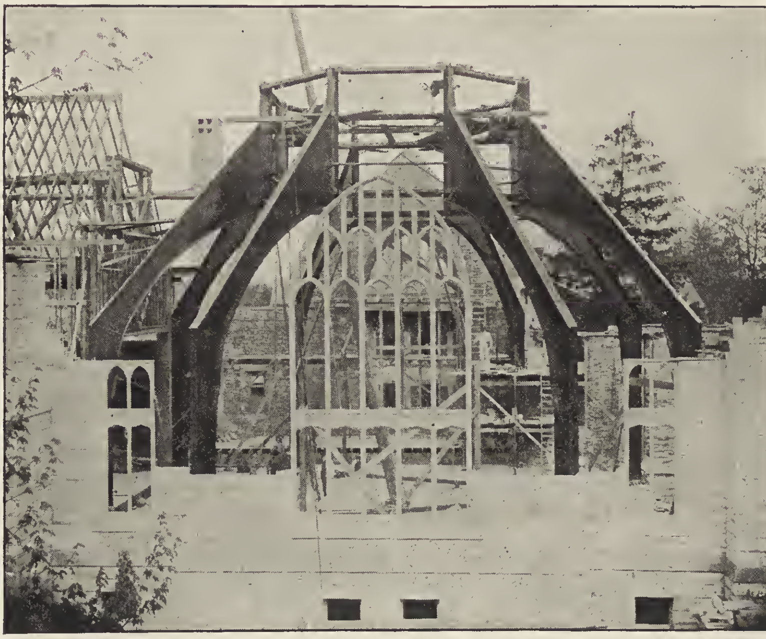


Fig. 198.—M. E. Church, Troy, Ohio.
S. R. Badgley, Architect.

frame of the lantern is formed by eight ribs built of 12-inch and 10-inch I's connected at the hips by gusset plates and angles, and tied together by portal bracing of angles, all of such dimensions and outlines as not to interfere with the ornamental interior finish and window openings.

At the four corners special-shaped box girders are attached to the main ribs to carry the arches and gable walls over the corners of the hall. The cross-beams *B* support the timber valley rafters, which in turn hold the rafters and roof covering. They are built of 18-inch channels of such outlines as to clear the ceiling line.

Any sway bracing between the main ribs would have been exposed in the interior and was not permissible. To give the necessary stability against the wind, the main ribs are therefore anchored at the base to concrete foundations. The anchorage and foundations are of such strength and weight as was necessary in addition to the weight of the wall and roof to resist the overturning action of the wind force against the roof.

A plan of the walls and steel framing is shown in Fig. 197.

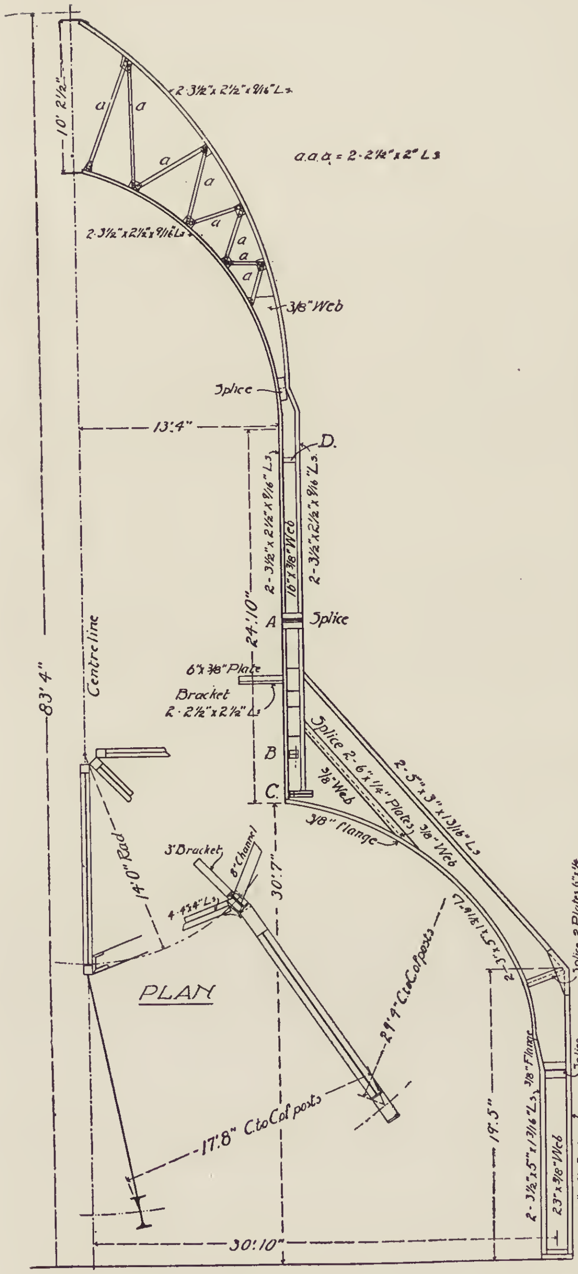


Fig. 199.—Detail of Steel Frame, Church at Troy, Ohio.

85. Figs. 198 and 201 show the steel frame of two other octagonal roofs constructed in the same manner. Both of these buildings were designed by Mr. S. R. Badgley, architect, of Cleveland, O., and the steel framework was manufactured and erected by the Rogers Iron Co., of Springfield, O., from whom the photographs and working drawings were obtained. Fig 199 shows one of the eight arched ribs of the church at Troy, with the principal dimensions, and also a partial plan of the steel frame. The relation of the sustaining posts to the outside walls is shown by the partial floor plan of the building, Fig. 200. As may be seen from the illustrations, the sustaining posts are not located at the corners of a true

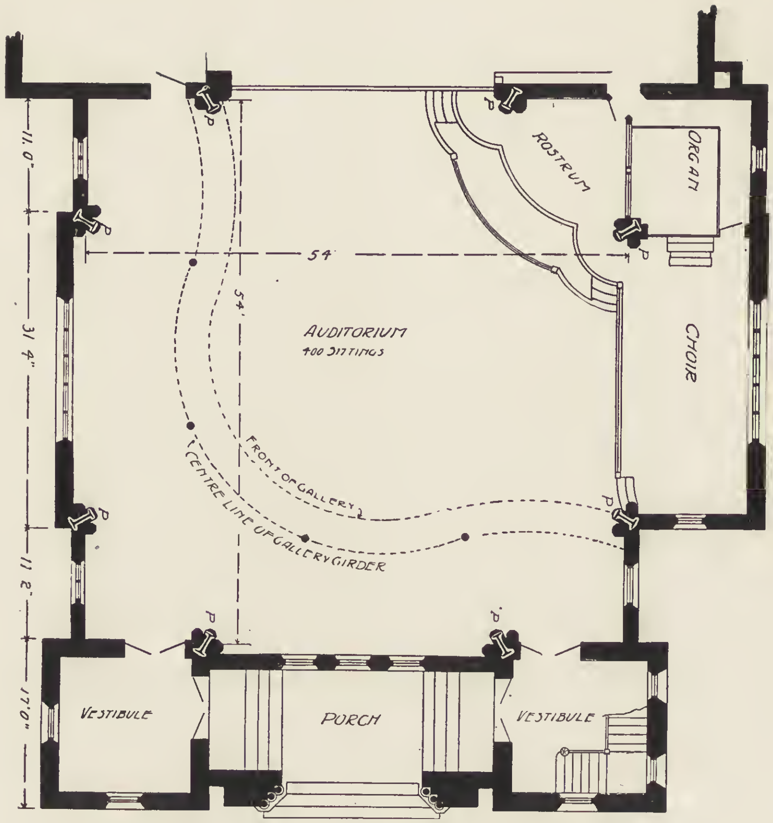


Fig. 200.—Partial Floor Plan of Church at Troy, Ohio.

octagon, but in the sides of a square having its corners cut off. The inner ends of these posts, however, are spaced so that the upper posts form a true octagon, as shown by the partial plan in Fig. 199, while the trussed ribs, at the top, form the angles of an octagonal dome. Fig. 201 shows the steel frame of the lantern and dome of the Washington church, as it appeared when completed ready to receive the wood furring, and Fig. 202 shows the finished exterior of the dome and lantern, and the general shape of the main roof. Fig. 203 shows the ceiling and base of the lantern of the church at Washington, the interior of the church at Troy being finished in the same way, except for the ornamentation. The object of the steel frame, in both of these churches, is to form a support for the lantern and dome, and also for the centre of the main roof and ceiling, without obstructing the interior view. The frame is entirely self-sustaining from the foundation, and imposes no strain whatever upon the outside walls.

In the Troy church, the foundations for the main columns were very near the main floor level. In the Washington church, the columns extended four or five feet below the floor. In both cases, the

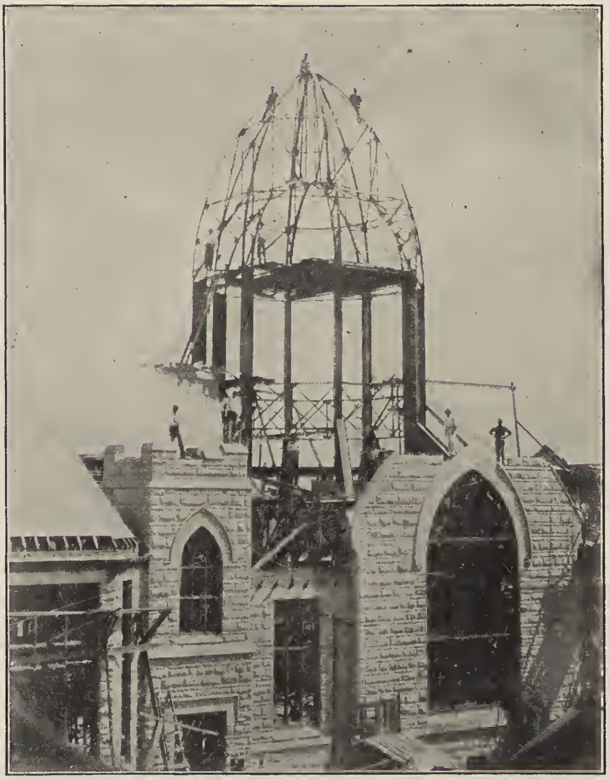


Fig. 201.—M. E. Church, Washington Court House, Ohio.
S. R. Badgley, Architect.

column foot plates were large and very substantial, and were well anchored by heavy bolts built into the masonry.

The top of the main posts are held in place by horizontal struts, at the levels *A*, *B* and *C*, Fig. 199, which form an octagonal ring, the connections at the posts being made very stiff. The base of the lantern, between the points *A* and *B* is also stiffened by diagonal bracing as shown in the view, Fig. 201. The projecting brackets on the inside of the lantern posts, support a gallery, a portion of which is shown in Fig. 203.

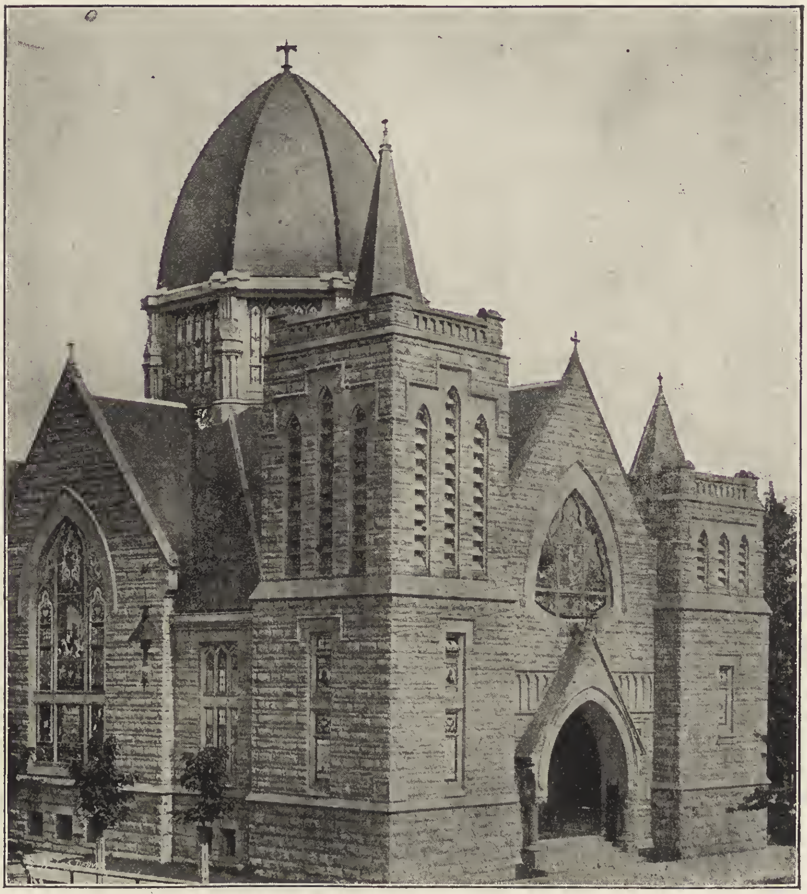


Fig. 202. Same Church, Completed.

In both of these churches the framework of the main roof was of wood, the purlins being supported by the eight steel posts. The dome roof is also formed of wood, attached to the steel ribs, and both the dome and main roof are covered with slate.

The walls of the lantern, where not filled with glass, are formed of wood framing, attached to the steel, sheathed with matched boards, and covered with galvanized iron. The ceiling of the lantern finishes on the line of the bottom of the steel ribs and is formed of lath and plaster, the whole interior of the lantern being plainly visible from the pews.

The external and internal effect of these buildings could not be

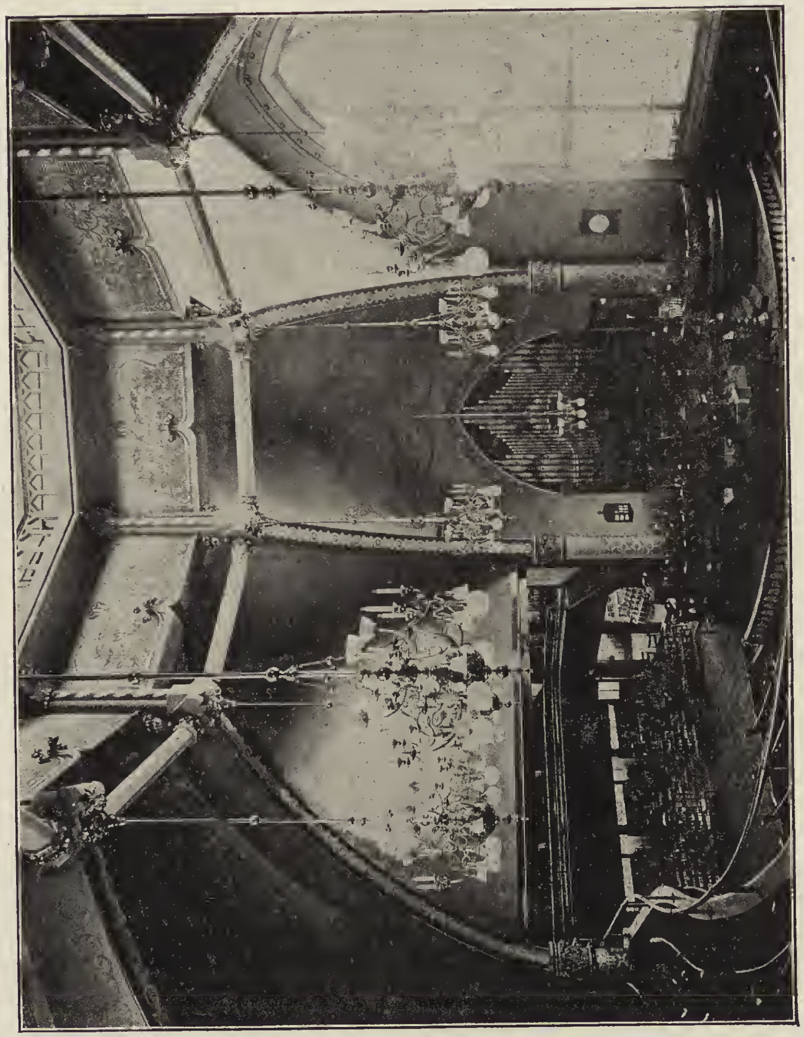


Fig. 203.—Interior of Church at Washington Court House, Showing Base of Dome.

obtained by any other method of construction, known to the author, as with any method of trussing, it would be necessary either to provide a greater space between the ceiling and roof, or else to expose the truss members.

The weight of the structural steel and iron in the Washington church was about 43 tons and in the Troy church about 75 tons, but

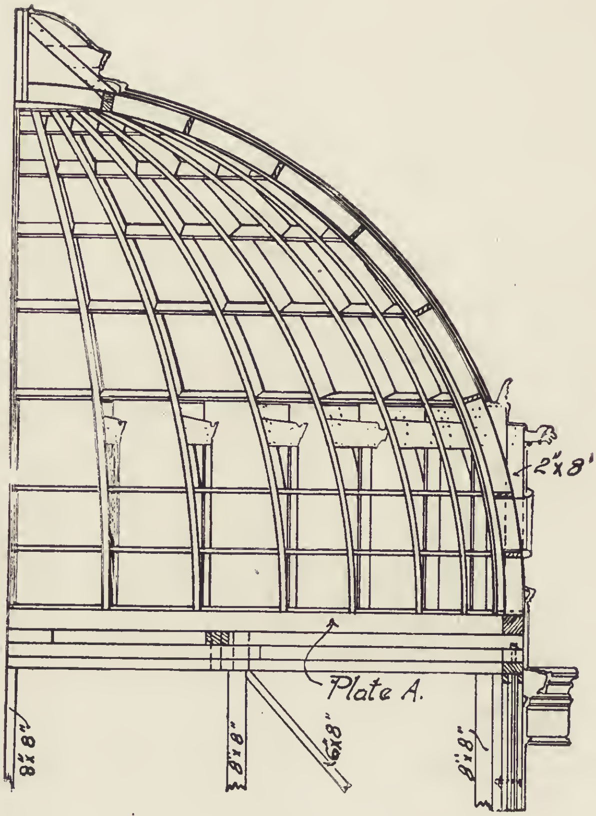


Fig. 204.

these figures include the entire steel used in the building, some in excess of that in the roof construction and columns.

The diameter of the dome of the Washington church, to outside of columns, is 32′ 6″ and the height above the top of upper columns, 29′ 11″. The height of the lantern above main roof is 19′ 5″.

The general plan of the Washington church is very similar to that of the Troy church, Fig. 200, the width of the square, figured from centres of the posts being 59.′6″. The main posts are built with a 24″x$\frac{1}{4}$″ web, and 3$\frac{1}{2}$″x2$\frac{1}{2}$″x$^5/_{16}$″ flange angles. The dome ribs are built of two 3″x2$\frac{1}{2}$″x$\frac{1}{4}$″ angles for the outer and inner chords, with single 3″x2$\frac{1}{2}$″x$\frac{1}{4}$″ angles for the bracing.

The engineers for the steel frame were The Osborn Co., of Cleveland.

DOMED ROOFS.

86. These are usually formed of a number of ribs or semi-trusses resting on a circular base and connected to a circular ring, plate or pole at the top. Each rib must possess sufficient stiffness to resist the load upon it, including its own weight, without suffering distortion. The outward thrust of the rib, which is comparatively small in a hemispherical dome, is taken up by a plate forming a ring about the base of the dome, and to which the ribs are attached. The sheathing of the dome, if the frame is of wood, or the angle purlins of a steel dome, also assist in taking up the outward thrust and in preventing the ribs from bulging outward. Occasionally domed roofs are supported by interior vertical supports, in addition to the outer support; under such conditions the ribs are merely curved

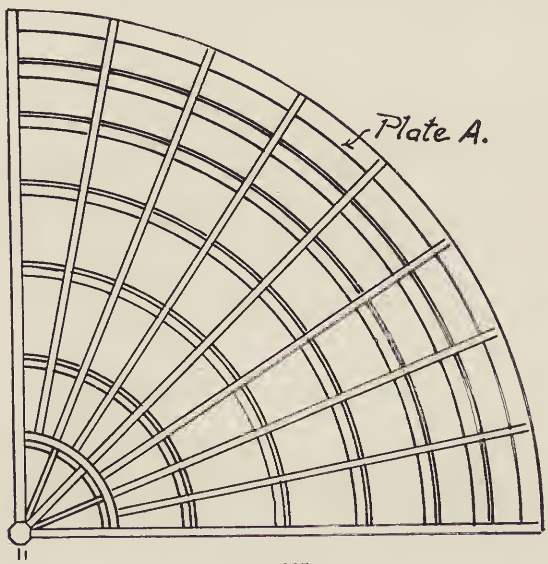


Fig. 205.

rafters, supported by curved purlins, and the mechanical principles involved are very simple.

EXAMPLES OF WOODEN DOMES.—Figs. 204 and 205 show about the simplest construction for a wooden dome. They were made from the working drawings of the dome of the Woman's Building at the Cotton States Exposition at Atlanta, in 1895, Elise Mercur, architect.

This dome was 41 ft, 6 ins. in external diameter and hemispherical in shape. The shell is formed of 32 ribs built up of 4 pieces of

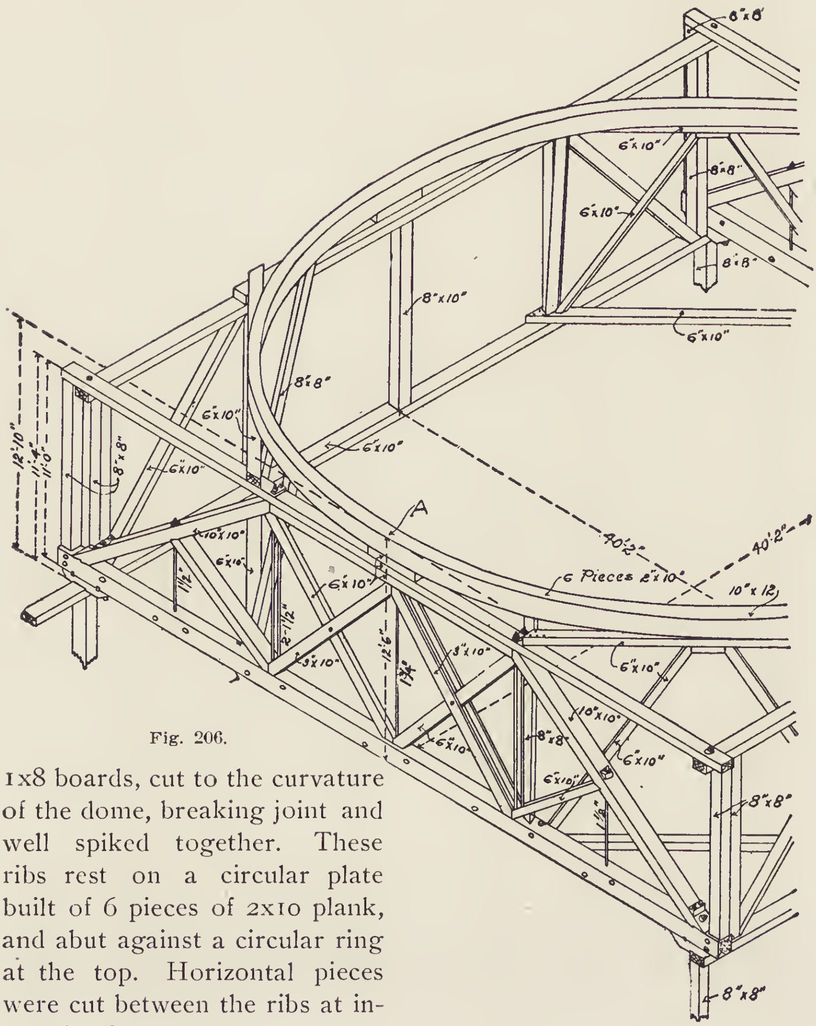


Fig. 206.

1x8 boards, cut to the curvature of the dome, breaking joint and well spiked together. These ribs rest on a circular plate built of 6 pieces of 2x10 plank, and abut against a circular ring at the top. Horizontal pieces were cut between the ribs at intervals of about $3\frac{1}{2}$ ft.

The strength of the shell was materially increased by the drum surrounding the base of the dome. Fig. 206 shows the manner in which the dome was supported over the square space below. The framework on two sides was supported by interior partitions and on the other two sides by Howe trusses, one of which is shown in the figure.

Fig. 207* shows a section through the wooden dome over the

*From the "American Architect," of May 29, 1897.

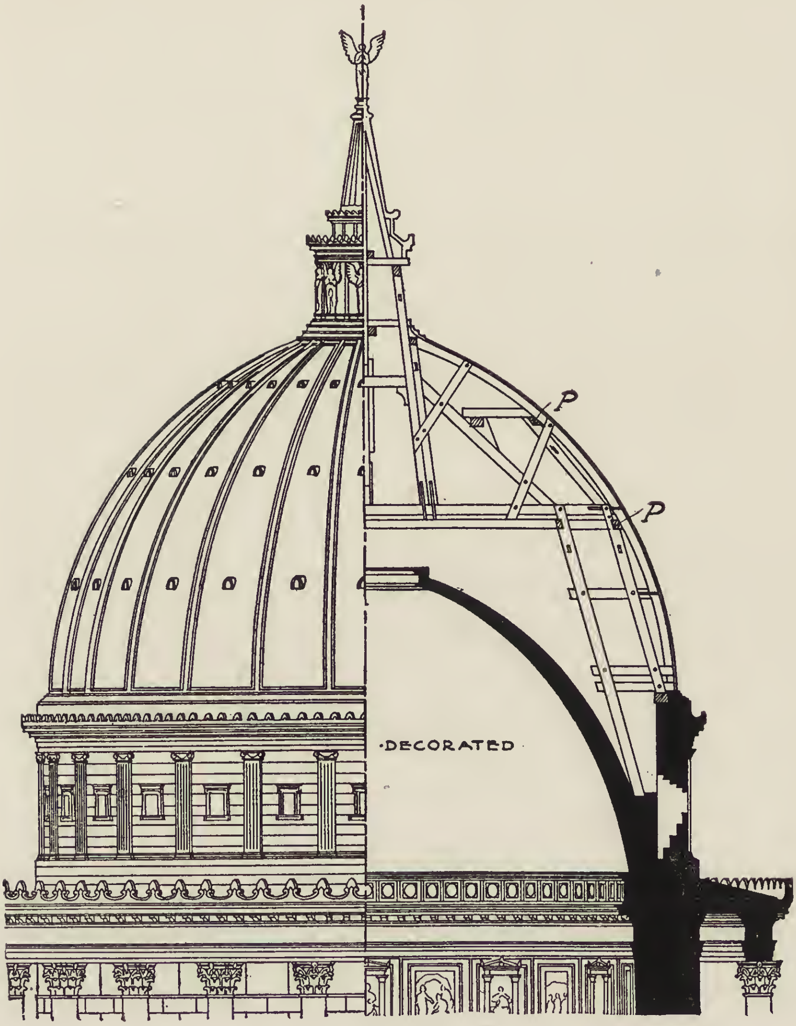


Fig. 207.

Nicolaikirche, Potsdam, Prussia, the external diameter of which is about 72 feet. No description was given of the construction, but it is probable that the etched plates, *P P*, extend all around the dome. and that there are perhaps three trusses similar to the one shown in the figure, extending across the dome in two directions, and that the outward thrust at the base is taken care of by iron rings.

Wooden domes can also be constructed by braced ribs, built up similarly to the steel ribs shown in the following illustrations.

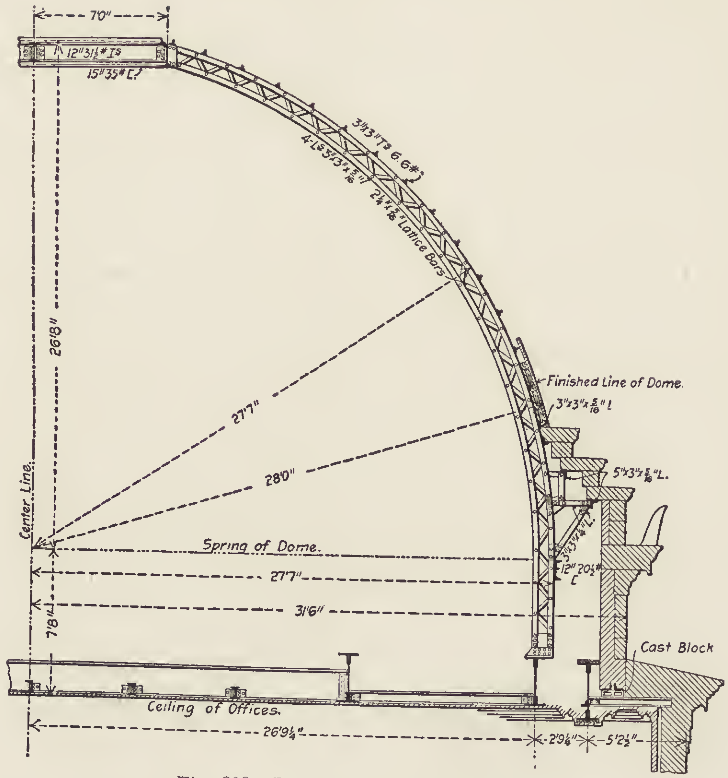


Fig. 208.—Dome of Buffalo Savings Bank.

87. EXAMPLES OF STEEL DOMES.—Fig. 208* shows a semi-section of the dome over the Buffalo (N. Y.) Savings Bank Building, Messrs. Green and Wicks, architects. "The framework of this dome is composed of 32 latticed radial segmental ribs, one of which is shown in elevation in Fig. 208 which also gives a section of the circular wall enclosing the foot of the dome, and shows the details of the granite blocks in its cornice, and their support on brackets attached to the ribs.

The foot of each rib is provided with horizontal angle flanges which are seated on and riveted to the tops of the main girders, and are made to offset from them and support the rib beyond the centre where necessary. The ribs are vertical up to the springing line, about 4 ft. above the base, and are there encircled by a circular 12-inch channel having its vertical web riveted to their

*This illustration and accompanying description is taken by permission from the "Engineering Record" of Nov. 25, 1899.

outside flanges. The tops of the ribs are connected by short vertical web angles to a drum, 14 ft. in diameter, made of a 15-inch channel and stiffened by two diametrical 12-inch beams at right angles to each other.

The roof was laid with 3x12x24-inch hollow book tile weighing 16 pounds per foot. This is covered with a bitumen damp course, lapped 3 ins. on which are bedded in slaters' cement semi-glazed terra-cotta tile, grooved, lipped and fastened with copper wire."

88. Figs. 209 and 210* show a partial section and plan of the

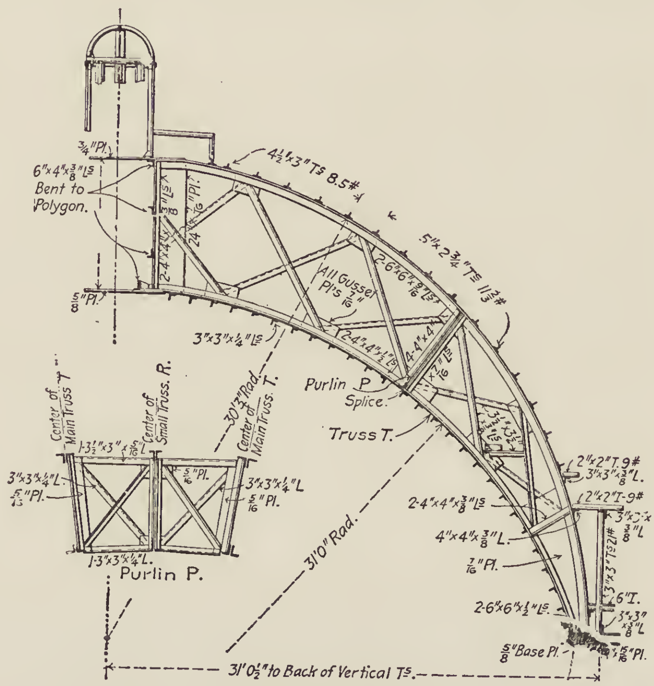


Fig. 209.—Section Through Dome, New York Clearing House.

steel frame of the dome over the New York Clearing House, Mr. R. W. Gibson, architect.

The dome has a nearly hemispherical exterior surface and is supported by an octagonal arrangement of web-connected plate girders. It rests on each of these girders and intermediately on 20-inch rolled beams across their angles, which form with them a sixteen-sided polygon.

*These cuts and the accompanying description are taken by permission from the "Engineering Record" of Sept. 29, 1900. Several other details of the steel work in this building are given in the same issue.

There are sixteen radial dome trusses seated on top of this polygonal framing and having their top and bottom chords riveted at the centre (top) to a pair of horizontal circular steel plates 6 ft. 4 ins. in diameter. Each truss is made in two sections, field riveted together through the flanges of web members which are perpendicular to the dome surface. Trussed purlins 6 ft. deep are connected to the trusses in the planes of these members and support at their centre points, intermediate curved rafters extending to the foot of the dome. The trusses are also connected by horizontal circular T and angle bars riveted to their top and bottom, to support the roof and ceiling respectively. The horizontal reactions at the lower ends of the radial trusses are provided for by a $14 \times {}^{15}/_{16}$-inch circular vertical steel plate, like a hoop around their feet.

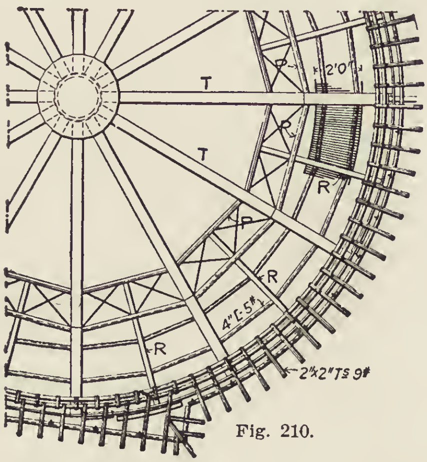


Fig. 210.

The dome is surmounted by a cupola or circular platform $11\frac{1}{2}$ feet in diameter and 20 ins. high. In the centre of this platform is a lantern 4 ft. in diameter and 6 ft. high with a hemispherical top. All the members of the cupola and lantern are made of pairs of $2\frac{1}{2}'' \times 2\frac{1}{2}''$ angles riveted together back to back, and connected by $\frac{1}{4}$-inch gusset plates and $\frac{5}{8}$-inch rivets.

89. Figs. 211 and 212 show a semi-section and plans of the steel dome above the administration building of the New Jersey State Reformatory at Rahway, built in 1897. This dome is 120 feet in diameter or 8 feet greater than the dome of St. Paul's, London, and only $4\frac{3}{4}$ feet less than the dome of the capitol at Washington. It is 130 feet high from the ground level to the top, and contains about 400 tons of open-hearth steel. The roof and ceiling are supported by 24 semi-arches, *A*, and 24 shorter ribs, *B*, interpolated between the main arches to carry the purlins of the lower part of the roof. Part of the arches rest directly on the top of the wall and part upon steel girders set across the corners of the hexagon. From the arches of the dome, a ceiling is suspended as shown in Fig. 209; this suspension work is in the nature of furring and does not form a part of the dome ribs.

The dimensions of the members of the main ribs of the dome are given in Fig. 211. The intermediate ribs *B* are of the same shape, but are built of lighter angles, the top and bottom chords being formed of pairs of $3\frac{1}{2}''$x$3''$-7.9 lb. angles with $2\frac{1}{4}''$x$2\frac{1}{4}''$x$\frac{1}{2}''$ web diagonals. The main ribs abutt at the top against a circular plate girder, and are riveted to it by means of connection angles.

In erecting the dome, a central tower was first built, on the top

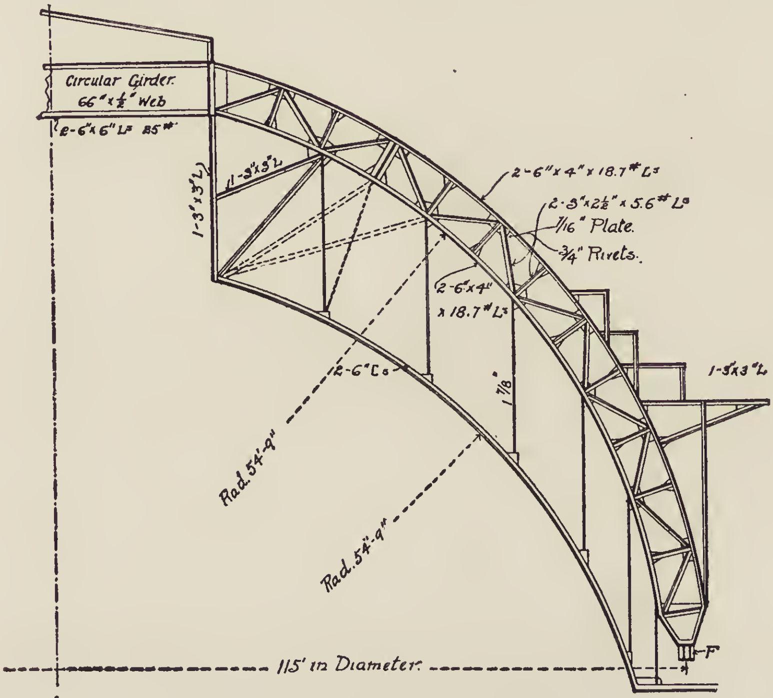
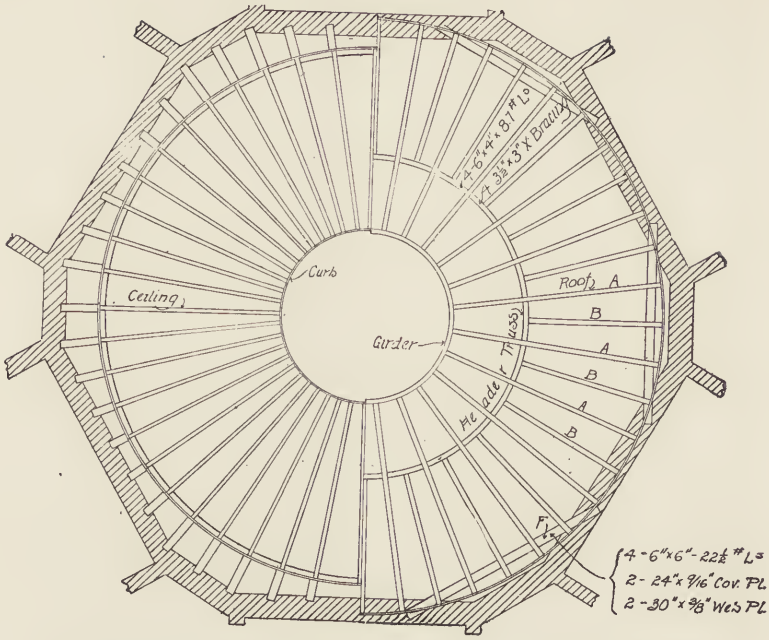


Fig. 211.—Semi-Section Dome at Rahway, N. J.

of which the circular girder was set in its exact position, and carefully levelled. On top of the tower and circular girder a revolving derrick was set, by means of which, and one six-spool hoisting engine, the ribs were raised into position. Each truss was received from the shop in three sections. The first, or lower section and the middle section were bolted together on the floor, then raised and held in place by tackle, while the third or top section was being raised and connected to the lower portion. After this connection was made the whole rib was dropped into place and became self-sustaining, being guyed to prevent turning over. These operations



One-half Plan of Ceiling Framing. One-half Plan of Roof Framing.

Fig. 212.

were simultaneously executed on diametrically opposite trusses and then the traveller was swung into the plane of another pair of trusses which were similarly erected.

Details of the erection plant and a complete description of the same may be found in the Engineering Record of May 14, 1898.

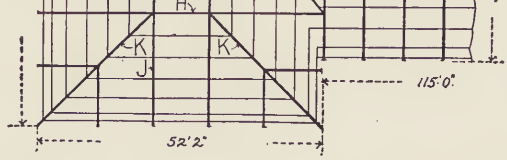


Fig. 213.—Plan of One Pavilion, South Park Conservatory.

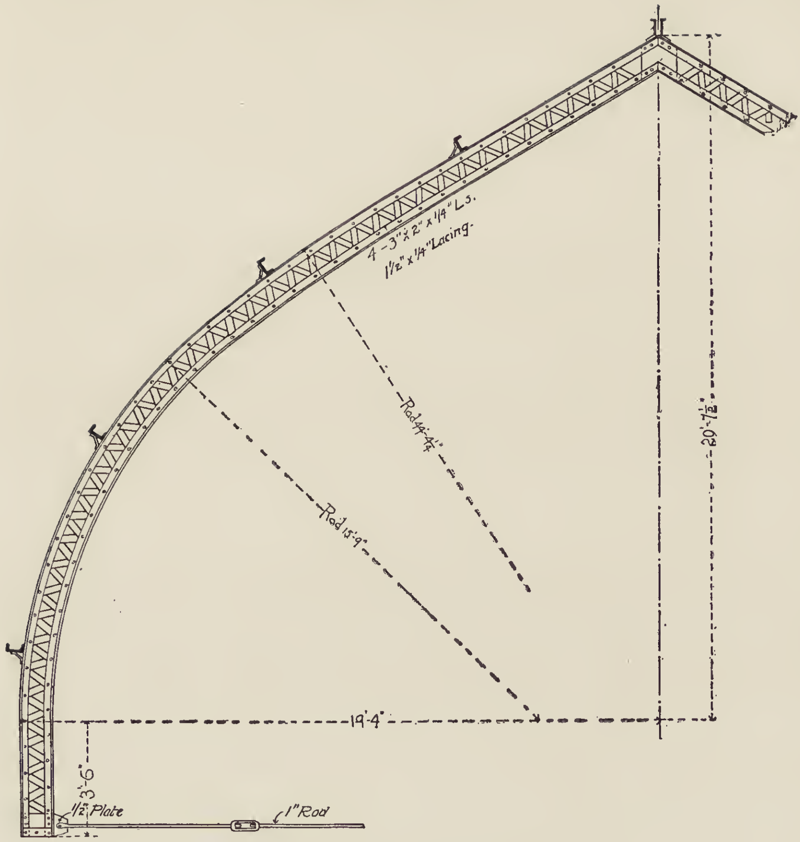


Fig. 214.—Truss F of Fig. 213.

90. VAULTED CONSERVATORY ROOF.—Fig. 214 shows an arched rib used in the construction of the vaulted roof of the South Park Conservatory, Chicago, Ill. This building is a steel and glass structure about 417x58 feet in extreme dimensions and 31 ft. in greatest height. It consists of a rectangular centre pavilion and two rectangular end pavilions with connecting wings 115 ft. long and 39 ft. wide. Fig. 213 shows a plan of one of the end pavilions and a portion of one of the connecting wings. The arched rib shown by Fig. 214 is one of the ribs shown in plan at *F*, *F*. These ribs were spaced about 7 ft. 8 ins. apart. The rafters or curved ribs, *H*, *H*, of the end pavilions are built in a similar manner to those at *F*, but their tops are connected by a horizontal top chord section, 10 ft. 4 ins. long, field riveted to them. The ribs *H*, *H*, were built of 4-$3\frac{1}{2}$x$2\frac{1}{2}$x$\frac{3}{8}$-inch L's, with $2''$x$^5/_{16}''$ lacing. The diagonal ribs,

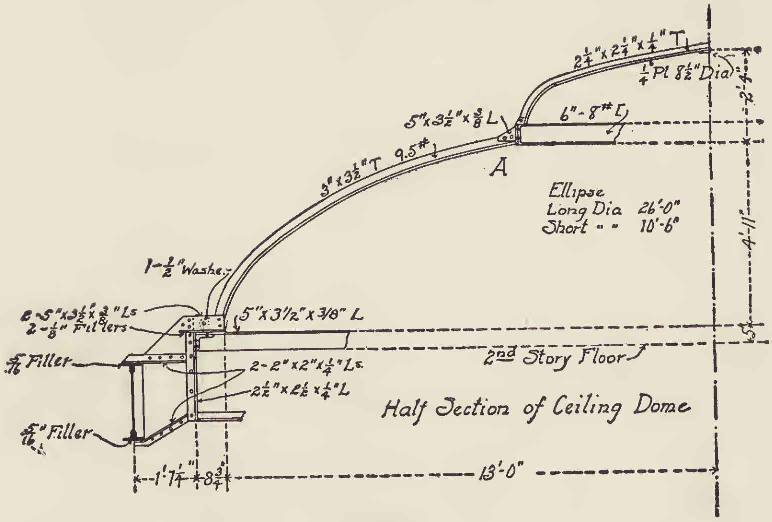


Fig. 215.

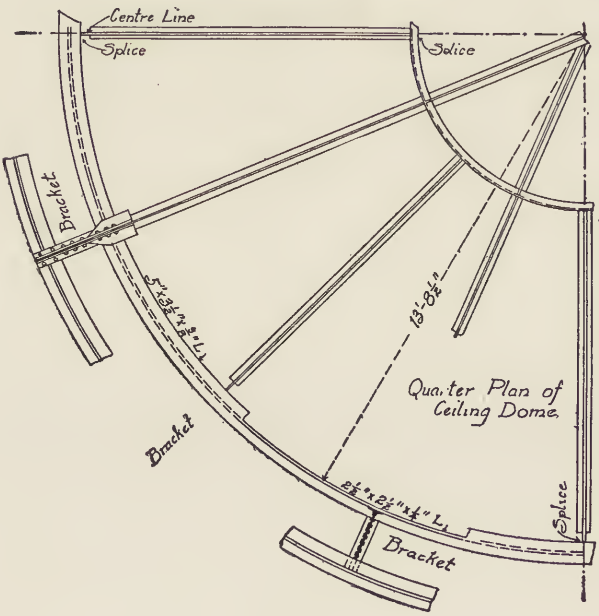


Fig. 216.

K, *K*, were built to the curve formed by the intersection of the vaulted sides. These ribs are 15 inches deep, and the members are of the same size as those used in the ribs *H*, *H*. The entire cover-

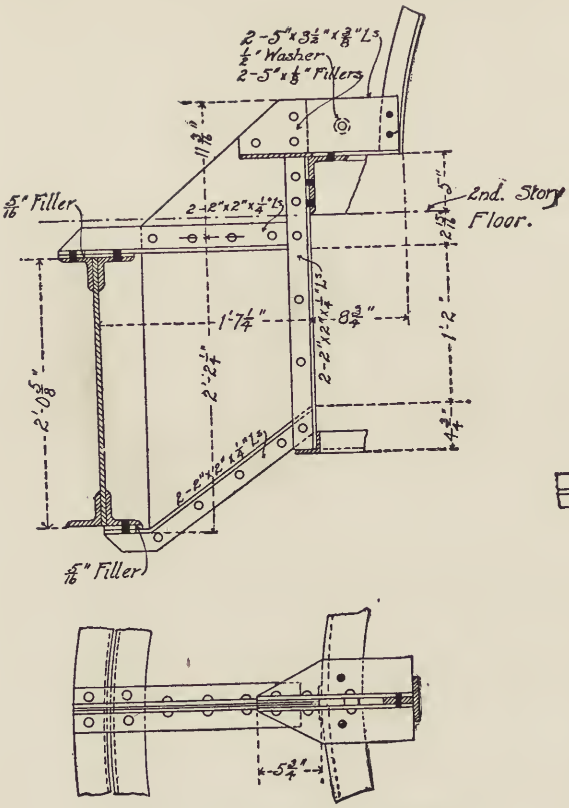


Fig. 217. Detail of Supporting Brackets.

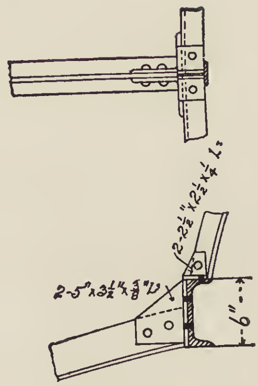


Fig. 218.
Detail at A, Fig. 215.

ing of the building is of glass. Details of the ribs *H* and *K*, were published in the Engineering Record for Dec. 24, 1898.

Mr. D. H. Burnham was the architect of the building and the steel work was designed and constructed by the Kenwood Bridge Co., Chicago.

91. Steel Ceiling Dome.—Figs. 215 and 216* shows a section and plan of a ceiling dome in the Appellate Court House, New York City. This dome is supported by a circular plate girder 24⅝" deep, and 30 ft. 8 ins. in diameter, to which sixteen solid web radial brackets are attached, which carry the framework of the dome. The dome itself is built of curved T-bars, arranged as shown by the quarter plan, Fig. 216, with a circular ring formed of a 5"x3½"x

*Redrawn from illustrations in the "Engineering Record" of April 14, 1900.

$\frac{3}{8}''$ L at the bottom of the ribs and another near the top formed of a 6-inch bent channel. Details of the supporting brackets, and of the connections to the upper ring are shown in Figs. 217 and 218.

The dome is glazed with stained glass and has a richly moulded and decorated cornice below it.

Chapter VI.

COLISEUMS, ARMORIES, TRAIN SHEDS, EXPOSITION BUILDINGS, ETC.

92. For drill rooms, skating rinks, etc., it is important that there shall be no posts or other obstructions to interfere with the movements of the men or skaters, while in the case of Coliseums or auditoriums it is equally important that the view from all portions of the building shall be unobstructed.

This necessitates a single span for the roof, and as buildings of this class are generally very large, the roof is usually the most costly portion of the building, and requires considerable engineering skill to properly design it.

As a rule the trusses are of steel, and their design is governed strictly by economy.

Wooden trusses could be used for spans up to 100 feet, but as they would require a greater height for the walls, to give the same clear height at the center, there is very little economy in using them, except in buildings where high walls are required on account of several galleries.

When a wooden truss must be used, a truss of the type shown by Figs. 58 and 59, will generally be most economical, where there are galleries, while for buildings without galleries a truss such as is shown by Fig. 61, may be used. The latter truss gives a very pleasing effect.

The roof of the Mormon Tabernacle, at Salt Lake City, is a very interesting example of an arched wooden roof. Timber was the only material available for roofing this building, because iron was very expensive in those days (1866-7), having to be freighted from the Missouri River. Nails cost about a dollar a pound and were used very sparingly.

The general floor plan of this building is shown by Fig. 219, while Fig. 220 shows an approximate elevation of one of the arched trusses.

The supports or walls of the building are 44 buttresses or pillars of sandstone masonry, each 3′ × 9′, and ranging in height from 14 feet at the east end to 20 feet at the west end. Each buttress has the foot of an arch to support, these arches forming the roof of the building. These buttresses are set with their axes normal to the sides of the structure, those at the ends being on radial lines. The arches are lattice trusses, the kind so often used during the early days of bridge construction, built of Utah red pine cut from the mountains near Salt Lake City.

Each arch consists of four curved chords or courses of timber braced together as shown. Each chord is composed of four pieces of $2\frac{1}{2} \times 12$-inch plank, two on each side of the lattice bars that pass between them. The lattice bars are also $2\frac{1}{2} \times 12$ inches. The two lines of plank forming each side of each chord are continuous. The joints are alternating, and as the strains are compressive, they

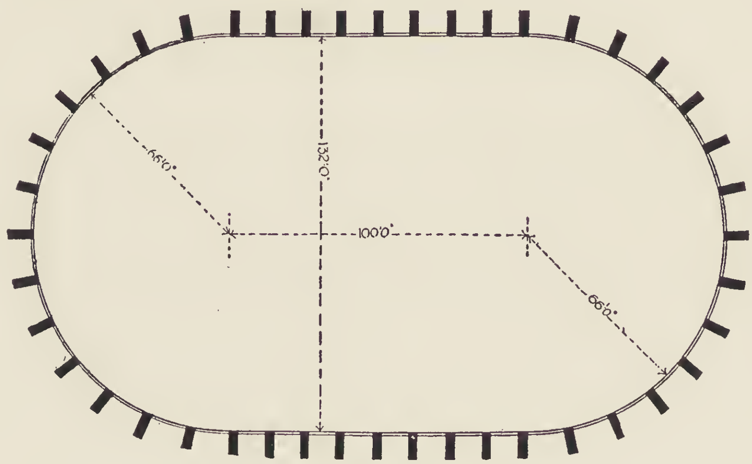


Fig. 219.—Plan of Tabernacle at Salt Lake City

are not spliced, but are simply butt joints, with an even bearing. The lattice bars and the chords intersect at the same points, and they are connected together by wooden pins about $1\frac{1}{2}$ inches in diameter. Each chord is thus 15 inches thick, and the depth of the arch is 10 feet.

As the arch has no horizontal chord or tie rod nor any abutment to thrust against, its strength depends entirely on its stiffness, and this is increased in the following manner: An extra stiffening brace, shown by dotted lines, starts from the outside chord at its foot and crossing the arch in a vertical plane it intersects the inside chord (being tangent to the inner curve line), and passing on it crosses the arch again and rejoins the top chord, well up on the flatter part of the arch. There are two braces, each of two $2\frac{1}{2} \times 12$-inch plank, one brace on each side of the arch, and they are fastened to each chord by a large iron bolt passing clear through the ten thicknesses of plank at these intersections. Very little iron besides these bolts was used. There is a full system of cross-bracing between the arches.

The half arches for forming the semi-circular ends of the building are similar in construction to the main arches. The tops of the half arches are connected to a small, nearly semi-circular construction of timbers built horizontally into the adjoining main arch. This main arch is well braced against the next one to it, but beyond

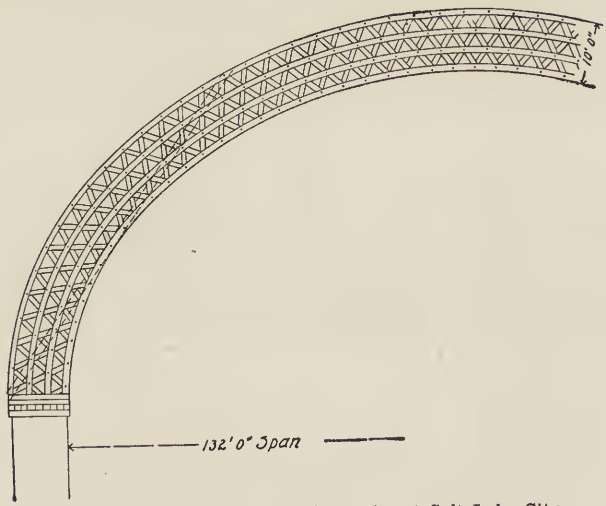


Fig. 220.—Arched Truss. Tabernacle at Salt Lake City.

this the regular cross-bracing is relied on to transmit the thrust from the half arches along the axis of the building.

The roof was originally of wooden shingles, but about the year 1892 these were replaced by a patent tin roof. This tin roof does not give as good satisfaction as the shingles, there being many leaks. The flatter part of the roof, over the center is of sheet iron.* The design of the tabernacle is said to have originated with Brigham Young, the necessary drawings being made by Architect W. H. Folsom, who also worked out the details.

93. Figs. 221, 222, and 223 show the ingenious method adopted by Mr. Carl Pfeiffer, a former architect of New York City, for roofing a riding school on Fifth Avenue, in that city. The rid-

*W. P. Hardesty, C. E., Engineering Record, Jan. 27, 1900.

ing room is 106′ 6″ long and 73 feet wide. This space is kept entirely clear of posts or columns, and the entire roof is supported by two arched trusses, one of which is shown in Fig. 222. The location of the trusses with reference to the plan is shown by Fig. 221. The roof between the trusses and on either side is supported by smaller trusses resting on these large trusses, but each of the arched trusses eventually supports a roof area of about 2,930 square feet and a great amount of extra frame work. It was desired to provide for the thrust of the main trusses without having rods exposed in the room, and the method adopted for taking up the thrust is rather unusual. Opposite the upper ends of the iron posts which receive the arched ribs are oak struts which are held in place by iron tie bars and heavy iron beams, which together form a horizontal truss

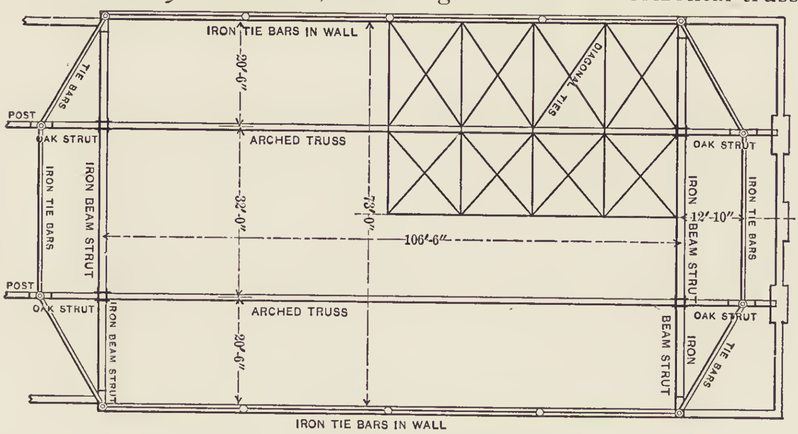


Fig. 221.—Plan Showing Horizontal Trusses.

at each end. These horizontal trusses are prevented from being pushed out by two 3-inch by 1-inch tie bars in each side wall, shown in the plan, Fig. 221. The bottoms of the two iron posts are also tied together by iron rods placed under the floor of the room. Altogether this gives for the tie rods of each truss two bars 3″ × 1″ and one rod 1½ inches in diameter.

Enlarged sections of the ribs and web members of the arched trusses are shown in Fig. 222. It should be noticed that the vertical web members have an iron rod through their center, so that they act as ties rather than struts.

Fig. 223 shows an enlarged detail of the iron skewbacks and posts which receive the ends of the arched trusses.*

*The construction of this building was originally illustrated in the American Architect of May 1, 1880.

94. When steel trusses are used for roofing buildings of this class, either the three-hinged braced arch or a truss like that shown in Fig. 103 is most commonly selected, because it requires no posts, and is comparatively easy of erection. There are several examples of segmental and elliptical braced arches, however (see Fig. 98), and for auditoriums having several galleries, the quadrangular truss has been frequently used, as in the Madison Square Garden, and the Kansas City Auditorium, see Figs. 88, 90, and 92.

For armories and gymnasiums not exceeding 80 feet in width the French truss, shown in Fig. 74, is an economical and desirable type.

Figs. 224 and 225 show a plan and elevation of the trusses and bracing of a drill hall, 80′ × 120′, roofed as in Fig. 74.

Fig. 103 shows one of the trusses used in roofing the Exposition Hall, at Providence, R. I., and Fig. 226 shows a plan of the roof framing.*

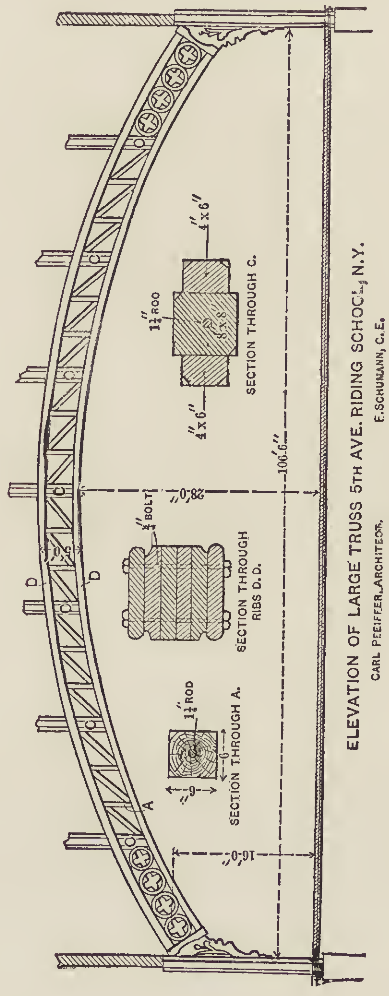


Fig. 222.

95. EXAMPLES OF ARMORY ROOFS. — As showing the quantities of steel required in roofs of from 82 to 118 feet in width, the following data, given by Mr. H. G. Tyrrell, C. E., is of much practical value:

*Figs. 74, 103, 224-226, are reproduced from drawings by Mr. H. G. Tyrrell, C. E., and published in the Architects' and Builders' Magazine for October, 1901.

"All of the following actual cases were proportioned for slate and plank roofing, resting on wood rafters 2 feet apart. Steel purlins occur about 10 feet apart. The unit stresses used were 12,000 and 15,000 pounds per square inch, in compression and tension, respectively. The trusses are all similar in general outline to that shown in Fig. 103, Chapter II. The spans given are center to center of side bearings, and are 4 to 5 feet less than the outside width of building.

"The assumed loads were as follows:

"For trusses, dead weight of roof and covering = 25 lbs. per sq. ft. of sloping surface.

"For purlins, dead weight of roof and covering = 18 lbs. per sq. ft. of sloping surface.

"Dead weight of snow = 10 lbs. per sq. ft. of sloping surface.

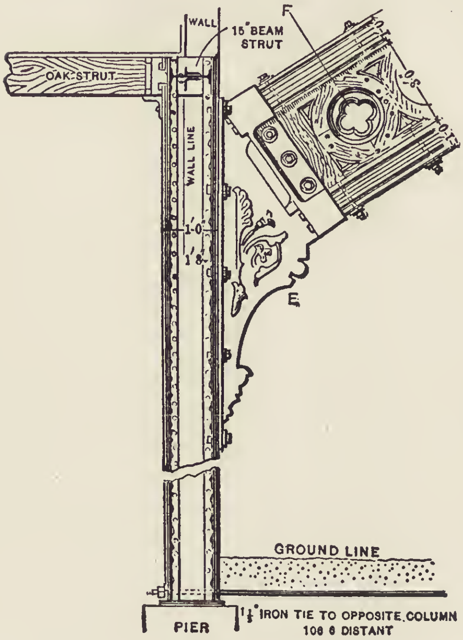


Fig. 223.

"Horizontal wind = 40 lbs. per sq. ft., = 28 lbs. normal."

PAWTUCKET ARMORY.—82 feet span. Length, 143 feet. Five main trusses, 24 feet apart. Pitch, 33°. Height to eaves, 16 feet. Height to ridge, 40 feet.

Quantities.	Lbs.
5 trusses	67,000
42 purlins	28,000
12 purlins	7,500
Bracing	6,100
5 ties	2,900
10 shoes	4,500
Total	116,000

This weight is equivalent to 8.7 lbs. per square foot of sloping roof surface.

PORTLAND ARMORY.—92 feet span. Length, 153 feet. Five main trusses, 25 feet apart. Height to eaves, 24 feet. Height to ridge, 50 feet.

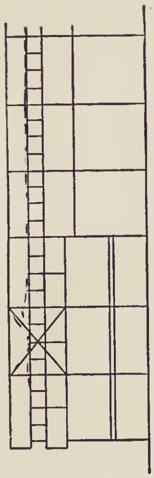


Fig. 225.—Elevation.

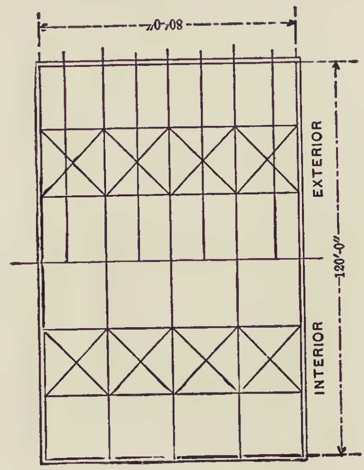


Fig. 224.—Plan.

Quantities.	Lbs.
3 trusses, at 17,860	53,580
2 trusses, at 19,700	39,400
6 cast shoes	2,100
4 cast shoes	1,400
3 tie rods	2,457
2 tie rods	1,980
28 purlins	19,100
8 "	9,600
18 "	22,400
8 "	5,184
4 "	2,876
44 struts in bracing	4,488
36 struts in bracing	3,300
72 rods	3,540
Total	171,400

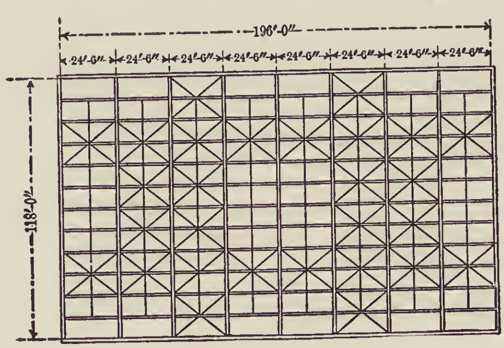


Fig. 226.—Plan of Roof Framing.

This weight corresponds to 9.7 lbs. per square foot of sloping roof surface, or 11.7 lbs. per square foot of ground covered.

The trusses in this case were built strong enough to carry a 13-foot gallery, on two sides and one end, to be added in future.

PHOENIX HALL, at Brockton, Mass., is 100 feet wide, and 144 long, outside. It has 5 main arches, 94 feet, c. to c. Distance between trusses is 24 feet. It is 33 feet high to eaves, and 67 feet to ridge. It has a gallery 17 feet wide. The only steel included for the gallery is the ten gallery brackets.

Quantities.	Lbs.
42 purlins	28,700
48 struts	6,400
Rod bracing	2,600
5 tie rods	4,680
5 arches	99,500
10 shoes	3,100
10 gallery brackets	6,060
Total	151,240

This weight is equal to 8.6 lbs. per square foot of sloping roof surface, or 10.6 lbs. per square foot of ground covered.

NORTHAMPTON ARMORY is 100 feet square. It has 3 main trusses, and 11 lines of trussed purlins. It has no gallery.

Quantities are as follows:	Lbs.
3 trusses, at 17,000	51,000
6 cast shoes, at 350	2,100
3 tie rods, at 780	2,340
44 purlins, at 670	29,500
Struts in bottom chord	5,200
Ties in bottom chord	2,100
Total	92,240

The sloping roof area = 11,600 sq. ft. ∴ Weight per sq. ft. = 92,240 ÷ 11,600 = 7.95 lbs.

THE PALACE AMUSEMENT CO.'S RINK, at Hartford, Conn., is 104 feet wide and 124 feet long. It has 4 main ribs 54 feet high, c. to c., pins. It is 24 feet high to eaves. It has a gallery 16 feet above the floor. There are 7 lines of trussed purlins. The gallery in this case is framed of steel, the main brackets being included with the trusses.

Quantities.	Lbs.
Trusses and rafters	132,400
Purlins	34,400
Rods	18,000
Total	184,800
Gallery	67,600

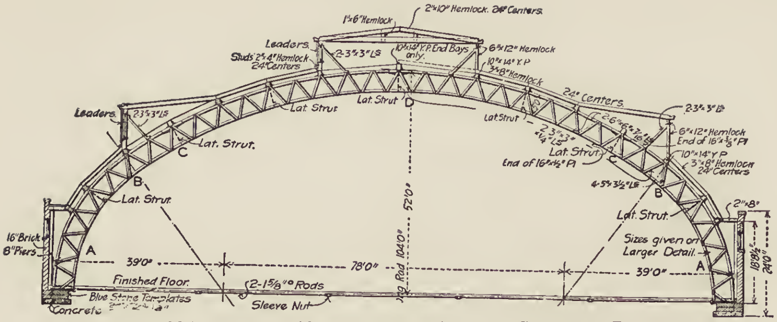


Fig. 227.—Section, 13th Regiment Armory, Scranton, Pa.

The total exposed roof area is 15,600 square feet. Hence the weight per square foot is:

Roof..................................	184,800 ÷ 15,600 = 11.8 lbs.
Gallery	67,000 ÷ 15,600 = 4.4 lbs.

PROVIDENCE EXPOSITION HALL.—This is 118 feet wide, and 196 feet long. It has 7 main trusses, 20 feet high to eaves. Figure 103 illustrates the design.

Quantities.	Lbs.
7 trusses, at 25,000....................	175,000
104 purlins, at 580....................	60,300
7 tie rods, at 1,100....................	7,700
Rod bracing on rafters....................	4,000
96 spacing struts, at 100....................	9,600
14 cast shoes, at 600....................	8,400
Total....................	265,000

This weight corresponds with 9.5 lbs. per square foot sloping, or 11.5 lbs. per square foot horizontal.

The following illustrations and descriptions give a good idea of the manner in which some of the largest armories and coliseums have been roofed:

96. DETAILS OF ARCHED ROOF OVER DRILL HALL OF THE THIRTEENTH REGIMENT ARMORY, SCRANTON, PA.*

The building consists of the usual administrative portion, about 56 feet deep, by 160 feet wide and of a drill hall covering the rest of the ground area and having an unbroken floor space of 35,000 square feet of asphalt. The side walls of the building are carried about 5 feet above the point where the roof meets them, and alongside this wall is a walk where soldiers can patrol from one tower to another, should it ever become necessary to defend the armory to that extent.

*From the Engineering Record, Aug. 24, 1902.

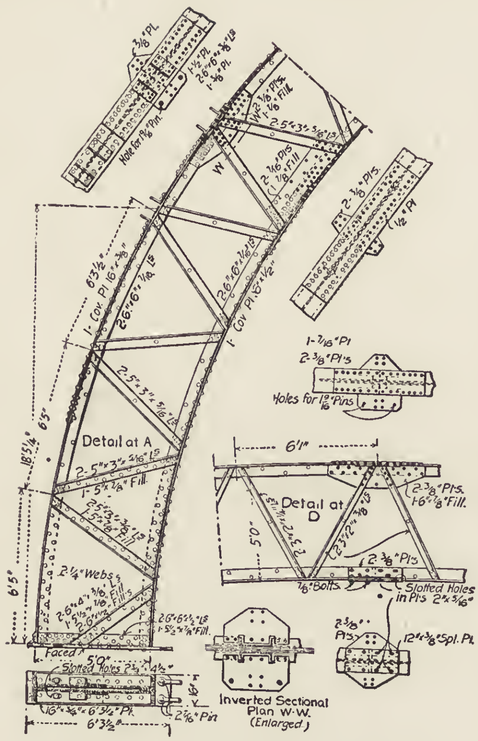


Fig. 228.—Detail of End Sections and Crown.

The drill hall has a roof supported by twelve steel trusses 20 feet apart on centers, which have a span of 156 feet, out to out, and a rise of 52 feet to the top of the upper chord. The trusses are hingeless arches, Fig. 227, with horizontal square lower ends seated on top of the abutment piers and connected by horizontal tension members under the floor. The parallel top and bottom chords are 5 feet apart, out to out, and are false ellipses, composed of circular arcs, described from three centers with a long radius of 104 feet and two short radii, of 39 feet each. The trusses are wholly within the side walls and are seated, just below the floor level, on extensions of the concrete wall footings capped with 2 x 6-foot blue stone pedestals. They are connected by seven lines of longitudinal lattice girder struts 5 feet deep with their webs in radial planes, which are field riveted to lateral connection plates on the top and bottom chord flange angles. The roof trusses are also braced together in alternate panels by lateral diagonal rods in the planes of the top chords, with pin connections and sleeve-nut adjustments.

Both chords of the arch trusses are made throughout of pairs of $6 \times 6 \times {}^7/_{16}$-inch angles riveted together back to back and reinforced with 16-inch flange cover plates from the haunches to the springing line. The panel lengths are nearly uniform, varying from about 4 feet 8 inches to 5 feet 10 inches, and all web members are pairs of zigzag angles from $5 \times 3 \times \frac{3}{8}$ inches at the skewbacks to $3 \times 2 \times \frac{3}{8}$ inches at the crown. Except where the truss sections are field

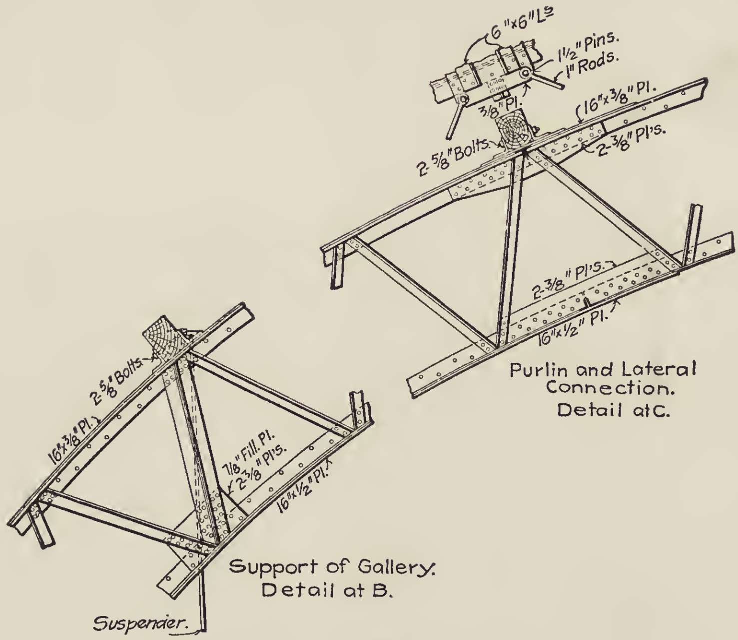


Fig. 229.

spliced, all web members are riveted directly to the vertical flanges of the chord angles without connection or gusset plates. Each truss was shipped from the bridge shops in six sections, which were field riveted together before erection. The splices were made at the panel points in the top chords and in the middles of the opposite panels of the bottom chords, and the joints were covered with flange and double web plates.

The end sections of the trusses are about 20 feet long and have a solid web plate in the two lower panels, as shown in the detail, Fig. 228. The $\frac{3}{4}$-inch planed base plate has two slotted holes for large anchor bolts, and projects beyond the inside of the truss and its pedestal to receive the connection pins for the two $1\frac{5}{8}$-inch horizontal bottom tie rods, which are made in five lengths connected by sleeve nuts. The base plate rests on a 1-inch bed plate nearly 4 feet long, and the anchor-bolt nuts are screwed down tight on ring washers which bear on the bed plates and clear the holes in the base plates, thus preventing any pressure or friction on the base plates. The lateral connection plates are in two pieces at each splice, riveted to the under side of the horizontal flanges of the chord angles, and are notched to clear the web members of the truss as shown in the detail sketch. At the crown splice the top chord joint is riveted solid, but the bottom chord splice is made with cover plates which are riveted fast to one section of the truss and have slotted holes for bolts in the other section so as to allow for temperature displacements. The roof is proportioned for a wind load of 30 pounds per square foot of the vertical projection, and the main trusses are designed for an additional load of 20 tons at each end from the gallery floor, which is calculated for a uniform loading of 150 pounds per square foot.

On every truss, at the sixth panel point from each end, there is riveted to the top chord a cast saddle from which are suspended two $1\frac{1}{2}$-inch vertical pin-connected rods with sleeve-nut adjustments. These rods clear each side of the truss and are bent to join each other under the bottom chord, where they are pin-connected to a single rod $2\frac{1}{8}$ inches square, which supports the side gallery. Pairs of short 6 × 6-inch angles are riveted across the top chord at intervals of about two panel lengths, and between them are secured by two bolts through their vertical flanges, 10 × 14-inch yellow pine purlins, about 12 feet apart. These purlins support 3 × 8-inch rafters 24 inches apart on centers, which are sheathed with matched $1\frac{1}{2}$-inch spruce boards planed and beaded on the under side. The boards are covered with five-ply slag roofing, laid by the Warren Ehret Company, except where the slopes are steep and in the gutters, where they are tinned, with standing seams.

97. ROOF OVER THE DRILL HALL OF THE FIRST REGIMENT NATIONAL GUARD ARMORY, NEWARK, N. J.*—This drill hall is 170 × 250 feet outside and about 86 feet in extreme height above the tops of the piers.

The framework is entirely of steel with seven main roof trusses 26 feet apart, and two 45-degree hip trusses [see Fig. 230]. These trusses and the four secondary hip trusses are all seated on brick piers with concrete footings which, for the main trusses, are 12 feet square and 3 feet thick. Each of the main piers has four $2\frac{1}{4}$-inch bolts 10 feet long attached to large steel plates bedded in the concrete, and has a granite capstone 5 feet square and 18 inches thick.

The main roof trusses are riveted three-hinged arch ribs of $163\frac{1}{2}$ feet span and 73 feet $5\frac{5}{8}$ inches rise, center to center of pins, the general shape of the trusses being as shown in Fig. 232. The lower

*From the Engineering Record of May 26, 1900.

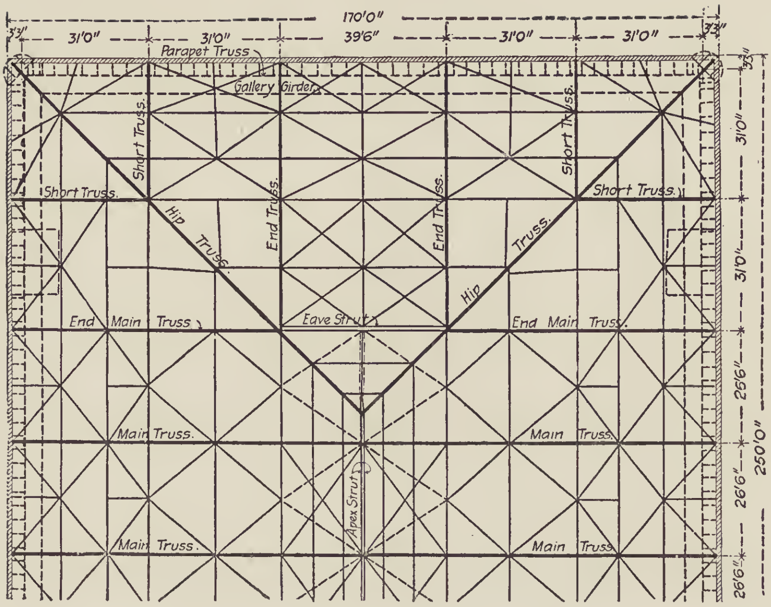


Fig. 230.—Half Plan of Roof, Newark Drill Hall.

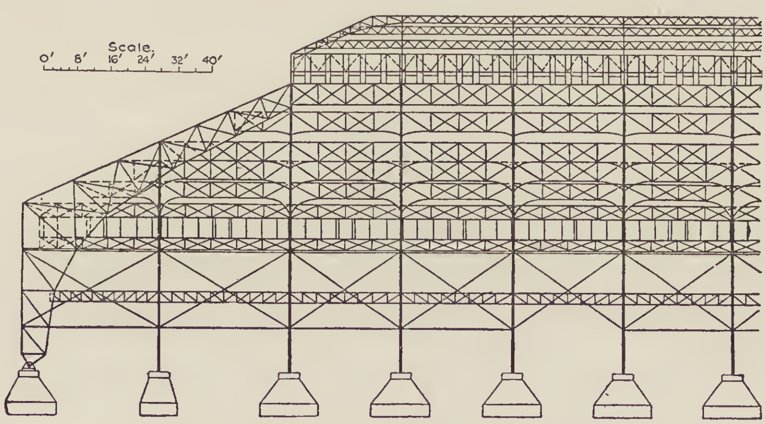


Fig. 231.—Half Elevation of Drill Hall Framing.

chord of the roof truss is approximately a parabolic line having its axis horizontal and about 3 feet below the centers of the end pins. The top chord is vertical to the roof line and thence is straight for a short distance and curves with a long radius to near the apex. This outline was designed with great care to avoid the tendency of the top chord to approach the bottom chord and form a neck below the apex. Each chord is made of two web plates, two angles and one or more cover plates. Each web member is made of two or four angles, latticed. All rivet holes were reamed, and where connections were bolted, finished bolts were used. The hinge pins are all of hard steel, 7 inches in diameter, and are locked in place to the ribs and pedestals by pairs of jaw plates with full holes on each member. The end pins of each main truss are connected together by a pair of horizontal round bars $2\frac{3}{4}$ inches in diameter with upset ends and sleeve-nut connections. These bars are supported at frequent intervals by iron stands on brick piers, and they are connected to the end pins by short sections of $5\frac{1}{4} \times 1\frac{1}{4}$-inch eye-bars. The hip

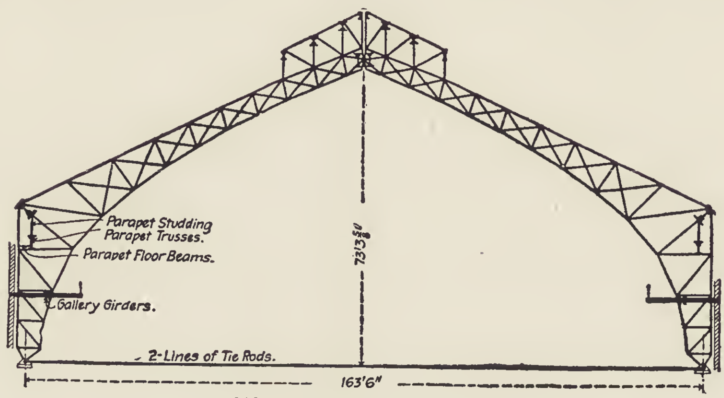


Fig. 232.—Diagram of Main Trusses.

trusses are similar to the semi-trusses of the main arch ribs, but have no bottom horizontal ties.

The main roof trusses are connected by twelve lines of purlin trusses and six lines of lantern purlins, including the double apex line which is covered by a ridge roll connecting the opposite halves of the lantern frames. There is a system of diagonal lateral rods in all panels of main trusses and purlins, in the planes of the top chords, and there are longitudinal lattice girders and diagonal angles in the vertical panels between the ends of the main trusses. Both ends of the main trusses are fixed to the pedestals and their anchor bolts are proportioned for the shear from wind strains. Temperature movements of the trusses are assumed to be compensated by the rise and fall of the crown. Wooden rafters, parallel to the top chords are carried on the purlins and receive the sheathing boards which are covered with slate. Photographs of the trusses and additional details are published in the Engineering Record.

98. THE CHICAGO COLISEUM ROOF.*—The Chicago Coliseum is about 160 feet wide and 302 feet long, and has a steel roof with eleven three-hinge arch trusses of the outline and dimensions shown by Fig. 233. The trusses are from 22 feet 10 inches to 25 feet apart, and are connected together by eight main lines of lattice-girder longitudinal struts, by the lateral diagonals, by the lantern framework, and by the beams supporting the side wall and galleries. The trusses are set entirely within the side walls, which rise several feet above the hips and support a line of dormer windows reaching back to the slope of the main roof. The

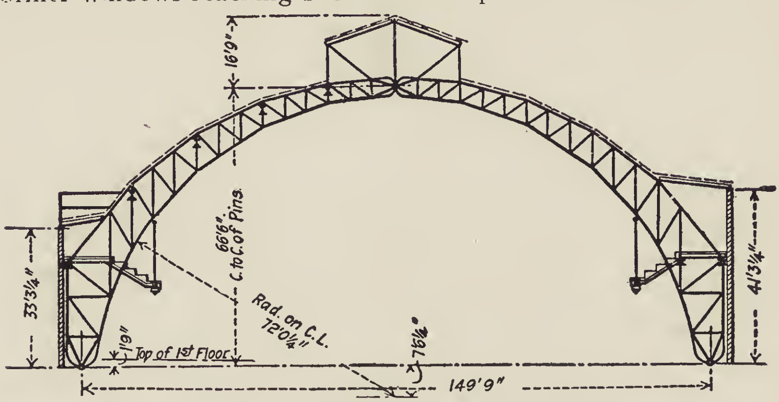


Fig. 233—Elevation of Truss; Chicago Coliseum.

end pins have semi-circular bearings in cast-iron pedestals on masonry piers, below the first floor level, and there is a lantern about 30 feet wide and 15 feet high which extends from end to end of the building on the center line of the crown pins. A gallery which extends around the four walls is 20 feet wide on the sides and about 30 feet wide at the ends, with octagonal inner sides 20 feet long at the corners.

At the walls the gallery is 25 feet high, level with the hips of the trusses, and for a width of ten feet the floor is horizontal and supported directly from the truss members. Beyond this width the floor slopes inward and downward to a fascia girder about 18 feet above the main floor and suspended from the main trusses. Across the ends of the building, about 20 feet of the width of the gallery is horizontal and supported by the gable walls and by transverse rows

*From the Engineering Record of June 29, 1901.

of columns. The arch trusses are made with the bottom chords approximately conforming to the chords of an arc of 72 ft. radius. The top chord corresponds to a larger radius, and is vertical to a height of about 23 feet above the end pins. Both chords are made throughout with two 12-inch channels, latticed, those in the top chord having a weight of 20½ pounds per foot everywhere, and those in the bottom chord varying from 20½ pounds at the crown to a maximum of 40 pounds in the second section above the bottom. In the end panels of the semi-trusses the chords converge to intersection on the hinge pins, Fig. 234, but their outline is continued by pairs of curved angles and cover plates which make semicircular rounded ends. The web members are pairs and double pairs of angles, which are 5 × 3 inches up to the second panel above the hips and 3 × 3 inches above that point. All of them are shop riveted to double gusset plates on the webs of the chord channels. In each semi-truss there are thirteen vertical members about 5¼ feet apart, which vary in length from about 20½ feet to 4 feet on centers of chords. On the west side of the building horizontal brackets are riveted to the vertical sections of the top chord to support the wall beams, and on all trusses there are pin connections at the fourth panel point of the lower chord for the 1¾-inch vertical suspenders for the gallery beams.

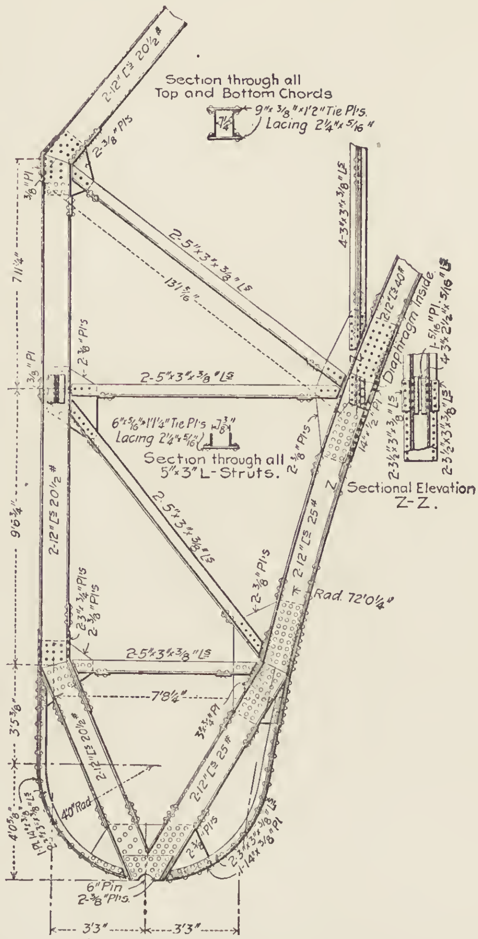


Fig. 234.—End Hinge of Truss.

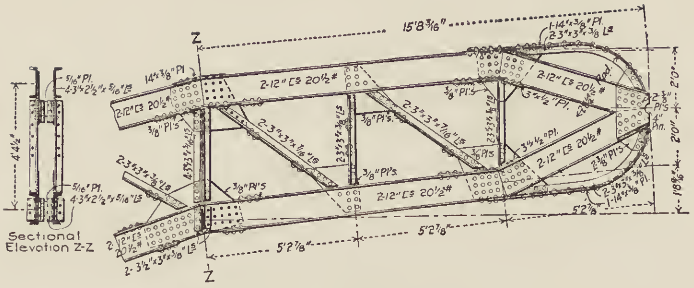


Fig. 235.—Center Hinge of Truss.

The longitudinal struts between the trusses are lattice girders 2 feet deep, having pairs of $3 \times 3 \times \frac{3}{8}$-inch chord angles and single $2\frac{1}{4} \times 2\frac{1}{4} \times {}^{5}/_{16}$-inch zigzag web angles. They are web connected to the upper ends of the vertical members in the trusses, and have 3×3-inch knee-brace angles to the lower chords at both ends. There are longitudinal struts at the top and bottom of the middle vertical posts in the lantern, and there are sway-brace diagonal rods in their vertical planes. The main trusses are braced together from end to end of the building by a system of lateral diagonals in the middle panel of the top chords, and there are lateral rods in every panel between the last two trusses at each end of the building.

The building was erected in the summer of 1899.

After all the trusses had been swung, and before all their braces had been completed, while the traveler was being taken down, the whole of the steel framework of the building fell to the ground, killing and injuring a number of workmen, as described in the Engineering Record of September 16, 1899. The building was reconstructed according to the original designs as soon as possible after the accident. Messrs. E. C. & R. M. Shankland, Chicago, were the engineers of the steel construction and the Pittsburg Bridge Company was the contractor for its manufacture and erection.

99. THE ST. LOUIS COLISEUM.—The Coliseum at St. Louis, Mo., is a rectangular brick building, 189×318 feet in size, intended for amusement purposes and as a place of popular assembly. The general construction of the building was described by Mr. E. W. Stern, M. Am. Soc. C. E., in a paper before the Engineers' Club of St. Louis, an abstract of which was published in the Engineering Record of Oct. 1, 1898. The building has an oval main hall, 186′ 2″ by 298′2″ in size, roofed by arched spans and unobstructed by col-

umns. Between the rounded ends of the steel framework of this auditorium and the outer brick walls are floors with iron columns and girders. The auditorium has a large central ground floor surrounded by ascending tiers of seats around the circumference of the structure, and there is a second inclined tier of seats forming a gallery. The arrangement and principal dimensions of the main trusses, girders, etc., is shown by plan and elevation diagrams in Fig. 236 and 237, respectively. The four main trusses are three-hinged arches, with the lower chord panel points above the floor line in the curve of a true ellipse. The end radial trusses correspond essentially to the semi-trusses of the main arch except for their top connection, where their top chords are attached to a semicircular frame supported by the end main trusses and designed to receive thrust, but no vertical reaction.

The roof covering of asphalt composition is laid on $1\frac{3}{8}$-inch boards, resting on wood joists $2\frac{1}{2} \times 16$ inches, 3 feet apart and ceiled underneath. These, in turn, are carried by the steel purlins of the structure, which are spaced about 16 feet apart. The gallery floor beams are carried on stringers of 8-inch channels spaced 3 feet 8 inches center to center, carried by girders running between, and supported by the arches. The rear stringer is a plate girder; the front one is a latticed girder, the gallery beams running through the latter and cantilevered out 5 feet 4 inches.

The loads, in accordance with which the trusses were figured, are as follows:

CASE I.

Wooden deck and gravel of roof	17.5 lbs. per sq. ft. vertically.
Steel	12.5 lbs. per sq. ft. vertically.
Snow and wind	25.0 lbs. per sq. ft. vertically.
Total	55.0 lbs. per sq. ft. vertically.
Add for floors, viz.:	
Main floors, banks and galleries	105.0 lbs. per sq. ft. vertically.
Attic floors	60.0 lbs. per sq. ft. vertically.

CASE II.

Wooden deck and gravel of roof	17.5 lbs. per sq. ft. vertically.
Steel	12.5 lbs. per sq. ft. vertically.
Snow	10.0 lbs. per sq. ft. vertically.
Total	40.0 lbs. per sq. ft. vertically.
Wind pressure over entire elevation of wall and roof of	30.0 lbs. per sq. ft. horizontally.

LOADS ON PURLINS.

Wooden deck and gravel of roof	17.5 lbs. per sq. ft. vertically.
Steel	3.5 lbs. per sq. ft. vertically.
Snow and wind	25.0 lbs. per sq. ft. vertically.
Total	46.0 lbs. per sq. ft. vertically.

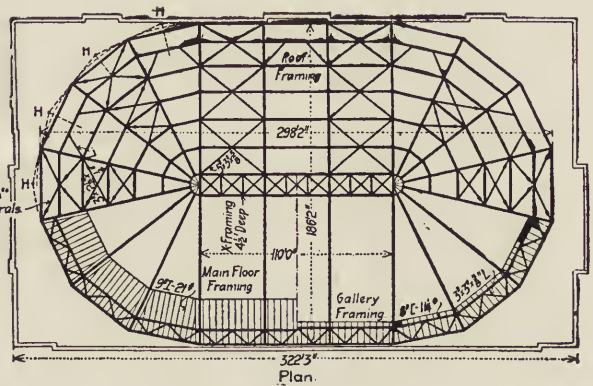


Fig. 236.—St. Louis Coliseum.

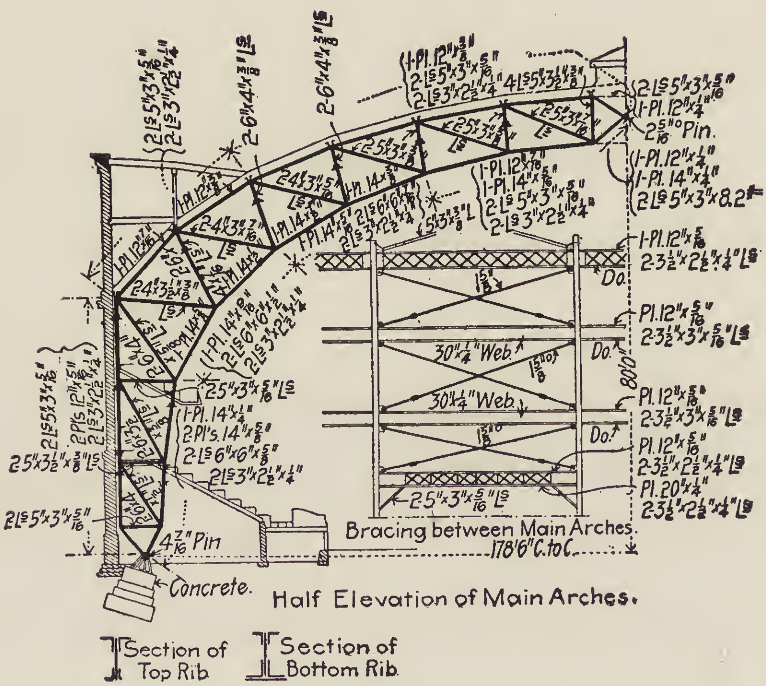


Fig. 237.

For the main trusses, in addition to the stresses of Case II., there was added the stress due to the wind bracing between these trusses.

For the radial trusses, in addition to loading of Case II., there was assumed an additional load of 50,000 pounds supposed to act up or down at the upper point of truss; this load being what was assumed probable in case there was slight unequal settlement of the footings.

For the half ring connecting the tops of the radial trusses there was another case assumed, besides Cases I. and II., viz., a thrust of 50,000 pounds at any point of the half ring; this being the thrust of a radial truss under its full live and wind load.

All the material used was of medium steel, excepting the rivets, which were made of soft steel. Both material and workmanship conform to manufacturer's standard specifications.

UNIT STRAINS.

Tension	16,000 lbs. per sq. in.
Compression, for lengths of 90 radii or under	12,000 lbs. per sq. in.
Compression for length of over 90 radii	17,100-57 l ÷ r.
Combined stress due to tension or compression and transverse loading	16,000 lbs. per sq. in.
Shear on web plates	7,500 lbs. per sq. in.
Shear on pins	11,000 lbs. per sq. in.
Shear on rivets	10,000 lbs. per sq. in.
Bearing on pins	22,000 lbs. per sq. in.
Bearing on rivets	20,000 lbs. per sq. in.
Bending, extreme fiber of pins	25,000 lbs. per sq. in.
Bending, extreme fiber of beams	16,000 lbs. per sq. in.

Lateral connections have 25 per cent. greater unit strains than the above.

In Case II. of trusses, the above unit strains were increased one-third.

The purlins are triangular trusses 4½ feet deep, made of angles. The bracing between main arch trusses terminates at the bottom with heavy portal struts of triangular box sections. The lateral rods are not carried to the ground on account of the obstruction they would make. The radial trusses are coupled together in pairs with lateral rods down to the ceiling line. The thrust due to wind is transmitted from them into the line of girders around the structure at this point, and into the adjoining systems. The compression ribs of the main and radial arches are stayed laterally by angle iron ties, connecting to the first panel-point in the bottom chord of the purlins. In the planes of the first diagonal braces of the trusses above the haunches, diagonal rods connect the bottom ribs of the trusses to the upper ribs of the next trusses. No struts were used between the bottom chords, as they would have been directly in the line of vision from the rear gallery seats to the farther end of the arena. The front and rear girders supporting the gallery and main floor beams are tied together with a triangular system of angle iron bracing.

To provide for expansion, the radial purlins and all the girders between the arches have slotted hole connections in every alternate bay. The diagonal rods between the two lines of ridge purlins were tightly adjusted on a hot day. To prevent secondary strains in the half ring to which the radial trusses are connected at their tops, there is $^1/_{16}$-inch clearance in all the pin holes. There is also clearance between the pin plates, so that the trusses and the ring can slide a little sideways on their pins. The lines of the arch trusses were laid out full size and the principal points checked by independent measurements in the template shop, and the work was accurately assembled.

The total weight of iron in the entire structure was 1,905,000 pounds, as follows: Main arches, each, 64,000 pounds; radial arches, each, 21,000 pounds; main floor stringers, each, 810 pounds; purlins between main trusses, each, 1,450 pounds; balcony floor stringers, each, 280 pounds; cast shoes, each, 3,000 pounds. There were 4,188 days' labor spent on the work in the shop and 3,550 days' labor during erection, the average number of men in the erecting force being about 50. The stress diagrams and detail plans of the steel frame were made under the supervision of Mr. Stern, in the office of the Koken Iron Works, who were contractors for the ironwork, and were submitted for approval to the consulting engineer, Mr. Julius Baier, Assoc. M. Am. Soc. C. E. Mr. C. K. Ramsey was the architect of the Coliseum and Mr. L. H. Sullivan was the consulting architect.

TRAIN SHEDS.

100. For the roofing of train sheds, two distinct systems are in vogue. The first and more expensive method is by means of a curved steel roof, supported on high arched trusses and covering the entire area, without intermediate supports.

The second method is to use short flat trusses, supported by one or more longitudinal rows of interior columns, as in Fig. 238.*

The first method has the advantage of allowing the tracks and platforms to be arranged in any way, involves no waste floor space,

*For Description see Engineering Record of March 19, 1898.

avoids any obstruction from intermediate columns, and presents an imposing interior. On the other hand, the long-span roof trusses are heavier and costlier than shorter ones supported on columns and covering the same area. It is difficult to keep the skylights and upper steelwork clean, and the curved roof sheathing is very expensive.

The second method requires less steel for a given length and

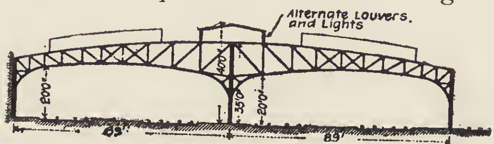


Fig. 238.—Train Shed at Providence, R. I.

width, the roof may be more quickly and cheaply erected, and the shed is more easily lighted, and it is easier to keep the skylights and steel work clean. Such roofs also lend themselves about as well to an attractive exterior treatment.

Notable examples of train sheds roofed by one span, are those at Pittsburg, Jersey City and Philadelphia for the Pennsylvania R. R.; at Philadelphia, for the Philadelphia & Reading R. R.; at Buffalo, for the N. Y. C. & H. R. R. R.; at Cleveland, for the Lake Shore R. R., and in the old depot of the Providence R. R. at Boston.

Examples of very large train sheds roofed on the second method are the Union Depot at St. Louis, the South Terminal Station at Boston (described in the Engineering Record of Jan. 14 and 21, 1899), and the station of the Pennsylvania Railroad at Camden, N. J., described in the issue of Sept. 14, 1901.

101. DESCRIPTION OF THE TRAIN SHED FOR THE PENNSYLVANIA R. R. at PITTSBURG, PENN.—The train shed for the new depot of the Pennsylvania Railroad at Pittsburg is about 555 feet long, 260 feet wide and 110 feet high over all. It is one of the largest in this country, and will have sixteen tracks with platforms between them, each track having a capacity for one twelve-car train or two five-car trains. The design of the shed is almost identical with that of the company's Jersey City terminal, and the dimensions are approximately the same, the two structures being practically alike except for some modifications of details.

The following description of the steel trusses and roof framing is taken by permission from the Engineering Record of Aug. 23, 1902:

The principal members of the framework are twenty-four three-hinge arch trusses, which are spaced alternately 9 feet and $40\frac{1}{2}$ feet apart on centers, and are braced together in pairs and connected by longitudinal girders and trussed purlins. The outline and general dimensions of the trusses are shown in the diagram elevations, Fig. 239, and the lateral bracing in the plane of the top chords is shown in the roof plan, Fig. 240. There are twenty-two lines of purlins which are lattice-girders in the planes of radial truss members. The short panels between the trusses of each pair have $\frac{3}{4}$-inch square pin-connected diagonal rods in every panel of the top chords and purlins, and the long panels between the pairs of trusses are braced by diagonal rods $1\frac{1}{4}$ inches square, which extend across one or two intermediate purlins.

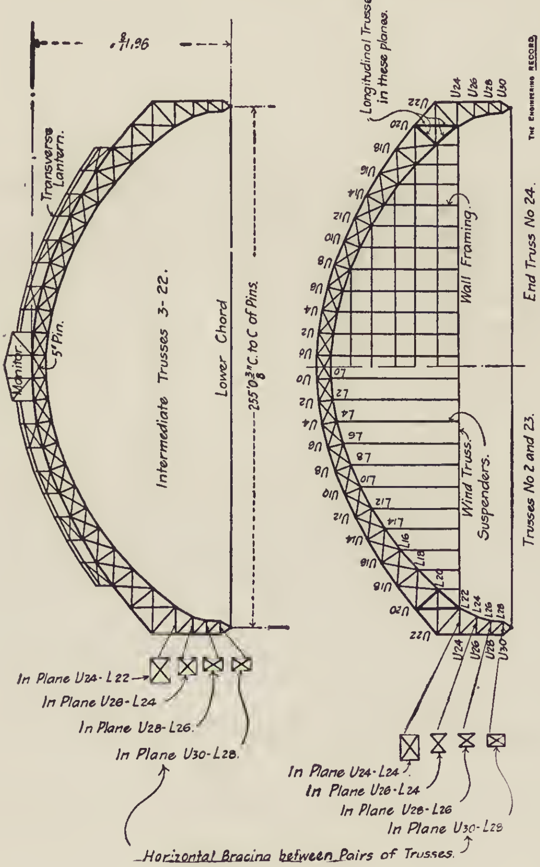


Fig. 239.—Outline of Trusses.

Each end of the building is braced against lateral and wind strains by a lattice-girder in a horizontal plane through the fourth panel point above the end pin of the arch truss, about 25 feet above the ground. Horizontal struts and X-braces at all the panel points up to the hip connect the trusses of each pair, and are made of angles with riveted connections. The vertical ends of the trusses up to the hips, about 35 feet above the end pins are braced on each

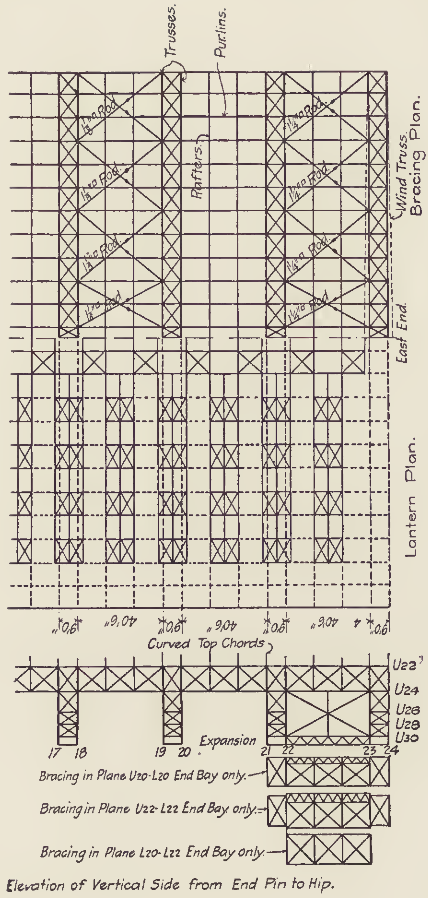
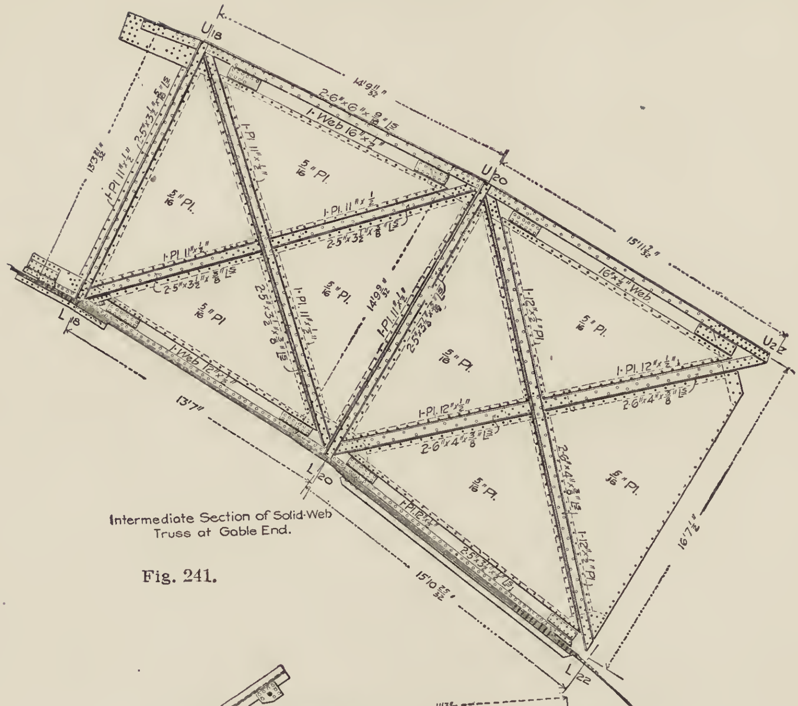


Fig. 240.—Roof Plan.

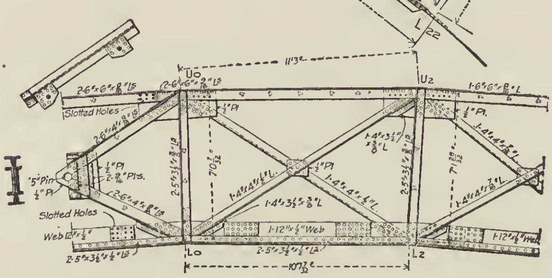
side of the building by a continuous line of lattice-girders about 12 feet deep, which reach from the hip to the next panel point below. Besides this the wall panel between the end two pairs of trusses is braced by a horizontal longitudinal lattice-girder at the bottom and by diagonal rods, and the vertical posts in each pair of trusses from the end pin up to the hip are braced with horizontal struts at panel points and X-brace angles in each panel thus formed (see Fig. 240). Besides these braces, each pair of struts is additionally braced at the hips by longitudinal horizontal struts and diagonals in the three planes of the web members, which are indicated by heavy lines in the elevation of the arch truss, Fig. 239.

There is a monitor with clerestory windows on the center line of the roof, which extends to within about 10 feet of each gable, and there is a transverse lantern 8 feet high and 15 feet wide over each pair of trusses except the end pairs, and in the center of each panel between the pairs. Both monitor and lanterns have opaque roofs and glazed sash on horizontal pivots in the sides. The purlins support two intermediate jack-rafters parallel to the top chords of the arch trusses in each panel, and these carry longitudinal I-beams not shown in the roof diagram, on which are sheathing boards covered with copper. Both gable ends of the building are open for a height of about 25 feet, up to the horizontal wind truss, and above that are closed with flat plates and corrugated galvan-



Intermediate Section of Solid-Web Truss at Gable End.

Fig. 241.



Part of Crown Section of Intermediate Truss.

Fig. 242.

ized iron on angle iron framing; the latter is supported by vertical and horizontal struts attached to the suspenders which carry the wind truss from the lower chord of the end truss. These members divide the space into rectangular panels from 10 to 15 feet wide and high.

The arch trusses have a rise of 93 feet and span of 255 feet on centers of pin, and have a clear height of about 87 feet above the rail base. They are alike in the intermediate and end panels except that in the latter the weights of the angles are lighter and the connections vary. They are 7 feet deep at the crown and about $6\frac{1}{2}$

feet deep on a radial line at the hips. Each semi-truss was shipped in six sections of two or more panels each, varying according to the depth of the truss and the length of the panels. The adjacent sections have their chords spliced with field-riveted cover plates like ordinary lattice-girder bridge work, except at the crown. This joint is made in the center of a panel and the diagonals are made extra heavy to carry the chord stresses to the 5-in. center pin, which engages reinforced jaw plates locking the two sections together (see Fig. 242). The abutting vertical surfaces at the pin are milled to ¼-inch clearance, and the top and bottom chords are spliced with bolts through slotted holes in the webs to allow for temperature distortions. The chords are straight and slightly divergent between panel points, and each chord is bent to a slight angle at every panel point.

From Uo to U22, eleven panels down from the crown to the hip, the truss is made similar to the panels shown next to the crown. From the hip to the end pin the four panels were shop riveted complete, the top chord is replaced by an intersecting vertical post in the plane of the exterior wall and the radial web members are replaced by vertical ones. The chord web plates also disappear in the two lowest panels and the members are lighter and proportioned chiefly for direct vertical loads. Connections not shown in the drawing are made to the horizontal truss members for the longitudinal and diagonal braces to the next trusses. In the gable trusses 11 × ½-inch plates are riveted between the pairs of angles in the web members and, projecting beyond both edges of them, make flanges to which and to the inner edges of the chord web

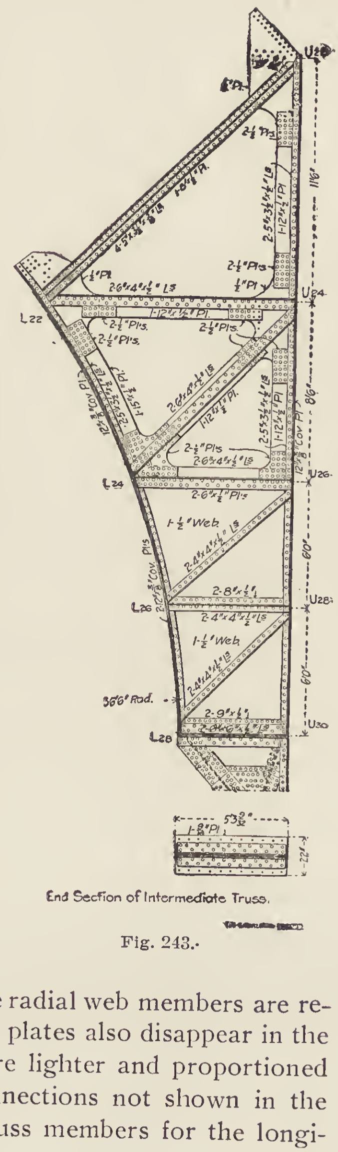


Fig. 243.

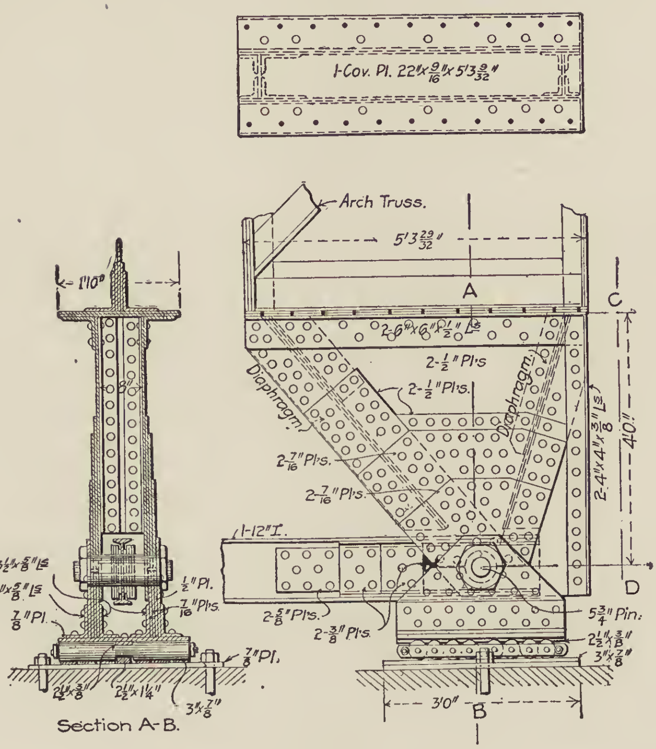


Fig. 244.

plates, $^5/_{16}$-inch solid web plates are field-riveted, as shown in the drawing of section U18, U22, Fig. 241 to form a wall surface, closing the whole area of the truss.

A double web shoe is field-riveted to the horizontal lower flange at the end of the truss (Fig. 244), and has a semi-cylindrical bearing and jaw plates engaging the lower hinge pin and locking it to the pedestal. The web plates of the shoe are heavily reinforced and are connected by two oblique transverse diaphragms, which converge from the feet of the truss chords to the pin centers. The pin receives the end of the horizontal lower chord or tie which takes the thrust from the foot of the arch truss. This chord is a single 12-inch 100-pound I-beam with the web reinforced to $5\frac{1}{2}$ inches in thickness for the pin bearing. It crosses the train shed in a closed trough under the floor level, and is composed of 30-foot sections spliced with sixty-two $\frac{7}{8}$-inch field rivets through double web and single flange cover plates.

The shoes are alike at both ends of the trusses, but the pedestal at one end has a center rib riveted to the base plate to lock and guide it on a nest of six 3-inch rollers 2 feet long, which travel on 3 x 7-inch bearing strips on a ⅞-inch bed plate 30 inches square. At the opposite end of the truss the fixed pedestal is like the roller one, except that it is about 5 inches higher to compensate for the absence of rollers, and is seated directly on the pier masonry, to which it is anchored by two 2-inch bolts upset to 2¾ inches. The foundations of the pedestals are shown by Fig. 245.

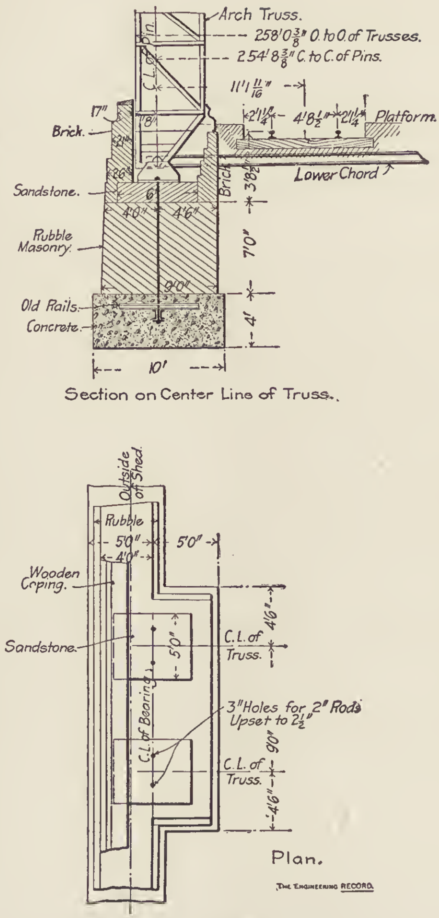


Fig. 245.

The train shed was designed in the engineering department of the Pennsylvania Railroad Company, Mr. W. H. Brown, chief engineer. The drawings were made and the work executed under the direction of W. A. Pratt, M. Am. Soc. C. E., now assistant to the chief engineer. Mr. Geo. C. Clarke was the assistant engineer in charge of construction. The steel work weighs about 2,350 tons, and was made by the Edgemoor branch of the American Bridge Company and erected by the employees of the Pennsylvania Railroad Company in charge of Mr. A. Braun.

EXPOSITION BUILDINGS.

102. Buildings of this class are usually large, one-story buildings consisting of floor, walls and roof, and with one or more level galleries around the outside walls. Posts in such buildings are not as objectionable as in the other class of buildings considered in this

chapter, but it is generally desirable to keep the center clear of columns. At the Columbian Exposition, 1893, nearly all of the main buildings were roofed by means of steel trusses and purlins. The roof construction of many of the buildings is described and illustrated in Volumes XXVI. and XXVII. of the Engineering Record. Figs. 101, 102 and 111, Chapter II., show the types of trusses used in three of the buildings.

The buildings at the Expositions held at Omaha in 1898-9, and at Atlanta, Ga., in 1895, were entirely roofed with timber trusses. Most of the buildings of the Exposition held at Buffalo in 1901 were constructed with wooden trusses, and the main buildings of the Louisiana Purchase Exposition, held in 1904 at St. Louis, had wooden trusses.

The type of truss most commonly used where the buildings have been roofed with wooden trusses is that of the Howe Truss, as this is the most economical and practical truss for such buildings. Details of the roof construction of the Horticultural and Forestry Building, the Agricultural Building and of the Music Hall at Buffalo, all wooden structures, are published in the Engineering Record of Aug. 24, Jan. 12, and Feb. 9, 1901.

In roofing large empty buildings of this character, the principal consideration, aside from the strength of the trusses, is to secure transverse stiffness to resist racking under wind pressure. With a wooden roof, this is best secured by an arrangement of posts and trusses, such as is shown in Fig. 246. By using double rows of posts on the outside, they can be braced so as to give great transverse stiffness, and they also reduce the span of the main truss, while the inner rows of posts may be utilized for the support of galleries. As an example of an economical and desirable construction for large exposition buildings with a wooden roof, the author has chosen the Agricultural Building at Buffalo (1901), the description being taken from the Engineering Record of Jan. 12, 1901.

ROOF OF AGRICULTURAL BUILDING, PAN-AMERICAN EXPOSITION.

103. This was one of the four largest buildings of the Exposition, being 150 feet wide and 500 feet long. The framework is supported on columns arranged in four longitudinal rows, as shown in Fig. 246, thus giving a clear central hall 97 feet wide and side aisles each about 17 feet wide, between columns. Two of the four

longitudinal rows of columns are 12 × 15½ inches in cross section and the other two are 14 × 40 inches, enlarged sections of the columns being shown by Fig. 247.

The small columns are built up of Norway pine planks laid flat and breaking joint every 3 feet, each having two ⅞-inch staggered bolts 12 inches apart, one through the column on each side of each joint. The edges of the planks are protected by a plank on each side of the column spiked to every plank with sixty-penny spikes

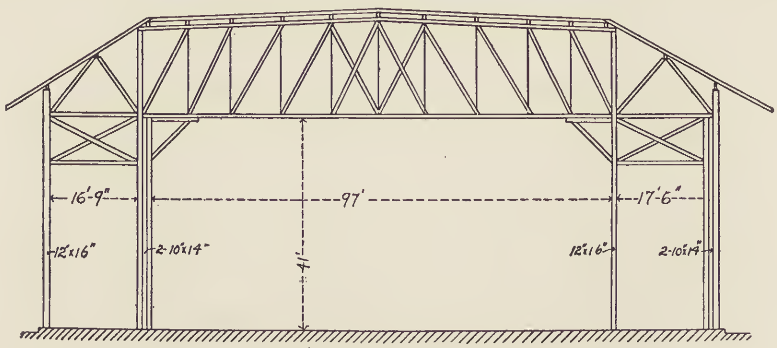


Fig. 246.—Transverse Section.

18 inches apart. Each of the large columns has two principal members each similar to the cross-section of the small column, and spaced 26½ inches apart on centers on the plane of the roof trusses. The separate members of each column are united by solid diagonal sheathing of 2 × 12-inch boards, inclined in different directions on

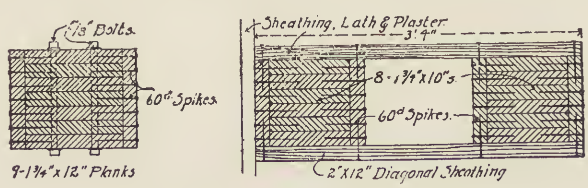


Fig. 247.—Enlarged Sections of Posts.

the opposite sides of the column and secured by six sixty-penny spikes in each end of each board.

The main roof trusses are supported by the intermediate rows of columns, a small column at one end of each truss and a large column at the other end. One member of the large column is cut off square to form a seat for the end of the lower chord, and the other member is continued up to form a seat for the top chord. At the other end of the truss the small column is continuous to the seat for the top chord, and a special seat is keyed and bolted to it on the inside to receive the bearing for the lower chord. The truss is extended at each end to connect with the outside column and the inner column is knee-braced to the lower chord and X-braced to **the** outside column, as shown in Fig. 248, thus forming a rigid

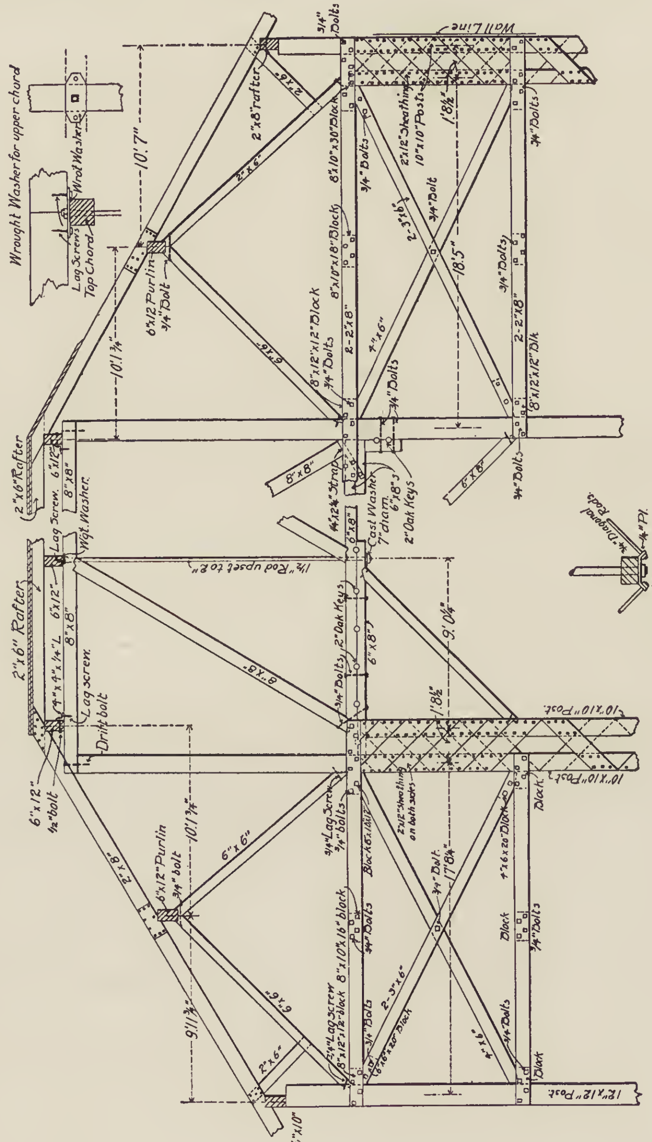


Fig. 248.—Detail of Bracing Between Outer Posts.

transverse framing to stiffen the columns against flexure and to resist wind stresses.

The trusses are made with 10-foot panels and a depth of 13 feet on centers. All compression members are of long-leaf southern yellow pine, and tension members are medium steel round rods with screw ends, most of which are upset. The trusses are seated on framed brackets which project from the intermediate columns and are keyed and bolted to the whole length of the end lower chord panels so as to virtually form long corbels and stiffen the trusses or have some effect of reducing the span.

Excepting the counters in the middle panels, all top-chord and diagonal members are single timbers nearly square in cross-section. At the middle point of the top chord, there is a reinforcement piece keyed and bolted to the under side to receive the diagonal members; at the adjacent top chord panel points and at the middle bottom chord panel points there are cast-iron angle blocks for the diagonals; elsewhere the diagonal members are notched or tenoned into the chords. The bottom chord is lap-jointed in the middle, where it is spliced with two $8 \times {}^5/_{16}$-inch steel splice plates 46 inches long with twenty-four $1\frac{1}{8}$-inch bolts. All the nuts on the lower ends of the vertical rods take bearing on standard cast-iron washers. The rods pass through holes bored through the chords and their upper ends are received in holes countersunk in the under sides of the purlins. The upper nuts take bearing on $\frac{1}{2}$-inch steel plates which project on both sides of the chords and form seats and splices for the purlins, to which they are spiked. At the ends of the trusses the steel bearing plates are bent to receive the sway-braced diagonal rods. The purlins support 2×6-inch rafters, 18 inches apart on centers, which are sheathed with 1-inch boards covered with metal tile and rubberoid. The trusses are cambered 3 inches in the center, and, where reinforcement pieces are keyed to the chord, they have 2-inch oak pins driven tight in holes bored half in each piece and provided with a $\frac{3}{4}$-inch through bolt at each key. In the middle of the roof there is a horizontal skylight 40 feet wide covered with a translucent material.

Where the roof trusses have very narrow seats on one side of the 10×10-inch column their yellow pine lower chords are reinforced with an oak angle block deeply notched to receive the square end of the inclined end post. A corresponding oak shoe is put opposite to it on the under side of the lower chord and supports it for a length of about 4 feet, thus reducing the unit pressure on the pine and giving a solid bearing on the end of the column member. Both shoe and angle block are keyed to the lower chord, and are secured to it with the same straps and through bolts, which are arranged so as to avoid unnecessary cutting of the cross-section.

Mr. George F. Shepley was the architect of this building, and the construction was detailed and supervised by the staff of the Exposition.

Chapter VII.

COMPUTING THE PURLIN AND TRUSS LOADS AND SUPPORTING FORCES.

104. The various steps to be pursued in designing a trussed roof and proportioning its parts, are as follows:

1. Laying out the roof and trusses on plan and section.
2. Determining the size of rafters and purlins.
3. Computing the truss loads and drawing the stress diagrams.
4. Computing the size of the truss members.
5. Detailing the joints.

The first step involves selecting the type of truss to be used, determining its shape, height and span, the spacing of the trusses and the manner in which the roofing, ceiling or any special loads are to be supported.

These points will be determined largely by the shape, size and character of the building and by the judgment of the designer, based upon previous study and experience.

The best "lay-out" will be that which is the simplest and most economical, while meeting the required conditions, but any lay-out showing a proper form of truss can be executed and given the necessary strength by using sufficient materials. The preceding chapters, if carefully studied, should enable one to lay out the roof and trusses with a reasonable degree of skill.

To compute the truss loads, a section through the roof must first be drawn, showing the slope of the roof, the outline of the truss with location of the purlins, if purlins are to be used, and the exact position of the trusses must be located on the plans. This will determine the roof area supported by the purlins and also by the trusses. The next step will be to estimate the dead load on the trusses, and to determine the allowance to make for wind and snow.

105. THE DEAD LOAD on a roof truss consists of the weight of all of the materials supported by the truss and also of the truss itself. Wind and snow are considered as live loads, because they

are not always present. The common practice in figuring roof loads is to compute the roof area supported at each joint of the truss, and then multiply by the load or loads *per square foot*, for which the truss is to be designed. Ordinarily the dead load includes the weight of roof covering, sheathing, rafters, purlins and truss. If the rafters are supported directly by the trusses, the purlins are omitted, and sometimes the sheathing and rafters are omitted and the roofing, if of slate, tile or corrugated iron, supported directly on the purlins. It is therefore necessary to estimate the weight of roof per square foot in each instance.

TABLE IV.—DATA FOR ESTIMATING ROOF LOADS, PER SQUARE FOOT OF ROOF SURFACE.

The weight per square foot of any roof may be quite closely estimated from the following data:

Shingles, common, $2\frac{1}{2}$ lbs.; 18 ins., 3 lbs.

Slates, 3-16 in. thick, $7\frac{1}{4}$ lbs.; $\frac{1}{4}$ in. thick, 9.6 lbs. (the common thickness is 3-16 in. for sizes up to 10″×20″).

Plain tiles or clay shingles, 11 to 14 lbs.

Roman tiles, old style, two parts, 12 lbs.; new style, one part, 8 lbs.

Spanish tiles, old style, two parts, 19 lbs.; new style, one part, 8 lbs.

Improved Oriental tiles, 11 lbs.

Ludowici tile, 8 lbs.

For tiles laid in mortar add 10 lbs. per square foot.

Copper roofing, sheets, $1\frac{1}{2}$ lbs.; tiles, $1\frac{3}{4}$ lbs.

Tin roofing, sheets or shingles, including one thickness of felt, 1 lb.

Corrugated iron, painted or galvanized, No. 26, 1 lb.; No. 24, 1.3 lbs.; No. 22, 1.6 lbs.; No. 20, 1.9 lbs.; No. 18, 2.6 lbs.; and No. 16, 3.3 lbs.

Standing seam steel roofing, 1 lb.

Five-ply felt and gravel roof, 6 lbs.

Four-ply felt and gravel roof, $5\frac{1}{2}$ lbs.

Three-ply ready roofing (elaterite, ruberoid, asphalt, etc.), 0.6 to 1 lb.

Skylights with galvanized iron frame, $\frac{1}{4}$ inch glass, $4\frac{1}{2}$ lbs.; 5-16 in., 5 lbs.; $\frac{3}{8}$-in., 6 lbs.

Sheathing, 1 in. thick, 3 lbs. per square foot for white pine, spruce, or hemlock; 4 lbs. for yellow or pitch pine.

The dimensions and weight of wooden rafters may be taken from tables V. and VI.

TABLE V.—MAXIMUM SPAN FOR WOODEN RAFTERS.

A. SHINGLED ROOFS NOT PLASTERED.*

Total load, 48 pounds per square foot.

Size of Joists.	Dist. on Centre.	Hemlock.		White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins	Ft.	Ins.
2 x 4	16	7	4	7	9	8	4	9	6	10	10
2 x 4	20	6	7	6	10	7	6	8	6	8	10
2 x 6	16	11	1	11	7	12	6	14	2	15	0
2 x 6	20	9	11	10	4	11	2	12	8	13	4
3 x 6	16	13	7	14	2	15	3	17	5	18	3
3 x 6	20	12	2	12	8	13	8	15	7	16	4
2 x 8	16	14	9	15	6	16	8	18	11	20	0
2 x 8	20	13	3	13	10	14	11	16	11	17	10
2 x 8	24	12	1	12	7	13	7	15	6	16	3
2 x 10	16	18	6	19	3	20	10	23	8	25	0
2 x 10	20	16	7	17	3	18	8	21	2	22	3
2 x 10	24	15	1	15	9	17	0	19	3	20	4

B. SLATE ROOFS NOT PLASTERED, OR SHINGLE ROOFS PLASTERED.*

Total load, 57 pounds per square foot.

Size of Joists.	Dist. on Centres.	Hemlock.		White Pine.		Spruce.		Oregon Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
2 x 4	16	6	9	7	1	7	7	8	8	9	2
2 x 4	20	6	0	6	4	6	9	7	9	8	2
2 x 6	16	10	2	10	7	11	6	13	0	13	8
2 x 6	20	9	1	9	6	10	2	11	7	12	3
3 x 6	16	12	6	13	0	14	1	15	11	16	9
3 x 6	20	11	1	11	8	12	7	14	3	15	0
2 x 8	16	13	7	14	2	15	3	17	4	18	3
2 x 8	20	12	2	12	8	13	8	15	6	16	4
2 x 8	24	11	1	11	7	12	6	14	2	14	11
3 x 8	16	16	7	17	4	18	9	21	3	22	5
3 x 8	20	14	10	15	6	16	9	19	0	20	1
3 x 8	24	13	7	14	2	15	3	17	4	18	4
2 x 10	16	17	0	17	8	19	2	21	7	22	10
2 x 10	20	15	2	15	10	17	1	19	4	20	6
2 x 10	24	13	10	14	6	15	7	17	8	18	8

* These tables allow for a snowfall of 2 feet. In the Southern States the spans in section A will be safe for slate or gravel roofs, if the joists are full to dimensions.

C. SLATE ROOFS PLASTERED, OR GRAVEL ROOFS NOT PLASTERED.*

Total load, 66 pounds per square foot.

Size of Joists	Dist. on Centres	Hemlock.		White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine.		Georgia Pine.	
	Ins.	Ft	Ins	Ft.	Ins.	Ft.	Ins	Ft.	Ins.	Ft.	Ins.
2 x 6	16	9	5	9	10	10	8	12	1	12	9
2 x 6	20	8	6	8	10	9	6	10	9	11	5
3 x 6	16	11	7	12	1	13	1	14	10	15	7
3 x 6	20	10	4	10	10	11	8	13	3	14	0
2 x 8	16	12	7	13	2	14	2	16	2	17	0
2 x 8	20	11	3	11	9	12	9	14	5	15	2
2 x 8	24	10	3	10	9	11	7	13	2	13	10
3 x 8	16	15	5	16	1	17	5	19	9	20	10
3 x 8	20	13	9	14	5	15	3	17	8	18	8
3 x 8	24	12	7	13	2	14	2	16	2	17	0
2 x 10	16	15	9	16	6	17	9	20	2	21	3
2 x 10	20	14	1	14	8	15	11	18	0	19	0
2 x 10	24	12	10	13	5	14	6	16	6	17	5
2 x 12	16	18	10	19	9	21	4	24	2	25	6
2 x 12	20	16	10	17	8	19	1	21	8	22	10
2 x 12	24	15	5	16	1	17	5	19	9	20	10

* These tables are intended for climates where a snowfall of 2 feet may be expected. In the Southern States. where there is no snow to speak of, the spans in Section A will be safe for slate or gravel roofs if the joists are sawn full to dimensions.

TABLE VI.—WEIGHT OF RAFTERS PER SQUARE FOOT.

Size of Rafters, Inches.	Spruce, Hemlock, White Pine, Spacing in Inches. Centre to Centre.			Hard Pine, Spacing in Inches, Centre to Centre.		
	16	20	24	16	20	24
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
2 x 4	1½	1.2	1	2	1.6	1⅓
2 x 6	2¼	1.8	1½	3	2.4	2
2 x 7	2⅝	2.1	1¾	3½	2.8	2⅓
2 x 8	3	2.4	2	4	3.2	2⅔
2 x 10	3¾	3	2½	5	4	3⅓

Wooden purlins will weigh about 2 lbs. per square foot of roof surface when the span is between 12 and 16 ft.

For steel roofs the size and weight of the purlins and rafters should be computed for each particular case.

For a rough approximation the weight of steel trusses, purlins, and bracing in a roof covered with corrugated iron with no ceiling will run from 4 to 6 lbs. per square foot of horizontal surface covered. The steel work for slate roofs with suspended ceilings below will run about $7\frac{1}{2}$ lbs. per square foot when the span does not exceed 50 ft.

Steel roofs supported by arched trusses will weigh from 8 to 12 lbs. per square foot of roof surface. Examples of the actual weight of several steel roofs are given on pages 186 and 189.

WEIGHT OF CEILINGS.—For computing the weight of ceilings, the weight of the joists may be taken from Table VI., or computed on the basis of 3 pounds per foot board measure for soft woods and 4 pounds for hard woods.

For lath and plaster allow 10 pounds per square foot. For $\frac{3}{4}$-inch ceiling $2\frac{1}{2}$ pounds, and for metal ceilings with furring $1\frac{1}{2}$ to 2 pounds.

When the attic space is accessible the author usually adds from 3 to 5 pounds per square foot for occasional loads on the ceiling, such as persons walking or climbing over it, or for the storage of odds and ends.

106. WEIGHT OF FLOORS AND FLOOR LOADS.—When a floor is supported by the truss, the dead weight of the floor should be computed, the same as for a ceiling, and if there are any partitions these must not be overlooked. Four-inch stud partitions, plastered both sides, weigh about 20 lbs. per square foot. An allowance must also be made for live load on the floor. If the floor is to be used for sleeping or living rooms, an allowance of 40 lbs. per square foot for live load will be ample. For committee rooms, etc., allow 75 lbs. per square foot, and for assembly rooms and dancing not less than 120 lbs. If used for storage of any kind, an estimate should be made of the probable maximum load that the special class of goods will give. Data for this purpose may be found in Chapter XXI. of the Architect's and Builder's Pocket Book.

For certain types of trusses the live floor load may be considered as a dead load, while for other types the stresses should be computed both with and without the live load. This subject is more fully considered in Chapter VIII.

107. WEIGHT OF TRUSS.—To the weight of the roof construction proper should be added an allowance for the weight of the trusses. If trusses could be built in exact accordance with the theoretical requirements their weight would be directly propor-

tional to the roof load and span, but as there is always some extra material, it is impossible to determine the weight of any proposed truss exactly until it is completely designed. Several tables for the weight of wooden trusses and formulas for steel trusses have been published, but hardly any two of them are alike.*

Tables VII. and VIII. compiled by the author, from a comparison of other tables and formulas, and from the weight of actual trusses, are sufficiently accurate for the purpose of determining stresses. The weights given are probably slightly in excess of the actual weights of average trusses, as the author prefers to have the error, if any, on the safe side. It should be noted that the weights are for each square foot of *roof* surface, and not for the horizontal area.

TABLE VII.—WEIGHT PER SQUARE FOOT OF ROOF SURFACE FOR WOODEN TRUSSES.†

Span.	½ Pitch	⅓ Pitch	¼ Pitch	Flat.
	lbs.	lbs.	lbs.	lbs.
Up to 36 ft.	3	3½	3¾	4
36 to 50 ft.	3¼	3¾	4	4½
50 to 60 ft.	3½	4	4½	4¾
60 to 70 ft.	3¾	4½	4¾	5¼
70 to 80 ft.	4¼	5	5½	6
80 to 90 ft.	5	6	6½	7
90 to 100 ft.	5¾	6¾	7	8
100 to 110 ft.	6½	7½	8	9
110 to 120 ft.	7	8½	9	10

* The following are some of the formulas given for weight of steel trusses, W being weight per horizontal square foot, S = span in feet, P = capacity of truss in pounds per horizontal square foot, and A the distance centre to centre of trusses in feet :

Charles Evan Fowler, C. E., for Fink trusses :

$W = .06S + .6$ for heavy loads ;

$W = .04S + .4$ for light loads.

H. G. Tyrrell, C. E. :

$$W = .05S + \frac{12}{\text{dist. centre to centre.}}$$

C. W. Bryan, C. E. :

$$W = .04S + 4.$$

M. S. Ketchum, C. E.:

$$W = \frac{P}{45}\left(1 + \frac{L}{5\sqrt{A}}\right)$$

† For scissors trusses increase one-third.

TABLE VIII.—WEIGHT PER SQUARE FOOT OF ROOF SURFACE FOR STEEL TRUSSES.

Span.	½ Pitch.	⅓ Pitch.	¼ Pitch.	Flat.
Up to 40 ft.	5.25	6.3	6.8	7.6
" 50 ft.	5.75	6.6	7.2	8.0
" 60 ft.	6.75	8.0	8.6	9.6
" 70 ft.	7.25	8.5	9.2	10.2
" 80 ft.	7.75	9.0	9.7	10.8
" 100 ft.	8.5	10.0	10.8	12.0
" 120 ft.	9.5	11.0	12.0	13.2
" 140 ft.	10.0	11.6	12.6	14.0

108. ALLOWANCE FOR SNOW.—In making an allowance for snow, one's judgment must be exercised to a considerable degree, as the maximum snow fall varies widely in different localities, and the amount of snow that may lodge upon a roof will depend in a great measure upon the inclination of the roof and its exposure to the wind, also somewhat upon the roof covering and whether or not snow guards are used.

The weight of dry, freshly fallen snow is commonly given at 8 pounds per cubic foot, while saturated snow or snow mixed with hail or sleet may weigh as much as 32 pounds per cubic foot. Dry snow may attain a depth of 3 feet and possibly more in some localities, but snow weighing as much as 32 pounds per cubic foot will hardly ever be found more than 16 inches in depth, even on a flat roof.

It is generally assumed that a sloping roof cannot be exposed to the maximum snow load and wind pressure at the same time, but as a high wind may follow a sleet storm, some allowance should be made for sleet, under any method of computing stresses, i. e., for roofs in the Northern States. For flat roofs, and for such others for which it may be deemed desirable to determine the stresses due to maximum snow load, the values given in Table IX. may be considered as the maximum possible loads for different portions of the country.

When the wind pressure is treated as a vertical load, the values given in Table X. should be used as representing the maximum load due to wind and snow combined.

109. WIND PRESSURE.—For roofs having a pitch of 4 ins. or more to the foot, the effect of the wind must be taken into

TABLE IX.—ALLOWANCE FOR SNOW IN POUNDS PER SQUARE FOOT OF ROOF SURFACE.

Location.	Pitch of Roof.				
	1/2	1/3	1/4	1/5	1/6 or less.
	* †	* †	* †		
Southern States and Pacific Slope..	0—0	0—5	0—5	5	5
Central States....................	0—5	7—10	15—20	22	30
Rocky Mountain States...........	0—10	10—15	20—25	27	35
New England States..............	0—10	10—15	20—25	35	40
Northwest States.................	0—12	12—18	25—30	37	45

Columns headed by an asterisk (*) are for slate, tile, or metal; those headed by a dagger (†) are for shingle roof. When *snow guards* are to be placed on the roof, the same allowance should be made for a half pitch as for one-third pitch, and the larger figures should be used.

account in determining the stresses. Two ways of doing this are in vogue. The more common method for wooden trusses and for steel trusses of the Fink or Fan types is to include the wind pressure with the vertical loads and to make a single allowance for both wind and snow.

This is not a correct assumption, as the wind acts on a roof in a direction normal (at right angles) to its surface and the stresses produced by such a force are quite different from those produced by a vertical force of the same or even greater intensity.

As a matter of practical experience, however, it is found that for ordinary types of wooden trusses, *having an inclination not exceeding* 45°, and for steel trusses of the Fink or triangular type, this method is sufficiently accurate.

Mr. Bryan, the designing engineer of the Edgemoor Bridge Works, states that: "In the Fink trusses a partial load due to wind or snow never causes any maximum stresses, so that it is customary to calculate these trusses for a uniform load over the entire truss, the wind and snow loads combined being usually assumed at 30 pounds per square foot of *area covered*," *i. e.*, horizontal surface. "It is not generally assumed that the maximum wind pressure and the snow load can act on the same half of the roof at the same time."

To be absolutely safe, however, the author recommends that the allowance for wind and snow combined be not less than indicated in Table X.

TABLE X.—ALLOWANCE FOR WIND AND SNOW COMBINED IN POUNDS PER SQUARE FOOT OF ROOF SURFACE.

Location.	Pitch of Roof.					
	60°	45°	1/3	1/4	1/5	1/6
Northwest States	30	30	25	30	37	45
New England States	30	30	25	25	35	40
Rocky Mountain States	30	30	25	25	27	35
Central States	30	30	25	25	22	30
Southern and Pacific States	30	30	25	25	22	20

110. The other and more exact method is to find the stresses for all of the different loadings to which the truss may be subjected, separately and then combine them so as to obtain the greatest possible stress that may occur in each member under any possible combination of loads.

This method should be followed in determining the stresses for all trusses having an inclination of more than 45° or a span of more than 100 ft. (except for flat roofs), also for all trusses in which a partial load may produce maximum stresses or call for counter bracing, as is the case in quadrilateral trusses and trusses with curved chords.

FORCE OF THE WIND.—For determining the stresses due to wind pressure alone the force of the wind is usually assumed to act in a direction normal, i. e., at right angles to the slope of the

TABLE XI.—NORMAL AND HORIZONTAL WIND PRESSURE ON ROOFS FOR 30 POUNDS HORIZONTAL PRESSURE AGAINST A VERTICAL SURFACE.

Inclination.	Norm.	Hor.	Inclination.	Norm.	Hor.
	lbs.	lbs.		lbs.	lbs.
5°	3.9	0.3	30°	19.9	10.0
10°	7.2	1.2	33°—41′ (1/3 pitch)	22.0	12.0
15°	10.5		35°	22.6	
18°—26′ (1/6 pitch)	13.0	4.0	40°	25.1	15.9
20°	13.7	4.5	45° (1/2 pitch)	27.1	19.0
21°—48′ (1/5 pitch)	15.0	6.0	50°	28.6	21.9
25°	16.9		55°	29.7	
26°—34′ (1/4 pitch)	18.0	8.0	60°	30.0	25.5

roof. This force is commonly based on a horizontal wind pressure of 30 lbs. per square foot, although quite often it is taken at 40 lbs. per square foot, depending somewhat upon the exposure and the shape or construction of the roof and truss.

The normal and horizontal pressure per square foot of roof surface corresponding to a horizontal pressure of 30 lbs. against a vertical surface is given in Table XI.

For a horizontal wind pressure of 40 lbs. per square foot the pressure given in the table should be increased one-third.

111. ROOF AND CEILING AREAS SUPPORTED BY THE TRUSS JOINTS.—Calculations for the stresses in a truss are always based on the assumption that the loads are transferred to the joints, and that the various members of the truss are free to move at the joints, as though connected by a pin. Before anything can be done towards determining the stresses, therefore, it is necessary to compute the load which is supported, theoretically at least, at each joint. This load is obtained by multiplying the roof or ceiling area contributory to that joint by the weight per square foot previously determined on. If the rafters are supported by purlins and these are supported at the joints of the truss, one half of the load on each purlin will be transmitted to the joint. If the purlins are supported at some distance from the joints, then part of the load from the purlins will come at one joint and part at another, as will be explained later. When there are no purlins, and the rafters bear directly on the top chord of the truss as in Figs. 41 and 113, then the different sections of the chord act as beams, and one half of the load on each section will be transferred to the joint between. As a rule, the joint loads will be the same in either case, unless the purlins are located away from the joints.

The following examples will show how to determine the roof or ceiling areas contributory to a joint:

EXAMPLE I. Let Fig. 249 represent the section through a roof and ceiling, showing the shape of the truss and the position of the purlins, and Fig. 250 a plan of the same roof showing the location of the trusses. Then the roof area supported by joint 2 of truss No. 3 will be that portion enclosed within the rectangle *h i m l*, Fig. 250. As the area of this rectangle is supposed to be measured on the slant of the roof, the true length of the lines *l m* and *h i*, is the distance *a*, Fig. 249, and the area of the rectangle will be the

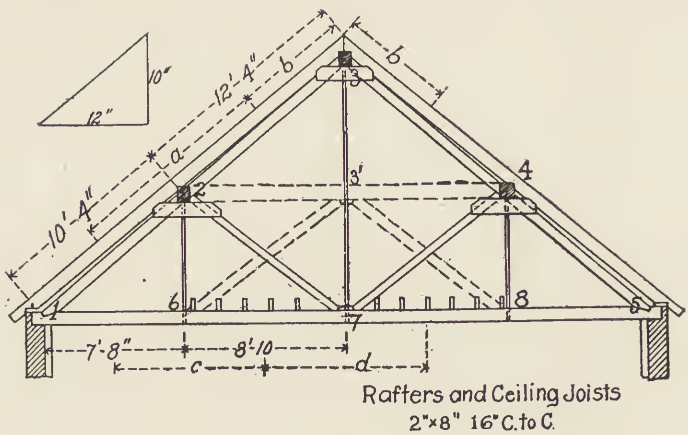


Fig. 249.

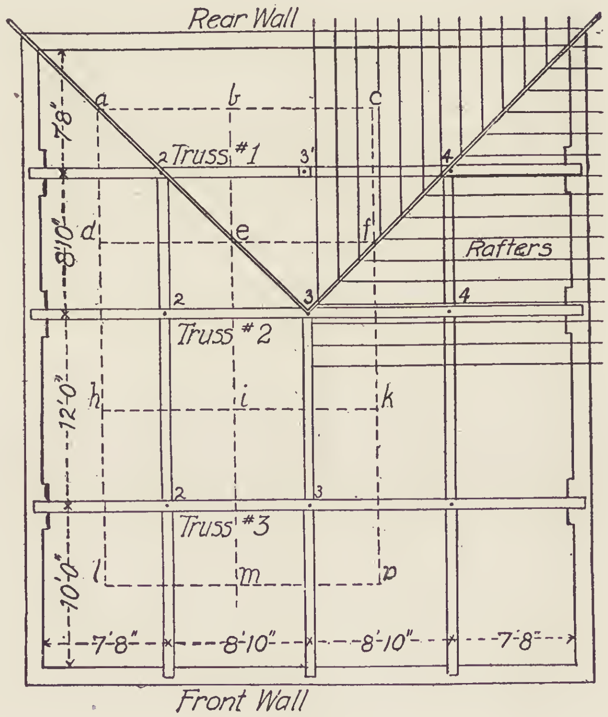


Fig. 250.

product of $h\ l$ by the distance a. The point h should be taken half way between truss 2 and truss 3, and the point l half way between truss 3 and the front wall. The distance a should be one half of the distance from 1 to 2, Fig. 249, plus one half of the distance from 2 to 3. In this particular example, $a = \frac{10'4'' + 12'4''}{2} =$ $11'\ 4''$ and $h\ l = 5' + 6' = 11$ ft., hence the roof area supported at joint 2 of truss $3 = 11' \times 11\frac{1}{3}' = 124\frac{2}{3}$ sq. ft.

The roof area supported by joint 3 of truss 3 is the portion enclosed within the rectangle $i\ k\ m\ n$, whose area $= i\,m \times 2\,b$ (Fig. 249), which in this example $= 11' \times 12'\ 4'' = 135\frac{2}{3}$ sq. ft.

The lengths a and b apply to all trusses of the same roof, but the horizontal distances may vary as in Fig. 250. Unless there is some particular reason for spacing the trusses unevenly, they are generally spaced uniformly, so that the trusses will all have the same loads.

Where the roof is hipped or has valleys, the valleys and hips are apt to change the loads slightly from what they would be if the roof were straight, but as a rule the difference is so slight that it is customary to estimate the loads as though the roof were straight. Thus the roof area supported at joint 3 of truss 2 may be taken as the area $c\ f\ k\ i = (4'\ 5'' + 6') \times 12'\ 4'' = 128\frac{1}{2}$ sq. ft.

The roof area at joint 3 of truss 1 may be taken as $b\ c\ f\ e = b\ c \times b\ e$. In this case $b\ c = a = 11'\ 4''$ and $b\ e =$ half the horizontal distance between joints 2 and $4 = 8'\ 10''$. [Note.—Truss 1 is shown by the dotted lines in Fig. 249.] The roof loads on the right hand side of the truss are the same as those on the left hand side.

112. EXACT METHOD OF FIGURING ROOF AREA SUPPORTED AT JOINTS WHEN THE ROOF IS HIPPED. —When the spacing between trusses and purlins is 16 ft. or more, and the roof rather flat, the exact loads transmitted by the hips should be calculated after the following manner:—

Let Fig. 251 represent the plan of a portion of a hip roof, the dot and dash line passing through joints 2 and 4, being half way between side walls, and the line passing through $k\ l\ s$, being half way to the next truss.

The dimensions between arrow heads are the *horizontal* dimensions. The pitch of the roof is such that the length of the lines *b d*, *e d*, *i h*, *n m*, etc., measured on the slant of the roof, is exactly 6 ft. Assuming that the rafters are well spiked to the hips, then the latter will act as beams transferring the roof load to the joints in the same manner as a horizontal beam. On this assumption

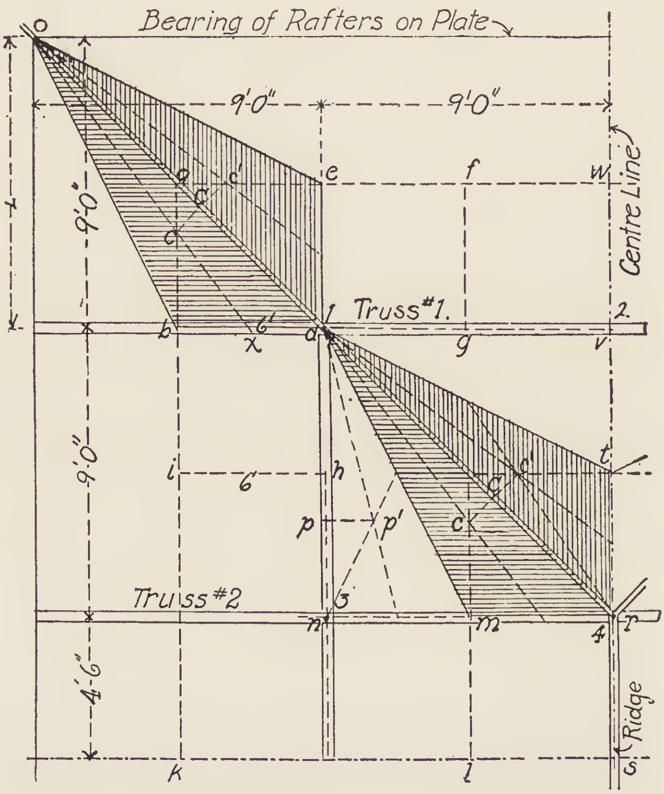


Fig. 221.—Partial Plan of High Roof.

the roof areas supported by the hips are shown by the shaded portions.

The joints of the trusses are supposed to be at the points 1, 2, 3 and 4. The roof area supported at joint 1 is made up of the reaction of the two hips; of the rectangles *b d h i* and *e f g d*, and also of a portion of each of the triangles *d n m* and *d t v*. The area supported by the hip 0-1 equals the sum of the triangles $o\ b\ d$ and $o\ e\ d$. The area of the first is equal to $\frac{b\,d \times L}{2} = \frac{6 \times 9}{2} = 27$ sq. ft.

As the triangles have the same area, the total area supported by the hip = 54 sq. ft. To find what portion of this load is transmitted to 1 and what to 0, we must find the centre of gravity of the triangles. This can readily be done by drawing a line from two angles to the centre of the opposite sides, as $b\ a$ and $o\ x$. The point where they intersect will be the centre of gravity of the triangle. Joining the two points c and c' by a line, the point C where the line crosses the hip will be the centre of gravity of the entire shaded portion. By the principle of the lever, the load at 1 is to the entire load as $o\ C$ is o1, or load at $1 = \frac{\textit{entire area} \times o\,C}{01.}$.

For the lengths of $o\,C$ and 01 we can take the horizontal measurement, as all that we require is the ratio. $o\,C$ measures 89″ and 01, 153″. Then the area supported at $1 = \frac{54 \times 89}{153} = 31.4$ sq. ft.

As the roof area supported by the hip 1-4 is the same as that supported by 0-1, the reaction at 4 from one hip will also be 31.4, and the reaction at 1 from hip 1-4 must be 54—31.4 = 18.6 sq. ft. Consequently joint 1 supports 54 sq. ft. of roof area transmitted by the two hips, which is just one half of what it would have to support if the roof were straight.

The area of each of the rectangles $b\ d\ i\ h$ and $d\ e\ f\ g = 6' \times 4\frac{1}{2}' =$ 27 sq. ft. The purlin 1-3 must support the roof area represented by the triangle $d\ n\ m$, and the top chord of truss 1 the triangle $d\ v\ t$. The centre of gravity of triangle $d\ n\ m$ is at p', and the proportion of the area transmitted to joint 3 is equal to $\frac{d\,p}{d\,n}$, and as in a right triangle the point p is at $\frac{1}{3}$ the distance $d\ n$ from 3, $\frac{d\,p}{d\,n} = \frac{2}{3}$. The area of $d\,n\,m = \frac{6 \times 9}{2} = 27$, and $\frac{1}{3}$ of this is transmitted to joint 1. By the same analysis, $\frac{1}{3}$ of *d v t* is transmitted to joint 1. Then total roof area supported at joint 1 =

Reaction of two hips	=	54
Areas of rectangles b d h i and d e f g	=	54
⅓ of each of triangles d n m and d v t	=	18
Total area		126 sq. ft.

Roof Area Supported at Joint 3.—The roof area supported at this joint will be $\frac{2}{3}$ of the area of the triangle *d n m* plus the area of the L-shaped figure *h i k l m n*. The area of *d m n* we found before to be 27 sq. ft., and $\frac{2}{3}$ of this is 18 sq. ft.

The area of the L-shaped figure equals three times the area of *b d h i* $= 3 \times 27 = 81$ sq. ft. Therefore the entire roof area supported at joint $3 = 81 + 18 = 99$ sq. ft.

Roof Area Supported at Joint 2.—As but one half of the roof area supported at this joint is shown in Fig. 251, we will find half the area and multiply by two. From an inspection of the figure we find that the area at one side of the centre line is made up of the rectangle *f w v g* = 27 sq. ft. + $\frac{2}{3}$ of the triangle *d v t* = 18 sq. ft. Hence the half area = 45 sq. ft., and the whole area 90 sq. ft.

Roof Area Supported at Joint 4.—The roof area contributory to joint 4, on each side of the centre line, is its proportional part of the shaded area supported by the hip, which we found before to be 31.4 sq. ft., and the area of the rectangle *l m r s*, which is 27 sq. ft. Consequently, the entire roof area supported at joint 4 = 116.8 sq. ft.

Comparison of Actual Roof Areas With What They Would Be For a Straight Gable Roof.—From the above calculations we find the entire roof area which must be supported by the six joints of trusses 1 and 2 to be—

Area at joint 2	=	90.0 sq. ft.
Area at joint 4	=	116.8 "
Areas at joints 1 and 1′	=	252.0 "
Areas at joints 3 and 3′	=	198.0 "
		656.8 sq. ft.

[Note.—Joints 1′ and 3′ are the joints corresponding to joints 1 and 3 for the other half of the roof, not shown.]

If the roof were straight without hips the area supported at each of the six joints would be $9' \times 12' = 108$ sq. ft., or a total of 648 sq. ft. Therefore the actual area is 8.8 sq. ft. in excess of what it would be if the roof were straight.

Taking the trusses separately, truss 1 has to support an actual roof area of 342 sq. ft., or 18 sq. ft. more; and truss 2, 314.8 sq. ft., or 9.4 less than if the roof were straight. These differences, however, amount to only $5\frac{1}{2}\%$ for truss 1 and less than 3% for truss 2, so that except where the trusses are spaced 16 ft. or more apart, and the roof is of heavy construction, the error in figuring the roof areas as in Example 1, is not sufficient to effect the actual construction of the trusses.

With steep roofs, moreover, it is probable that the hips would not take their full share of the load, so that the joint loads obtained by assuming the areas to be the same as for a straight roof would be as close to the actual conditions as can be estimated.

The method above explained for figuring the reaction from hips also applies to valleys, only that with valleys the bottom of the rafters are supported and the full estimated load is sure to come upon them, and possibly more.

EXAMPLE 2—DECK ROOF.—Let Fig. 252 be a half section through a deck roof, supported by trusses with purlins located as indicated, and the trusses spaced 12 ft. on centres. The roof areas supported at the joints are computed precisely as in Example 1, except that the flat and sloping areas should be kept separate as they will probably have different weights per sq. ft.

Thus roof area at joint 2 = 9′ 10″ × 12′ = 118 sq. ft. sloping roof.

" " " joint 3 = $\begin{cases} 4' \ 11'' \times 12' = 59 \text{ sq. ft. sloping roof.} \\ 4' \ 6'' + 1' \times 12' = 66 \text{ sq. ft. flat roof.} \end{cases}$

" " " joint 4 = 9′ 3″ × 12′ = 111 sq. ft. flat roof.

" " " joint 5 = 9′ 6″ × 12′ = 114 sq. ft. flat roof.

In figuring the area of deck supported at joint 3 one foot in width is added to allow for the projection beyond centre of purlin.

EXAMPLE 3.—For this example we will take a case where the purlins rest on the chord of the truss, other than at the joints, as in Fig. 253. For a roof of this span the type of truss shown is the most economical one that can be used, but if the purlins were spaced at the joints the rafters should be 2″ × 6″, while by spacing the purlins as shown 2″ × 4″ rafters will answer even with slate roofing, and on a large roof this will effect quite a saving in cost.

We will assume that the trusses are spaced 12 ft. apart on centres. The roof area supported by each of the purlins A, B, C, will therefore be equal to $8' \times 12' = 96$ sq. ft. The roof area supported by each of the purlins D, E $= \frac{8'}{2} + 2' 4'' \times 12 = 76$ sq. ft.,

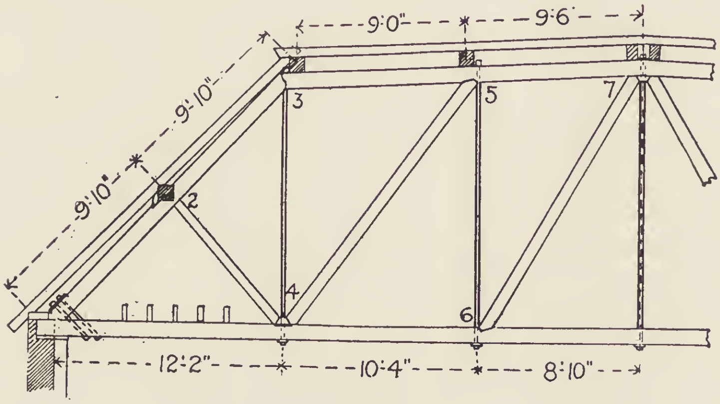


Fig. 252.

hence the roof area directly supported at joint $5 = 152$ sq. ft. As purlin A. comes at a joint its load is transmitted directly to the joint. The reaction of purlin B, coming at a point between two

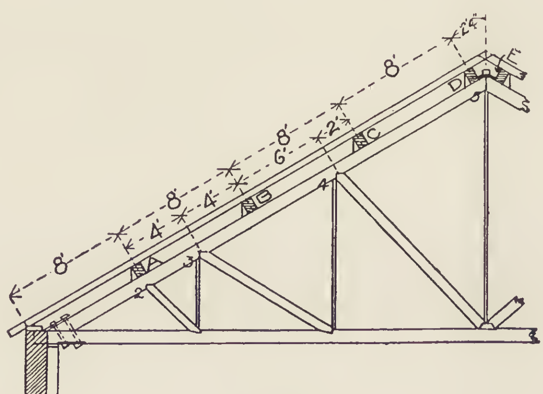


Fig. 253.

joints, will be supported in part by each joint, and the same is true of purlin C. We must therefore find the amount of the purlin loads that will be borne by each of the joints. When a beam sustains a concentrated load applied at some point other than the centre, as in Fig. 254, the portion of the load which is transmitted to each support may be found by the equations given for P1 and P2 in Fig. 254. If the load is applied $\frac{1}{3}$ of the span from P1, then ⅔ of the load will be supported at P_1 and ⅓ at P_2. When the load is applied at the centre, then each support receives one half.

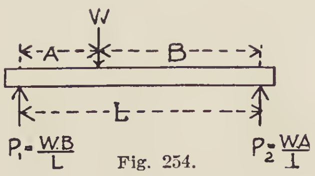


Fig. 254.

To return now to Fig. 253, it is evident from the above that joint 3 must support $\frac{6}{10}$ of the load on B, or 57.6 sq. ft. Joint 4 must support $\frac{4}{10}$ of the load on B, and also $\frac{8}{10}$ of the load on C, which together equal 115.2 sq. ft. Joint 5, in addition to supporting purlins D and E, must also support $\frac{2}{10}$ of the load on purlin C and on the corresponding purlin on the other side of the roof, hence the roof area supported at joint 5 = 152 + 4-10 of 96 = 190.4 sq. ft. It will be seen from the above computations that with the arrangement of joints and purlins shown, joint 5 receives more load than joints 3 and 4 combined. As a load at the apex, however, brings no stress on the rods, this is really a more economical arrangement than if the roof area was evenly distributed on the joints.

114. EXAMPLE 4—TO COMPUTE THE ROOF AREAS SUPPORTED BY DIAGONAL TRUSSES.—Let Fig. 255 represent (in the upper portion) the intersection of two pitched roofs with the ridges at right angles to each other, and also in the lower portion a cross section of the roof. In its horizontal dimensions this roof is the same as that shown in Fig. 162, but to facilitate the computations the pitch of the roof has been changed. As far as computing the roof loads is concerned, however, Fig. 255 will apply as well to a roof supported by triangular trusses as by scissors trusses, but if the ceiling below is to be level it will be simpler and generally cheaper to support the roof over the intersection or crossing as shown in Fig. 168. Where the ceiling is raised or vaulted it will generally be found necessary to use diagonal trusses. In this example only one of the diagonals is a full truss, the other diagonal being formed of two half trusses supported at the center of the full truss as shown by Fig. 164.

We will now proceed to compute the roof area supported at each of the joints of the three trusses.

The entire roof area supported by the diagonal trusses is that portion included within the heavy broken line. This area is sup-

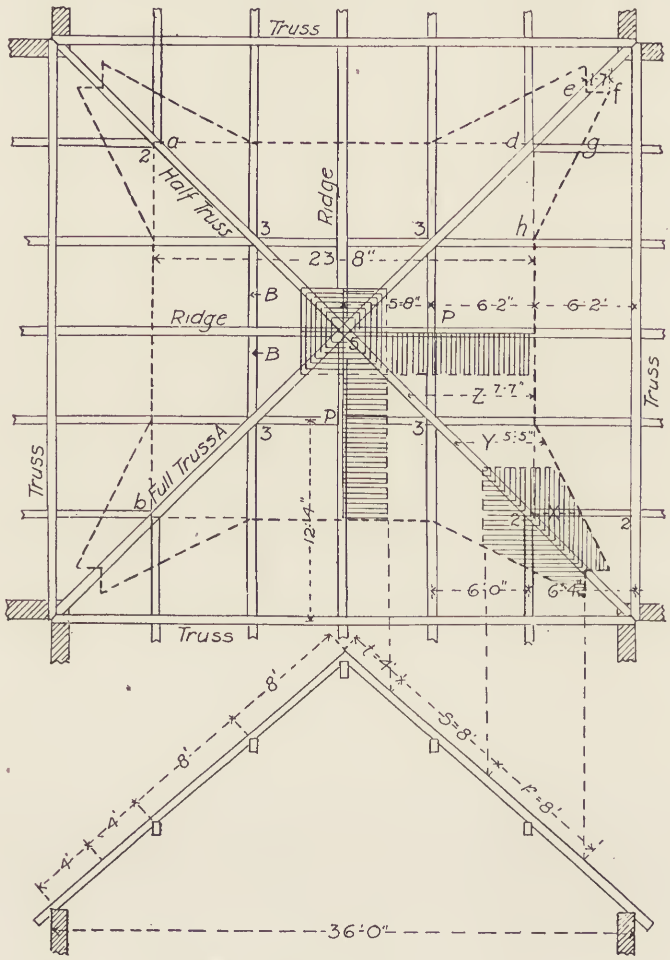


Fig. 255.—Plan and Elevation of Roof.

ported at the eight points where the purlins intersect the diagonals, and also at the apex. The roof area supported at joint 2 of each truss is shown by the shaded portion in the lower right-hand corner. This area is equal to twice the length at X multiplied by the slant height, r, which is 8 ft. X $= \frac{1}{2}$ the distance from the centre of the valley to the centre of the side truss, or $3' \, 2''$, therefore the entire area $= 6' \, 4'' \times 8' = 50\frac{2}{3}$ sq. ft. The roof area supported at joints 3, 3, is the portion included between the shaded portions and the boundary lines. To compute this area it is necessary to consider it as made up of two portions, one measured by Z and the other by Y. The entire area $= 2\,Y \times 4 + 2\,Z \times 4 = 104$ sq. ft. The ridge purlins are supported at the points P, P, by braces extending to the tie beams of the trusses directly below joint 3, which we will designate as joint 4. As this joint (4) will support ½ of the load at the points P, P, on each side of it, the area supported is equal to twice $\left(6' \, 2'' + \frac{5' \, 8''}{2}\right) \times 4 = 72$ sq. ft. The remaining roof area, $5' \, 8'' \times 8'$, should be considered as supported at the apex, or joint 5 of the full truss. We will now add together the joint areas that we have found, and check them by the total area.

The sum of the joint areas is made up as follows:

4 times the area at joint 2....................	=	202⅔	sq. ft.
4 times the area at joint 3....................	=	416	"
4 times the area at joint 4....................	=	288	"
area at joint 5....................	=	45⅓	"
Total area	=	952	sq. ft.

To compute the area included within the heavy broken line, first find the area included within the square a b c d, and then add the additional areas at the corners.

The area a b c d $= 23' \, 8''$ multiplied by twice the slant height, which is about $16' \, 3''$, or $769\frac{1}{2}$ sq. ft. To this should be added 8 times the area included within the points d e f h. This area is equal to $\frac{\text{e f} + \text{d g}}{2} \times 4$ plus $\frac{1}{2}$ d g $\times 8 = 22$ 1-6 sq. ft. Multiplying by 8, we have $177\frac{1}{3}$ sq. ft. Adding the area a b c d, we have for the total area 947 sq. ft. or 5 ft. less than the sum of the joint areas. This slight discrepancy is due to some inaccuracy in figuring the slant heights, and is not of sufficient amount to be considered as an error.

In figuring the loads on the full truss, the reactions of the half trusses must be added to the load at the center.

These four examples should be sufficient to explain the method of figuring roof areas contributory to joints.

115. EXAMPLES IN ESTIMATING JOINT LOADS.—The roof areas multiplied by the roof load per square foot will give the joint loads.

EXAMPLE 5.—What roof loads should we use in determining the stresses for the trusses Figs. 249 and 250?

Ans.—The dead weight per square foot would be about as follows, assuming that the roof is to be covered with slate, 3-16-in. thick:

For slate	7¼ lbs.
For sheathing	3 "
For rafters	3 "
For purlins	2 "
For truss (Table VII.)	3¼ "
Total dead weight	18½ lbs.

For the wind and snow combined, we should allow by Table X. 27½ lbs., the pitch of the roof being a mean between 45° and a ⅓ pitch. Therefore the roof load per square foot should be taken at 46 lbs. The roof areas supported at the joints were found in Example 1, to be as follows:

For truss 1, area at joint 2, = 93½ sq. ft.; and at joint 3, 100 sq. ft. For truss 2, area at joint 2, = 118 sq. ft., and at joint 3, 128½ sq. ft. For truss 3, area at joint 2, = 124⅔ sq. ft., and at joint 3, 135⅔ sq. ft. Multiplying these areas by 46, we obtain the following loads:

For truss 1, at joint 2, 4,301 lbs.; at joint 3, 4,600 lbs.

For truss 2, at joint 2, 5,428 lbs.; at joint 3, 5,911 "

For truss 3, at joint 2, 5,734 lbs.; at joint 3, 6,240 "

EXAMPLE 6.—Estimate the joint loads for the truss shown by Fig. 253, the roof being shingled on ⅞-in. sheathing nailed to 2 × 4 rafters.

Ans.—The dead weight will be made up as follows:

Shingles	2½ lbs.
Sheathing	3 "
Rafters	1½ "
Purlins	2 "
Truss (56-ft. span, ⅓ pitch)	4 "
Total	13 lbs.
Add for wind and snow, Table X	25 "
Total load per square foot	38 lbs.

In Example 3 the roof areas were found to be as below; multiplying these by 38 lbs. we obtain the respective weights:

Roof areas in sq. ft.	Joint 2, 96;	Joint 3, 57.6;	Joint 4, 115.2;	Joint 5, 190.4
Loads in lbs.	Joint 2, 3,648;	Joint 3, 2,189;	Joint 4, 4,377;	Joint 5, 7,235

EXAMPLE 7.—What joint loads should be used for the truss shown by Fig. 252, assuming that the sloping roof is to be covered with slate and the deck roof with tin, and that the building is located in one of the central states?

Ans.—The weight per sq. ft. of the sloping roof will be

For slate	7¼ lbs.
For sheathing	3 "
For 2 × 6 rafters	2¼ "
For purlins	2 "
For truss, 64′ span	4 "
Wind and snow	30 "
Total	48½ lbs.

For the deck roof:

Tin	1 lb.
Sheathing, rafters and purlins	6¼ lbs.
Truss	5¼ "
Snow	30 "
Total	42½ lbs.

Multiplying the roof areas supported at the joints, as found in Example 2 by these loads, we have

Load at joint 2..	=	118 sq. ft. × 48½ lbs. = 5,723 lbs.	
Load at joint 3..	=	59 sq. ft. × 48½ lbs. = 2,862	= 5,667 lbs.
		66 sq. ft. × 42½ lbs. = 2,805	
Load at joint 5..	=	111 sq. ft. × 42½ lbs. = 4,718 lbs.	
Load at joint 7..	=	114 sq. ft. × 42½ lbs. = 4,845 lbs.	

EXAMPLE 8.—What load per sq. ft. should be taken in computing the stresses in a truss of 64-ft. span supporting a flat roof in the New England states, the rafters to be 2″ × 8″, 16″ on centres, resting directly on the trusses, and the roofing to be 5-ply tar and gravel?

Ans.—The weight per square foot should be estimated as follows:

Roofing	6 lbs.
Sheathing	3 "
Rafters	3 "
Truss	5 "
Snow	40 "
Total	57 lbs.

116. JOINT LOADS FROM THE CEILING.—Buildings which are plastered, such as halls, schools, churches, etc., generally have a ceiling supported either directly by the tie beams of the trusses, as in Fig. 249, or by purlins, which are themselves supported at the lower joints, as in Figs. 256 and 160.

The weight of the portion of ceiling supported at each joint must be computed, in the same manner as the roof loads.

In figuring ceiling loads, however, no allowance is to be made for the truss itself, as that is included in the roof loads. The weights to be allowed for ordinary ceilings are given in Section 105.

The ceiling areas are computed in precisely the same manner

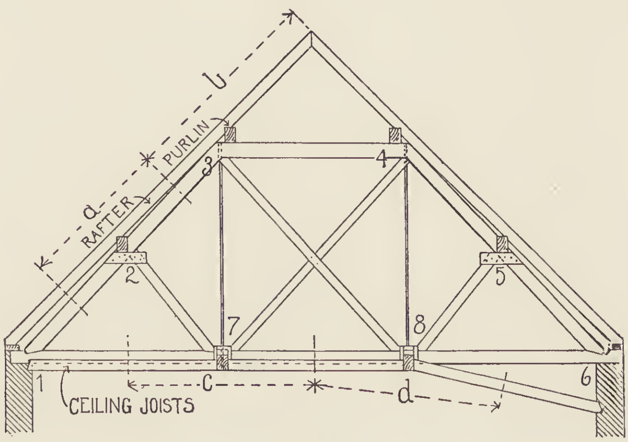


Fig. 256.

as roof areas. Thus in Fig. 249, if the trusses are 12 ft. apart, the ceiling area supported at joint 6 will be 12 × c., and at joint 7, 12 × d. The distance $c = \dfrac{7' \, 8'' + 8' \, 10''}{2} = 8' \, 3''$, and $d = 8' \, 10''$.

Therefore the roof area supported at joint 6 = 99 sq. ft., and at joint 7, 106 sq. ft.

For the weight of the ceiling joists, lath and plaster, we should allow 13 lbs. per sq. ft., and it would be wise to allow 7 lbs. additional for any odds and ends that might be stored in the attic. If the attic is floored over, an allowance of from 30 to 40 lbs. per sq. ft. should be made, in addition to the dead weight, according to the purpose for which the attic space is to be used.

Allowing 20 lbs. per sq. ft. for the weight of ceiling and light storage, we have for the load at joint 6, 1,980 lbs., and at joint 7, 2,120 lbs.

With the construction shown by Fig. 256, the ceiling joists are supported by purlins, but the ceiling areas are the same as if the ceiling joists rested directly on the trusses. Thus the area at joint 7 will equal c times the distance between trusses, and the area at joint 8 equals d times the distance between trusses.

117. SUPPORTING FORCES, OR TRUSS REACTIONS. —Before the stress diagram of a truss can be drawn, it is necessary to know not only the loads which the truss has to support, but also the supporting forces or reactions.

These are calculated on the principle that *when a beam or truss supports any number of loads acting vertically, the supports of the beam or truss, taken together, must offer an upward resistance equal to the sum of the loads.*

Thus if we have a beam loaded as in Fig. 257, and supported by two posts, the load on the two posts will be equal to the sum of the weights, and the posts are assumed to push against the under

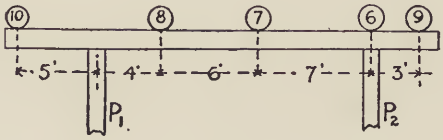


Fig. 257.

side of the beam just as much as the loads bear down. If we assume that the balls in Fig. 257 have the weights indicated by the numbers, then the total load on the beam is 40 lbs., and the two posts, together, must push upwards, as it were, with a force of 40 lbs. If the weights on the beam were symmetrically disposed in respect to the supports, then the supports would each offer the same resistance; but when the weights are unsymmetrical, either in amount or position, the posts will not receive the same load, but one will be more heavily loaded than the other.

The supports of beams and trusses are usually passive, so that they cannot really push upwards, but in considering the forces which act on a truss, they are considered as an active force, as much so as the loads, and must be determined both in direction and amount with accuracy. The supports of a beam or truss are considered as *reacting* against the under side, and the weight or pressure which comes on the supports is called the *reaction.* The algebraic sum of the reactions must always be equal to the sum of the loads, but in the case of cantilever beams or trusses, it may sometimes happen that one of the supporting forces must pull down on the beam in order to balance it. In that case the reaction is considered as a minus or negative quantity.

Before describing the method of determining the amount of each supporting force, it is necessary to take up the subject of moments.

A *moment* in mechanics is the tendency to cause rotation, and when we speak of a moment we must have in mind some fixed point about which the moment is taken. Moments can only be produced by forces, hence we speak of the moment of a force, or have some particular force in mind.

The moment of a force about any given point may be defined as the product of the force into the perpendicular distance from the

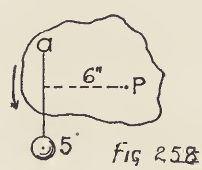


Fig. 258.

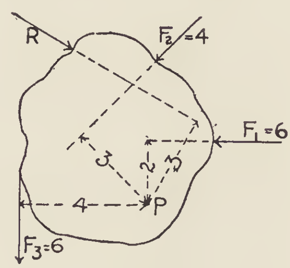


Fig. 259.

point to the line of action of the force, or, in other words, as *the product of the force by the arm with which it acts.* The arm must always be measured square or at *right angles* to the *direction* of the force.

As an example of moment, we will assume that the body, shown by the irregular figure, Fig. 258, is pivoted at the point P, and a weight of 5 lbs. is suspended from the point *a* by a string. The weight is a force, or, more strictly, represents the force of gravity, and as gravity always acts vertically, the force is a vertical one. Consequently its arm must be measured horizontally. Now, it is evident that a body in the condition shown in Fig. 258 will rotate about the point P in the direction of the arrow until the point *a* is directly under P, assuming that the body itself has no weight. The tendency to rotate is the moment of the weight, and the amount of this moment is the product of the weight into the arm shown by the dotted line, or $5 \times 6 = 30$ inch pounds. A body may have several moments acting upon it at the same time, and every force exerts a moment, except when the force is applied at the pivot point.

In Fig. 259 we have three forces, F_1, F_2 and F_3, acting on the body, and all tending to turn it in the same direction about the point P. Assuming that the forces have the values represented by the numbers, and that the lengths of arms in inches are as indicated, then the sum of the moments tending to rotate the body in the direction of the arrow is 48. Now it is evident that the body can be kept from rotating by a force applied in the direction R, and to just balance the body, so that it will not rotate in either direction the moment of R about P must equal the sum of the other three moments, or 48. The perpendicular distance between P and the direction of the force R is 3, therefore if the moment of R must be 48, and the arm is 3, R must equal $\frac{48}{3}$, or 16. This leads us to a second proposition, viz.:

If any number of forces act on a body, then for the body to be in equilibrium, the sum of the moments tending to turn the body in one direction must equal the sum of the moments tending to turn the body in the opposite direction about any given point.

Also, if a force and its moment are known, the arm may be found by dividing the moment by the force, or if the moment and arm are known, the force must be the quotient obtained by dividing the moment by the arm. By means of the two propositions given in this section, the supporting forces for any form of truss or variations in loads may be computed.

119. SUPPORTING FORCES FOR TRUSSES SUPPORTED AT EACH END.—The larger number of roof trusses are of this class, and, as a rule, the loads are considered as acting vertically, as is always the case with dead loads, so that the computations for the supporting forces are very simple.

In every truss with end supports and vertical loads symmetrically disposed with reference to the span, the supporting forces are equal, and each is equal to one-half the sum of the loads.

By symmetrically disposed, we mean that every load on one side of the center must be balanced by an equal load on the other side, and applied at the same distance from the center. It does not matter whether or not the truss itself is symmetrical nor what its shape may be, provided that the loads are symmetrical.

The trusses shown in Figs. 249, 252, 253, and 256 are both symmetrical and symmetrically loaded, and this is true of most of

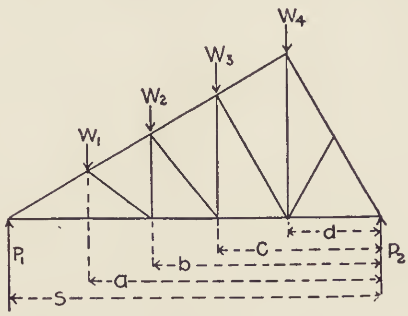


Fig. 260.

the trusses that have been shown thus far. Fig. 116 is an example of an unsymmetrical truss, symmetrically loaded.

TRUSSES UNSYMMETRICALLY LOADED.—For trusses unsymmetrically loaded, the simplest method of finding the amount of each supporting force is by the method of moments, which is as follows:

First draw a diagram of the truss, representing the center lines of the members.

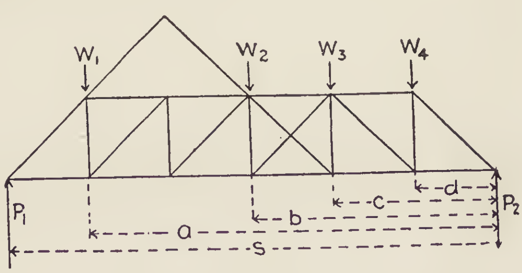


Fig. 261.

2d—Indicate the loads in their correct position, by arrows, with the amount of the load in figures above, as in Figs. 260 and 261, and then scale the horizontal distances from the loads to the right-hand support (corresponding to the distances a, b, c, etc.) as accurately as possible.

3d—Multiply each load by its distance from the right support, add together the products, and divide their sum by the span; the quotient will be the amount of the left support, or P_1.*

4th—From the sum of the loads subtract the value of P_1, and the remainder will be the amount of P_2. Thus in either of Figs. 260 or 261,

$$P_1 = \frac{W_1 \times a + W_2 \times b + W_3 \times c + W_4 \times d}{s}$$

$$P_2 = (W_1 + W_2 + W_3 + W_4) - P_1.$$

It is important to take the measurements a, b, c, etc., from the truss diagram, or from the intersections of the center lines, because it is these lines which are used in drawing the stress diagram, and that the latter shall work out correctly, the supporting forces must be determined as accurately as possible. The method of drawing the truss diagram is more fully explained in Section 129.

120. EXAMPLE 9.—Find the supporting forces for the truss shown by Fig. 262, the loads being in pounds.

Ans.—Multiplying each load by its distance from the right support, and adding together the products, we have:

3,800 × 28	=	106,400
4,300 × 20	=	86,000
11,500 × 20	=	230,000
4,800 × 10	=	48,000
24,400		470,400

Dividing the sum of the products (470,400) by the span (36), we obtain for the quotient 13,066 lbs., which is the value of P_1. Subtracting this amount from the sum of the loads, we have 11,334 lbs. as the value of P_2.

EXAMPLE 10.—A good example of an unsymmetrical truss, unsymmetrically loaded, is shown in Fig. 263, which represents one of the half diagonal trusses, shown on the plan, Fig. 162. [Although these trusses have been called half trusses, each of them is a full and complete truss, the term half truss being used because they only extend from the supporting corner to the center of the through truss.] Computing the roof and ceiling areas contributory to the joints, in the manner illustrated by Example 4, and multiplying by 42 lbs. per sq. ft. for the roof, and 15 lbs. for the

*It is immaterial whether the moments are taken about the right or the left support, but if taken about the left support, the quotient will be the amount of the right support.

ceiling, we obtain the joint loads indicated in the figure. The horizontal distances are measured to the intersection of center lines.

Taking moments about P_1, we have for moments

(2,360 + 450) × 7½′	=	21,075
(4,800 + 5,000) × 15′	=	147,000
Sum of loads = 12,610 lbs.; sum of moments..	=	168,075 ft.-lbs.

Dividing the sum of the moments by the distance between supports, $23\frac{2}{3}$ ft., we obtain 7,100 lbs. as the reaction at P_2, and P_1

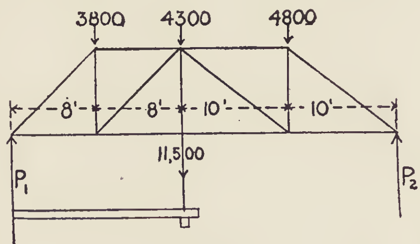


Fig. 262.

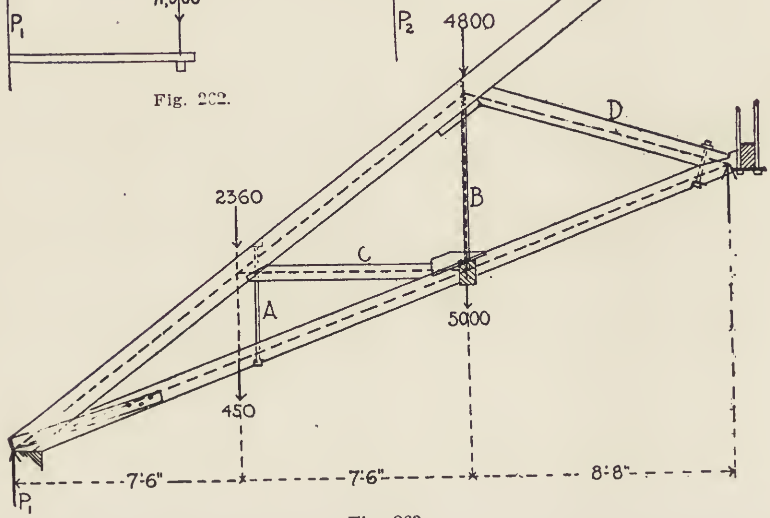


Fig. 263.

must be the difference between the sum of the loads and P_2, or 5,510 lbs. As there are two of the half trusses, the through truss must be figured for a load at the center of 14,200 lbs., in addition to the weight of the roof and ceiling which it supports directly.

It should be remembered that in computing the moments of a force, the arms must always be measured perpendicular to the direction of the force, and as dead loads always act vertically, the arm must always be measured on a horizontal line.

Other examples of finding the supporting forces are given in Chapter VIII.

121. SUPPORTING FORCES FOR CANTILEVER TRUSSES.—The supporting forces for cantilever beams or trusses may be found in the same way as for trusses supported at the ends, but to make the method perfectly plain we will give a few examples.

EXAMPLE 11.—To find the respective loads on P_1 and P_2, Fig. 257.

Ans.—We can take moments about any point in the beam, but in this particular case it will be simplest to take moments about a point directly over the center of P_2. The weights to the left of P_2 tend to turn the beam down to the left, and we will call their moments plus. The weight to the right of P_2 tends to turn the

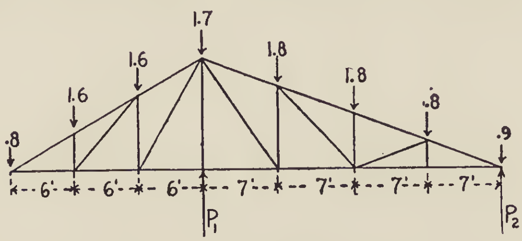


Fig. 264.

beam in the opposite direction, and we will call its moment minus.

The weight directly above P_2 has no moment, as its arm is 0. Multiplying each weight by its arm, we have:

10×22	$= +220$
8×13	$= +104$
7×7	$= +49$
9×3	$= -27$

Total moment $= +373 - 27 = 346$.

This moment must be resisted by P_1 acting with an arm of 17 ft.

Consequently P_1 must equal $\dfrac{346}{17} = 20.35$ lbs. P_2 must be the difference between the sum of the loads and P_1, or $40 - 20.35 = 19.65$ lbs.

EXAMPLE 12.—To determine the supporting foces for the truss shown by Fig. 264, the loads being in tons.

In the case of cantilever trusses it is necessary to figure the loads at the end joints, because at one end they do not come over a support, and therefore they affect the stresses. In this example it will be simpler to take the moments about P_2. Commencing with the load at the extreme left, and multiplying each load by its respective horizontal distance from P_2, we have

.8 × 46	=	36.8
1.6 × 40	=	64.0
1.6 × 34	=	54.4
1.7 × 28	=	47.6
1.8 × 21	=	37.8
1.8 × 14	=	25.2
1.8 × 7	=	12.6
.9 × 0	=	.0
Sum of loads = 12.0	Sum of moments =	278.4

Now the moments of the loads tending to rotate the truss to the left, about P_2, must be balanced by the moment of P_1, and as

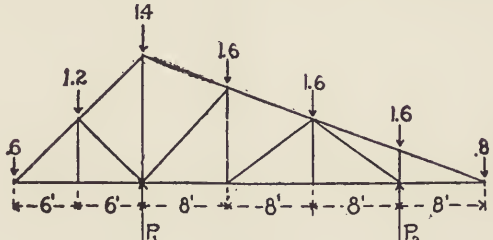


Fig. 265.

its moment must be equal to the sum of the moments of the loads, and its arm is 28, the reaction exerted by P_1 must equal $\frac{278.4}{28} =$ 9.94 tons. As the sum of the reactions must equal the sum of the loads, P_2 must equal 12 — 9.94, or 2.06 tons.

EXAMPLE 13.—To determine the supporting forces for the truss shown by Fig. 265.

Take moments about P_2, the moment of the load at the extreme right being marked minus (—) because it acts in the opposite direction from the other moments.

.6 × 36	=	+21.6
1.2 × 30	=	+36.0
1.4 × 24	=	+33.6
1.6 × 16	=	+25.6
1.6 × 8	=	+12.8
1.6 × 0	=	.0
.8 × 8	=	— 6.4
Sum of loads = 8.8	Sum of moments =	+123.2

$$P_1 = \frac{123.2}{24} = 5.133 \text{ tons.} \quad P_2 = 8.8 - 5.13 = 3\tfrac{2}{3} \text{ tons.}$$

To prove the correctness of the answer, the sum of the moments tending to turn the truss to the left about P_1 must just equal the sum of the moments tending to turn the truss to the right.

The forces tending to turn the truss to the left are P_2 and the two loads on the left of P_1. The four loads to the right of P_1 tend to turn the truss to the right. The load directly above P_1 has no moment. The sum of the moments tending to turn the truss to the left about P_1 is 102.4, and of those tending to turn the truss to the right is also 102.4; therefore the values obtained for P_1 and P_2 are correct.

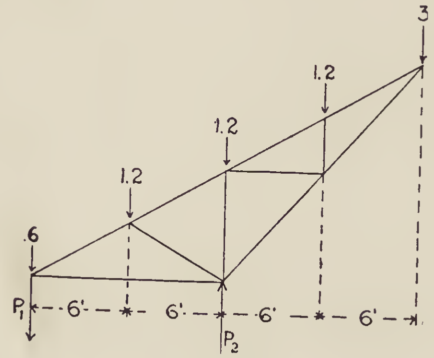


Fig. 266.

EXAMPLE 14.—To find the supporting forces for the truss shown by Fig. 266, which is similar to that shown in Fig. 109, the overhanging end supporting a center king rod truss. In this example we will take the moments of the loads about P_1. Doing so, we have

.6 × 0	=	.0
1.2 × 6	=	7.2
1.2 × 12	=	14.4
1.2 × 18	=	21.6
3.0 × 24	=	72.0
Sum of loads = 7.2	Sum of moments =	115.2

$$P_2 = \frac{115.2}{12} = 9.6 \text{ tons.} \quad P_1 = 7.2 - 9.6 = -2.4,$$

showing that P_1 must pull down, to form an anchorage for the overhanging arm. If now we take the sum of all the moments about P_2, we find them to be + 43.2 and — 43.2, showing that our solution is correct. After working these examples, the reader should have no difficulty in finding the supporting forces for any truss or beam under vertical loads only.

Chapter VIII.

STRESS DIAGRAMS FOR VERTICAL LOADS.

A. TRUSSES SYMMETRICALLY LOADED.
B. TRUSSES UNSYMMETRICALLY LOADED.

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122. EXPLANATION OF THE PRINCIPLES OF GRAPHIC STATICS.

Except for a few forms of trusses, it is much easier to determine the stresses by means of a diagram drawn accurately to a scale than by mathematical calculation, while the representation to the eye of the forces which exist in the several parts of a frame or truss gives one a better understanding of the actual conditions which exist than when the stresses are worked out by means of formulas.

In determining stresses by the graphic method, also, any mistakes or errors are much more likely to be discovered, than in mathematical calculations, while with ordinary care the stresses may be determined as accurately as the several parts of the frame can be proportioned.

Statics is that branch of mechanics which treats of forces in equilibrium, i. e., as balanced.

Graphic Statics is the representation of the different forces graphically, or by means of lines.

The loads and supporting forces of a truss, which are commonly designated as the *external forces,* produce *stresses* in the members of the truss, which are also represented as forces.

A Stress Diagram is a drawing made to a scale, usually of pounds to the inch, which represents the stresses and also the external forces which act in and on a frame or structure.

123. Stress diagrams, for trusses and framed structures, are based upon the following propositions, which, although quite simple in themselves, must be well understood before one can draw a stress diagram intelligently.

(1) *Any force may be represented by a straight line, drawn to a scale.* The length of the line, measured by the scale, gives its magnitude; the position of the line shows the line of action of the force, and an arrow head, the direction in which the force acts.

(2) If two forces applied at a point and acting in the same plane

be represented in direction and magnitude by two intersecting straight lines, the resultant force will act in the direction of, and be equal to, the diagonal of the parallelogram formed on these lines.

Thus, if the lines A B and A C (Fig. 267) represent two forces acting on one point A, and in the same plane, then, to obtain the force which would have the same effect as the two forces, we complete the parallelogram A, B. C. D, and draw the diagonal A. D. This line will then represent the resultant of the two forces. It also gives the direction and magnitude of the single force that would balance the two given forces, only the balancing force must act in the opposite direction.

Also to obtain the diagonal, it is not necessary to draw the full parallelogram, for as we know the magnitude and direction of the two forces, we can draw one, as A C (Fig. 268), and from the end of the line, draw the other, as C D, and the line connecting the points A and D will be the required diagonal. It is also evident that it makes

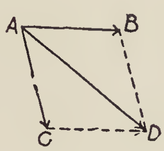


Fig. 267.

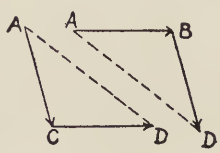


Fig. 268.

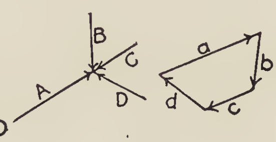


Fig. 269.

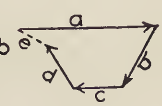


Fig. 270.

no difference which force we draw first, thus, if we draw A B and then B D we obtain the same diagonal as in the first operation. The principle elucidated in this proposition is the same as explained in § 4, page 16.

124. (3)—*If any number of forces acting at a point can be represented in magnitude and direction by the sides of a polygon taken in order, they will be in equilibrium.* Thus, let A, B, C and D (Fig. 269) represent four forces applied at one point and acting in the direction of the arrow heads; now, if the four forces balance each other, if we draw a line a, equal and parallel with A, and from the end of a, a line b, equal and parallel to B, and so on with c and d, the line d will just close the polygon. If it does not close the polygon the forces are not balanced and either one or more of the forces must be changed, or another force added. Thus, if the forces were of such magnitude that when drawn in order to a scale, they formed an open polygon, as in Fig. 270, either the magnitude of two of the forces must be changed or an additional force added, equal and

parallel to e, to produce equilibrium, also if all but two of the forces are known, we can obtain the magnitude of those two forces, provided their direction is known. Thus, in Fig. 269, if we know the magnitude of the forces A and B, and the direction of C and D we can find the magnitude of each of these forces required to produce equilibrium by means of the polygon, for we can draw the lines a and b (Fig. 269) to a scale, and parallel to the line of action of the forces, and from the end of b draw a line parallel to C and from the end of a, a line parallel to D. These two lines will intersect and the intersection will give the length of c and d, and by measuring the length of these lines we can obtain the required magnitude of the forces C and D. It is upon this principle particularly that the stress diagrams are drawn.

125.—*How to tell the character of a force.* But two kinds of forces are involved in the stress diagram of a truss—compressive forces, and tensile or pulling forces, the former indicating compression and the latter indicating tension. In § 4, it was shown that if a piece of material is subject to compression, each end of the piece pushes against the opposing force, or the force acts *outward* at each end, and if it is in tension, there is a pull at each end, or the force acts *inward* at each end. Consequently, when we have a force, with the arrow head pointing *towards* the point of application, it must be a compressive force, and if the arrow head points *from* the point of application, it must be a tensile force.

In a truss the forces are considered as applied at the joints and as the forces in the equilibrium polygon must follow each other in rotation, i. e., the arrow heads must point as in Figs. 269 and 270, around the polygon, we can readily tell in which direction each force acts in relation to any given joint, and if the arrow head points toward the joint it indicates compression. If it points from the joint it indicates tension. In most trusses one can tell by general principles which pieces are in tension and which are in compression, but it sometimes happens that the only way in which one can tell is by the direction of the arrow head, so that the direction of the arrow heads should always be noted in the mind if not drawn on the stress diagram. The foregoing principles in conjunction with those given in §117 and 118 are the only ones ordinarily involved in the construction of a stress diagram, and if these are well understood there should be no difficulty in understanding the diagrams.

126.—APPLICATION OF GRAPHIC STATICS TO SIMPLE TRIANGULAR FRAMES WITH BUT ONE EXTERNAL LOAD.

To show the application of graphic statics in its simplest form, we will take a simple triangular frame, composed of a horizontal beam and two pairs of eye bars, with pin connections, sustaining a single load at the bottom, as in Fig. 271. For convenience of illustration we will assume a load of 100 lbs., the application being precisely the same for any load. Now, at the lower joint d, we have three forces applied, viz., the load, the stress in A and the stress in B. The amount of the load we know, and we also know that its direction is vertical and downward. To find the amount of the stress in A and B, draw a vertical line, 1—2, equal to one hundred lbs. at a scale of, say one hundred lbs. to the inch, with the arrow head at

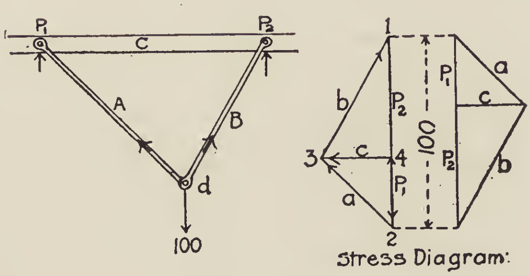


Fig. 271.

the bottom. Then by proposition 3, § 124, for the three forces at d to be in equilibrium they must be represented by the sides of a triangle, taken in order, and the line 1—2 must be one side of the triangle. To find the other sides from one end of the line, representing the load, draw a line parallel to A and from the other end a line parallel to B and continue the lines until they intersect. It will make no difference in the result whether we draw the line parallel to A from the top or bottom of the load line, as is illustrated in Fig. 271, the lines a. a. and b. b. being of the same length in both diagrams. For trusses, however, it is usually more convenient to draw the stress lines so that they will be to the *left* of the load line, as in the left half of the stress diagram, Fig. 271, and in future examples the stress diagram will be drawn in that way. Now, if we letter the lines in the stress diagram to correspond with the lettering of the frame, using small letters for the stress diagram and capital letters for the frame, then the length of the line a, measured by the scale of 100 lbs. to the inch, will give the stress in A, and the length of the line b, gives the stress in B.

Moreover, the arrow heads must follow each other in rotation, and, as the arrow on the load line points down, those on a and b must point as shown in the stress diagram. Now, if we place arrow heads on A and B, pointing in the same way as those on a and b, we see that the heads point from the joint d, and hence, as explained in §125, these pieces must be in tension, as is also obvious from the manner of loading.

Stress in C.—If the beam C had a slot through its center, and the pins at P_1 and P_2 were free to move, it is obvious that the load would cause the pieces A and B to come together until P_1 and P_2 touch. Therefore C must exert a compressive resistance, to hold P_1 and P_2 in place. The amount of this compression is found by drawing a horizontal line from 3 to the load line, and measuring its length by the scale of 100 lbs. to the inch. The line c also divides the load line in the proportion of the vertical reactions at P_1 and P_2. To show that this is true we will consider the forces acting at P_1. At this joint we have three forces, viz., the stress in A, compression in C, and the vertical reaction represented by the arrow, and these three forces must form a triangle in the stress diagram. From the triangle of forces acting at d, we obtained the stress a, so that we have one side of the triangle of forces for P_1, one of the other sides of the triangle must be parallel to C and the third must be parallel to the reaction. Consequently the triangle of forces at P_1 must be 3—2—4 and the distance from 2 to 4 must be the reaction. The arrow heads for this triangle must point in the opposite direction from those in the triangle 1—2—3, or from 3 to 2, 2 to 4 and 4 to 3, so that P_1 and c are both pointing towards the joint and therefore indicate compression. Scaling the lines in the stress diagrams we obtain the following values for the different stresses, viz., a = 52, b = 73, c = 36½, P_1 = 36½, P_2 = 63½ lbs.

If the beam C is inclined, as in Fig. 272, the stresses will be found in precisely the same way, the line c in the stress diagram being drawn parallel to the beam. (Note—The lines in the stress diagrams must *always be drawn parallel* to the corresponding lines of the frame.) If the beam is notched so as to have horizontal seats, and anchored at the top, the reactions at P_1 and P_2 will be vertical.

If the beam is supported at the bottom only and abuts against a corresponding beam at the top, as in the case of the rafters at the top, then the entire reaction at the top will be horizontal, and at the bottom there must be both a horizontal and vertical reaction, the vertical reaction being the full amount of the load, and the horizontal reaction being equal to the product of the load by one-half of the span, divided by the rise, both measurements being in inches.

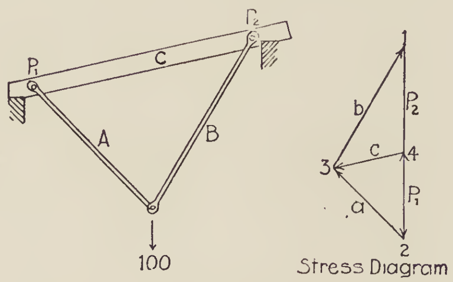


Fig. 272.

If, instead of a suspended load, the load is supported by two struts, notched into a beam at the bottom, as in Fig. 273, the stresses in the struts and beam are found in precisely the same way as in Fig. 271, but in this case it will be found that the arrow heads on a and b point towards the joint, and therefore indicate compression, while there will be a pull in C to hold the ends of the struts together.

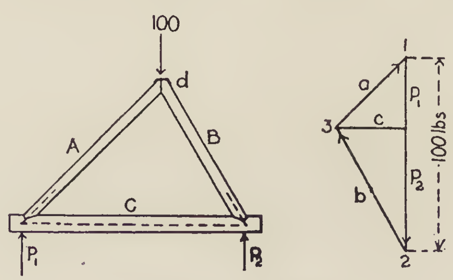


Fig. 273.

127.—*Stresses in a Derrick.*—Fig 274 is intended to represent a boom derrick, with a load of 100 lbs. suspended from the end of the boom. Now, at the point x, three stresses are applied, viz., the load a, tensile stress in C, and a compressive stress in B. To find the stresses in B and C draw a vertical line 1—2 equal to 100 lbs. (the load) at a scale of, say, 100 lbs. to the inch, and from the upper end of this line draw a line parallel to B and from the lower end a line parallel to C, the lines intersecting at 3. Then the line b gives the stress in B and the line c in the stress C. To find the stress in the guy rope D, *when it is in the same plane as the boom,* draw a line from 3 parallel with the guy until it meets the load line at 5. Then the triangle of forces for the point y must be 3—2, 2—5, 5—3, and the arrow heads should point in the direction indicated by the notation. The distance from 2 to 5 must represent the compression in the mast. If we draw a horizontal line from 3 to the load line at 4, then the distance 2—4 represents the vertical component of the stress in C and the distance 4—5, the vertical component of the stress in the guy, and it is evident that the sum of these components must be the stress in the mast, or in other words, the stress in the mast, is the downward pull of C and D. The reaction at P must be the

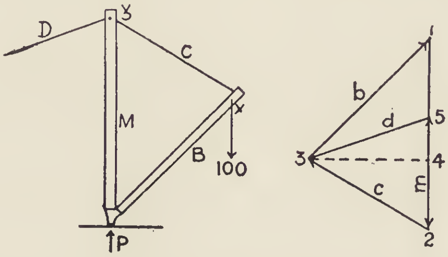


Fig. 274.

stress in the mast, plus the vertical component of b (1 — 4) or the amount of the load, plus 4 — 5 the vertical component of D. The distance 4 — 5 in this example measures 22 lbs., consequently the reaction P must be 122 lbs.

In § 117 it was stated that the sum of the parallel forces acting in one direction must equal the sum of those actions in the opposite direction, and the reader might at first think that the reaction at P should equal the load of 100 lbs., but there is a third vertical force not shown in the figure and that is the anchorage of the guy D. The guy must pull up on its anchorage to the same amount that it pulls down on the mast, which is the distance 4—5 or 22 lbs. To counteract this uplift at the end of the guy there must be a weight or force of 22 lbs. acting downwards, consequently the sum of the weights or forces acting downwards is 122 lbs., which must be resisted by the single reaction at P.

128.—*Points to be observed in connection with the foregoing ex-*

amples.—If we stop to consider the process by which the stresses in the foregoing examples, Figs. 271-274, were obtained, we will notice, first, that the amount or magnitude of the stress in the different members of the frame depends only upon the load and the inclination of the piece, and is not affected by the length of the member. Second, that the stresses are proportional to the load. Thus, in any one of the foregoing examples, if the load is doubled, the magnitude of each line in the stress diagram will be doubled, for doubling the load will be the same as increasing the scale in the same ratio, i. e., a line one inch long represents 100 lbs. at a scale of 100 lbs. to the inch, or 200 lbs. at a scale of 200 lbs. to the inch.

APPLICATION OF GRAPHIC STATICS TO TRUSSES WITH VERTICAL LOADS.—Before we can show the application of graphic statics to trusses having loads applied at several points, it is necessary to first describe the truss diagram and the notation employed in lettering both the truss diagram and the stress diagram.

129.—*Truss Diagram.*—After the truss has been drawn out, and the joint loads computed, the truss diagram should be carefully drawn, with the loads and supporting forces indicated, and then lettered as described in § 130.

In the truss diagram the members *must meet* at the joints, otherwise it will be impossible to draw the stress diagram. Theoretically the truss diagram should coincide with lines drawn longitudinally through the centre of each member, but in practice it is generally impracticable to locate all the members of a wooden truss so that their center lines will meet at the joints, and in such cases, the truss diagram is drawn by taking the centre lines of the principal members and usually of the rods, and then connecting the joints for the braces.

Thus, to draw the truss diagram for the truss Fig. 249, draw lines through the centre of the tie beam, rafters and rods, and for the braces connect the points where the lines through the centre of the rods intersect the rafters and tie beam, as in Fig. 276, which is the truss diagram for this truss. For the queen rod truss, Fig. 256, draw centre lines for the tie beams, rods and straining beam. Obtain the intersections for the end joints by drawing short lines through centre of rafters, intersecting centre line of tie beam and connect these points with the intersections of centre lines of rods, with centre lines of straining beam. For the truss shown in Fig.

253, draw lines through the centre of the beam, rafters and rods, and connect the points of intersection for the braces.

For Howe Trusses, draw lines through the centre of the chords and rods, and connect the points of intersection for the braces, the end points being obtained as explained above for the queen rod truss.

Sometimes the lines of the truss diagram will vary considerably from the actual centre lines of the truss, as for instance, in Fig. 263. For this truss we would draw centre lines through the tie beam and rafter, and the rod B, but instead of drawing a line through the centre of rod A, it will be better to locate the line for this rod directly under the point where the load is applied. As a rule any reasonable variation between the truss diagram and the actual centre lines will not materially affect the amount of the stresses, but to some extent it produces a bending movement in the rafters or tie beams, which should be avoided as much as possible. For steel trusses, the truss diagram and actual centre lines should practically coincide with each other. After the lines of the truss diagrams are drawn all of the external loads and the supporting forces should be indicated by lines and figures as in the following examples. If the truss is symmetrical and symmetrically loaded, it is only necessary to show a little more than one-half of the truss. If the truss is not symmetrical, or is not symmetrically loaded, then the diagram for the entire truss must be drawn.

130.—METHOD OF LETTERING THE TRUSS DIAGRAM. —The method of lettering the truss and stress diagrams used in the following examples is after what is known as "Bow's Notation," and aids the student very materially in tracing the stresses in the stress diagram, and in avoiding errors, besides affording a means for designating the various lines and a ready comparison of the stress diagram with the truss diagram.

The essential feature of this notation is that in the truss diagram a letter is given to the *Spaces* between the lines or forces, and each line or force is designated by the letters *on each side* of it. Thus in Fig. 275, the entire space cut off by the supporting force P_1, the rafter and the load line at joint 2 is designated by A, the space between the load lines at 2 and 3 and bounded by the rafter is designated by B, and the entire space beneath the truss and between P_1 and P_2, by O. The force P_1, being between spaces O and A, may be designated as O A. The load at 2, represented by the arrow, is between A and B, and hence is designated as AB. The lower half of the rafter is designated as AE, and the left half of the tie beam as EO.

In the stress diagram, the same letters come at the ends of the corresponding lines, so that the line ae in the stress diagram shows the magnitude of the stress in AE. *Capital Letters* are always used on the truss diagram and small letters on the stress diagram, hence in the description capital letters always refer to the former and

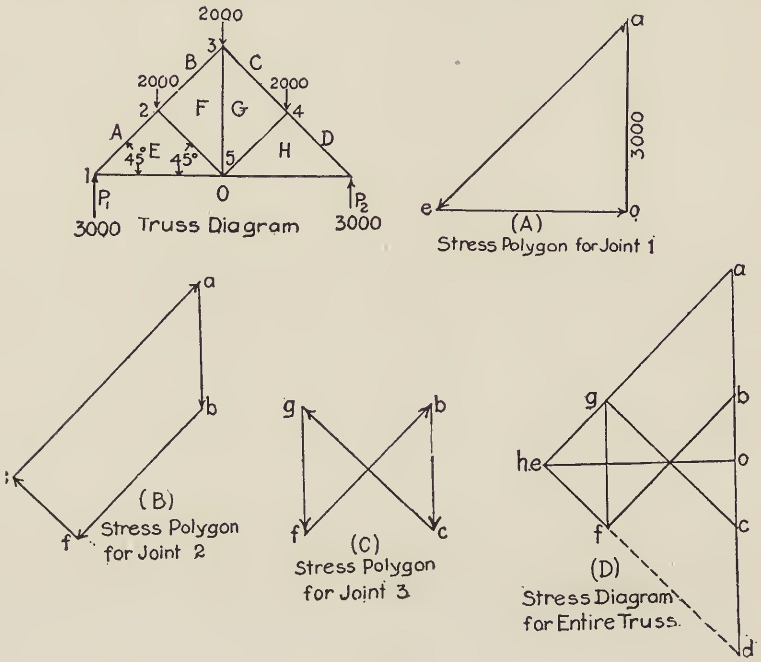


Fig. 275.

small letters to the latter. It does not make any difference what letters are used but the author finds it convenient to use the first letters, as A B C, etc., to denote the spaces above the truss, and between the forces, the first left hand space being always designated as A.

Whenever it is found that the stress diagram cannot be lettered to correspond with the truss diagram, then we may be sure that some mistake has been made either in the lettering of the truss diagram or in drawing the stress diagram.

Problem 1, Fig. 275.—As the first problem in drawing the stress diagram for a truss, we will take a simple king rod truss, with three external loads. The truss diagram should be carefully drawn to a scale of 1/4-inch or 1/8-inch, to the foot, as most convenient, and lettered as in Fig. 275. The stress diagram should be drawn on the same sheet of paper, and as close to the truss diagram as practicable without encroaching upon it. In this first problem a separate stress diagram for each joint is shown in order to more clearly explain the principle, but in practice one diagram is drawn to show the stresses for the entire truss.

In drawing the stress diagram, the first line to be drawn (except in a few special cases) is a vertical line representing one of the supporting forces, and for trusses supported at each end it is more convenient to start with the left hand support P_1, and to take the stresses acting at that point (joint 1) in order from left to right. In this instance P_1 = 3,000 lbs., and a convenient scale will be 2,000 lbs. to the inch. (Note.—For measuring the stress diagram an engineer's scale, graduated to 20ths, 30ths, 40ths, 50ths, etc., of an inch should be used.) The stress diagrams in Fig. 275 were drawn to a scale of 20ths (2,000 lbs. = 1 inch or 100 lbs. = 1/20 of an inch) and reduced by the photo-engraving process to 2/3ds the original size, so that in the engraving the scale is about 3,000 lbs. to the inch. The scale to be used in any particular case will depend upon the amount of the loads. If P_1 is less than 8,000 lbs., a scale of 2,000 lbs. to the inch is best. When P_1 is between 8,000 and 18,000 lbs., a scale of 3,000 lbs. to an inch is more convenient. As a rule the scale should be such that no line in the stress diagram will exceed 14 inches in length.

We will commence our stress diagram, therefore, by drawing a vertical line oa (Diagram A) = 3,000 lbs. or 1½ inches in length.

As the supporting force acts *upwards,* the arrow head should be at the top, and the line must be lettered oa. The letter designating the space to the right of P_1, being O, o must be at the bottom of the line and a, at the top. The next force acting at joint 1, going around as the hands of a clock, is the stress in AE, and as the arrows must always point from each other, the line in the stress diagram representing AE must start from a. The fact that we already have the letter a, also shows that ae must start from a, then, from a draw a line of indefinite length parallel with the line AE. The next force acting on the joint is the stress in E O, which, with oa and ae must form a closed triangle in order to preserve equilibrium. Therefore, the stress line for EO must pass through o, and to obtain the point e, draw a line through o, parallel with OE, until it intersects the slanting line drawn through a. The point of intersecsection of these two lines gives the point e, and the stress diagram for joint 1, is the triangle oae. The arrow head on eo must point towards o, to preserve the rotation. If now we notice the direction of the arrow heads we see that the head on ae points towards joint 1, and that on eo from joint 1, consequently the former denotes compression and the latter tension.

Stress Polygon for Joint 2.—At joint 2 we have two known forces—the stress ea and the load, and two unknown forces. To show the stress polygon for joint 2, separately, draw a line ea parallel to AE, and of the same length as in diagram A (Fig. 275), and from a, draw a vertical line equal to 2,000 lbs.—the amount of the load. The arrow head on ea will now point in the opposite direction from that in diagram A (see § 125), or towards a, and the polygon begins with the point e. To obtain bf and fe, from b and e draw lines parallel to BF and FE, respectively, and letter the point of intersection f. Polygon B then represents in direction and magnitude all of the forces acting at joint 2.

As the stresses for corresponding members on each side of the stress must be alike, diagram A and B give the stresses for all of the members of the truss except the centre rod, FG. To obtain the stress in this member we may draw the diagram for either joint 3 or joint 5, but that for joint 3 is the simpler. *At joint 3* we have the known forces fb and bc (the load) and the unknown stresses in CG and GF, which we obtain by drawing lines through c and f (diagram C), parallel respectively to CG and FG. The arrow head on fb must point in the opposite direction from that in diagram B, or towards b, and the heads on the other lines must point from each other as shown. We see that the head on gf points from the joint, indicating tension. By scaling the lines in the three diagrams A, B and C, with the scale to which oe was drawn, we obtain the magnitude of the stresses, or 4,440 lbs. for ae, 3,000 lbs. for eo, 1,420 lbs. for ef, 2,830 lbs. for bf and 2,000 lbs. for fg.

As the truss rafters are drawn at an angle of 45 degrees, it is evident that oe must be equal to oa, and that $ea = 3000 \times \sqrt{2}$

Stress Diagrams for Entire Truss.—In practice it is much easier to show all of the stresses in one diagram, as at D, which contains all of the lines shown in diagrams A, B, and C, and is drawn in the same order as given above for the separate diagrams. In the single diagram, however, it is not practical to show the arrow heads, as each line would require two heads, and they become confusing in the engravings. The direction in which the forces act, however, must be kept in mind, and a few of the heads may be indicated in pencil, if necessary.

In diagram D, if having drawn the stresses at joint 3, we wish to draw those for joint 4, we start with the line gc, already drawn, which must act towards c, lay off the distance cd = 2000 lbs., and from d, draw a line parallel to DH, and from g, a line parallel to GH, the two lines intersecting at h, and we have the completed diagram for the entire truss, which must be symmetrical about the line eo. As DH and FE are parallel, also AE and GH, dh and gh lie over fe and ae, and the points h and e coincide. This often happens in stress diagrams, but each line should be considered as a *separate line* for its entire length, and properly lettered, as the lettering assists very effectually in avoiding mistakes.

Problem 2. Fig. 276. In this figure, we have shown the truss diagram for the truss shown in Fig. 249, and for which the roof loads are computed on page 234, example 5, truss 3. This truss in addition to the roof loads also has three ceiling loads which must be considered. *When the ceiling loads are directly supported by vertical rods* these loads may be considered as applied at the top of the truss and added to the roof loads, without affecting any of the stresses *except* those in the rods supporting the respective ceiling loads. As the stress diagram is a little simpler, when the loads are applied at the top only, the author generally follows this method of adding the ceiling loads to the roof loads directly above, and drawing the stress diagram as if there were no ceiling loads. When this is done, however, the ceiling load must be added to the stress given by the stress diagram, to obtain the true stress in the rods.

In the truss in question there will be no stress in the outer rods from the roof loads, and the tension in them will be just the amount of the ceiling loads, or 1800 lbs. In the center rod the real stress will be that shown by the stress diagram, or 7,200 lbs. plus the ceiling load of 1,950 lbs.

That the ceiling load may not be overlooked in computing the total stress, it is well to put the loads beside the member in the truss diagram, preceded by the sign +.

It is also a good plan to put on the sheet the loads per sq. ft. for which the truss is designed, and also the spacing of the trusses or if the trusses are not spaced evenly, the length of roof which the truss supports. All of the data relating to the strength of the truss will then be on one sheet.

In the truss diagram, members which do not enter into the stress diagram should be shown by dotted lines, and the diagram lettered

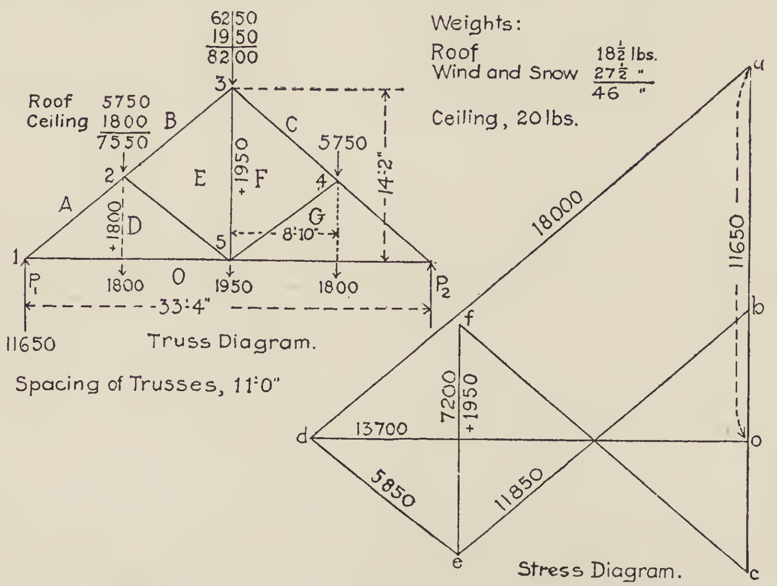


Fig. 276.

as though these members did not exist. Thus in the truss diagrams, Fig. 276, the letter D refers to the entire triangle 1-2-5.

The stress diagram is drawn in precisely the same way as the diagram in Problem 1, commencing by drawing the line oa = to P_1, 11,650 lbs. at a scale* of, say, 3,000 or 4,000 lbs. to the inch. From a, draw a line parallel to AD, and from o, a horizontal line, the two intersecting at d. At joint 2 we have the stress da, and measure downwards from a, the load AB = 7,550 lbs. giving the point b. From b, draw a line parallel to BE and from d, the point

*In the engraving the scale is about 6,000 lbs. to the inch.

of beginning, a line parallel to ED, and lines intersecting at e, then the lines da, ab, be, and ed, form the stress polygon for joint 2. For the stress polygon at joint 3, we have eb, and measure down from b, the load of 8,200 lbs. giving the point c. From c, draw a line parallel to CF, and from e, the point of beginning, a line parallel to FE, the two intersecting at f. We now have the stresses for all of the members of the truss, the magnitude of the stresses being indicated by figures on the stress diagram.

It is not worth while to figure the loads or stresses of a roof truss closer than 50 lbs., as the truss cannot be proportioned with a greater degree of accuracy.

Checking the Stress Diagram. The accuracy of the stress diagram in the case of triangular or Howe trusses can readily be checked by a few simple computations, which it is often well to make. As the lines of the triangle dao are parallel respectively to those of the triangle 1-3-5, the triangles must be similar, consequently,

$$1\text{—}3 : 3\text{—}5 :: da : ao, \text{ or } da = \frac{1\text{—}3 \times ao}{3\text{—}5}$$

and $$1\text{—}5 : 3\text{—}5 :: do : ao, \text{ or } do = \frac{1\text{—}5 \times ao}{3\text{—}5}$$

In this case, $1\text{—}5 = 16'\ 8'' = 200''$; $3\text{—}5 = 170''$; $1\text{—}3 = \sqrt{200^2 + 170^2} = 262\tfrac{1}{2}$, and $ao = 11{,}650$ lbs.

Therefore, $$da = \frac{262.5 \times 11{,}650}{170} = 17{,}989 \text{ lbs.,}$$

and $$do = \frac{200 \times 11{,}650}{170} = 13{,}706 \text{ lbs.}$$

The other stresses can also be checked in a similar manner, but if the principal lines of the stress diagram scale within 100 lbs. of their computed values, then the rest of the diagram will be likely to be fully as accurate.

Data for easily checking the stresses in end panels of all triangular and Howe Trusses, supported at each end, *and having the main tie horizontal.*

In the above examples we used the full half span, and the full rise to obtain the stress in do, but we would have obtained the same result by using any two numbers having the same ratio to each other, as 100 and 85, or 20:17, or in other words, it is the proportion which the sides bear to each other, or the inclination of the strut which determines the stresses, and not the actual dimensions of the panel.

For trusses supported at the ends and with the main tie horizontal, the stresses in the end panel are governed solely by the supporting force, and the angle of the strut, hence *for any given angle the stresses may be found by multiplying the supporting force by the factors for that angle.*

Table XII gives the factors for a number of angles, expressed in degrees, and also by the rise in 12 ins. These factors may be used for unsymmetrical, as well as symmetrical trusses, and also for trusses unsymmetrically loaded, if the supporting forces are correctly computed, but they *do not apply* to trusses in which the main tie is inclined. They also give the stress for the end panel only. For angles not given in the table, it will be necessary to check the stresses by proportion as in the foregoing example.

TABLE XII.—FACTORS FOR COMPUTING STRESSES IN END PANELS OF TRUSSES WITH HORIZONTAL TIE BEAM.

Inclination of strut.	Rise in 12 Ins.	Factor for strut.	Factor for tie beam.
60°	20.78 Ins.	1.155	.5773
51° — 20′	15 "	1.28	.8
49° — 24′	14 "	1,317	.857
47° — 17′	13 "	1.36	.923
45°	12 "	1.414	1.
42° — 30′	11 "	1.48	1.091
41° — 11′	10½ "	1.518	1.143
39° — 48′	10 "	1.562	1.2
36° — 52′	9 "	1.666	1.334
33° — 41′	8 "	1.803	1.50
30°	6.93 "	2.	1.732
26° — 34′	6 "	2.236	2.
22° — 30′	4.97 "	2.611	2.414

Problem 3. Fig. 277. For this problem we will take the same truss as in Problem 2, and show how the stress diagram is drawn when the ceiling loads are not combined with the roof loads. In this case the outer rods will be represented in the stress diagram, and must be shown in the truss diagram by full lines, and each triangular space must have a separate letter. In order to facilitate a comparison between the diagrams in Figs. 276 and 277, the same letters have been retained, and the letters D′, R and S, added in Fig. 277.

As the total loads is the same in each problem the supporting forces must be alike.

At joint 1, we have precisely the same forces acting as in Fig. 276, consequently the triangle of forces for this joint will be the same in both problems, or in Fig. 277, we start with oa = 11,650 lbs. and draw ad and od, precisely as in Fig. 276, and if drawn to the same scale, the corresponding lines must be of the same length. At joint 3, Fig. 277, we now know the stress in AD and the load AB, but we have three forces that we do not know, viz., the stress in BE, ED′ and D′D, and *when there are three unknown forces* we cannot draw the stress diagram for that joint until one of the forces has been found or assumed. At joint 2, we have but two unknown forces, viz., the stress in DD′ and D′R, therefore, we can draw the stresses for this joint. In drawing the stress diagram for any joint, always commence with the known force, furtherest around to the right, which in this case is the load RO. Now, as we already have

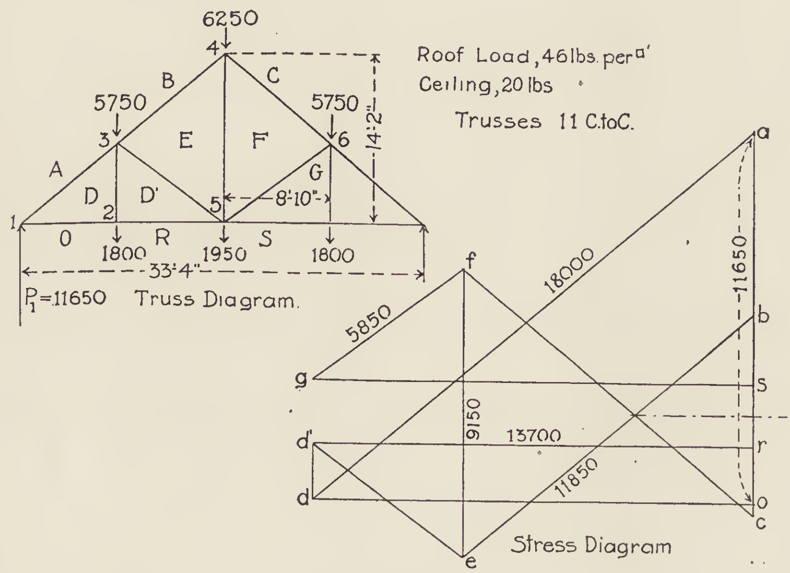


Fig. 277.

the point o, we know that one end of ro must be at o, and as the force acts down, the other end must be *above*, therefore to find the point r, we measure upwards from o, 1,800 lbs., and r is the beginning point of the stress polygon for joint 2. We now have ro, od, and from d, draw a line parallel to DD′ and from r, the point of beginning, a line parallel to D′R, the two intersecting at d′. As we now have the stress d′d, we can proceed to complete the stress diagram for joint 3. The first known force at joint 3 is d′d, acting down, or away from the joint, next, da, then measure down from a, 5,750 lbs., which gives the point b, and the point e is obtained by drawing lines through b and d′ parallel respectively to BE and ED′. Next, complete the stress polygon for joint 4. Here we have the lines eb and measure down from b, the load of 6,250 lbs., which gives the point c. Through c and e draw lines parallel respectively to CF and FE which will intersect at f. As we now have the stress in the center rod and all of the members to the left, it is not necessary to carry the diagram further, but to show that the completed diagram would be symmetrical, we will carry it one step farther, by completing the stress diagram for joint 5. The first known force at this joint,

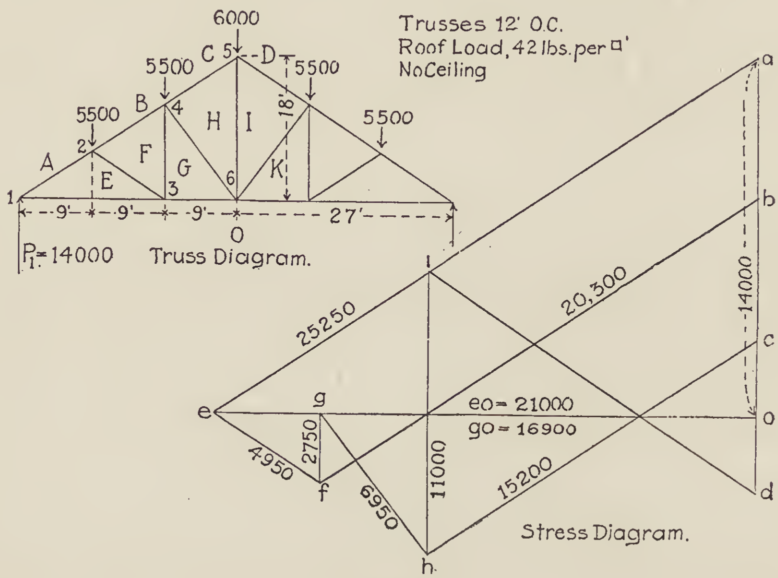


Fig. 278.

around to the right, is the load of 1,950 lbs., which termintes at r, and as it acts down, s must be above it, a distance of 1,950 lbs. We then have sr, rd′, d′e, ef, and draw gs and fg. Now if the diagram has been accurately drawn a horizontal line drawn halfway between the points s and r will pass through the intersection of eb and cf and bisect fe, and if the stress diagram is carried out one step farther so as to show the forces acting at joint 6, the diagram will be found to be symmetrical about this center line.

If now we compare the stress diagram in Figs. 276 and 277, we will see that the slanting and horizontal lines are of the same length in both diagrams, and that fe in Fig. 277 is just 1,950 lbs. longer than in Fig. 276, which is the amount of the ceiling load sustained by the rod. The length of dd′ is the same as ro, or the stress is just the amount of the ceiling load. The same result is obtained, therefore, by either method.

Problem 4. Fig. 278. Six panel queen truss with roof loads only. Commence the stress diagram by drawing the line oa = P_1 = 14,000 lbs., then draw ae and oe as in the preceding problems. Complete the polygon for the different joints in the order in which they

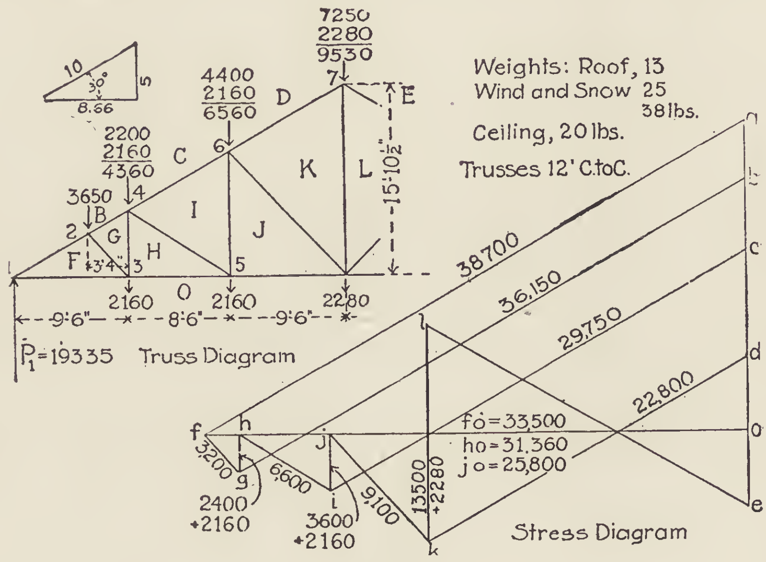


Fig. 279.

are numbered. The rise of the rafter is 2′ in 3′ or 8″ to the foot, and by Table XII. eo must equal 14,000 × 1½ or 21,000 lbs.

Problem 5, Fig. 279. This represents the truss shown by Fig. 253 and for which the roof loads are computed in example 6, page 234. It is assumed that the truss also supports a ceiling for which the joint loads have been computed on a basis of 20 lbs. per sq. ft. In this problem the ceiling loads are first added to the roof loads directly above, and also added to the stresses for the rods given by the stress diagram, as explained in Problem 2. The stress diagram is drawn precisely in the same manner as that for problem 4, taking the joints in the order in which they are numbered, fo should equal $\frac{19{,}335 \times 8.66}{5}$ or 33,488 lbs., and if it scales within 100 lbs of this value, it will show a good degree of accuracy. The stress diagram should be drawn to a scale of 4,000 lbs. to the inch.

Problem 6, Fig. 280. This truss has the same span and rise as that shown by Figs. 253 and 279, and is figured for the same roof and ceiling loads, but in this truss the struts or braces are arranged to

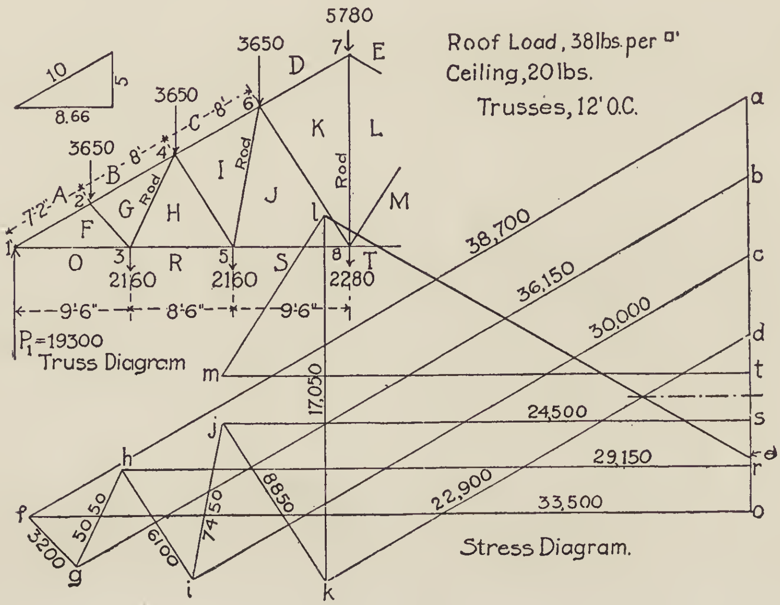


Fig. 280.

come under the purlins so as to avoid a transverse strain in the rafter and the rods are inclined. As the total roof and ceiling areas supported by the truss are the same as those supported by the truss Fig. 279, the supporting forces should have the same value (the difference of 35 lbs. is due to the fact that the joint loads have been taken slightly in excess of the actual computations). As the rods in this truss are inclined, we cannot add the ceiling loads to the roof loads but must show them separately in the stress diagram, as in Prob. 3. When the ceiling loads are shown separately in the stress diagram the spaces between the ceiling loads must each have a special letter.

Commence the stress diagram as in all of the preceding problems, by drawing a vertical line oa equal to P_1 at a scale of say 4,000 lbs. to the inch, and from a and o draw lines parallel respectively, to AF and FO intersecting at f. Then complete the stress polygon for joint 2. Here we have fa, measure down ab = 3,650 lbs. and from b and f draw lines parallel to BG and GF, intersecting at g. For the stress polygon for joint 3, we must start with the load RO, the point r being obtained by measuring upwards from o, 2,160 lbs. We then have ro, of and fg, and from r and g draw lines parallel to HR and GH, intersecting at h. For joint 4 the stress polygon is hg, gb, bc, ci and ih. Continue in the same manner for joints 5, 6 and 7. For joint 8 we start by measuring upwards from s, 2,280 lbs. which gives the joint t. We then have ts, sj, jk and kl, and from l and t draw lines parallel respectively to LM and MT intersecting at m. If the diagram has been accurately drawn the point m will be directly above j, and a horizontal line bisecting ts will pass through the intersection of el and kd, and bisect lk.

As the inclination of the rafter and the amount of supporting force is the same as in Prob. 5, the stress triangle for joint 1 should be the same in the two stress diagrams (if drawn to the same scale) or the stress in AF and FO should be the same in both cases.*

Problem 7, Fig. 281. For this problem we have the case of an unsymmetrical truss, symmetrically loaded. The outline of the truss is similar to that of the truss on the right, Fig. 123. The loads are approximately what they would be for a spacing of 14′ 6″ center to center of trusses.

As the loads are applied at equal distances from the supports, the reactions must be equal and each equal to one-half of the total load, or 10,500 lbs.

The stress diagram is drawn in precisely the same manner as those for the preceding examples, commencing with the supporting force P_1 = oa = 10,500 lbs. The stresses should be drawn in the order in which the joints are numbered. For the stress polygon at joint 5, we have the lines og and gh, and draw hi, intersecting og

*In order to show the stress diagram in Fig. 280 more clearly, it was drawn to a larger scale than that in Fig. 279, but the result is the same. The stresses in the other members of the truss, however, vary considerably, because of the difference in the inclination of the web members. It should be noticed that the stress in the rods GH and IJ is considerably greater in Fig. 280 than in Fig. 279.

prolonged. For joint 6, we now have ih, hc, cd = 7,000 lbs. and a line drawn from d parallel to the rafter must pass through the point i, to close the polygon. If a line from d parallel to DI, does not pass through i, it shows that either the stress diagram or the truss diagram has not been drawn with sufficient accuracy. As a matter of fact, a very slight inaccuracy in the measurements of the truss diagram, or in drawing the lines of the stress diagram perfectly parallel with those of the truss diagram will cause the line through d to pass

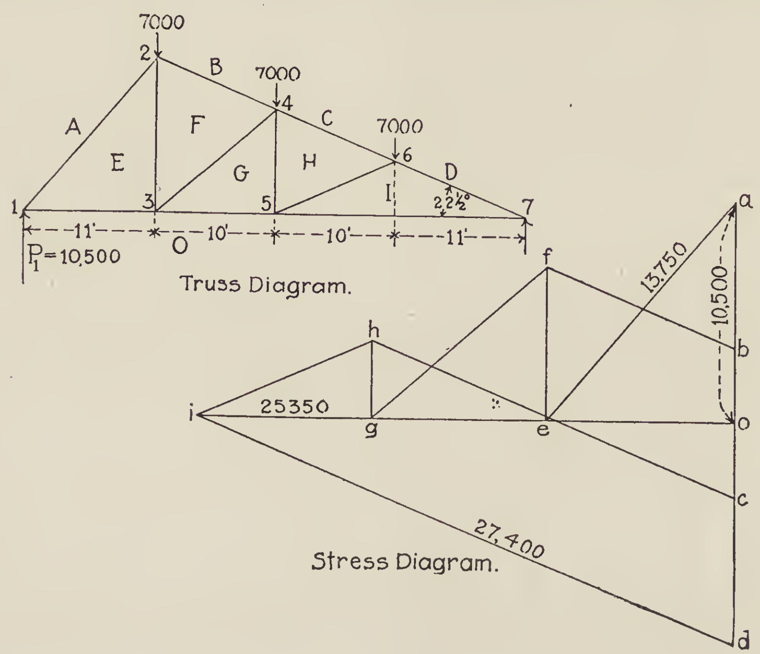


Fig. 281.

to one side of the point i. It is well to notice in this problem that the stresses at the right end of the truss are much greater than those at the left, due to the lesser height on that side. As the angle of the long rafter is 22½ degrees, the length of the line oi should equal 10,500 × 2.414 (Table XII.) = 25,347, which very closely corresponds with the scale.

Problem 8, Fig. 282. The truss diagram is that of the truss shown by Fig. 256, the distance a, Fig. 256 is 9′ 9″, b 13′ 4″, and c, 12 ft. As the trusses are spaced 15 ft. on centers, the roof area supported at joint 2 = 9′ 9″ × 15′, = 146¼ sq. ft. At joint 3, the roof area supported = 13-1/3 × 15 = 200 sq. ft., and the ceiling area supported at joint 7 = 12′ × 15′ = 180 sq. ft. Multiplying these areas by the respective loads per sq. ft., we have in round numbers, the loads indicated on the truss diagram.

In Fig. 256, counter braces are shown in the center panel, but as explained in § 7, there will be no stress in these braces under a symmetrical load; they cannot be shown in the stress diagram and should therefore be omitted from the truss diagram. In this example we have added the ceiling loads to the roof loads directly above, so

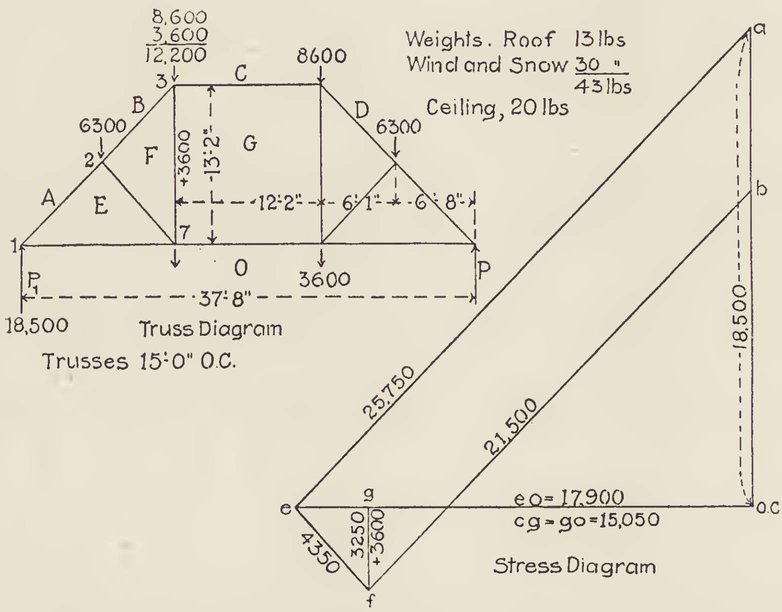


Fig. 282.

that the stress diagram will be drawn as though there were no ceiling loads. To draw the stress diagram commence by drawing the vertical line oa, of a length equal to P_1, or 18,500 lbs. at a scale of 4,000 lbs. to the inch (the scale of the engraving is 7,500 lbs. to the inch) and from a and o, draw lines parallel respectively to AE and EO, intersecting at e. As the supporting force acts upwards, the arrow head on oa will be at a, on ae at e and on eo at o, showing that ae is in compression, and eo in tension.

Next, complete the diagram for joint 2, the first known force at this joint is the stress in the rafter, represented by the line ea, which now acts towards a, next the load of 6,300 lbs., which acts down, and which when measured off to the scale gives the point b, then from b and e draw lines parallel respectively to BF and FE, intersecting at f. The force bf acts towards f, and fe towards e, showing that both members are in compression (as the forces act towards the joint). Next, complete the diagram for joint 3. The first known force at this joint is fb, which now acts towards b, next the load of 12,200 lbs. which takes us to the point o, where we also put the letter c. As the top chord CG is horizontal, and the point c coincides with o, the stress line for C G will lie over or coincide with that for EO, and its length will be obtained by drawing a line from f parallel to GF, until it intersects eo at g. Then cg will be the stress line for CG, with the arrow head pointing towards g and denoting compression. If now we trace the polygon of forces for joint 7, we find that we have oe, ef, fg and that go must close the polygon or that the stress in GO, is just equal to that in CG.

Scaling the stress diagram, we obtain the stress indicated. We should not forget that the true stress in the rod FG is that indicated by the line fg (3250) + the ceiling load of 3,600 lbs. or 6,850 lbs.

Problem 9, Fig. 283. Stress diagram for a truss like that shown by Fig. 19, with dimensions as shown on the truss diagram Fig. 283. It is assumed that the portion of the attic included within the heavy black line is to be finished and used for ordinary purposes. The roof load at 46 lbs. per sq. ft., will allow for a slate roof. With the trusses spaced 12 ft. on centers, the roof loads will be about as indicated by the upper set of figures. Included in the roof loads at joints 2, 4 and 5, is the weight of the plastered ceiling, the stress on the center rod being 1,010 lbs. The load at point 8 is estimated as follows:

42 sq. ft. of floor, at 20 lbs., equals	840
42 sq. ft. of floor, at 60 lbs, equals	2,520
60 sq. ft. of studding, lath and plaster; at 15 lbs., equals	900
Total	4,260

Load at 3 = 10½′ x 12′ x 60 = 7,560 lbs.

The stress diagram for joints 1 and 2 is drawn precisely as in Fig. 281, but in this truss we must complete the stress diagram for joint 3 before we can draw that for joint 4.

At joint 3 the first known force is the stress in OE, represented by the line oe, which acts towards e, next the line ef, and to find the stress in FG and GO draw a line from f parallel to FG, intersecting oe at g; then fg represents the stress in FG, and go the stress in GO, both being in tension. Now, at joint 4, we have gf, fb, obtain the point c by measuring downwards 13,760 lbs, and from c draw a line parallel to CH, and from g, the point of beginning, a line parallel to HG, until it intersects the line from c at h. hg overlays eo, but it should be considered as a separate line. This gives us all of the stresses in one-half of the truss. It should be noticed that in this truss the stress in GH, is less than that in OG, by an amount

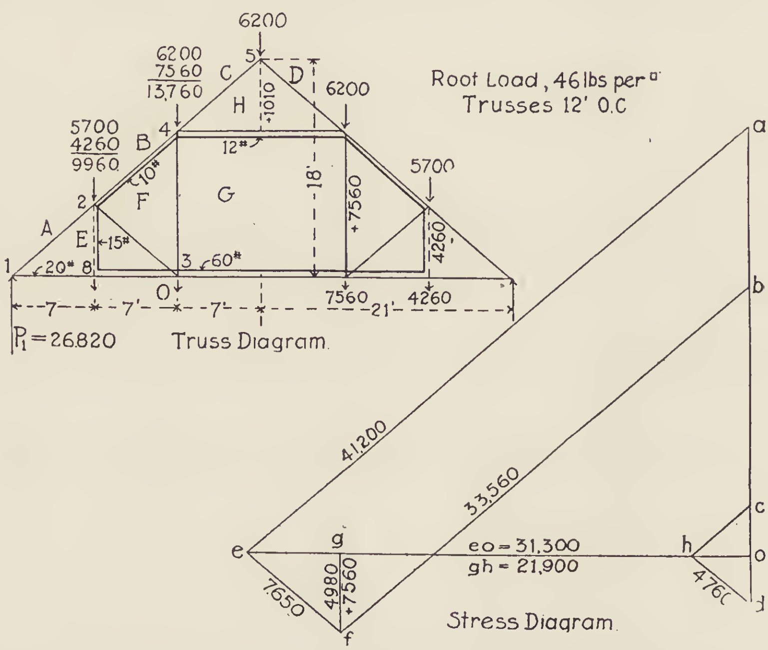


Fig. 283.

equal to the horizontal thrust, or component of CH and DH, that is, the apex load produces a tension in GH, which reduces the compressive stress by that amount.

Problem 10 Fig. 284 Five Panel Howe Truss, supporting a gravel roof, and plaster ceiling. Ceiling loads added to roof loads. To draw the stress diagram commence with the vertical line oa equals P_1, 13,300 lbs. at a scale of 4,000 or 5,000 lbs. to the inch. From a draw a line parallel to AD, and from o a line parallel to DO, the two lines intersecting at d. To complete the stress diagram for joint 2, we have the line da, acting towards a, measure down from a the load of 6,600 lbs. which gives the point b, and from d and b draw lines parallel respectively to ED and BE, intersecting at e. Although the line ed is drawn from d, it acts from e towards d, as the arrow heads must follow each other in succession around the polygon. For joint 3, the stress polygon is od, de, ef and fo. For joint 4, we have fe, starting from the point f, eb, bc equals 6,600 lbs., and the line cg which brings us to the starting point f, leaving no room for a line parallel to GF, thus showing that there is no stress in the rod GF, except that due to the ceiling load of 1,600 lbs. It should

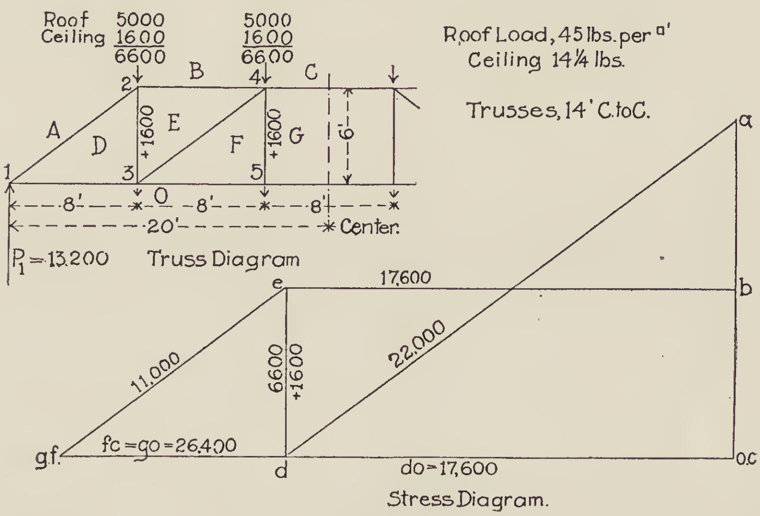


Fig. 284.

not be forgotten that to the stress found by scaling the line ed (6,600 lbs.) should be added the ceiling load to obtain the true stress which in this case is 8,200 lbs.

To check the stress diagram, by Table XII, the rise of end strut being 9″ in 12″, the length of do, should be equal to 13,200 × 1-1/3 equals 17,600.

It should be noticed that the stress in the top chord between joints 2 and 4 is just equal to the tension in the tie beam between joints 1 and 3.

Problem 11, Figs. 285 and 286. 286 shows the truss and stress diagrams for the six panel Howe Truss shown in Fig. 285. The stress diagram is drawn in the same manner as that in problem 10, only carried one step farther, and presents no difficulties. There is no stress in the center rod except that produced by the ceiling load.

Problem 12, Fig. 287. Here we have a very shallow Howe Truss loaded at alternate panel as in Fig. 115. The loads are approximately what they should be for a spacing, c to c of trusses of 14½ ft., with no ceiling loads. The stress diagram for joint 1 is drawn pre-

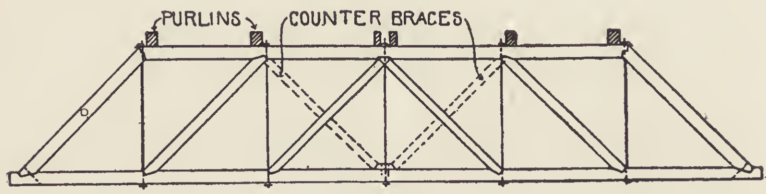


Fig. 285.

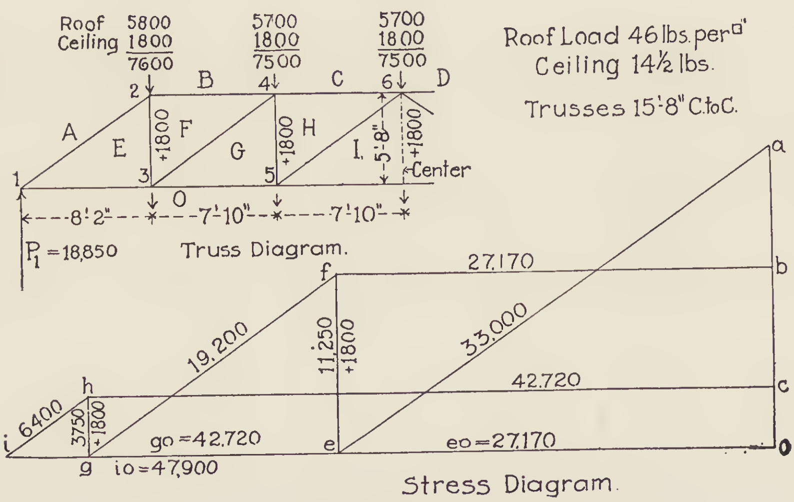


Fig. 286.

cisely as in all of the foregoing problems. For joint 2 we have the stress FA represented by the line fa and as there is no load at this joint, the next force must be the stress in AG, which will be represented by a horizontal line drawn through a, and the figure must close by a vertical line, representing the stress in GF drawn through f, the intersection of these two lines gives the point g. To complete the stress diagram for joint 3, we already have of, and fg, and draw gh and ho parallel, respectively to GH and HO. For joint 4, we have hg, and ga; measure down from a the load of 8,000 lbs., giving the point b, and from b draw a line parallel to BI, and from h (the point of beginning) a line parallel to IH, the two intersecting at i.

For joint 5, the stress polygon is oh, hi, ij and jo.

For joint 6, the stress polygon is ji, ib, bk and kj.

For joint 7, the stress polygon is oj, jk, kl and lo.

For joint 8, the stress polygon is lk, kb, bc and a line from c parallel to CM, which brings us back to the point of beginning, so

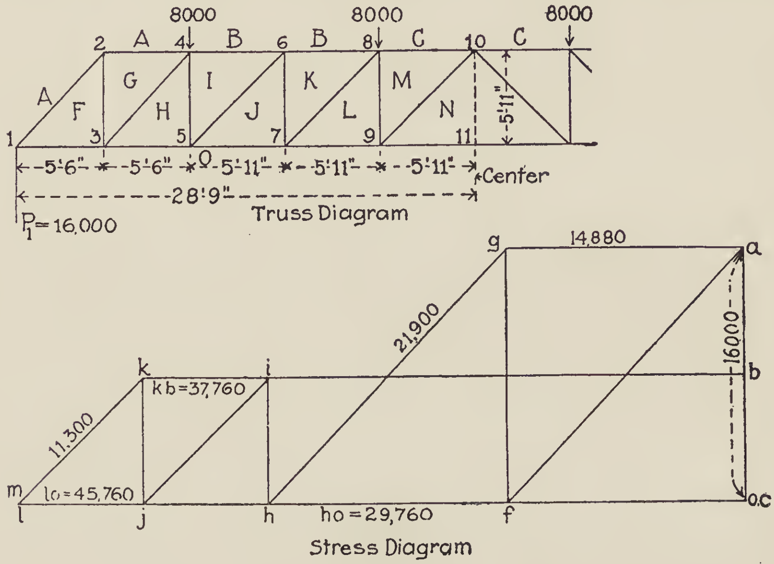


Fig. 287.

that cm coincides with ol, and ml is represented by a dot, showing that there is no stress in ML.

For the stress polygon for joint 9, we have the line ol, and as lm has no length, and the figure must close by a horizontal line through o, it is evident that there can be no stress in MN, and the stress in NO must equal that in MO.

For resisting the given loads therefore, the rod LM and brace MN could theoretically be omitted, but in practice it is a good idea to put in light members to support the chords and keep them from bending under their own weight. With a ceiling load at joint 11, there would be a stress in these members.

Problem 13, Fig. 288. Howe Truss with but one load at the center. For this method of loading the stress diagram is similar to one-half of the truss diagram, the stress in the rods and braces being uniform while the stress in the chords increases with the number of panels. The maximum bending moment for a beam or truss loaded at the center is $\frac{WL}{4}$, L denoting the span.

In this case $\frac{WL}{4} = \frac{4000 \times 32}{4} = 32{,}000$. The stress in the tie beams in the center panel of the truss we find from the stress diagram, to be 8,000 lbs. Dividing the bending moment by this stress

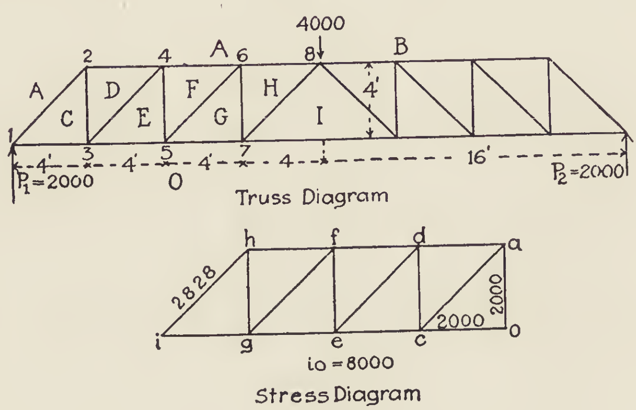


Fig. 288.

we have 4 for the quotient, which is the height of the truss—or the maximum stress in the tie beam of a Howe Truss, is equal to the maximum bending moment in foot pounds, divided by the height of the truss in feet and this is true for all Howe Trusses and also for lattice trusses. For example, the maximum bending moment in the truss, Fig. 284 equals $13{,}200 \times 20 - (6600 \times 12 + 6600 \times 4)$ equals 158,400 foot-pounds. Dividing by the height we have 26,400 lbs. for the stress in the tie beam at the center.

Problem 14, Fig 289. Howe Truss, with inclined top Chord. For this problem we have taken a truss like that shown by Fig. 116. The roof loads were computed at 47 lbs. per square foot and a spacing center to center of trusses of 17 ft. There is no ceiling load. The portions of the top chord beyond joints 2 and 6 (see Fig. 116) receive no strain and are extended merely to facilitate the construction and to stay the truss. The center rod also has no stress, being used

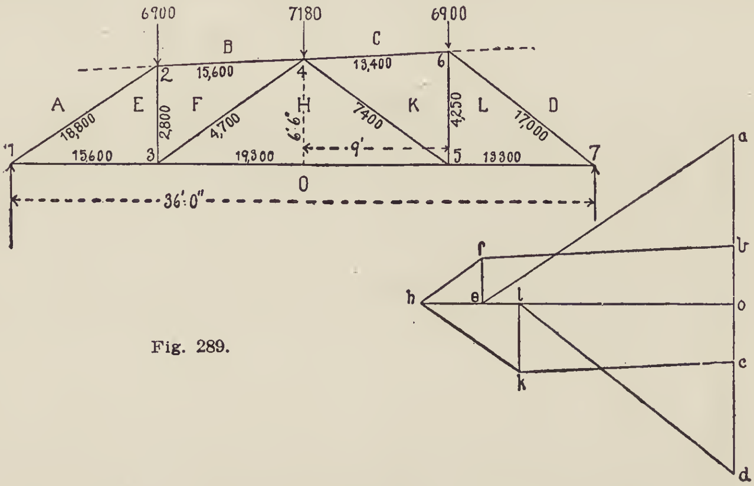


Fig. 289.

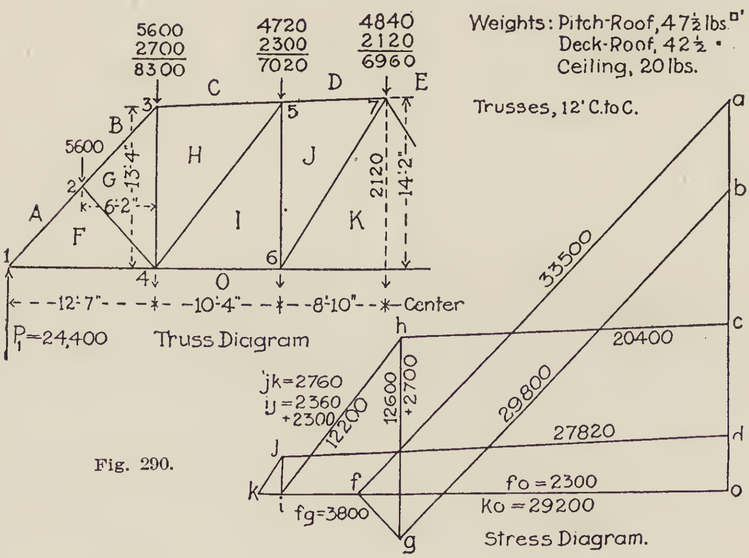


Fig. 290.

merely to prevent the tie beam from sagging. The entire space between the inner brace is therefor designated by a single letter.

The stress diagram is drawn in exactly the same manner as that in problem 11, except that the lines representing the stresses in the top chord are drawn parallel to the slant of the chord instead of horizontal, and as is the case with all unsymmetrical trusses, it is necessary to complete the diagram for the entire truss. In this problem the stresses are given on the truss diagram, instead of on the stress diagram.

Problem 15, Fig. 290. For this problem we will take the truss shown by Fig. 252, and for which the roof loads were found in example 7, page 235.

The stress diagram for joints 1, 2 and 3 are drawn precisely as in problem 8, except that at joint 3 the top chord is inclined. For joint 4 the stress polygon is of, fg, gh, hi, and io. For joint 5, ih, hc, cd, dj, and ji, and for joint 6, oi, ij, jk and ko. It will be

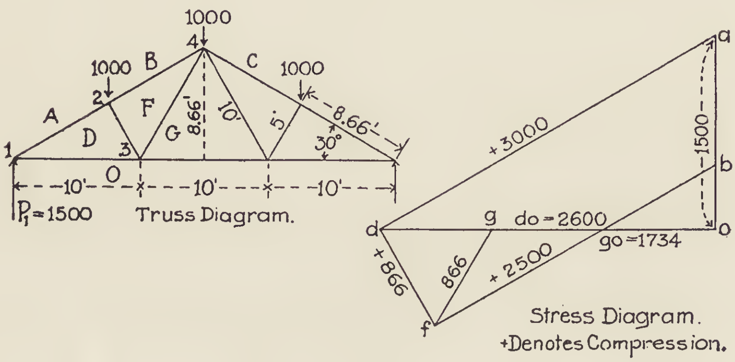


Fig. 291.

seen that inclining the top chord towards the center reduces all of the stresses beyond joint 3.

Problem 16, Fig 291. Simple Fink truss. The panel loads in this example are for convenience, taken at 1,000 lbs. for any other load the stresses will be increased or decreased in the proportion that the load bears to one thousand. Thus, for panel loads of 3,000 lbs. the stresses will be just three times those given on the stress diagram, *provided* the panel loads are all equal, which is commonly the case with steel trusses.

The drawing of the stress diagram presents no difficulties. The general shape of the stress diagram will be the same for any inclination of rafter, but the greater the inclination, the less will be the stress in the principal members.

Problem 17, Fig 292. Simple Fan Truss, similar to Fig. 66. The stress polygons should be drawn in the order in which they are numbered. The span and rise of this truss are the same as those of the truss in Fig. 291, and the panel loads correspond to the same weight per square foot of roof. It will be noticed that the principal stresses are a little greater in Fig. 292 than in Fig. 291, because more of the roof load is supported by the truss and less by the wall. The rafter being shorter in Fig. 292, however, the actual amount of steel required will be about the same in the two trusses. Which of the two trusses would be the most economical for any given roof will depend upon the kind of roof, and how the roofing is supported, as explained on page 58. The truss dimensions in Figs. 291 and 292 are given to show the relative lengths of the members, with a 30 degree pitch.

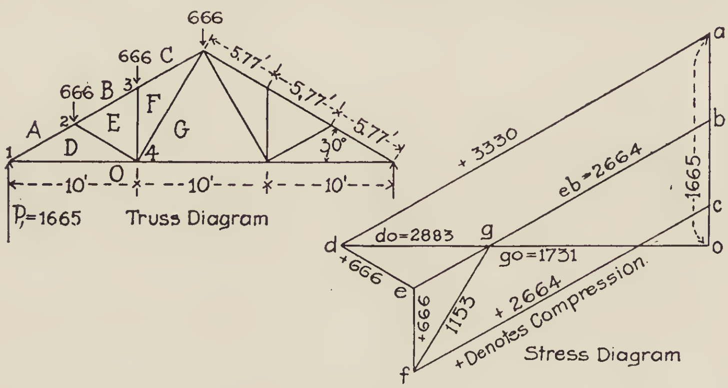


Fig. 292.

They have no bearing on the stress diagram, i. e., *with the same panel loads* the stresses will be the same for a truss of twice the span, as long as the inclination of the members remain unchanged.

Problem 18, Fig. 293. In this example we have a simple Fink truss with cantilever projections beyond the supports. As usually built, the posts supporting the truss extend to joints 3 and 9, to give lateral stiffness, but for determining the stresses under vertical loads the seats of the truss should be considered to be at joints 2 and 10. In this truss there will be a load at the outer joints equal to one-half of the panel loads, if the distances 1—3, 3—4 and 4—6 are equal.

The stress diagram in this case is commenced by drawing the line ab (Diagram A) to represent the load AB at joint 1, with the arrow heads pointing down, and from a and b, drawing lines parallel respectively to EA and BE. The stress be acts from b towards e, or from the joint, denoting tension, while the stress ea acts towards the joint denoting compression. Next complete the stress polygon for joint 2. We already have the line ae, and the point a, and as the supporting force is OA, its upper end must be at a, and o will be a distance below it equal to 4,500 lbs. We then have for our stress polygon, oa, acting up, ae, ef and fo, the point f being found by drawing a line through o parallel to FO. In this case all of the forces act towards the joint hence all the four members are in compression. Next complete the stress polygon for joint 3. We already have fe and eb, and measure down bc equals 1,500 lbs. From c draw

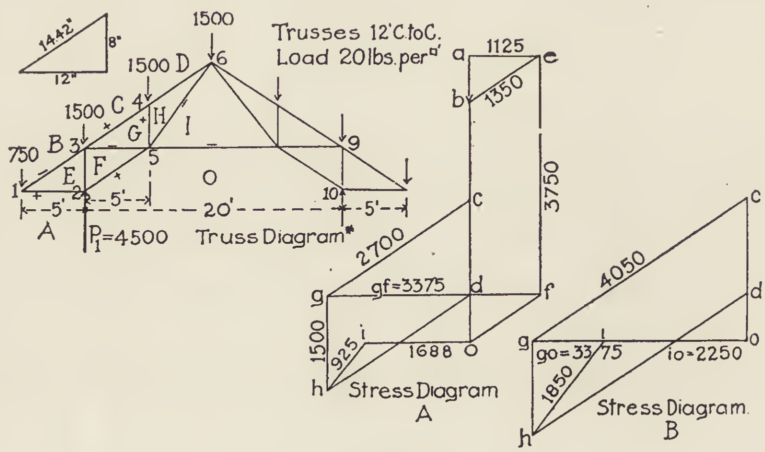


Fig. 293.

a line parallel to CG and from f a line parallel to GF, giving the point g.

For joint 4 the stress polygon is gc, cd, dh and hg.

For joint 5 the stress polygon is of, fg, gh, hi and io.

Diagram B shows what the stresses would be if the overhanging projections were omitted. Comparing diagrams A and B we see that the overhanging projections decrease the stress in the rafter above joint 3, by the amount of the stress in BE. The tension in the tie beam between joints 3 and 5, is the same in both cases, but the tension in the center, io, is less in diagram A, also the stress in hi.

For this truss the stresses due to a horizontal wind pressure of 30 lbs. per sq. foot should be computed, and added to those produced by the dead load.

Problem 19, Fig. 294. Eight panel Fink Truss with rafter divided into equal panel lengths.

No difficulty should be experienced with the stress diagram until we come to joint 4, where there are three unknown forces. There are two or three ways of getting over this difficulty, the simplest one, in the opinion of the author, being to assume, for the moment that a strut extending from joint 5 to joint 6 is substituted for members HI and IK, which would leave but two unknown forces at joint 4. Proceeding then, with the stress polygon for joint 4, we have

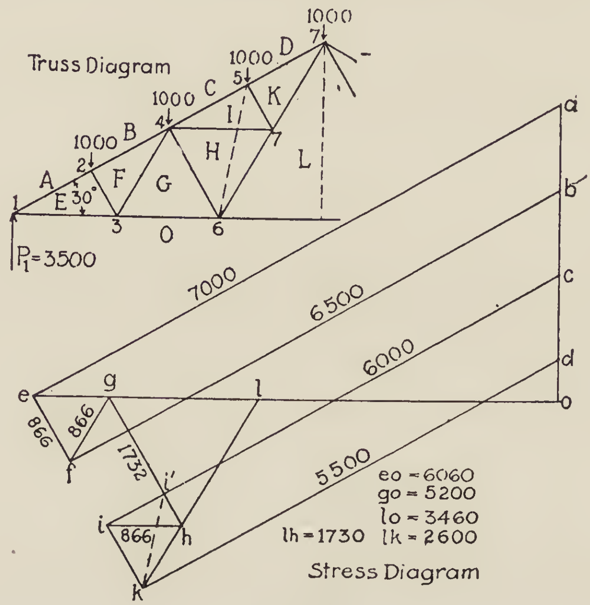


Fig. 294.

already drawn the lines gf and fb, and we measure down from b, a distance of 1,000 lbs., which gives the point c. Then from c and g draw lines parallel respectively to CI and HG, intersecting at i′. Next, complete the stress polygon for joint 5 on the assumption that the strut IK is placed, as shown by dotted line, instead of by the full line. We have for the stress polygon for this joint the line i′c and proceed to measure down from c the load of 1,000 lbs. giving the point d. From d and i′ draw lines parallel respectively to DK, and to the dotted line, intersecting at k. We now proceed to complete the stress diagram for joint 6, with the strut as shown by dotted line.

We have already drawn the lines og, gi′, i′k, and from k draw a line parallel to KL intersecting og at l. Now the stresses in KL and LO are not affected by changing the position of the strut, so that the points k and l are in their correct position. Returning to the stress polygon for joint 6, and proceeding as though the dotted line had not been drawn, we have the correct lengths for og and lo, and if we prolong gi′ until it intersects lk at h, the lines gh and hl will correctly represent the stresses in GH and HL. Returning to joint 4 and considering that the tie IH is now in place, we have hg, gf, fb, bc and from h draw a line parallel to IH, until it intersects ci′, prolonged, giving the true position of the point i. For joint 5, we now have ic, cd, dk, and a line from i parallel to KI which should pass through the point k, which completes the stress diagram for one-half of the truss.

As a matter of fact, when the panel loads and divisions of the rafter are all equal, as in this example, the line ik will lie in the continuation of ef, so that the points i and k can be found directly by continuing the line ef, but if the panel loads or divisions of the rafter are not equal, or there are loads at joints 3 and 6, then the points i and k will not be in the continuation of ef and it will be necessary to find them by the process described above, which applies to any manner of loading, or any inclination of the rafter.

The stresses given on the stress diagram are given closer than they can be scaled, in order to show the exact stresses for panel loads of 1,000 lbs., and an inclination of 30 degrees. For any 8 panel Fink truss, evenly divided, and with this inclination, the stresses will be increased or diminished from those shown in direct proportion to the panel loads. Thus for a truss of 70 ft. span, and panel loads of 4,800 lbs., the stresses will be 4.8 times those given in Fig. 294. It is interesting to note that the stresses EF, FG, HI and IK are all equal, and that the stress in HG, is just twice that in EF or IK.

Problem 20, Fig. 295. Eight panel Fink Truss with loads at the lower joints.

As a rule, those buildings in which Fink trusses are used have no ceiling, but the trusses are often required to support shafting, tracks or pulleys, for hoisting machinery, etc., and sometimes a gallery except for the change due to the ceiling loads the stress diagram is drawn exactly as in Fig. 294. The line oa represents one-half of the total load on the truss. For the stress polygon for joint 3 we start by measuring up from o, a distance equal to the load at the joint which gives the point r. We then have ro, oe, ef and from r and f draw lines parallel, respectively to GR and FG, intersecting at g. For joints 4, 5 and 6 the stress diagram is first drawn on the assumption that the dotted line is substituted for HI and IK, as explained in Problem 19. For the stress polygon at joint 6 we start by measuring up from r the load of 500 lbs. giving the point s. We then have sr, rg, gi′ and i′k, and from k draw a

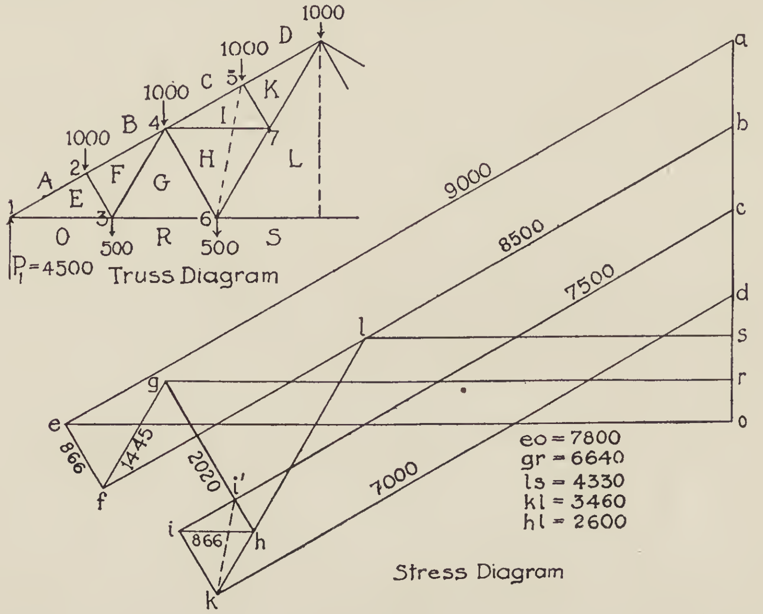


Fig. 295.

line parallel to KL until it meets a horizontal line drawn through s, at l. Then working backwards, we have for joint 6, ls, sr, rg, and gh and hl must close the figure. Knowing gh we can readily obtain ih and ik.

Problem 21, Fig. 296. Eight Panel French Truss, like Fig. 69, except that the tie from joint 1 to joint 6 is inclined, the stress diagram is drawn precisely as explained in Prob. 19.

It should be noticed that inclining the tie, materially increases the stresses in all members except the struts.

Problem 22, Fig 297. Fink Truss with vertical struts. The stress diagram is drawn in the same manner as described in Prob. 19, first

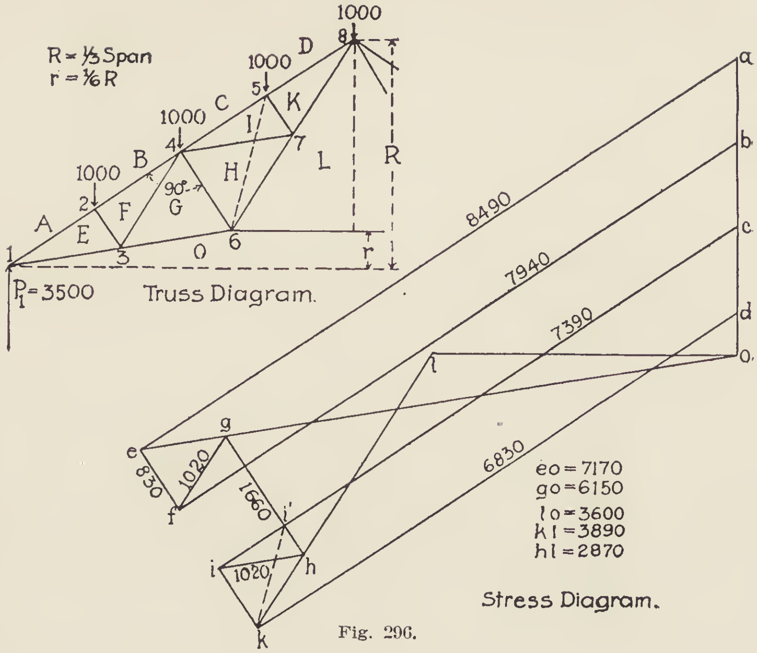


Fig. 296.

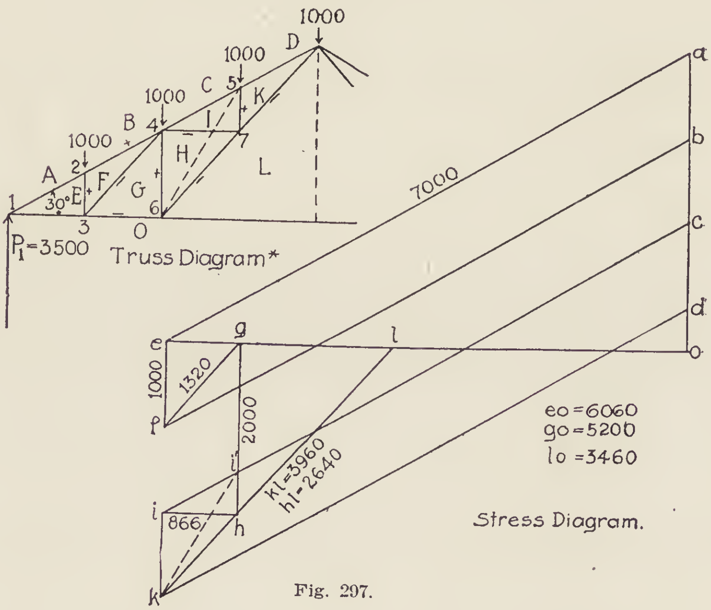


Fig. 297.

finding the point k by means of the dotted line, and working backwards from l and g to find the point h. It is interesting to note that with vertical struts the stress is uniform throughout the rafter, and that the stress in all of the web members is increased. As the vertical struts are shorter than the inclined struts, the actual amount of steel required for the truss in Fig. 297 will be about the same as for the truss in Fig 294, when the span is not over 70 ft. and with a rise of 30 degrees or more.

Problem 23, Fig. 298. Twelve-panel Truss (similar to Fig. 73). The stress diagram presents no difficulties until we come to joint 8, when it is necessary to assume that the members ST and TU are

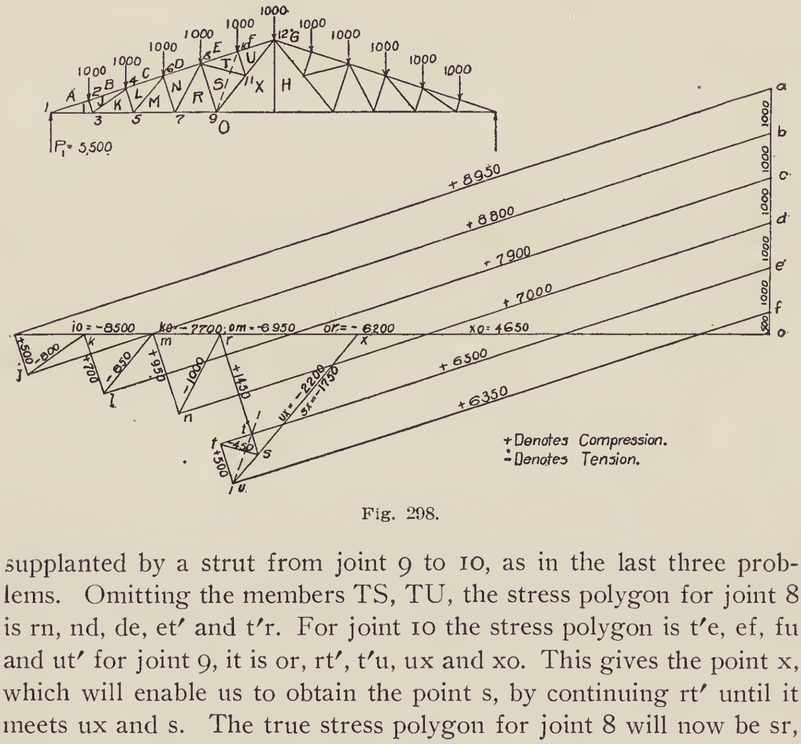


Fig. 298.

supplanted by a strut from joint 9 to 10, as in the last three problems. Omitting the members TS, TU, the stress polygon for joint 8 is rn, nd, de, et′ and t′r. For joint 10 the stress polygon is t′e, ef, fu and ut′ for joint 9, it is or, rt′, t′u, ux and xo. This gives the point x, which will enable us to obtain the point s, by continuing rt′ until it meets ux and s. The true stress polygon for joint 8 will now be sr, rn, nd, de, et and ts, and for joint 10, te, ef, fu and ut.

Problem 24, Fig. 299. Truss like Fig. 75.* The stress diagram is

*The stresses in the monitor should, theoretically, be determined before attempting the truss diagram. For they will not have just the same effect at the truss joints, to which the monitor is attached, as would a uniformly distributed roof load. Practically the loads at the truss joints mentioned may be considered to be those due to panel loads the same as at the other joints.

readily drawn until we come to joint 6, when it is necessary to find t′ u, and v, as in Prob. 23. At joint 9 we also have three unknown forces, viz., FA′, A′X and XV. To find the point a′ it is necessary to first find the stress in XA′. This can be done by drawing a stress diagram for the portion of the truss included within the lines 9-15,

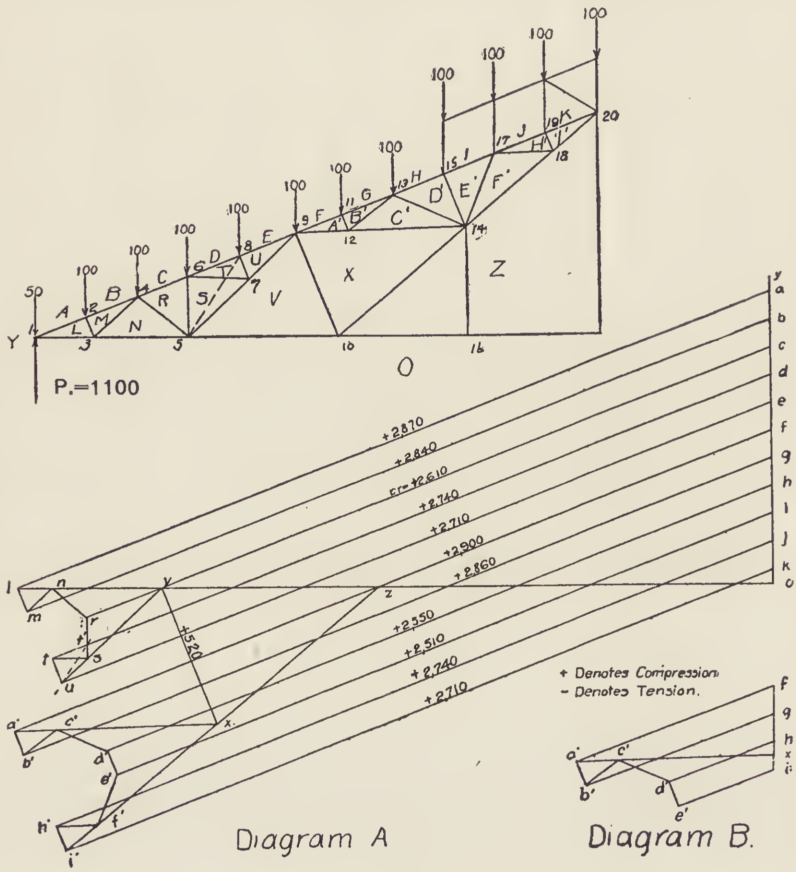


Fig. 299.

Stresses not given on the engraving.
lo, 2625, no, 2500, vo, 2125, zo, 1375
xz, 740, zf 1335 ; zi 1475. vs. 375 ; vu, 500, Mn = st = 125.
b′c = f′h = 147. lm, = tu = a′b = d′e = h′i′ = 928.
nr = rs = 155. c′d = e′f = 195.

15-14 and 14-9, considered as an independent truss. The stress diagram for this portion of the truss is shown by diagram B, which gives the correct stress for all of the web members. Now, for the stress diagram for joint 9, we have vu, ue and ef. Draw a line from f parallel to FA′ and a line from v parallel to VX. The stress line for A′X must connect these two lines and must be equal in length to

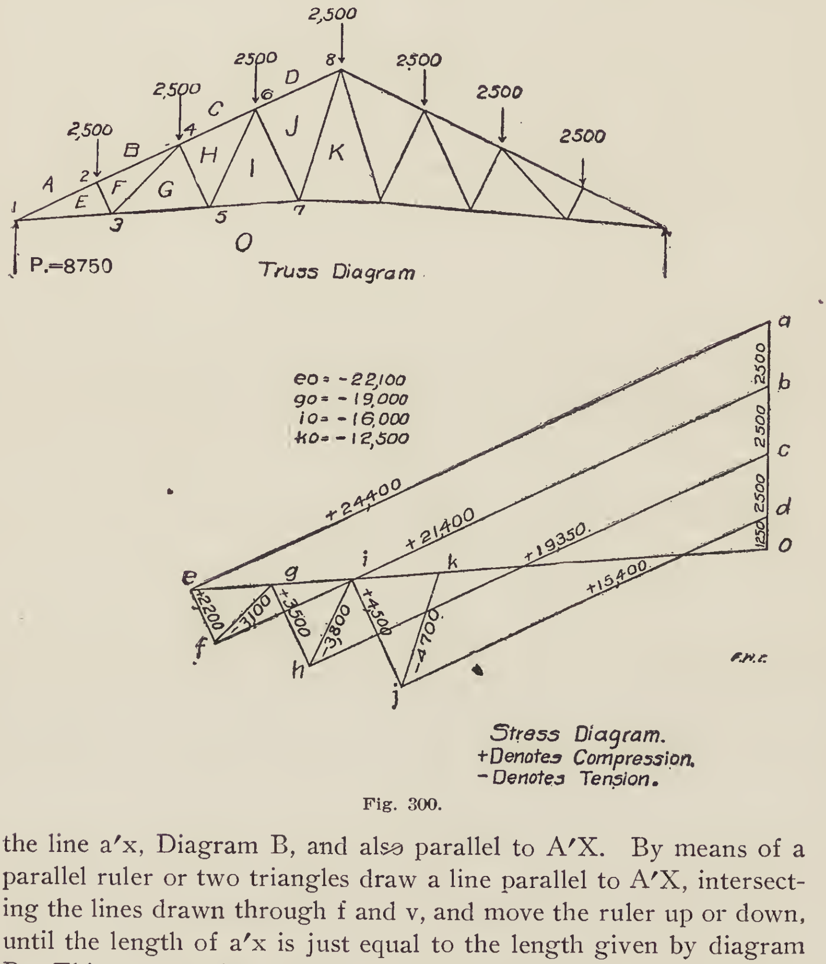


Fig. 300.

the line a′x, Diagram B, and also parallel to A′X. By means of a parallel ruler or two triangles draw a line parallel to A′X, intersecting the lines drawn through f and v, and move the ruler up or down, until the length of a′x is just equal to the length given by diagram B. This can readily be done by means of dividers. We thus obtain the points a′ and x in the full stress diagram after which it is easy to draw the stress polygons for all remaining joints. When the truss rafter is divided into equal lengths, and the load is uniform, as in this example, the point a′ will be in line with u and v, but if the rafter is not divided evenly, then the point a′ will not be in a line with u and v, but the process for finding the point a′ applies to any condition of loading or division of the rafter.

Problem 25, Fig. 300. Warren Triangular Truss. By drawing the

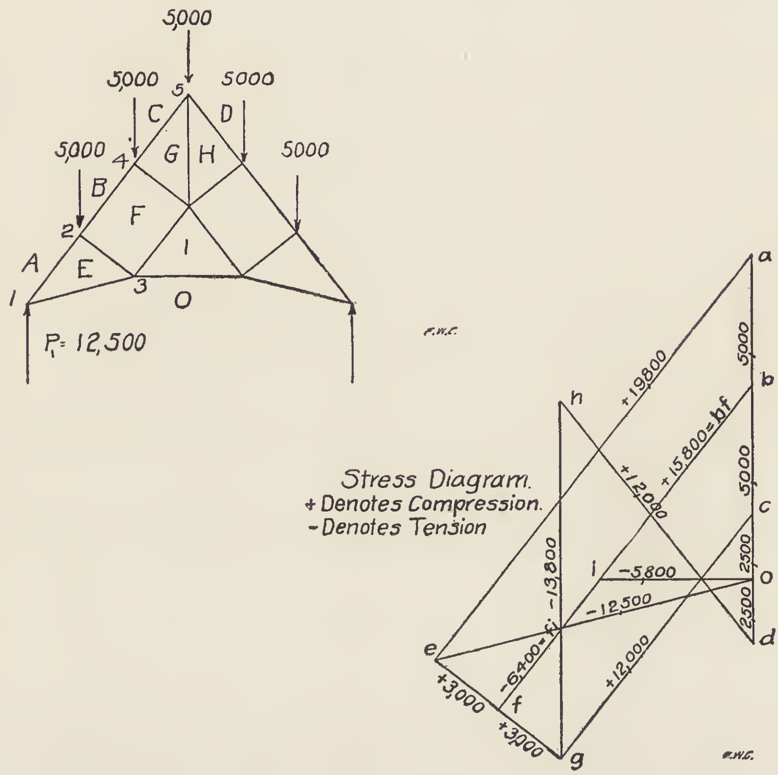


Fig. 301.

stress polygons in the order in which the joints are numbered, no joint has more than two unknown forces, so that there should be no difficulty with the stress diagram.

Problem 26, Fig. 301. Truss like Fig. 77. The stress diagram for this truss is drawn in the same manner as described in the preceding problems, going from joint to joint in the order in which they are numbered.

Problem 27, Fig. 302. Truss like Fig. 80. In starting the stress diagram for this truss the force next in order to the supporting force going around like the hands of a clock is the stress in AF, which, being horizontal, gives a different stress diagram from that

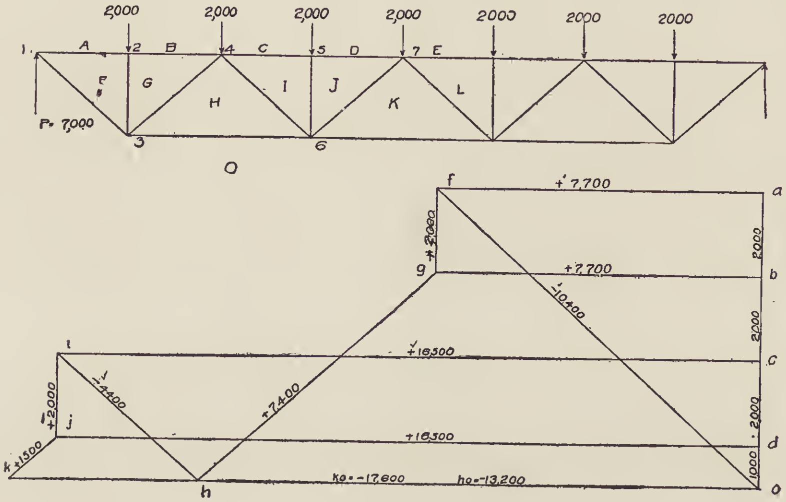


Fig. 302.

of any of the trusses thus far considered. It should be remembered, however, that the process of drawing the stress diagram is the same, whatever the inclination of the members, and it is only in cases where there are three unknown forces that any difficulty arises.

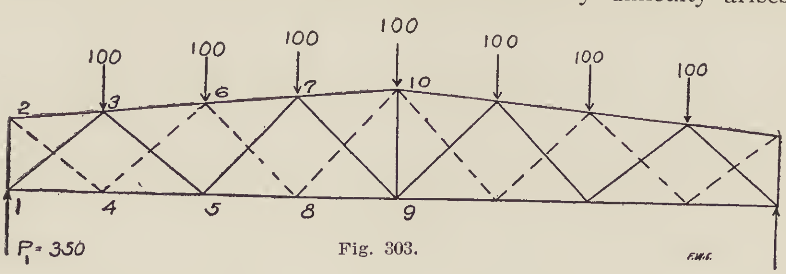


Fig. 303.

Proceeding with the stress diagram for this truss in the order in which the joints are numbered, no joint has more than two unknown forces.

Problem 28, Figs. 303 and 304. Lattice Truss. Fig. 303 shows the truss diagram for a steel lattice truss of similar design to that shown in Fig. 82, except that in Fig 303 we have introduced a vertical member at the center. This truss is really a combination of two trusses, one laid over the other, as it were, with the chords coinciding. Trust diagrams "A" and "B" in Fig. 304 show the two trusses of which Fig. 303 is composed, diagram "A" representing the portion of Fig. 303 shown by full lines, and a diagram "B" the truss

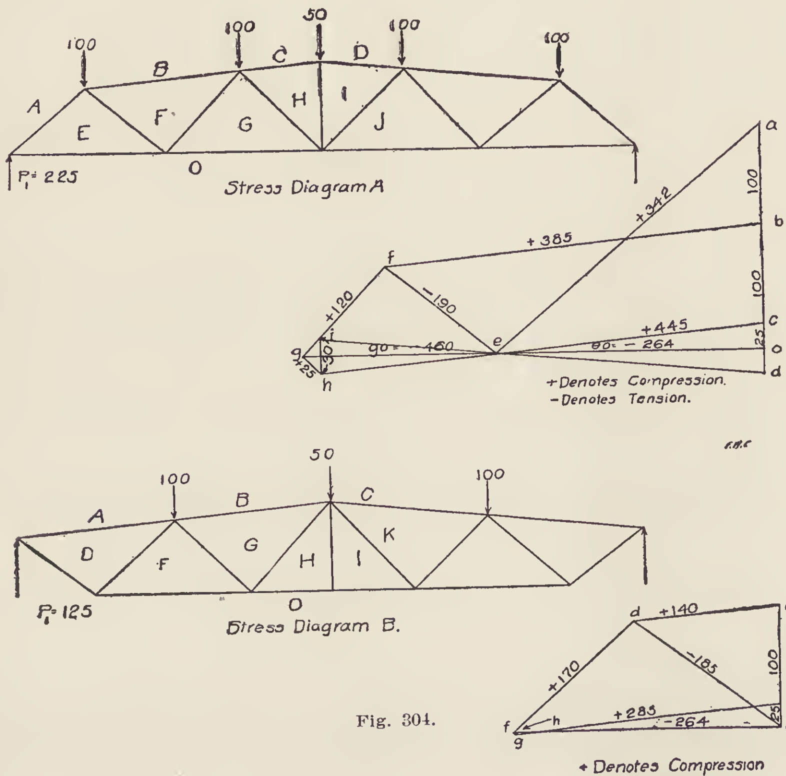


Fig. 304.

whose web members are shown by dotted lines. By referring to Fig. 303 it will be seen that the loads at joints 3 and 7 must be supported entirely by the truss shown by diagram "A", and the load at joint 6 must be supported by the truss shown by diagram "B". Joint 10, on account of the center vertical member, is common to both trusses, and we should, therefore, assume that 50 lbs. is carried by one truss, and 50 lbs. by the other. The joint loads on

Fig. 303 are therefore divided, as shown by diagrams "A" and "B", Fig. 304. Drawing the stress diagrams in the manner described for all of the preceding problems we obtain the stresses indicated on the diagrams.

To obtain the real stress in the chords it will be necessary to combine the stresses given by the two stress diagrams. Thus, the stress in the top chord between joints 2 and 3 (Fig. 303) is 140 tbs. from diagram "B" and 0, from diagram "A". Between joints 3 and 6 the stress is 385 tbs. from diagram "A" and 140 tbs. from diagram "B". Between joints 6 and 7 the stress is 385 + 285 tbs. and between joints 7 and 10 the stress is 507 + 285 tbs.

In the same way, the stresses for the bottom chord are 264 tbs. between joints 1 and 4, 528 tbs. between joints 4 and 5, 770 tbs. between joints 5 and 8, and 797 tbs. between joints 8 and 9.

It should be noted in connection with stress diagrams B, that there is no stress in the vertical center member, from the center load on this truss, and even in the truss shown by diagram "A" the stress on the center member is but 30 tbs. By considering that the entire load at joint 10 is supported by truss "B" the diagonal GH, Diagram A is put in compression, causing a tensile stress in HI, with the inclination of top chord shown by the diagrams.

The load at joint 10 could be divided between the two trusses in such a ratio, however, so that there would be no stress at all in GH and HI, and hence the center member can be omitted, and probably would be in light trusses. The ratio in which the centre load should be divided between the two trusses to give no stress in GH and HI will depend upon the inclination of the top chord, and when the inclination reaches a certain angle, a vertical tie in the centre becomes absolutely necessary to hold down the apex.

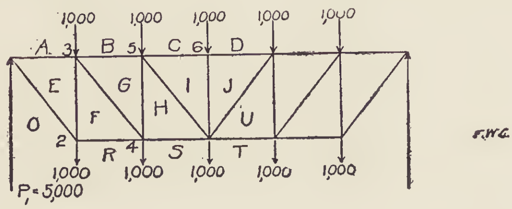
With a horizontal top chord there is no necessity or advantage in a vertical center member.

None of the diagonal members of the truss shown by Fig. 303, enters into both of the component trusses, hence each has only the stress shown by the stress diagram for the truss to which it belongs. Thus, the stress in diagonal 1-3, is 342 tbs., and in 2-4, 185 tb., the former being in compression and the latter in tension.

Problem 29, Fig. 305. Suspended Pratt Truss. The stress diagram presents no difficulties when drawn in the order in which the joints are numbered. The diagram, like many others, is given merely to show the reader its appearance when correctly drawn, and to serve as a check when drawing the stress diagram, for any particular truss.

When this truss sustains unsymmetrical loads the stress in the diagonals nearest the center may change to compression instead of tension.

Problem 30, Fig. 306. Simple Scissors Truss. For this prob-



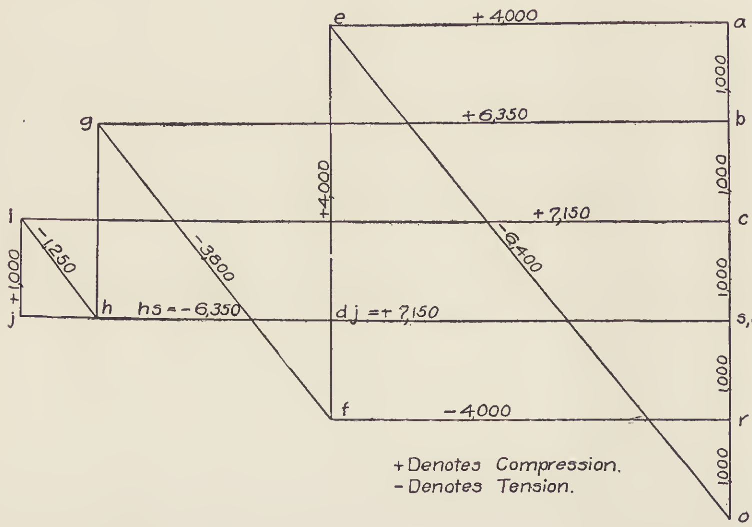


Fig. 305.

lem we have taken the center lines of the truss shown in Fig. 155, which supports a shingle roof and plastered ceiling. The actual weight of the materials in the roof and truss, exclusive of lath and plaster, is in the neighborhood of 13 lbs. per sq. ft. of roof surface. The loads given on the truss diagram were computed on the basis of 43 lbs. per sq. ft. of roof surface, and 12 lbs. per sq. ft. additional for plastered ceiling. It should be noted that half of the weight of the horizontal ceiling is supported at the apex joint. To draw the stress diagram, commence by drawing a vertical line oa = 7,140 tbs. (one-half of the load on the truss) and from a draw a line parallel to the rafter and from o, a line parallel to the tie beam. Where these two lines intersect place the letter d. Then ad represents the stress

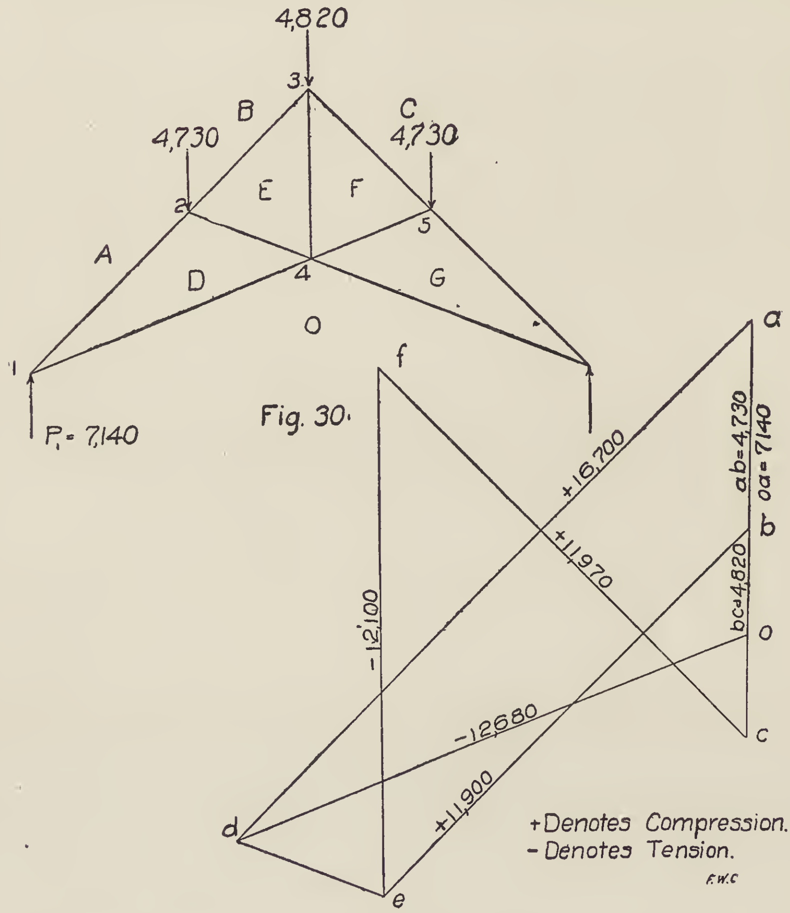


Fig. 306.

in AD and do the stress in DO. For the stress polygon at joint 2, we have da, and from a measure downwards the load of 4,730 tbs. giving the point b. From the point of beginning d, draw a line parallel to ED, and from b, a line parallel to the rafter BE, and at the point of intersection of these lines place the letter e. It should be noted that ed acts *towards* the joint denoting compression, while DO is in tension. For the stress polygon for joint 3, we have eb, measure downward, bc = 4,820 lbs., and from e and c draw lines parallel respectively to FE and CF, intersecting at f. We now have the stress lines for all of the members of the truss, as the stresses are the same in each half, and scaling the lines of the stress diagram we obtain the stresses indicated on the engraving.

"THE BARRETT SPECIFICATION" FOR STANDARD SLAG AND GRAVEL ROOFS.

A roof laid in accordance therewith will last from ten to twenty years with absolutely no repairs of any kind. Any reputable contractor will guarantee such a roof *for at least ten years.*

From the viewpoints of economy, fire protection and general satisfaction, a Barrett Specification roof will prove immeasurably superior to one of Tin, Slate, or Ready Roofings of any description.

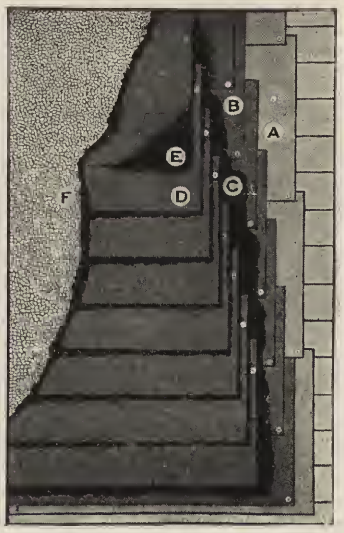
The Barrett Specification for Standard Slag or Gravel Roofing (*to follow description of Roof Sheathing*):

Over the foregoing shall be laid a five (5) ply Coal Tar, Pitch, Felt and Slag or Gravel Roof, to be constructed as follows:

The Rosin Sized Sheathing Paper or Unsaturated Felt to be used shall weigh not less than five (5) pounds per one hundred square feet.

The Tarred Felt shall weigh not less than fourteen (14) pounds per one hundred square feet, single thickness.

The Pitch shall be the best quality of straight-run coal-tar pitch, distilled direct from American coal tar, and there shall be used not less than one hundred and twenty (120) pounds (gross weight) per one hundred square feet of completed roof.



The nailing shall be done with threepenny barbed-wire roofing nails driven through tin discs.

The Slag or Gravel shall be of such a grade that no particles shall exceed five-eighths (5/8) of an inch or be less than one-fourth (1/4) of an inch in size. It shall be dry and free from dust or dirt. In cold weather it must be heated immediately before using. Not less than three hundred (300) pounds of Slag or four hundred (400) pounds of Gravel shall be used per one hundred square feet.

The materials shall be used as follows:

First lay one thickness of Rosin Sized Sheathing Paper or Unsaturated Felt (A), lapping each sheet one (1) inch over the preceding one, and nailing only so often as may be necessary to hold in place until covered with the Tarred Felt (B), and the nailing may be omitted entirely if practicable.

Over the Rosin Sized Sheathing or Unsaturated Felt lay two (2) full thicknesses of Tarred Felt (B), lapping each sheet seventeen (17) inches over the preceding one and nailing along the exposed edges of the sheets only so often as may be necessary to hold the sheets in place until the remaining Felt can be applied.

Over the entire surface of the Felt thus laid, spread a uniform coating of Pitch (C), mopped on. Then lay three (3) full thicknesses of Felt (D), lapping each sheet twenty-two (22) inches over the preceding one, and nailing, as laid, every three (3) feet, not more than ten (10) inches from the upper edge.

When the Felt is thus laid and secured, mop back with Pitch (E) the full width of twenty-two (22) inches under each lap. Then spread over the entire surface of the roof a uniform coating of Pitch, into which, while hot, embed Slag or Gravel (F).

NOTE.—When this roof is to be laid over a hydraulic cement concrete, as in fireproof construction, a special specification will be furnished for this kind of work.

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