Timber framing

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TIMBER FRAMING

BY

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PREFACE

The material in the following chapters has appeared, in the main, as a series of articles in *Western Engineering*. In being arranged for publication in the present form, this has been revised and enlarged. The matter contained therein is the result of some eleven years' experience in timber-framing, during which time I have been intimately connected with the design and the superintendence of construction of nearly two hundred million board feet of timber, most of this being represented by the structural features of two expositions.

In this work, I have found that the published record of timber construction is meagre. Especially is this statement true of details of design, and strength of timber joints. I have searched through all available engineering literature for the results of tests of timber joints and fastenings, and have been disappointed in finding so few recorded. To supplement these few tests, I have made additional ones on various types of timber joints.

I have, in my own work, always tried to design the particular structure, so that it would be effective and efficient in action, and at the same time be simple and direct for the carpenter to frame. These two conditions are not always possible to obtain; their correlation, however, is always to be sought. With this end in view, I have, whenever possible, followed my designs into the field, observed the framing and erection of the structure, its behavior under load, and the effect of time and the elements.

The results of this experience and study are presented in the following pages. As explained in the introductory chapter, this volume is in no sense a text-book, and does not cover equally all phases of timber-construction. Its many shortcomings are realized, but it is hoped that the contents may be of some benefit to those who may have occasion to design or construct timber-framing.

With some of the theories advanced, and conclusions drawn, there may be differences of opinion. I am frank to state that certain of these conclusions may have to be modified in the light of future tests. It is always unwise to attempt to extend the results of tests too far. However, until such further tests are made, it is imperative that working-values be established for present use. The best that can be done under those circumstances is to use the most reasonable theory that can be found, utilizing the available tests as a guide. This method is certainly better than a blind guess, or a rule-of-thumb method. As an illustration of the condition just mentioned, the present method or methods of designing bolted joints may be cited.

I wish to acknowledge the assistance of W. L. Huber, E. L. Cope, and C. H. Munson. Especial acknowledgement is due to A. W. Earl and T. F. Chace for aid in making calculations, and for valuable criticism and suggestions in the preparation of the text, and to Robert S. Lewis of the University of Utah, for the material on timber mine-structures contributed by him. I desire, also, to express my appreciation of the courtesy of the publishers of *Engineering Record, Engineering News, Engineering and Mining Journal,* and to the American Railway Engineering Association for permission to reproduce material from their publications.

HENRY D. DEWELL.

San Francisco, May 1, 1917.

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TIMBER FRAMING

CHAPTER I

Introduction

While timber as a structural material has been largely supplanted by steel and concrete, especially in permanent work, there are still many occasions where it may be employed advantageously in bridges and buildings, and other structures of a somewhat permanent nature. A knowledge of the properties of timber, its capabilities, and its limitations for use in construction, is therefore an essential part of the education of a civil engineer.

The old-school bridge engineer was a past master in the art of timber framing. Many of his structures, it is true, were framed more by experience and judgment than by considerations of theory and of computed stresses, yet the number of timber railroad bridges still giving service testifies to the soundness of his design. The results of his experience have been handed down to his successors and are represented today in the accepted standards of the railway engineer's office. Outside of this class of engineers, however, it may be truthfully said that neither is the art of timber framing generally understood, nor is the value of such knowledge appreciated.

For the design of wooden buildings of exceptional size or of unusual proportions, a structural engineer is now generally retained; otherwise, the plans for framing are prepared in the architect's office. In the latter event the work is usually given to an architectural draftsman possessing little experience in actual construction, and only a superficial and therefore frequently dangerous knowledge of structural mechanics. This practice results from the commonly accepted ideas that timber designing consists in computing the sizes of beams and girders, or in solving the stresses in a roof truss, and that, given the required sizes of the principal structural members of a frame, the carpenter is fully capable of designing the joints. This conception of the scope of timber designing is erroneous. There is no timber structure of an appreciable size which will not justify a careful and intelligent study of the framing details, not alone on the ground of safety, but also from the consideration of economy. Important details should not be left to the judgment of the contractor or carpenter. With all respect for the ability of the experienced carpenter, there is at times nothing so impractical as a so-called 'practical man.' I have seen instance after instance where it would seem that the carpenter had gone out of his way to frame a joint in the weakest possible manner.

Obviously, the method of finding the stresses in a structure is the same whether the material be timber or steel or concrete, and timber joints are as susceptible to analysis for strength as are details in any other material. The cause of the weak details so often seen in timber trusses has been largely the failure on the part of the designer to realize that the joints needed attention. As a test for the display of ingenuity and as a problem to develop one's knowledge of practical and efficient construction, the design of an ordinary mill building in timber and the superintendence of its framing and erection has few equals.

For an intelligent design in timber, a knowledge of sawmill and timber-yard methods is essential. The difficulties of actual framing and erection must also be anticipated and provided for; the designer must imagine himself in the carpenter's place and realize, for example, what cuts will be most difficult to make and what holes will be hard to bore; in other words he must foresee in what details careless work is most likely to occur. The possibility of the timber being green and the consequent shrinkage must be recognized, and if such shrinkage is detrimental to the strength of the structure, means must be provided for tightening the joints after the shrinkage has taken place. As possible incipient causes of failure by shear, the checks due to seasoning must not be neglected. In short, all the limitations of the material must be fully realized.

In the case of structures of steel, the majority of the details can be made in accordance with the standards of present-day practice, fully treated in the text-books and the handbooks of the steel companies and bridge shops. In the realm of reinforced concrete design, certain standards for detailing are being formed rapidly. For timber, however, there are no such standards, except those for bridge and trestle-work generally followed by the railroad engineer. Or, it may be said that in timber construction, many details called standard can be justified by no consideration of efficiency. Even the standards used by the old-school bridge engineers cannot be employed indiscriminately. Certain of these, while entirely suitable for the woods obtainable in the Eastern States, have been transferred to the West and applied without modification to timbers with entirely different properties from those for which the details were designed. The most notable example of this practice is the use of the standard cast or malleable-iron washer with wood as soft as Douglas fir.

For both steel and concrete design and construction there are many good text-books and standard specifications, but for timber framing, such as heavy building and bridgework there are only a few text-books and, to my knowledge, no standard specifications. Among the few books dealing with this subject, Jacoby's 'Elements of Heavy Framing' and Howe's 'Simple Roof Trusses' are notable for their excellence, and their contents should be mastered by anyone interested in the design of timber structures. It is with the view of supplementing these and other existing works, by bringing into correlation the drafting-room design and the requirements of the field, rather than covering the whole subject of structural design in timber that the present treatise has been undertaken.

A general knowledge of structural design on the part of the reader has been assumed and no attempt has been made to cover the whole field of timber framing, but by discussing the advantages and disadvantages of different typical details, I have tried to point out the structural limitations of the material and the difficulties which arise during construction. Only by a thorough understanding of the many elements that enter into the design of joints and details in timber framing is it possible to make the finished structure safe, efficient, and economical.

A set of general specifications for timber-work is given in the concluding chapter. These specifications are intended primarily for buildings, but with certain obvious modifications are applicable to any timber structure. Since the greatest forests occur in the West, these specifications apply particularly to Douglas fir, but with different unit stresses they may be used for any other timber. The properties of Douglas fir are not far different from those of long leaf yellow pine, so that the specifications may be used with but slight changes for structures built of the latter timber. With these specifications, I hope to establish to some extent certain workable standards for timber framing in general, and for building construction in particular.

CHAPTER II

Mill and Yard Specifications

The strength of individual sticks of timber varies greatly. For this reason the statement is sometimes made that refinement in calculation of timber framing, and even the computation of stresses, is unnecessary. However, the variation in strength of timbers classed under any one grade is not so great but that definite working stresses can be established with the certainty of such stresses being safe.

It is of the utmost importance then that the designer should be familiar with the probable qualities of the timber of which his structure will be built. He must know the allowable variation in size due to sawing, sizing, and surfacing, also the allowable number and size of the knots and other defects. For this reason, there follow extracts from the 'Standard Classification, Grading, and Dressing Rules for Douglas Fir, Spruce, Cedar, and Western Hemlock Products' as adopted by the West Coast Lumber Manufacturers Association. These specifications while local to the Pacific Coast are typical of the grading rules for any timber.

General Grading Rules

1. All lumber is graded with special reference to its suitability for the use intended.

2. With this in view each piece is considered and its grade determined by its general character, including the sum of all its defects.

3. What is known as "Yard Lumber," such as dimension common boards, finish, etc., is graded from the face side, which is the best side, except that lumber which is dressed one side only is graded from the dressed side.

5. The defects in lumber are to be considered in connection with the size of the piece, and for this reason wider and longer pieces will carry more defects than smaller pieces in the same grade.

6. No arbitrary rules for the inspection of lumber can be maintained with satisfaction. The variations from any given rule are numerous and suggested by practical common-sense, so nothing more definite than the general features of different grades should be attempted by rules of inspection.

7. Lumber must be accepted on grade in the form in which it was shipped. Any subsequent change in manufacture or mill-work will prohibit an inspection for the adjustment of claims, except with the consent of all parties interested.

8. A shipment of any grade must consist of a fair average of that grade, and cannot be made up of an unfair proportion of the better or poorer pieces that would pass at that grade. A shipment of mixed widths shall contain a fair assortment of each width. A shipment of mixed lengths shall contain a fair assortment of each length.

9. Material not conforming to standard sizes shall be governed by special contract.

11. The grade of all regular stock shall be determined by the number, character, and position of the defects visible in any piece. The enumerated defects herein described admissible in any grade are intended to be descriptive of the coarsest piece such grades may contain, but the average quality of the grade should be midway between the highest and lowest pieces allowed in the grade.

12. All dressed lumber shall be measured and sold at the full size of rough material used in its manufacture.

13. All lumber one inch or less in thickness shall be counted as one inch thick.

14. In determining the seriousness of the pitch pocket as a defect both its width and length must be

considered. The tighter the pocket the longer it may be.

15. Size and number of pockets admissible in any piece must be left largely to the judgment of the grader and a reasonable deviation from the number of pockets specified in the rules will be permissible.

16. Pitch shakes are clearly defined openings between the grain of the wood, are either filled with granulated pitch or not, but are in either case a serious defect, and must not be admitted in any grade above No. 2 common.

17. A pitch streak is a well defined accumulation of pitch at one point in the piece and when not sufficient to develop a well-defined streak, or where fibre between grains is not saturated with pitch, it shall not be considered a defect.

18. A small pitch streak shall be equivalent to not over one-twelfth the width and one-sixth the length of the piece wherein it is found.

19. A standard pitch streak shall be equivalent to not over one-sixth the width and one-third the length of the piece it is in.

20. Splits and checks shall be considered as to length and directions.

21. Wane is bark or lack of wood on edges of lumber from any cause.

22. Chipped-grain consists in part of the surface being chipped or broken out in small particles below the line of the cut, and as usually found should not be classed as torn-grain and shall be considered a defect only when it unfits the piece for the use intended.

23. Torn-grain consists in a part of the wood being torn out in dressing. It occurs around knots and curly places and is of four distinct characters, slight, medium, heavy, deep.

24. Slight torn-grain should not exceed $\frac{1}{32}$ in. deep, medium, 1/12 in., and heavy $\frac{1}{8}$ in. Any torn-grain more than $\frac{1}{8}$ in. shall be termed deep.

25. Loosened grain consists of a point of one grain being torn loose from the next grain. It occurs on the

heart side of the piece and is a serious defect, especially in flooring.

26. In standard manufacture of factory flooring, decking, or thick-dressed and matched stock and stock grooved for splines, and for shiplap, the finished width shall be ½ in. less over all than the count or measured width of the rough material used in manufacturing and the tongue and lap shall be measured to determine the finished width.

27. Equivalent means equal, and in construing and applying these rules, the defects allowed, whether specified or not, are understood to be equivalent in damaging effect to those mentioned applying to stock under consideration.

Defects

28. Recognized defects are knots, knot-holes, splits, checks, wane, rot, rotten streaks, pin and grub-worm holes, dog and picaroon holes, pitch seams or shakes, pitch pockets, chipped, torn and loose-grain, solid, pitch, stained heart, sap-stain and imperfect manufacture.

Knots

29. Knots shall be classified as pin, small, standard and large as to size; round and spike as to form; and tight, loose, and rotten as to quality.

30. A pin knot is tight and not over ½ in. diam.

31. A small knot is tight and not over ¾ in. diameter.

32. A standard knot is tight and not over 1½ in. diameter.

33. A large knot is tight and any size over 1½ in. diameter.

34. A round knot is oval or circular in size.

35. A spike knot is one sawn in a lengthwise direction.

36. A tight knot or sound knot is one solid across its face, is as hard as the wood itself, and is so fixed by growth or position that it will retain its place in the piece.

37. A loose knot is one not held firmly in place by growth or position.

38. A rotten knot is one not as hard as the wood itself.

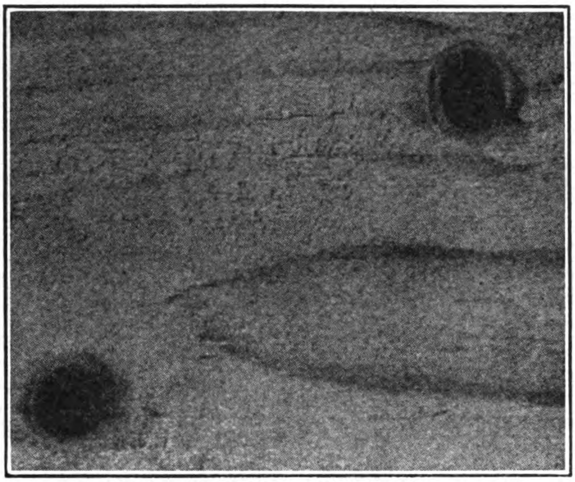


FIG. 1. PIN KNOT.

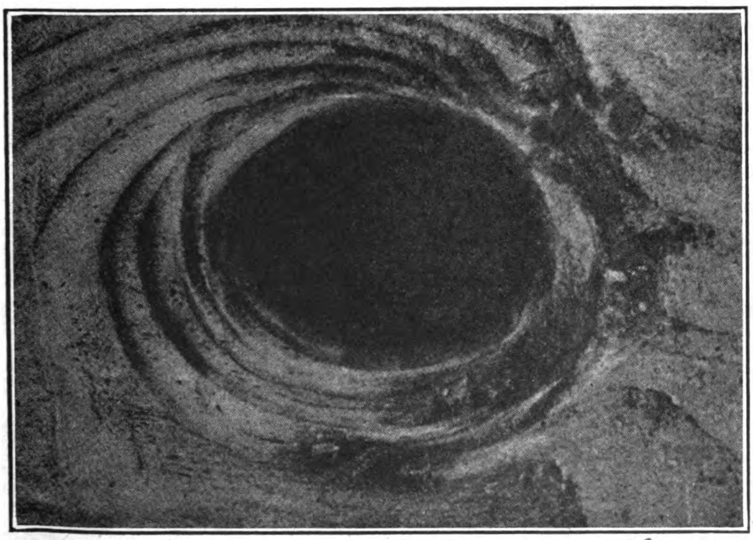


FIG. 2. STANDARD KNOT.

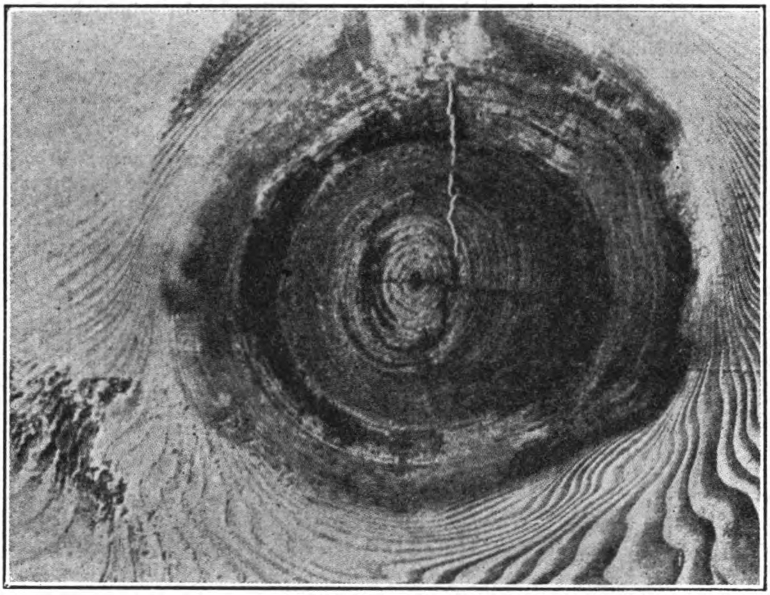


Fig. 3. LARGE KNOT.

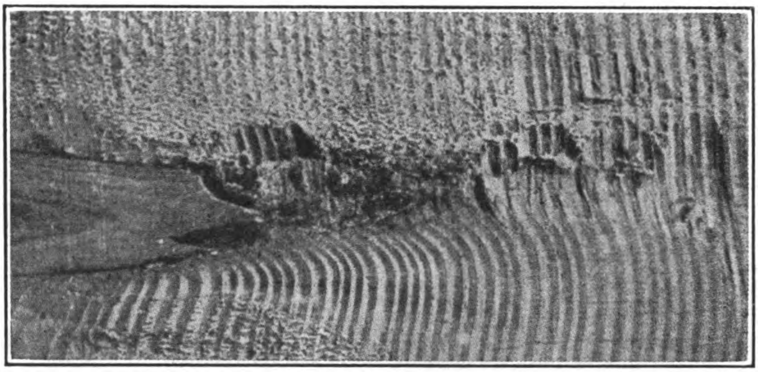


Fig. 4. SMALL SPIKE KNOT.

39. The mean or average diameter of knots shall be considered in applying or construing the rules.

Pitch

40. Pitch pockets are openings between the grain of

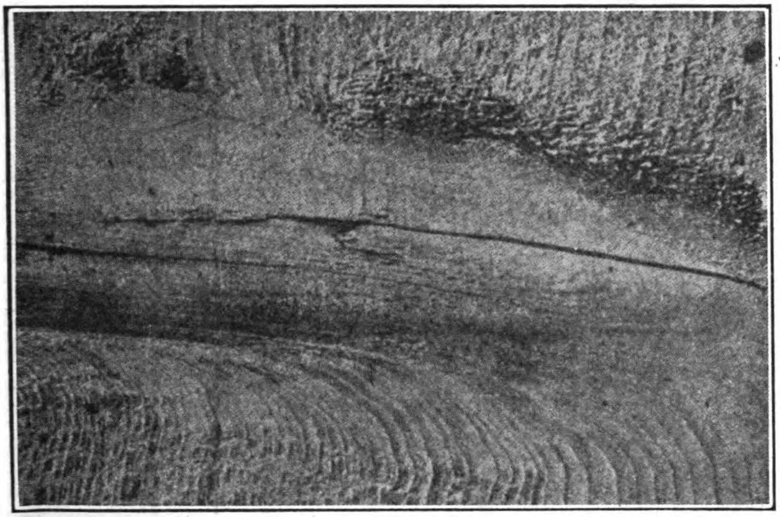


FIG. 5. LARGE SPIKE KNOT.

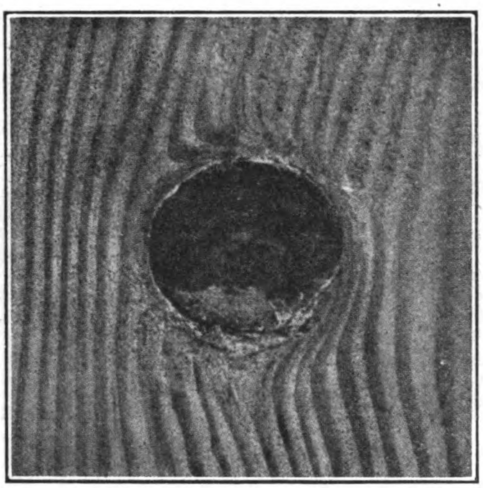


FIG. 6. LOOSE KNOT.

the wood, containing more or less pitch and surrounded by sound grain wood.

Sap

41. Bright sap shall not be considered a defect in

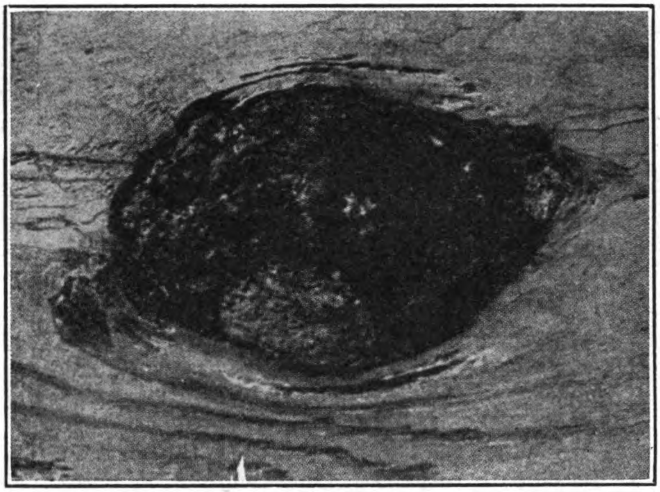


FIG. 7. ROTTEN KNOT.

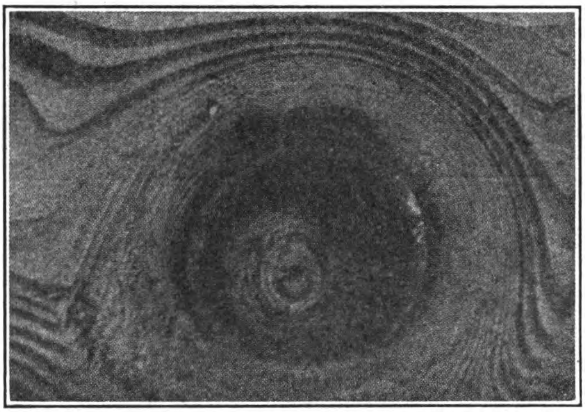


FIG. 8. PITCH KNOT.

any of the grades, except as specially provided for in the following rules.

42. Sap-stain shall not be considered a defect except as herein provided.

43. Discoloration of heart-wood or stained heart must not be confounded with rot or rotten streaks. The

FIG. 9. CLUSTER OF KNOTS.

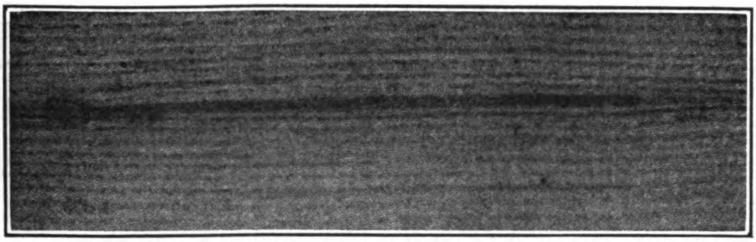


FIG. 10. CLOSED SMALL PITCH POCKET.

presence of rot is indicated by a decided softness of the wood where it is discolored, or by small white spots resembling pin-worm holes.

Standard Sizes

46. In the absence of a special agreement between the buyer and seller for each order, all dressed lumber is finished to the following sizes.

47. Flooring: 1 by 3 in., finished size $\frac{13}{16}$ by $2\frac{1}{2}$-in. face; 1 by 4 in., finished size $\frac{13}{16}$ by $3\frac{1}{4}$-in. face; 1 by 6 in., finished size $\frac{13}{16}$ by $5\frac{1}{8}$-in. face; $1\frac{1}{4}$ by 3 in., finished

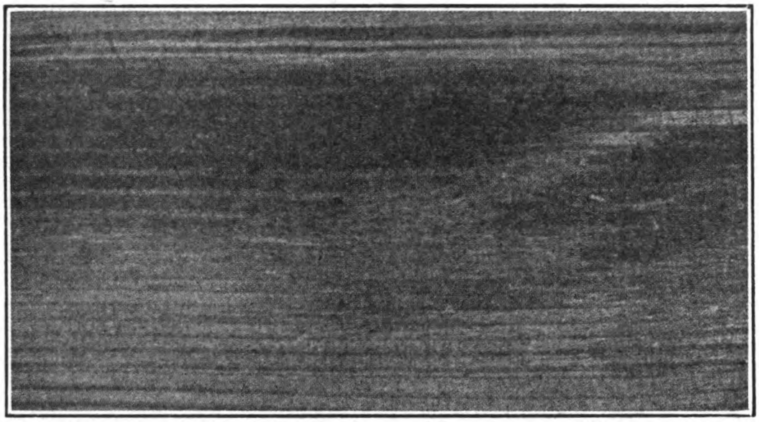


FIG. 11. SOLID PITCH.

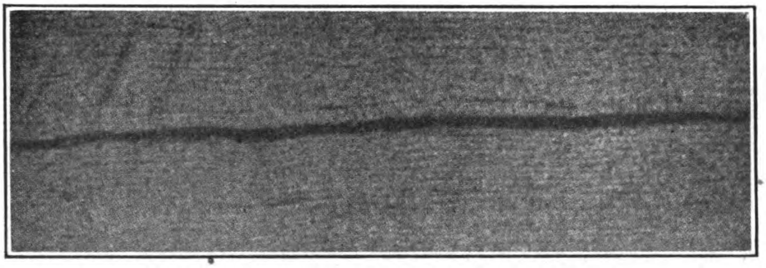


FIG. 12. LARGE OPEN PITCH POCKET.

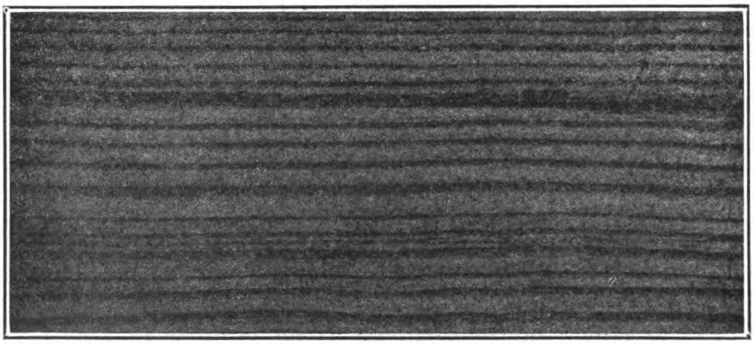


FIG. 13. SMALL PITCH STREAK.

size $1\frac{1}{16}$ by $2\frac{1}{4}$-in. face; $1\frac{1}{4}$ by 4 in., finished size $1\frac{1}{16}$ by $3\frac{1}{4}$-in. face; $1\frac{1}{4}$ by 6 in., finished size $1\frac{1}{16}$ by $5\frac{1}{8}$-in. face; 1 by 6 in. flat-grain flooring, finished size $\frac{3}{4}$ by $5\frac{1}{8}$ in. Standard lengths are multiples of one foot.

53. Widths if dressed on one or both edges: 4 in. to $3\frac{1}{2}$ in.; 5 in. to $4\frac{1}{2}$ in.; 6 in. to $5\frac{1}{2}$ in.; 8 in. to $7\frac{1}{4}$ in.; 10 in. to $9\frac{1}{4}$ in.; 12 in. to $11\frac{1}{4}$ in.; 14 in. to 13 in.; 16 in. to 15 in. Standard lengths are multiples of one foot.

59. Common boards, S1S, or shiplap to $\frac{3}{4}$ inch.

60. Grooved roofing, $\frac{3}{4}$ by $7\frac{1}{4}$ in., $9\frac{1}{4}$ or $11\frac{1}{4}$ in. face; $\frac{1}{2}$-in. groove, $1\frac{1}{2}$ in. from each edge.

61. Shiplap and Dressed and Matched: 1 by 8 in., finished size $\frac{3}{4}$ by 7-in. face; 1 by 10 in., finished size $\frac{3}{4}$ by 9-in. face; 1 by 12 in., finished size $\frac{3}{4}$ by 11 in. face. Standard lengths are multiples of two feet.

62. Dimension, S1S1E, or S4S: 2 by 4 in. to $1\frac{5}{8}$ by $3\frac{5}{8}$ in.; 2 by 6 in. to $1\frac{5}{8}$ by $5\frac{5}{8}$ in.; 2 by 8 in. to $1\frac{5}{8}$ by $7\frac{1}{2}$ in.; 2 by 10 in. to $1\frac{5}{8}$ by $9\frac{1}{2}$ in.; 2 by 12 in. to $1\frac{5}{8}$ by $11\frac{1}{2}$ in.; 3 by 6 in. to $2\frac{1}{2}$ by $5\frac{1}{2}$ in.; 3 by 8 in. to $2\frac{1}{2}$ by $7\frac{1}{2}$ in.; 3 by 10 in. to $2\frac{1}{2}$ by $9\frac{1}{2}$ in.; 3 by 12 in. to $2\frac{1}{2}$ by $11\frac{1}{2}$ inches.

63. Timbers, S1S1E, or S4S, 4 by 4 in. and larger, $\frac{1}{2}$ in. off each way. Standard lengths are multiples of two feet unless otherwise specified.

64. All sizes in dimensions and timbers are subject to natural shrinkage.

Fir Common

Boards and Shiplap and Dressed and Matched:

117. One-inch select common, 4 to 12 in.; shall be square edged; will admit sound knots not over 1 in. diameter in 4 in. and 6 in. and not over $1\frac{1}{2}$ in. in 8 in. to 12 in., but situated away from edge; medium-sized pitch pockets and slight stain, but should be of a sound strong character. Hemlock permitted in this grade.

118. Common: Will admit of any two of the following, or their equivalent of combined defects: Wane $\frac{1}{2}$ in. deep on edge, 1 in. wide on face, extending not over one-sixth of the length of the piece; knots not more in diameter than one-third of the width of the piece; stain;

torn grain; pitch streaks; pitch pockets; seasoning checks; one straight split not longer than the width of one piece or a limited number of worm-holes well scattered. These boards should be firm and sound and suitable for use in ordinary construction without waste. Hemlock permitted in this grade.

119. No. 3 common boards or sheathing: Will admit of all stock below the grade of common that is suitable for cheap sheathing and will allow: Coarse knots, knot-holes, splits, rotten sap, and any number of grub or pin-worm holes. Hemlock permitted in this grade.

Dimension

121. Common dimension: Generally speaking, this stock must be suitable and of sufficient strength for all ordinary construction purposes without waste. Will admit of coarser knots than 1-in. common, which in a 2 by 4-in. should not be larger than 2 in. Spike knots not over two-thirds the width of the piece; wane not over ½ in. deep on edges and 1 in. wide on face up to 2 by 6 in., and ½ in. deep on edge and 1½ in. wide on face on 2 by 8 and wider, extending not more than ¼ the length of the piece; stain; solid pitch; pitch pockets; seasoning checks; one straight split, not more than the width of the piece, 2 or 3 grub-worm holes, a limited number of pin-worm holes and torn grain. Hemlock permitted in this grade in 4 and 6-in. widths.

122. No. 2 common dimension: This grade must be suitable for use in a cheaper class of construction than common. Will allow coarse and unsound knots and knot holes that do not unfit the piece for use intended, rotten streaks, pitch seams, pitch pockets, a reasonable amount of rotten sap and pin-worm holes, a few grub-worm holes well scattered. It is understood that no culls or stock that will not work without waste will be allowed in this grade. Hemlock permitted in this grade in 4 and 6-in. widths.

Fir timbers

123. Selected common: 2 by 4 in. to 2 by 12 in. and

3 by 4 in. to 4 by 6 in. shall be square-edged. Will admit any quantity of sound knots not over 1 in. diam., or small pitch pockets not over 4 in. in length. Sizes larger than 4 by 6 in. will admit sound knots not to exceed $1\frac{1}{2}$ in. diameter; pitch pockets not to exceed 6 in. in length.

124. Common: Rough timbers, 4 by 4 in. and larger, shall not be more than $\frac{1}{4}$ in. scant when green, or $\frac{1}{2}$ in. scant when S1S1E or S4S, and be evenly manufactured from sound stock and must be free from knots that will materially weaken the piece.

125. Timbers 10 by 10 in. may have a 2 in. wane on one corner, or its equivalent on two or more corners, one-fourth the length of the piece. Other sizes may have proportionate defects. Season checks and checks extending not over one-eighth the length of the piece admissible.

126. No. 2 common timbers: This is a grade of timber that will admit of large, loose, or rotten knots, shakes, or rot that do not impair its utility for temporary work. Hemlock and white fir will be allowed in this grade.

Fir Car Material

149. Railroad ties: Shall be sound common lumber.

Fir Bridge-Stringers

150. Common: Shall be sound common lumber, free from large unsound knots or knots in clusters, or other defects that will materially unfit the piece for the purpose intended.

151. Select common: Sap shall not show on any one corner more than 10% of any side or edge measured across the surface anywhere along the length of the piece. Shall be free from shake, splits, or pitch pockets over $\frac{3}{8}$ in. wide or 5 in. long. Knots greater than 2 in. diam. will not be permitted within one-fourth of the depth of the stringer from any corner nor upon the edge of the piece; knots shall in no case exceed 3 in. diameter.

Western Hemlock

214. Western hemlock is a wood well adapted to many uses. It is strong, holds nails well and therefore makes good framing lumber. It is hard and wears well as flooring. It is easily dressed to a smooth surface, and takes a fine polish, which, together with the beauty of grain and color, makes a fine interior finish. Western hemlock is entirely free from the 'wind shake' so common in the hemlock of the East. This lumber has been sold in the East under various names, such as 'Alaska pine,' 'Columbia pine,' 'gray fir,' 'Washington pine,' etc., and has given good satisfaction.

215. In a general way the rules for grading fir and spruce are applied to hemlock.

The preceding specifications apply to lumber as shipped in carload lots to the various retailers. When lumber is purchased from the retail lumber yard, it is usually classified as 'No. 1 Common,' or 'Merchantable.' The specifications governing this grade are as follows, taken from the 'Domestic List No. 6 of the Pacific Lumber Inspection Bureau.' Edition of 1912.

No. 1 Common

This grade shall consist of lengths 8 ft. and over (except shorter lengths be ordered) of a quality suitable for ordinary constructional purposes. Will allow small amount of wane, large sound knots, large pitch-pockets, colored sap one-third the width and one-half the thickness, slight variation in sawing, and slight streak of solid heart-stain.

Defects to be considered in connection with the size of the piece.

Discoloration through exposure to the elements or season checks not exceeding in length one-half the width of the piece shall not be deemed a defect excluding lumber from this grade, if otherwise conforming to the grade of No. 1 Common.

No. 2 Common

This grade shall consist of lumber 6 ft. and over (except shorter lengths be ordered) having defects that prevent it being graded as No. 1 Common, but must be suitable for a cheaper class of construction than the preceding grade.

Will admit large coarse knots, knot holes, and splits that do not render the piece unfit for use; colored sap, or wane on corner leaving a fair nailing surface, worm-holes, large pitch-pockets, and solid heart-stain one-half the piece.

The quality of the material purchased under these conditions, it is hardly necessary to state, depends largely upon the standards of the local yard, and in general lumber purchased as No. 1 Common will contain a considerable amount of No. 2 Common.

The accompanying photographs, reproduced from the 1915 Manual of the American Railway Engineering Association, Report of Special Committee on Grading of Lumber, illustrate some of the defects in Douglas fir timbers.

CHAPTER III

Unit Working Stresses

The allowable unit stresses to be used in any material are always a matter of individual judgment in the end. They should be decided from considerations of probable quality of material, nature of the loading, that is, whether live or dead, and if live, whether accompanied by impact, whether a constant load or one occurring at rare intervals; the particular detail under consideration, the purpose which the structure is to serve, the probabilities of future increase in loading or modifications of use, of the structure, and the character of the superintendence. Theoretically, the last condition may not influence the design, since the engineer is not necessarily responsible for the field inspection and must many times in his design assume competent and conscientious superintendence. Practically, however, no reputable engineer would use high working stresses did he know that the field inspection would be of questionable amount and quality.

For steel and concrete, the working stresses have been quite definitely established both by tests and experience. Of all structural materials, steel is the most uniform in quality, skilled labor being employed at all stages of its manufacture, fabrication and erection. In the case of concrete, the quality and strength of the finished product can be determined in general by selecting and proportioning the ingredients and by careful workmanship. Hence for different proportions of material, working stresses are now quite definitely established. On the other hand, sawed lumber is a finished product, and its strength must be judged by its physical appearance alone. The diversity of opinion as to the proper unit stresses or design may be demonstrated by even a cursory examination of the building codes of the different cities of the United States. This lack of similarity is especially striking in the case of the specified working stresses for timber in bending, these stresses varying by 150%.

However, in comparing the different unit working stresses adopted by the various building codes, it must be remembered that the specified loadings also vary, and to make a true comparison all stresses must be reduced to the same load-base. The working unit stresses given in building ordinances are generally rather low, in accordance with the average low grade of timber used.

Probably the best guide in selecting stresses for timber,

Kind of Timber	Bending: Extreme Fiber Stress: Average Ultimate	Bending: Extreme Fiber Stress: Working Stress	Bending: Modulus of Elasticity: Average	Shearing: Parallel to Grain: Average Ultimate	Shearing: Parallel to Grain: Working Stress	Shearing: Long Shear in Beams: Average Ultimate	Shearing: Long Shear in Beams: Working Stress	Compression: Perpendicular to Grain: Elastic Limit	Compression: Perpendicular to Grain: Working Stress	Compression: Parallel to Grain: Average Ultimate	Compression: Parallel to Grain: Working Stress	Compression: For Cols. under 15 diameters. Work. Stress	Compression: Formula for Working Stress in Long Columns over 15d	Ratio of length of Stringer to depth
Douglas Fir	6100	1200	1 510 000	690	170	270	110	630	310	3600	1200	900	1200(1-l/60d)	10
Longleaf Pine	6500	1300	1 610 000	720	180	300	120	520	260	3800	1300	980	1300(1-l/60d)	10
Shortleaf Pine	5600	1100	1 480 000	710	170	330	130	340	170	3400	1100	830	1100(1-l/60d)	10
White Pine	4400	900	1 130 000	400	100	183	70	290	150	3000	1000	750	1000(1-l/60d)	10
Spruce	4800	1000	1 310 000	600	150	170	70	370	180	3200	1100	830	1100(1-l/60d)	
Norway Pine	4200	800	1 190 000	590*	130	250	100		150	2600*	800	600	800(1-l/60d)	
Tamarack	4600	900	1 220 000	670	170	261	100		220	3200*	1000	750	1000(1-l/60d)	
West Hemlock	5800	1100	1 480 000	630	160	270*	100	440	220	3500	1200	900	1200(1-l/60d)	
Redwood	5000	900	800 000	300	80			400	150	3300	900	680	900(1-l/60d)	
Bald Cypress	4800	900	1 150 000	500	120			340	170	3900	1100	830	1100(1-l/60d)	
Red Cedar	4200	800	860 000	...	...	...		470	230	2800	900	680	900(1-l/60d)	
White Oak	5700	1100	1 150 000	840	210	270	110	920	450	3500	1300	980	1300(1-l/60d)	12

TABLE I—UNIT STRESSES IN STRUCTURAL TIMBER IN POUNDS PER SQUARE INCH.

Adopted by the American Railway Engineering and Maintenance of Way Association upon recommendation of the Committee on Wooden Bridges and Trestles.

*Partly air-dried.

l = length in inches.

d = least side in inches.

The stresses are for a green condition of timber and are to be used without increasing live-load stresses for impact.

The working stresses given in this table are intended for railroad bridges and trestles. For highway bridges and trestles, the unit stresses may be increased 25%. For buildings and similar structures in which the timber is protected from the weather, and practically free from impact, the unit stresses may be increased 50%. To compute the deflection of a beam under long-continued loading instead of that when the load is first applied, only 50% of the corresponding modulus of elasticity given in the table is to be employed.

is the Report of the Committee on Wooden Bridges and Trestles of the American Railway Engineering Association. Table I gives a summary of their recommendations. H. S. Jacoby, himself a member of this committee, says:* "These unit stresses are the result of an extended study of all the full-sized tests of structural timber available, as well as the unit stresses which have been in use in designing wooden bridges and trestles and have been demonstrated to be safe by extensive experience. As indicated in the footnote (see Table I), the values are based on a green condition of the timber, but in a few cases where no data for green timber were available, those for partly air-dry timber were inserted.* * * The table contains no working unit stresses for pure tension. Wood has a greater resistance to tension than to any other kind of stress, and it is found to be difficult to break it in a true tensile test. As there is more or less cross-grain, it is advisable to use the same unit stress in designing tensile members as for bending."

The working stresses as given by this table (increased 50% for buildings) are the highest that should be used for any structural timber. In fact they would seem to be too high for the material commonly used for ordinary building construction. The specifications given for determining the quality of timber for which these stresses are recommended are much more strict than those of the lumber manufacturers.

In 'Properties and Uses of Douglas Fir,'† there is given the result of bending-tests on 175 green bridge-stringers, purchased in the open market, 8 by 16-in. cross-section, and graded according to various standard specifications. It is interesting to note that only 54 of these stringers fell within the No. 1 railroad grade of the Specifications of the American Railway Engineering and Maintenance of Way Association. Table 10 of this

*"Structural Details," pages 555, 560.

†Forest Service Bulletin No. 88, U. S. Department of Agriculture, p. 43-45.

bulletin shows the modulus of rupture of the high 10%, embracing five tests, to be 8468 lb. per square inch; that of the low 10%, embracing five tests, 5750 lb. per square inch. The average modulus of rupture of all of these tests was 7108 lb. per square inch. The corresponding stresses at the elastic limit were 5800 lb., 3374 lb., and 4516 lb. per square inch.

When graded according to the specifications for No. 2 railroad, 67 stringers came within the limit. The high 10%, comprising eight stringers, had a modulus of rupture of 7430 lb. per square inch and a fibre stress at the elastic limit of 5006 lb. per square inch. Eight stringers also constituted the low 10%, having fibre stresses of 4761 lb. and 3109 lb. per square inch at the ultimate strength and elastic limits, while the general average fibre stresses were 6116 lb. and 4057 lb. per square inch.

In a similar manner the stringers were graded according to the export grading rules of the Pacific Coast Lumber Manufacturer's Association, adopted 1903. In the essentials, these are the same as the present specifications of the West Coast Lumber Manufacturer's Association. Thirty stringers fell within the grade of 'merchantable.' The high 10% had a modulus of rupture of 6353 lb., the low 10% 3710 lb., while the average modulus was 4946 lb. per square inch. The stresses at the elastic limits were 4597 lb., 2580 lb., and 3532 lb. per square inch, respectively.

Taking the safe working fibre stress in bending for timbers of the grade of Railroad No. 1 as 1800 lb. per square inch, it is interesting to compute the corresponding fibre stresses for timber of the grade of 'merchantable' by the Pacific Coast export grading rules. On the basis of the average ultimate strength this fibre stress would be $\frac{4946}{7108} \times 1800$ or 1250 lb. per square inch. On the basis of the elastic limit the stress would be $\frac{3532}{4516} \times$ 1800 or 1410 lb. per square inch.

Similarly, assuming that a working stress of 1800 lb. per square inch is satisfactory for timber of the grade of Railroad No. 2, the corresponding stresses for merchantable timber would be $\frac{4946}{6116} \times 1800$ or 1450 lb. per square inch, based on the respective moduli of rupture, and $\frac{3532}{4057} \times 1800$ or 1560 lb., per square inch, based on the respective elastic limits.

On the basis of the above comparison it is believed that 1500 lb. per square inch is the highest working unit-stress for bending that should in general be allowed for ordinary building construction with good inspection. Where the inspection is likely to be either poor or else wholly lacking, it would seem that the value of 1200 lb. per square inch should not be exceeded for timber in bending.

This statement is made for the following reasons: first, generally poor grade of timber as furnished by the local lumber-yard; second, undersize of timbers, especially of joists, due to surfacing or resawing, since any material over 1½ in. thick sells as 2-in. stock; and, third, holes bored, or notches cut in joists to accommodate conduits and pipes, these holes or notches often being placed in the worst possible position as regards the strength of the joist.

It will be noted that the table of unit stresses of the American Railway Engineering and Maintenance of Way Association gives no value for the elastic limit of timber in bending. While it is true that timber has not the definite elastic limit of steel, yet there is a definite yield point and no working stresses should be determined without a consideration of this property. The elastic limit is especially important in the case of bearing perpendicular to the fibres of the timber, and the allowable stress for cross-bearing should be based on this limiting resistance and not on the ultimate strength. The folly of small washers of insufficient size, for rods or bolts taking tension, has been mentioned before and will be treated more fully in a succeeding article. There are reproduced here (Fig. 14 and 15) two diagrams taken from Forestry Bulletin No. 88, showing the variation in the ultimate stresses and the stresses at the elastic limit of the Douglas fir bridge-stringers tested.

The working stresses of Table I and the results quoted from Forest Service Bulletin No. 88 both represent values for green timber. The effect of seasoning on timber is, in general, an increase in strength. For example, Forest Service Bulletin No. 88 gives the results of bending tests on green and air-dried halves of ten 8 by 16-in. by 32-ft. stringers. That is to say, ten green stringers, 8 by 16 in., 32 ft. long, as nearly uniform in quality throughout their lengths as possible, were selected. "One-half of each 32-ft. piece was tested in a green condition, and the other half tested after air-seasoning. * * The average moisture content of the air-seasoned material was 16.4%." The average ultimate strength in bending of the green material was 5440 lb. per sq. in., while the same value for the air-seasoned timber was 6740 lb. per sq. in., or an increase of 24%. The corresponding values for the elastic limit were 3740 and 5478 lb. per sq. in., showing an increase in strength due to seasoning of 47%. Also, "a number of tests were made on various grades of Douglas fir stringers seasoned from six to eight months; the grades select, merchantable, and seconds being those defined in the export grading rules of the Pacific Coast Manufacturers Association adopted in 1903. In this group of stringers the fibre-stress at elastic limit and the modulus of rupture, in the case of select material, are increased, respectively, 8% and 5% by seasoning, the modulus of elasticity remaining practically unchanged. In the merchantable material the increase in these functions is respectively 19%, 33%, and 6%. In the seconds the fibre-stress at elastic limit increased 6%, while the modulus of rupture and modulus of elasticity show, respectively, a decrease of 12% and 2%." And, "The failures in seasoned Douglas fir stringers and car-sills were similar to those in green material, except that failures in horizontal shear were more common."

"Failure in horizontal shear is more common in seasoned than in green timbers, because the net areas resisting shear along the neutral plane is often considerably decreased by checks. It seldom occurs in weak, low-grade material, which fact is doubtless due to the dowelling-pin action of the knots invariably associated with low-grade timbers."

"This summary of failures, as well as that for green material, . . . indicates conclusively that in general the point of greatest weakness in Douglas fir beams is the part subjected to the highest stresses in compression parallel to the grain. The principal exceptions to this rule are beams that have large knots on or near the tension-face, beams that have bad diagonal or cross-grain, and beams that contain deep checks along the neutral plane. The elastic limit of the beam is closely related to the strength of the wood in compression parallel to the grain, while the modulus of rupture is most dependent upon the quality of the wood that is subjected to tensile stresses."

Probably the most troublesome part of detailing connections in timber is to make the necessary provision that the cross-bearing strength be not exceeded. Indeed, it will usually be found upon examination of typical structures that this point has not been considered. I have found that a great many designers consider that the crushing of the fibres of the timber in side-bearing is not a serious matter. The idea is prevalent that, after an initial crushing, no further deformation will take place, and that the structure will still be in a working condition. The fallacy of this idea will be realized by anyone who has seen an actual test made on the crushing strength of timber across the fibres. In designing timber framed structures, this weakness of timber must be carefully considered, otherwise, the consequent deformation may unduly stress other parts. Fig. 14 shows that the average elastic limit is about 570 lb. per square inch, therefore the working stress may be taken at 285 lb. per square inch. Here, again, these values are for green material. Seasoned material should show a marked increase of strength, and for air-seasoned Douglas fir, protected from moisture, the working stress may well be increased from 285 lb. per square inch to 350 lb. per square inch.

Reference to Fig. 14 shows that the average elastic limit of Douglas fir for end compression or pressure against the ends of the fibres is 3612 lb. per square inch. The working stress as recommended by Table I for buildings is 1800 lb. per square inch, which gives a safety factor of two on the basis of the elastic limit. I prefer in general to limit the pressure to 1600 lb. per square inch. When providing for the stresses of compression, tension, and shear, the nature of the detail should always receive consideration in deciding the exact amount of working stress. Under some conditions 1800 lb. per square inch and even slightly more will provide a sufficient factor of safety. For example, the basic* working compressive unit-pressure of a lag screw in timber may be well taken at 1800 lb. per square inch, since there is a close and uniform fit of the lag screw in the bored hole. On the other hand, the actual pressure of the toe of the batter-post of a truss against the shoe-plate may be almost as great as the ultimate strength of timber in end compression, depending altogether upon how accurately the carpenter shapes the batter-post to fit the shoe. It is evident, then, that in details involving difficult cuts, a relatively low unit pressure should be used, and in straight cross-cuts a corresponding higher working stress may be justly employed.

This intentional variation in unit stress applies with greater force to the consideration of details involving the stress of tension. Timber has a high tensile resistance, and where it is certain that no other stress exists than simple, uniformly distributed tension, the working stress of 1800 lb. per square inch or even higher is not excessive. However, as will be shown later, secondary

*Basic, as distinguished from average unit stress on diametral section of lag-screw, to be discussed in Chapter IV.

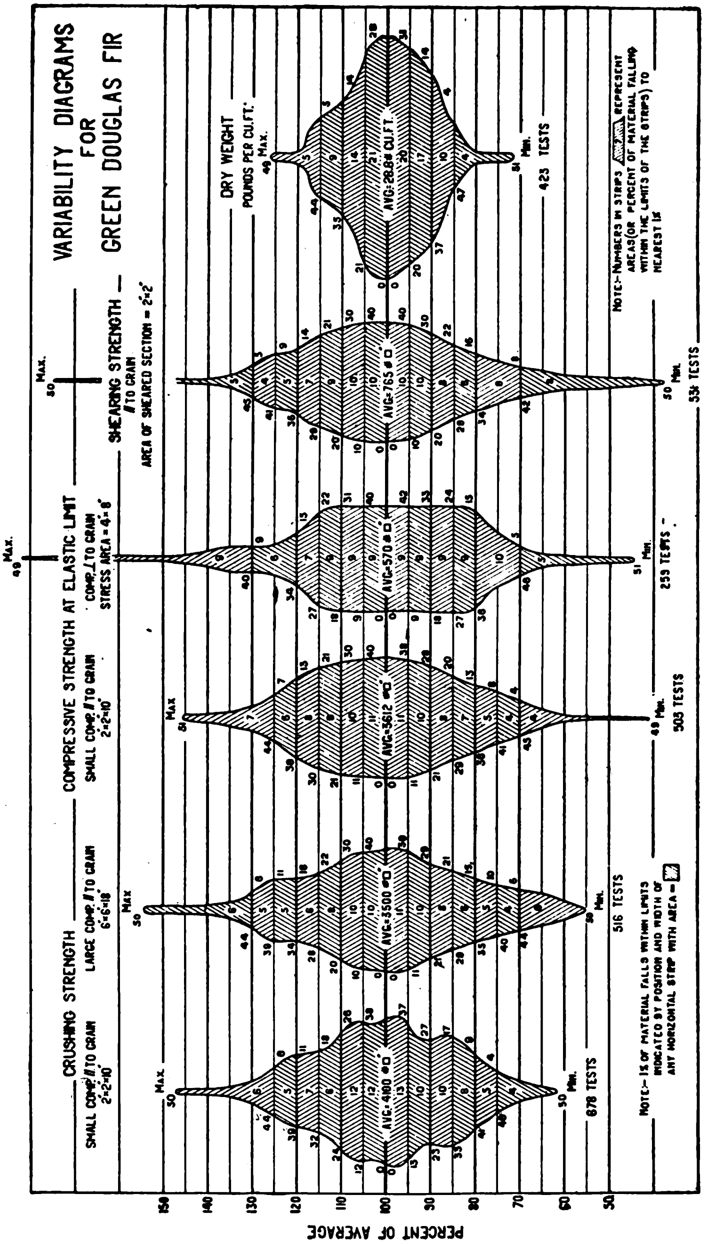


FIG. 14—ILLUSTRATING VARIABILITY OF GREEN DOUGLAS FIR IN CRUSHING STRENGTH, COMPRESSIVE STRENGTH AT ELASTIC LIMIT, SHEARING STRENGTH, AND DRY WEIGHT.

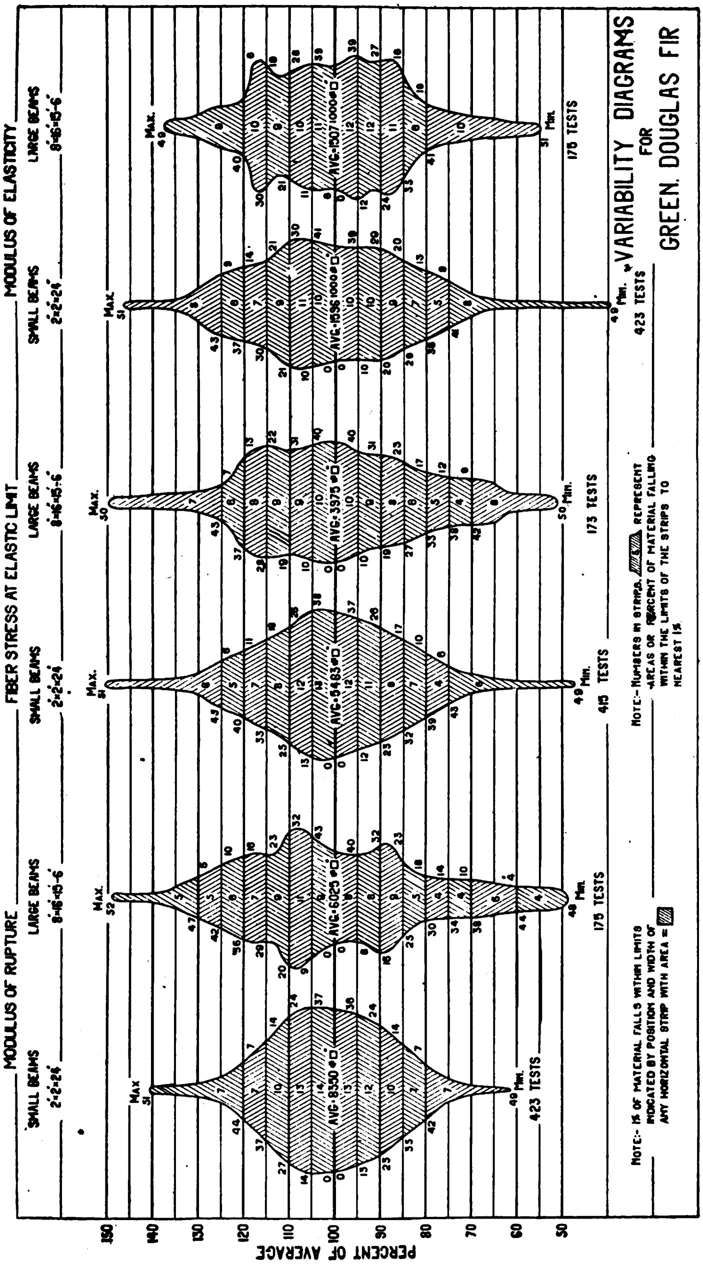


FIG. 15—ILLUSTRATING VARIABILITY OF GREEN DOUGLAS FIR IN MODULUS OF ELASTICITY, MODULUS OF RUPTURE, AND FIBRE STRESS AT ELASTIC LIMIT.

stresses of indeterminate but nevertheless large amount may exist, as in the case of tension chords in trusses. For this reason I prefer 1500 lb. per square inch as a general limit for Douglas fir in tension.

The ever-present season-crack in timber as an incipient cause of failure by shear along the fibres is sufficient justification for limiting the working stress in shear to 150 lb. per square inch, and it is wise to decrease this stress to 100 lb. per square inch, unless it is necessary to exercise the strictest economy in the design of the particular detail.

Time Element as Affecting the Strength of Timber

No mention has yet been made of the effect of time on timber; in other words, the effect on the ultimate strength of long-continued loads. This is a quality characteristic of timber alone, as compared to the other materials ordinarily used in construction, and may well be referred to as fatigue, although the term 'fatigue,' as usually understood in a technical sense when applied to metals refers to a different thing, namely, the effect on the strength of the material of a great number of repetitions of loadings, all within the ultimate strength.

J. B. Johnson in his 'Materials of Construction' notes that "Timber is entirely different from other forms of building material in this, that it constantly yields under heavy loads, and will finally fail under little more than half of the load required to break it on a short time test, such as is ordinarily given in a testing machine. R. H. Thurston reported a few time tests on small wooden beams, 1 in. square and 4 ft. long, in the Transactions of the American Association for the Advancement of Science for 1881. He found that 60% of the breaking load would break the beams if left on some nine months* * * the author has made about 75 tests in crushing endwise. * * * Longleaf yellow-pine sticks, 40 in. long and 2 in. square, were cut from a single plank, and these had seasoned three years in the dry. Each stick was dressed to about 1.5 in. square, and then cut into specimens 3 in. long. The alternate specimens were tested in compression endwise in a testing machine, as is ordinarily done, and the strength was found to be exceptionally uniform. The intervening specimens were then loaded in succession, with various percentages of the average ultimate strength of the two adjoining specimens, and these loads left upon them until failure occurred." Mr. Johnson concludes "But little more than one-half the short-time ultimate load will cause a column to fail if left upon them permanently. Or, the ultimate strength of columns under permanent loads is only about one-half the ultimate strength of these same columns as determined by actual tests in a testing machine."

A paper reporting the behavior of timber under long-continued loads was read before the American Society for Testing Materials in June 1909 by H. D. Tiemann. The tests were on twenty beams 2 by 2 by 40 in. of longleaf yellow-pine on a 36-in. span, and were made at the Yale Forest School. An abstract of this paper appears in *Engineering News* for August 26, 1909, Vol. 62, No. 9, pages 216-217.

Mr. Tiemann concluded that "dry longleaf pine beams may be safely loaded permanently to within at least 75% of their immediate elastic limit, provided no increase in dampness occurs, and deflection will ultimately cease (practically) under this load. No perceptible deflection will occur because of the time-effect and loads up to within 20% of the immediate elastic limit. ('Immediate' signifies 'caused by an immediate load or live load as by an ordinary machine-test.') Loads greater than the immediate elastic limit are dangerous, and will generally result in rupture if continued long enough," The length of time necessary to cause such failure is shown to be as small as a year or less. It is evident that the 'critical' load, in either beams or columns, does not correspond to the elastic limit.

The time element is the cause of the recommendation of the American Railway Engineering Association, as given in Table I. "To compute the deflection of a beam under long-continued loading instead of that when the load is first applied, only 50% of the corresponding modulus of elasticity given in the table is to be employed.''

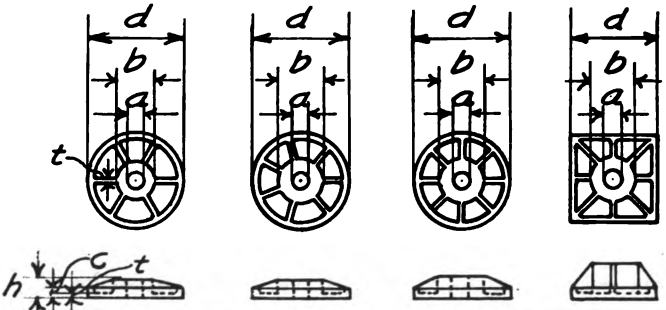
CHAPTER IV

Washers and Pins

Compression on Surfaces Inclined to the Direction of Fibres. Resistance of Timber to Pressure from Cylindrical Metal Pins. The comparative weakness of soft timbers like Douglas fir to compression across the fibres has been mentioned, likewise the prevailing use of the standard cast-iron and malleable-iron washers with bolts and rods. This practice is altogether too common, and results mainly from an unwarranted and ignorant confidence on the part of the designer and constructor in the word 'standard.' Standard details in steel construction are usually the result of careful and intelligent study, and long experience, but even standard details in steel will not be suitable for every case. In timber construction the term 'standard' means even less, and many details to which this term is applied are unfit for even the average case.

If the use of the standard O. G. cast-iron washer and the standard malleable-iron washer is to be condemned, what may be said of the employment of small circular wrought-steel washers with timber of low cross-bearing strength? These latter washers were originally designed for use with hard woods, yet they are constantly employed in ordinary construction, with the result that the washers are usually found to be drawn far into the wood, and the resistance to further tension in the bolt is nil.

It is not intended to make the assertion that all bolts require large washers to fulfill their function properly. Relatively short bolts acting in shear, as in splice joints, may be designed with small washers with safety. All bolts and rods, however, which act in tension should be



Size Bolt	Size Upset	a	b	c	d	h	t	Shape Base	No. Ribs.	Wgt.
5/8	Not U.	3/4	1 9/16	1/8	3 1/8	3/4	1/8	C	6	.56
3/4	"	7/8	1 3/4	3/16	4	1	1/4	C	6	1.10
7/8	"	1	2 1/4	3/16	4 1/2	1 1/8	1/4	C	6	1.80
3/4	1	1 1/8	2 3/8	1/4	5 1/4	1 1/4	1/4	C	6	2.79
13/16	1 1/8	1 1/4	2 5/8	1/4	5 5/8	1 5/16	1/4	C	6	3.29
7/8	1 1/4	1 3/8	3	5/16	6 1/2	1 7/16	5/16	C	7	5.30
1	1 3/8	1 1/2	3 1/4	5/16	7	1 9/16	5/16	C	7	6.34
1 1/8	1 1/2	1 5/8	3 1/2	3/8	7 7/8	1 11/16	3/8	C	7	9.04
1 1/4	1 5/8	1 3/4	3 3/4	3/8	8 5/8	1 15/16	3/8	C	7	11.30
1 3/8	1 3/4	1 7/8	4	3/8	9 1/8	2 1/4	3/8	C	7	13.59
1 7/16	1 7/8	2	4 3/8	7/16	10	2 3/8	7/16	C	8	18.66
1 1/2	2	2 1/8	4 5/8	7/16	10 3/8	2 7/16	7/16	C	8	20.39
1 5/8	2 1/8	2 1/4	4 3/4	1/2	11 1/4	2 5/8	1/2	C	8	25.99
1 3/4	2 1/4	2 3/8	5 1/8	1/2	12 1/8	2 3/4	1/2	C	8	30.62
1 7/8	2 3/8	2 1/2	5 1/8	5/8	11 1/4	4 1/4	5/8	Sq.	8	48.23
2	2 1/2	2 5/8	5 5/8	3/4	12	5 1/8	3/4	Sq.	8	69.32

FIG. 16. DIMENSIONS OF SPECIAL CAST-IRON WASHERS.

provided with washers of ample area, and even those joints in which the bolts act principally in shear will be greatly strengthened and their effective life increased by the use of washers of generous size. Except for temporary work, the employment of larger washers than the standard will always be economical, when all the factors are considered. Especially is this true where the timber work is exposed to the weather, as the tightening of the nuts on the bolts when washers of insufficient size have been used will crush the timber, exposing it to the weather with consequent decay.

F. L. Bixby made a series of tests on the efficiency of the standard O. G. cast-iron washer in the course of thesis work at the University of California in 1904. These tests have been reported by me.* Mr. Bixby found that the standard washer would develop from 30% to 45% only of the strength of the corresponding rod. Special O. G. washers were then designed with the following diameters:

$\frac{3}{4}$-in. bolt, 4.38-in. diameter washer
$\frac{5}{8}$-in. bolt, 3.44-in. diameter washer
$\frac{1}{2}$-in. bolt, 2.96-in. diameter washer.

Tests were made on these washers as on the standard washers. The results, rated on one-half the stress required to strip the thread of the rod in tension, showed average efficiencies from 86% to 91% as against the low efficiencies noted above for the standard washers. A typical load-compression curve of Mr. Bixby's tests is shown in Fig. 17. The timber was Douglas fir in all cases.

The tests show conclusively that for all washers with tension rods pulling across the fibres of Douglas fir, the area of the washer should be such that the unit bearing-stress will be the same fraction of the elastic limit of the timber for cross-bearing as the unit stress in the rod is of the elastic limit of the rod in tension. By reference to Fig. 14 of the preceding chapter, the average elastic limit of green Douglas fir for compression across the fibres is seen to be 570 lb. per sq. in.† Consequently, if the elastic limit of steel rods is assumed to be 32,000 lb. per sq. in., the corresponding unit working-stress for the washers, when used with green timber, should be 285 lb. per sq. in. This low stress may seem to be extravagant, inasmuch as many timber trusses of Douglas fir are giving service with much higher washer-pressure. It

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**Engineering News*, Vol. 71, No. 13.

†Mr. Bixby does not note whether the timber of his tests was green or air seasoned. The elastic limit as found by his tests for bearing across the fibres was from 410 to 677 pounds per square inch.

may be definitely stated, however, that in the event of a considerable overload, the washers would crush the timber of the truss chords to such an extent that excessive deflection with a probable consequent failure of the truss would occur, while the rods would not be dangerously overstressed. With such a truss, where the rods are designed for 16,000 lb. per sq. in., and the washers for a bearing pressure of 400 lb. per sq. in., the critical strength of the truss would not be lessened were the sizes of rods to be decreased until their unit stress became 22,400 lb. per sq. in. In other words, looking at the matter simply from the commercial standpoint, the engineer may increase the unit stress in the rods of a truss, and design the truss with smaller rods and larger washers than are ordinarily used, and still feel confident that the truss is as safe as one having conservative unit-stresses in the rods and high bearing-pressures under the washers.

In the framing of the Panama-Pacific International Exposition buildings, special ribbed cast-iron washers were designed and used. Their dimensions are shown in Fig. 16. These washers were designed for a bearing pressure on the timber of 350 lb. per sq. in. and a corresponding unit stress in the steel of 16,000 lb. per sq. in.

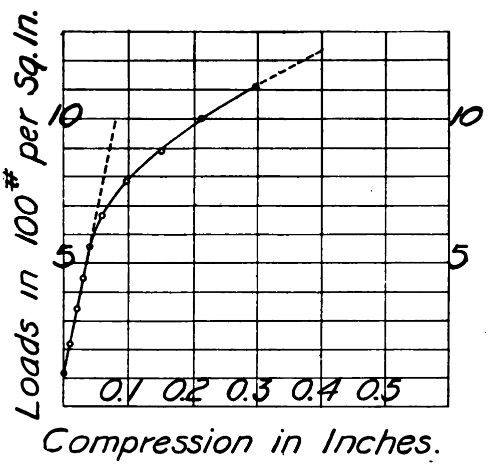

FIG. 17. TYPICAL WASHER CURVE.

In this case, the rather high unit-stress for cross-bearing on the timber was justified by the fact that all the work was of a temporary nature, and first cost, consistent with safety, was the governing factor in the design.

To check the strength of the washers as designed, some tests were made on the various sizes, at the University of California. The first washer tested was a $\frac{3}{4}$-in. washer. It was placed in the testing machine fitted with a short length of bolt and nut, the nut bearing against the head of the testing machine, and the washer bearing across the fibres of a short block of Douglas fir. The bolt fitted into a hole bored into the wood. Although a total load of 23,000 lb. was sustained without fracturing the washer, the latter sank into the timber $\frac{1}{2}$ in. under the pressure, and the test had to be discontinued without having reached the capacity of the washer in cross breaking-strength. The other washers were broken by making them bear against the ends of the fibres of the block of Douglas fir, the head of the testing machine resting directly upon the washer. The ends of the wooden block were not exactly parallel, so that the machine head bore eccentrically on the washer, producing bending stresses in the cast iron. As this condition often exists in actual construction, the bolts being not exactly normal to the timber, the results of the tests may be taken as typifying the strength of such washers under the most adverse conditions of construction.

Table II gives the results of the tests. The specimens tested were from two different foundries; in the table of results the two sets are designated as *A* and *B* washers.

The dimensions of the washers tested were all as shown in Fig. 16, except that in the case of $\frac{3}{4}$-in. washers, h $\frac{13}{16}$-in. instead of 1 in. and t was $\frac{3}{16}$-in. instead of $\frac{1}{4}$. Similarly, for the $\frac{7}{8}$-in. washers, t was $\frac{3}{16}$-in. instead of $\frac{1}{4}$-in. Due to the rather poor showing of the *B* foundry washers, the dimensions of the $\frac{3}{4}$-in. and $\frac{7}{8}$-in. washers were increased as noted in the preceding paragraph and to the dimensions shown in Fig. 16. The effect of shrink-

TABLE II

FAILURE TESTS OF EXPOSITION WASHERS

Size	Ultimate Load-Lb.	Remarks
	Washers from Foundry *A*	
$\frac{3}{4}$	20,000	Failure at edge of head.
$\frac{3}{4}$	28,000	Failure through head.
$\frac{3}{4}$	16,000	Failure through ribs: flaws.
$\frac{3}{4}$	17,000	Failure through ribs: flaws.
$\frac{7}{8}$	33,000	Failure through head.
$\frac{7}{8}$	23,500	Failure at edge of head.
1	23,500	Failure through head.
	Washers from Foundry *B*	
$\frac{5}{8}$	18,000	Failure through ribs near rim.
$\frac{5}{8}$	20,500	Failure through ribs near rim.
$\frac{3}{4}$	7,500	Failure through centre of hole.
$\frac{3}{4}$	9,500	Failure through head.
$\frac{3}{4}$	9,500	Failure through head.
$\frac{7}{8}$	16,000	Failure at edge of head.
$\frac{7}{8}$	14,000	Failure at edge of head.
1	25,600	Failure through head.
1	17,000	Flaws.
1	23,000	Small flaws.
1	26,000	Small flaw.
1	23,000	Small flaw.

age and flaws is much greater in the smaller sizes of washers than in the larger sizes, consequently there should always be more metal than might be called for by strict observance of theoretical dimensions. Further, in driving bolts and tightening nuts, the washers are subject to considerable hammering, and for these reasons any thickness of metal less than $\frac{1}{4}$ in. is not advisable.

In Jacoby's 'Structural Details,' there is published the result of a series of tests on cast-iron and malleable-iron washers by H. M. Spandau, which shows that ribbed cast-iron washers of equal strength with the O. G. type may be designed, and at a saving in metal of from 30 to 50%. There are also given details of ribbed cast-iron washers used as standard by the Atchison, Topeka & Santa Fe and the Union Pacific railroads. Further tests of cast-iron washers were made by L. R. Rodenhiser in the course of thesis work at Cornell University.*

***Cornell Civil Engineer*, Vol. 23, No. 2. The tests herein

Jacoby in his 'Structural Details,' page 246, allows a 25% increase in the allowable unit bearing-pressure, when the bearing does not cover the full width of the member. While there may exist a theoretical reason for such increased resistance, I do not believe that actually such a condition will exist, and I do not recommend any such increase in bearing-pressure. On small jobs, it will usually be found more economical to use small square steel plate-washers instead of special cast-iron washers. The proper sizes of plates needed to give the desired unit bearing-pressures may be easily computed, and the information given to the contractor by means of typical sketches and tables. The thickness of such plates should not be less than one-half the nominal diameter of the threaded portion of the rod or bolt. For example, if a unit bearing-pressure of 350 lb. per sq. in. be used, a $\frac{3}{4}$-in. bolt in tension should be provided with a $\frac{3}{8}$ by $3\frac{3}{4}$ by $3\frac{3}{4}$-in. plate.

Compression on Surfaces Inclined to the Direction of Fibres. The preceding discussion relates only to washers bearing normally on and across the fibres of the timber. Where the direction of pressure is inclined to the direction of the fibres, the area for bearing need not be so large, and it will usually be found convenient and economical to use washers of special design, either of cast iron or of plate steel.

The resistance to compression offered by the fibres of the timber when the pressure is exerted at an inclination with the direction of the fibres, namely, neither normal nor parallel to the fibres, is a subject on which there is some difference of opinion. H. S. Jacoby, in his 'Structural Details' develops the following formula,

$$n = p \sin^2 \Theta + q \cos^2 \Theta$$

in which

n = the allowable unit stress on a surface which

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recorded were apparently made for washers bearing against the ends of the fibres of the wood. For this reason, the tests are not of practical benefit in establishing the proper size of washer for bearing across the fibres in timber like Douglas fir.

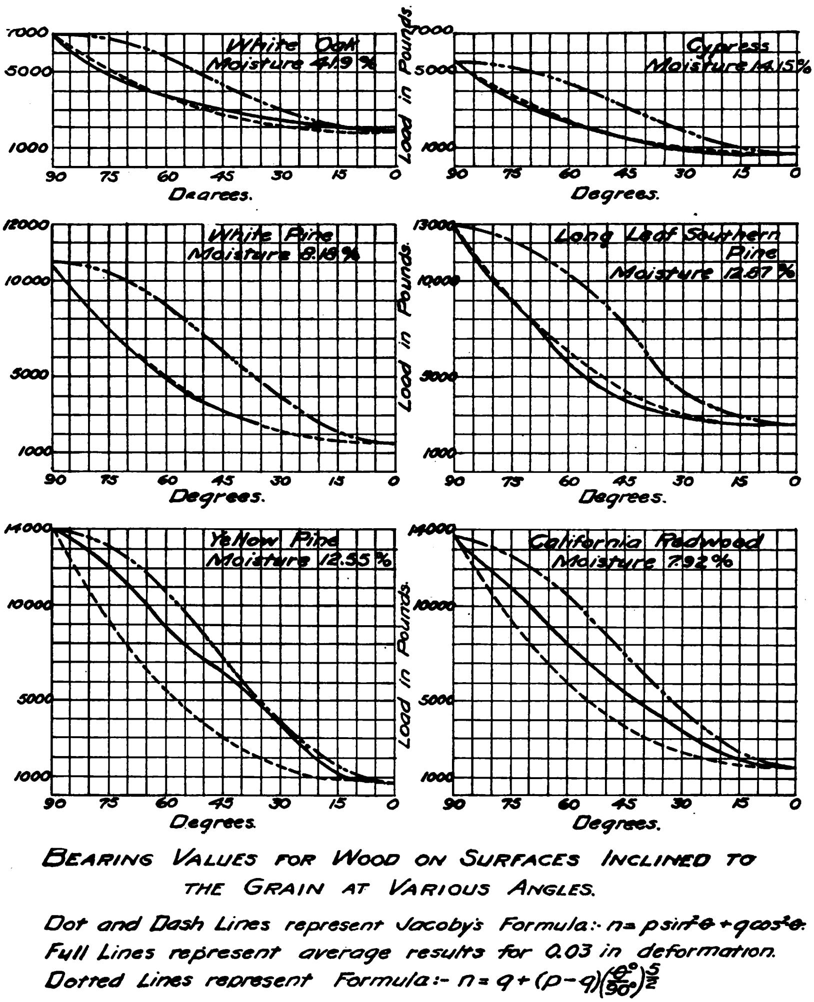

FIG. 18. BEARING VALUES FOR WOOD ON SURFACES INCLINED TO THE GRAIN AT VARIOUS ANGLES.

makes an angle Θ with the direction of fibres

p = the allowable unit stress against the ends of the fibres

q = the allowable unit stress on the sides of the fibres

Fig. 19 shows this equation plotted, using the values p = 1800 lb. per sq. in., and q = 285 lb. per sq. in. These values are approximately one-half the values for the elastic limits as shown on Fig. 14.

Malverd A. Howe published* the results of some tests made to determine the allowable bearing pressure on inclined surfaces for various timbers. These results are shown in the diagrams of Fig. 18, which are taken from the article just mentioned. Mr. Howe recommends the formula

$$n = q + (p - q) \left(\frac{\theta^\circ}{90^\circ}\right)^{5/2}$$

which equation corresponds closely to the values as determined by the tests. Fig. 20 shows this formula plotted, using the values $p = 1800$ and $q = 285$, as before.

The allowable unit stresses for bearing on inclined surfaces of timber, as shown by Fig. 19 and 20, differ materially. The effect on the design of a truss joint from using the two sets of allowable pressures will be shown in a subsequent article. Mr. Jacoby's curve is the more economical in material, but the results of Mr. Howe's tests are not to be ignored. The data available at present should be supplemented by further tests.

Details of beveled cast-iron washers are shown in Fig. 21. Attention is called to the thickness of the base of these washers. It is intended that the washers should be set into the timber to the depth of the base plate. For inclined bolts or rods, the use of these washers will give a neat and efficient detail, and one comparatively easy to construct.

Resistance of Timber to Pressure from a Cylindrical Metal Pin. The bearing of a round metal pin in a closely fitting hole in timber, as occurs in the case of a spike, screw, or bolt with a driving fit, is a special case of bearing on inclined planes. The subject is discussed in Jacoby's 'Structural Details,' Chapter II, Article 23, where the following statements are made:

"In framing, it is sometimes necessary to use metal pins or bolts as beams in transferring stresses from one timber to another. This involves the determination of

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**Engineering News*, Vol. 68, No. 5, and Vol. 68, No. 10.

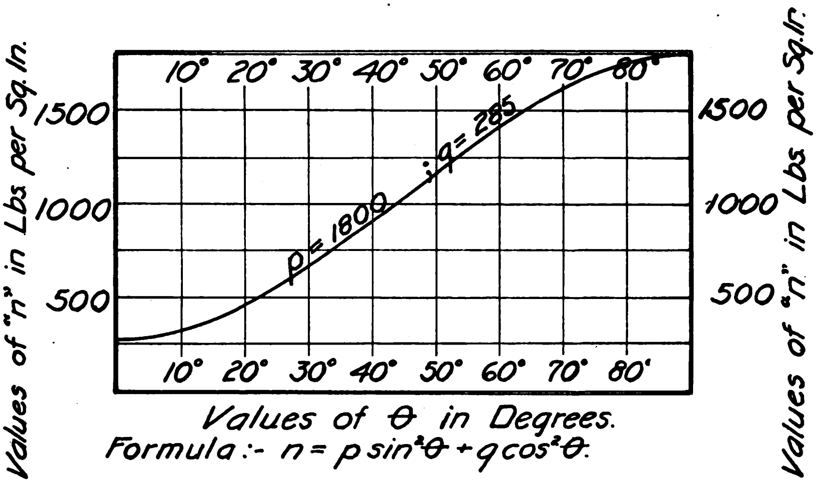


FIG. 19. CURVE FOR DOUGLAS FIR BASED ON JACOBY'S FORMULA.

n = normal intensity on inclined planes.
p = normal intensity on ends of fibres.
q = normal intensity across fibres.
θ = angle made by plane with direction of fibres.

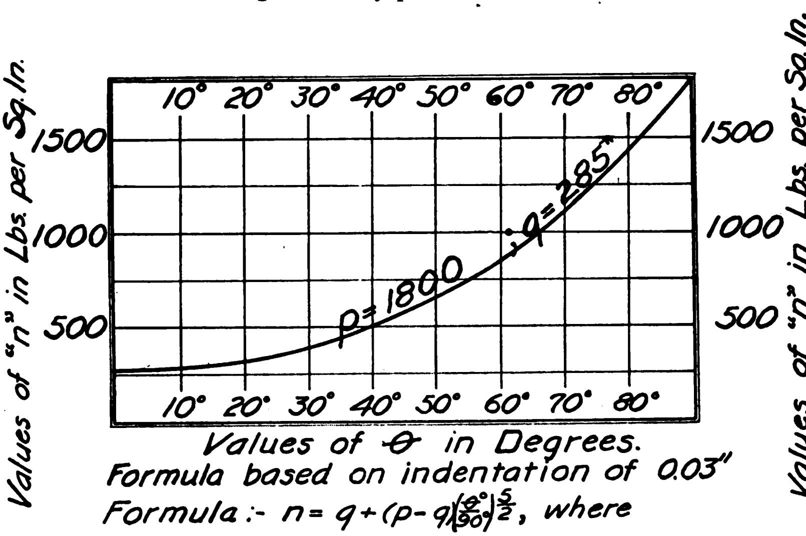


FIG. 20. CURVE FOR DOUGLAS FIR BASED ON HOWE'S FORMULA.

n = normal intensity on inclined planes.
p = normal intensity on ends of fibres.
q = normal intensity across fibres.
θ = angle made by plane with direction of fibres.

the pressure of the fibres of the wood upon the cylindrical surface of the pin. When the resultant of the pressure is perpendicular to the fibres of the wood, the magnitude of the resultant is the same as if the bearing surface were the diametral section of the pin. But when the direction of the resultant is parallel to the fibres of the wood, the case is entirely different because the resistance of the fibres to lateral compression is much less than to longitudinal compression."

The following formulæ are deduced by Jacoby:

Let

P = the total safe load on the pin

h = height of the timber bearing against the pin

d = the diameter of the pin

p = safe unit stress for compression parallel to the fibres, or for bearing on the ends of the fibres

q = safe unit stress for compression perpendicular to the fibres, or across the fibres

u = the unit pressure normal to the surface of the pin

Θ = the angle which u makes with the direction of the fibres (complement of angle Θ used above)

Θ' = the special value of Θ for which the transverse component of $u = q$

"For wood having a ratio of 0.25 between the safe unit bearing on side and on the end of the fibres, respectively, $\Theta' = 15°$, and $P = 0.62\ hdp$. An experimental determination for long-leaf yellow pine, but in which the timber was tested to its ultimate strength, gave an average coefficient of 0.63 for 5 tests. The specimens were prevented from splitting by means of clamps. The plane of division between the fibres crushed sidewise was marked in every case, and gave an average value for Θ' of $15\frac{1}{2}°$. When the resultant of the pressure of the wood on a round pin is perpendicular to the fibres, the magnitude of the safe bearing value is to be taken as hdq, that is, the pressure is the same as if the pin were square or rectangular in cross-section."

With the preceding theory, and in particular with the statement as to the bearing across the grain, I am not in accord. The tests on spiked, screwed, and bolted joints do not show the difference in strength between end and cross-bearing that the theory would indicate exists, as will be seen from the record of the tests to be given later.*

The following theory is presented to cover the case now under discussion. Fig. 22 shows the case of a cylindrical metal pin bearing against the ends of the fibres and across the fibres, respectively. Let the nomenclature be as before, with the additional terms, as follows:

$n =$ safe unit stress for compression on a surface inclined to the direction of the fibres.

$s' =$ the component of n, parallel to the direction of fibres.

$s'' =$ the component of n, perpendicular to direction of fibres.

If the assumption be made that the various differential inclined surfaces will resist pressure simultaneously, in radial lines, equivalent in amount to the allowable unit stress, n, and if the law of variation of pressures on inclined surfaces be known, the capacity of the timber or the safe load on the pin may at once be determined. For example, let it be assumed that the law governing the allowable pressures on inclined surfaces is according to Jacoby's formula, and that

$n = p \sin^2 \Theta + q \cos^2 \Theta$

Then, referring to Fig. 22.

$s = n \sin \Theta = (p \sin^2 \Theta + q \cos^2 \Theta) \sin \Theta$

The pressure s' acts upon the differential area $hr \cos \Theta \, d \, \Theta$

Therefore

$$P = 2phr \int_{0^\circ}^{90^\circ} \sin^2 \theta \sin \theta \, d\theta - qhr \int_{0^\circ}^{90^\circ} \cos^2 \theta \sin \theta \, d\theta$$

Integrating and substituting the limits,

$P = \frac{2}{3} phd + \frac{1}{3} qhd$

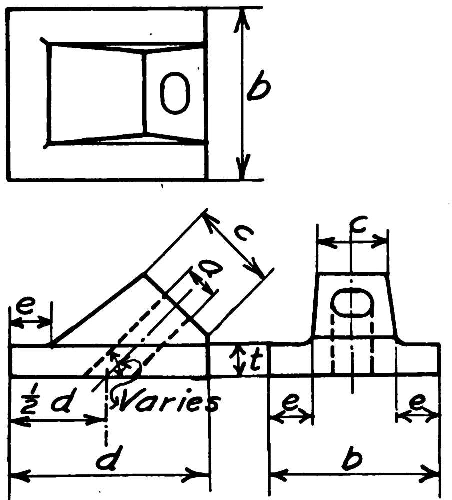
For a pin of diameter of 1 in. and length of 1 in., P will represent the average unit stress on the diametral section of the pin $= p'$

*See also 'Tests of Duplex Hangers,' Chapter XII.

or p' (unit stress in lb. per sq. in.) $= \frac{2}{3}p + \frac{1}{3}q$.

Similarly, for the case of Fig. 22, where the direction of P is perpendicular to the direction of the fibres, it may be shown that

$p'' = \frac{1}{3}p + \frac{2}{3}q$, where p'' = average unit stress on diametral section of the pin.



Size Rod	a	b	c	d	t	e
3/4	7/8	3½	1¾	4	5/8	3/4
7/8	1	4¼	2	4½	3/4	1
1	1⅛	4¾	2¼	5¼	7/8	1⅛
1⅛	1¼	5¼	2½	6	1	1¼
1¼	1⅜	6¼	2¾	6½	1	1½

FIG. 21. DETAILS OF BEVELED CAST-IRON WASHERS.

Using the values of 1800 lb. per sq. in. and 285 lb. per sq. in. for p and q, respectively, in the above formula,

$p' = 1295$ lb. per sq. in., and

$p'' = 790$ lb. per sq. in.

If, on the other hand, the variation of the pressure on inclined surfaces be taken from Howe's formula, and the same numerical values for p and q used as before, it may be shown that

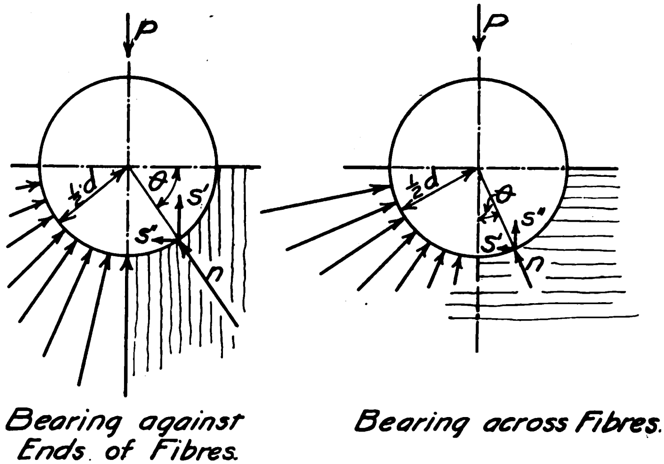


FIG. 22. DIAGRAM SHOWING PRESSURE OF ROUND METAL PIN IN TIMBER.

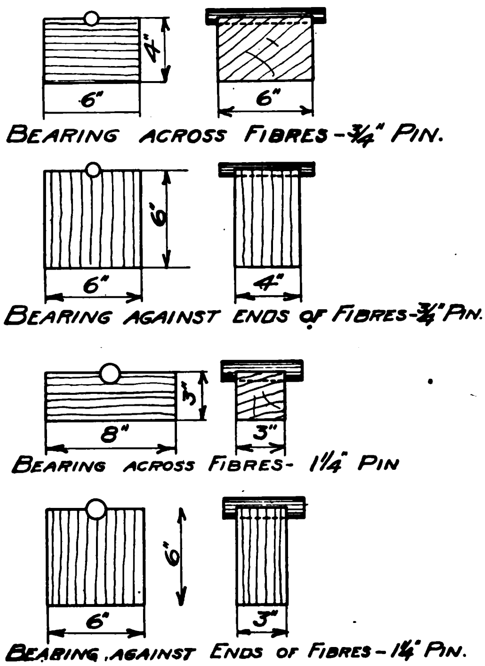


FIG. 23. DETAILS OF TEST-PIECES USED IN PIN EXPERIMENTS.

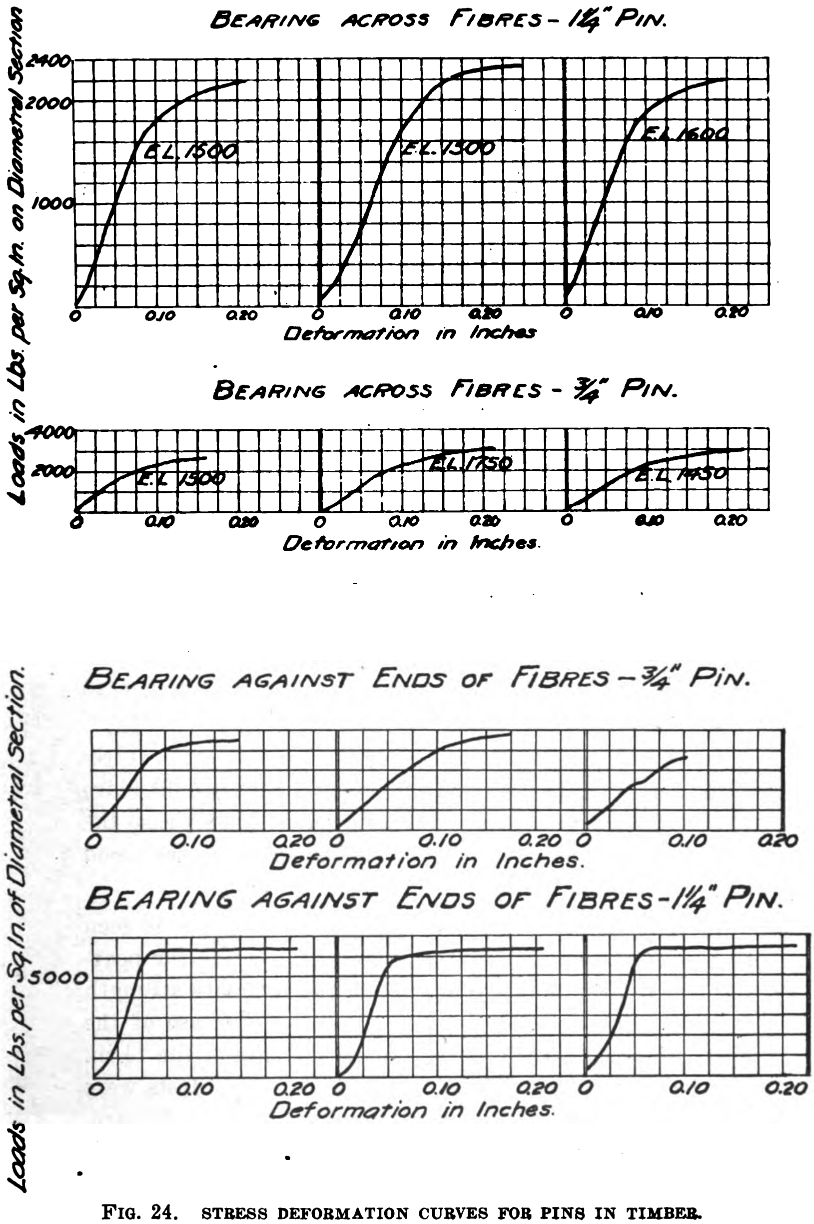


FIG. 24. STRESS DEFORMATION CURVES FOR PINS IN TIMBER.

$p' = 1120$ lb. per sq. in., and that

$p'' = 675$ lb. per sq. in.

In an effort to throw light on this question, I tested a number of small blocks of Douglas fir, made with half holes, and each fitted with a short piece of bolt having a tight fit in the holes. Two sizes of bolts were used, one $\frac{3}{4}$-in. diameter, and the other $1\frac{1}{4}$-in. diameter. The tests were made both for bearing against the ends of the fibres and across the fibres. The details of the test pieces are shown in Fig. 23, while the stress deformation curves are shown in Fig. 24.

The results are not determinate. While the stress deformation curves for the two sizes of bolts bearing across the fibres show the same value (approximately) for the elastic limit, there is quite a variation in the curves for end bearing, although the ultimate strengths are nearly the same. The blocks with the $1\frac{1}{4}$-in. pins were much better specimens, both in respect to quality of timber and grade of workmanship than those with the $\frac{3}{4}$-in. pins, as the end cuts of the latter were not true. However, all uneven blocks were shimmed, and it is not probable that the difference in strength was due altogether to either quality of timber or workmanship.

It is probable that the diameter of pin affects the results, and that the formulæ developed would hold more closely for pins of a large diameter, since in this case the effect of the alternate rings of spring and summer wood would not be so marked.

The elastic limit for bearing across the grain is seen to be approximately 1500 lb. per sq. in., while for bearing against the ends of the fibres, the ultimate strength is approximately 4000 lb. per sq. in. in the case of the $\frac{3}{4}$-in. pin, and 6000 lb. per sq. in. in the case of the $1\frac{1}{4}$-in. pin, or an average of 5000 lb. per sq. in. The tests were not sufficient in number, nor were the blocks made carefully enough to use the results as working data. They do indicate, however, that the pressure of a circular metal pin in a timber block is a function of the relative

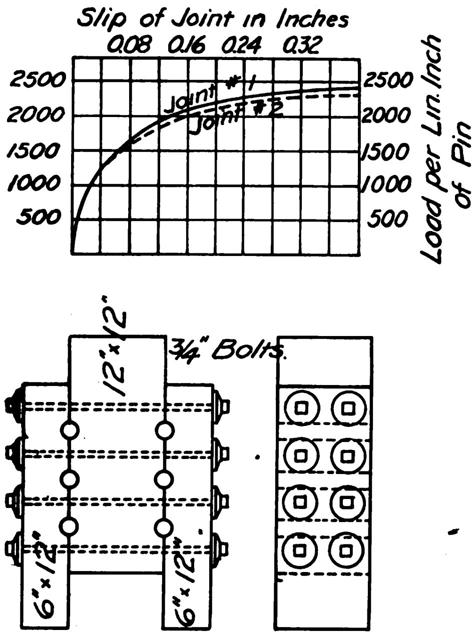


FIG. 25. CURVE FOR AUSTRALIAN IRON BARK AND TEST JOINT FRAMED FOR 2-IN. PINS. BOLTS AND PINS, 6-IN. CENTRES.

resistances of the timber for compression against the ends of the fibres and across the fibres.

The validity of the assumptions on which the formulae for average unit bearing-pressures on pins are based may be questioned. While it is not contended that the pressure distribution on a circular pin is accurately known, the formulae have a rational basis, and give results that are in approximate accord with such tests as have been described, and others which will be discussed in a subsequent chapter. The factors entering into any theoretical solution of the question are many and complex, and absolute working values may be assured only through further tests.

Joints Framed With Shear Pins. Fig. 25 shows a detail of a joint in which circular pins of metal or hardwood are used to transmit stress. The detail is one which was used extensively in the framing of Exposition buildings, for splicing tension members of trusses, fastening bolsters on columns to receive the ends of beams, girders, knee-braces or trusses. The resistance of such a joint involves several elements, namely, strength of the pins in shear, resistance of the pins to distortion, bearing resistance of the timber, both against the ends of the fibres and across the grain, and the strength of the bolts in bending, shear, and tension.

The method of framing the connection, when used as a tension splice is as follows. Two splice-pads are spiked on the sides of the main timber. The bolt holes are bored, the bolts are driven and the nuts tightened. The holes for the pins are then bored, and the pins driven. A close fit for all pins is thus assured with a minimum amount of labor. The joint has certain obvious advantages over some of the splice-joints which are in general use in timber framing. It does not depend in its action upon difficult and expensive cuts and daps of the timber, and is much cheaper than a joint composed of steel fish-plates with attached lugs.

A number of tests of such joints were made in 1913, and described in detail in *Engineering News,* Vol. 71, No. 12, and Vol. 72, No. 9. The materials used were oak, common gas pipe, full weight steel pipe, $1\frac{1}{2}$-in. extra heavy pipe, solid steel, Hawaiian Ohia, Australian hickory, and Australian ironbark. All pins were 2 in. diam. The ultimate strength of the joints expressed in pounds per linear inch of pin ranged from 1950 lb. for the gas pipe pins to 2800 lb. for the Ohia pins. The slip of the joints at these loads was about $\frac{3}{8}$ in. The average load at the apparent yield point was approximately 1800 lb. per linear inch of pin, with a slip of 0.03 in. From the results of the tests, it may be stated that joints of this kind may be framed using 2-in. pins of extra heavy steel pipe, solid steel pins, pins of Hawaiian Ohia and of Australian iron bark, with working loads of 800 lb. per linear inch of pin. Sufficient bolts must be provided to take a total stress of one-half the load on the joint.

Oak and gas-pipe pins are practically worthless. The disadvantage of this joint is the effect of shrinkage, which, if such occurs, may allow a slip. For this reason, this type of joint is best suited to seasoned timbers. Metal pins are to be preferred to hardwood pins, unless the latter are thoroughly seasoned. Furthermore, the construction is not suited to very thick joints, since the cross-shrinkage of the wood will allow the timbers to spread and loosen the pins.

CHAPTER V

Spiked, Screwed, and Bolted Joints

The strength of spiked joints in light timber framing, and of bolted joints in heavy timber framing is, perhaps, the most vital subject in timber design, yet it is a field in which but few tests have been recorded. This statement is particularly worthy of remark when applied to the resistance of bolts to lateral forces, producing bending and shear in the bolts and compression on the timber. Numerous tests have been made on the resistance of spikes and screws to withdrawal from timber, but these will not be discussed in the present article. Working values for such cases will be given in the specifications of the concluding article. For detailed information on the results of the tests that have been made, the reader is referred to the texts of Jacoby, Howe, and others.

Lateral Resistance of Spikes and Nails. The experiments on the lateral resistance of spikes and nails are in three sets, as far as can be determined.

The first series of tests was made by F. B. Walker and C. H. Cross in 1897 at the University of Minnesota. The results are published in the Journal of the Association of Engineering Societies, Vol. 19, December 1897.

The second series of tests was made by H. D. Darrow and D. W. Buchanan at Purdue University in 1898-1899, and the results are published in the Proceedings of the Indiana Engineering Society of 1900.

An abstract and review of both these sets of tests is given in Jacoby's 'Structural Details.'

The third series of tests was made in 1907 by C. K. Morgan and F. Marish, at the Iowa State College. These tests are described by M. I. Evinger in Bulletin No. 2, Vol. IV, of the Engineering Experiment Station of the Iowa State College, entitled 'Holding Power of Nails in Single Shear.'

Tables III, IV, and V show the results of these tests.

TABLE III—*Walker and Cross*

STRENGTH OF WIRE NAILS AT THE ELASTIC LIMIT OF JOINT FOR WHITE AND NORWAY PINE

Size of Nail	Strength of Nail, Lb.	Size of Nail	Strength of Nail, Lb.
6*D*	55	30*D*	226
8*D*	88	40*D*	275
10*D*	112	50*D*	342
16*D*	112	60*D*	362
20*D*	218	80*D*	500

In the experiments of Walker and Cross, the timber used was white pine, Norway pine, and oak, with average compressive strengths of 4840, 5820, and 6600 lb. per sq. in., respectively. The timber cleats in the tests were surfaced. This surfacing must have influenced the strength of the joints at the lower loads, and possibly at loads up to the elastic limit, as there is considerable friction in a tightly spiked joint with rough timbers.

TABLE IV—*Darrow and Buchanan*

STRENGTH OF WIRE NAILS AT ULTIMATE FAILURE OF JOINT FOR YELLOW PINE AND OAK

Kind of Nail	Size	Yellow Pine		Oak	
		Wire	Cut	Wire	Cut
Common	2*D*	...	130	...	160
"	3*D*	...	180	184	289
"	4*D*	198	213	211	344
"	6*D*	240	317	314	429
"	8*D*	361	427	454	573
"	10*D*	724	932	762	822
"	16*D*	855	1079	891	1066
"	20*D*	930	1112	1350	1631
"	40*D*	1450	1360	1745	1874
"	60*D*	2000	1860	1770	....
Finish	4*D*	106	163	186	262
"	6*D*	216	209	299	273
"	8*D*	264	282	359	405
"	10D	537	451	583	....
"	12*D*	498	...	637	....
Fence	8*D*	690	686	709	780
"	10*D*	855	912	1030	1092
Fine	3*D*	121	...	164	....

The timber of the tests of Darrow and Buchanan was well seasoned yellow pine and oak, and the average compressive strengths are given as 7000 and 10,200 lb. per sq. in. for the two sticks of yellow pine used and 5300 lb. per sq. in. for the oak.

The strength of the timber used in the tests of Morgan and Marish, white pine, yellow pine, spruce, fir, and oak, is not given, but the statement is made that the lumber was obtained from the stock piles in a local lumber yard, and that it was not thoroughly seasoned at the time the experiments were started.

At first inspection, the results of the different tests appear to differ by so great an amount that any deductions are worthless. However, when all the factors that enter into the tests are considered, the variation in the strength of the different joints can be largely explained. The ultimate strengths of the blocks of timber in the three sets of tests varied considerably. The method of measuring the slip of the joints was rough in all cases. In at least two of the series of tests, the slip of the joint was measured on one side of the joint alone. This method can never give the true values of the deformations, and erratic results are to be expected. My experience in testing timber joints has been that if measurements are made carefully, the platted load-deformation curves will be remarkably smooth. Also, in order to obtain a curve that will express fairly accurately the relation of load to deformation, the slips of the joints must be measured closer than $\frac{1}{16}$ inch.

For purposes of comparison of the results of three sets of tests Table VI has been prepared, which gives the resistance of the various sizes of nails at the elastic limit of the joints. To bring all experiments to the same basis, the values of Walker and Cross, which are for white and Norway pine, have been increased by the factor 1.45. This factor represents the ratio of the average or effective unit bearing pressures of yellow pine to white and Norway pine at the elastic limit for the case of a round metal pin bearing against the

TABLE V—*Morgan and Marish*

STRENGTH OF WIRE NAILS AT ULTIMATE FAILURE OF JOINTS FOR WHITE PINE, YELLOW PINE, SPRUCE, FIR, AND OAK

Size of Nails	Number of Nails Tested	Length of Nails in Inches	Gauge of Wire	Average Strength in Pounds per Nail for the Total Number of Nails Tested: White Pine	Yellow Pine	Spruce	Fir	Oak
8*D*	20	2½	10¼	230	294	187	478	656
10*D*	20	3	9	235	324	237	475	608
12*D*	8	3¼	9	318	625	332	360	665
16*D*	20	3½	8	299	494	263	456	750
20*D*	20	4	6	372	746	386	632	992
30*D*	20	4½	5	540	1221	629	832	1392
40*D*	20	5	4	656	1067	555	837	1580
50*D*	12	5½	3	712	1425	563	985	1925
60*D*	12	6	2	826	1783	794	...	2125
BARBED WIRE NAILS								
8*D*	10	2½	10¼	...	425	...	307	...
10*D*	10	3	9	...	460	...	318	...

ends of the fibres in a close fitting hole in accordance with the formula developed in the preceding chapter. Similarly, the values of the tests of Darrow and Buchanan, which are for the ultimate strength of the joints, have been reduced by the factor 0.41, which represents the ratio of the strength of the joints at the elastic limit to that at the ultimate strength multiplied by the ratio of the average unit bearing pressures at the elastic limit for the timbers of the two tests.

The values in the fourth column were selected as the average values for a slip of joint of $\frac{1}{16}$ in., and were taken from the curves in the bulletin of the tests. In all three sets of experiments, it was found that the elastic limit of the joints corresponded to a slip of the joints of approximately $\frac{1}{16}$ inch.

A. W. Muenster, in the Journal of the Association of Engineering Societies, discussing the tests of Walker and Cross, proposed expressing the shearing resistance of a wire nail at the elastic limit of the joint by the formula Cd^2, C being a coefficient dependent on the timber, and d being the diameter of the nail. For white and Norway pine, C would be approximately 5500. Further he proposed to take the working values at about 60% of the strength at the elastic limit, or a coefficient of 3300.

TABLE VI

LATERAL STRENGTH OF WIRE NAILS AT ELASTIC LIMIT OF JOINT IN YELLOW PINE

Nail	Strength in Pounds		
Size of	Walker and Cross	Darrow and Buchanan	Morgan and Marish
8*D*	128	148	120
10*D*	163	296	175
12*D*	...	...	250
16*D*	163	350	260
20*D*	316	382	375
30*D*	328	...	540
40*D*	400	595	480
50*D*	495	...	630
60*D*	525	764	775
80*D*	725	...	...

W. K. Hatt, in presenting the results of the experiments of Darrow and Buchanan to the Indiana Engineering Society, recommended that the safe working values of a wire nail in yellow pine be expressed by the formula:

$S = 9.5\ D$, where D is the 'penny' weight of the nail.

Table VII gives the working values for the strength of wire nails, (1) by Muenster's rule, but using a coefficient, $C = 4000$, and (2) by the rules $S = 8\ D$. The value of $C = 4000$ is obtained by applying a safety factor of two to the value $C = 5500$, and multiplying the result by 1.45, as explained previously, to bring the coefficient to the basis of yellow pine.

Mr. Hatt states that the rule which he proposes should not apply to nails heavier than 20*D*.

From consideration of all the tests, I recommend using for the case of nails in Douglas fir the formula $S = 4000\ d^2$, or for nails up to and including 20*D* nails the values of the third column of the preceding table, which represent $S = 8D$.

Since the preceding discussion of nailed joints was written, there has been published a partial report of some tests on the resistance to lateral forces of several

TABLE VII

SAFE WORKING VALUES FOR THE LATERAL STRENGTH OF WIRE NAILS IN YELLOW PINE

Comparison of Results by Methods of Muenster and Hatt, with Modified Coefficients

Size of Nail	$S = 4000\ d^2$	$S = 8D$
6*D*	53 lb.	48 lb.
8*D*	62 "	64 "
10*D*	88 "	80 "
12*D*	88 "	96 "
16*D*	110 "	128 "
20*D*	165 "	160 "
30*D*	194 "	240 "
40*D*	226 "	320 "
50*D*	268 "	400 "
60*D*	322 "	480 "
80*D*	364 "	640 "

sizes of common wire nails, made at the Forest Service Laboratory, Madison, Wisconsin.*

The timber used was thoroughly air-dry (average moisture 13.8%) long-leaf yellow pine. A deflectometer was used to measure the slip of the joints. For this reason, I feel that this set of tests should be given especial attention, and the complete report, when published, carefully studied. In framing the test-joints, $\frac{3}{16}$-in. holes were bored in the cleats for each nail, but the blocks were not bored. No nails were placed in checks, knots, or other defects. Table VIII gives the results of the tests. In the fifth column, I have inserted the value of C in the formula $S = Cd^2$. The average value of C is seen to be about 7000. Thus the working values for the nails of Table VII are but slightly over one-half the elastic limit as found by these tests.

Mr. Wilson makes the following comments on the tests:

1. The elastic limit is well defined, and at a comparatively small deformation.

2. No definite relation between size of nails and deformation at the elastic limit is apparent.

3. It seems probable that the load per nail is independent of the number of nails in the joint.

4. In efficiency per unit-weight, the smaller nails in general seem to have the advantage.

Until the complete set of tests has been made and published, it is unwise to make definite comments. The slip at the elastic limit is small, or, expressed in another manner, the elastc limit of the nail occurs at a low deformation, in fact, at a deformation of only about one-half that found by the other investigators quoted. It is possible that the same point on the stress-deformation curve has not been taken by the various investigators.

This discussion of the strength of nailed joints is not complete without mention of the tests of J. C. Stevens, consulting engineer, of Portland, Oregon, noted in the

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*'Tests Made to Determine Lateral Resistance of Wire Nails,' by Thomas R. C. Wilson, Engineer in Forest Products, Forest Service, *Engineering Record*, Vol. 75, No. 8, February 24, 1917.

Engineering Record of January 13, 1917, Vol. 75, No. 2. The tests are only briefly described. Ordinary 2 by 12-in. yellow-pine planks were nailed to fir sills and then sheared off. From the results of these tests a working load of 210 lb. for a 16D nail, and 250 lb. for a 20D nail were used in the heel-plates of wooden flumes. In the light of other tests these working values are high, but within the elastic limit.

The additional matter just given leads me to reaffirm my recommendation that the working strength of wire nails in lateral shear, when used with Douglas fir, be taken in accordance with the formula $S = 4000\ D^2$.

Common Wood-Screws. The strength of ordinary wood-screws in single shear was investigated as thesis work in Cornell University by Andrew Kolberk and Milton Birnbaum and discussed by them in the *Cornell Civil Engineer,* Vol. 22, No. 2, November 1913. "The screws were the ordinary cut, flat-head screws, made by the American Screw Co. of Providence, Rhode Island." The timber employed in the joints was cypress, yellow

TABLE VIII

LATERAL RESISTANCE OF NAILS IN AIR-DRY LONG-LEAF PINE

Size of nail	Load in lb. per nail: At elastic limit	Load in lb. per nail: At ultimate strength	Slip in inches: At elastic limit	Slip in inches: At ultimate strength	Test value of C at elastic limit in formula $S = Cd^2$
*30D	355	779	0.021	0.54	7340
*40D	394	845	0.026	0.58	6950
*50D	450	1261	0.019	0.69	6710
*60D	422	1144	0.018	0.70	5240
†30D	333	783	0.034	0.88	6880
†40D	389	1125	0.028	1.08	6860
†50D	544	1615	0.036	1.18	8120
†60D	589	1644	0.039	1.32	7300

*Timbers with grain parallel, load parallel to grain.

†Timbers with grain at right angles, load parallel to piece receiving points of nails.

In the first series each value is based on four tests of 3-nail and four tests of 6-nail joints. In the second series each value is based on two sets of 3-nail and two tests of 6-nail joints. The average value of C at the elastic limit for all tests is 6920.

pine, and red oak. The average strength in end bearing of the three timbers was found to be 4980, 7580, and 8440 lb. per sq. in., respectively. The thickness of the timber cleats varied with the length of screw used, but the test piece was always arranged to make the screws act in single shear. It was found by experiment that screws could not be driven closer than $2\frac{1}{2}$ in. from the edge perpendicular to the direction of fibres without danger of splitting the wood. It was also often found impossible to drive the screws without previously boring holes, and in such cases the size of hole was made equal to the diameter of the screw at the root of thread. In oak, and in the case of the large screws in yellow pine, separate holes had to be bored for the shank and for the threaded portion of the screw. A hole was also bored for the head of the screw, thus bringing it flush with the surface of the wood. Every part of the screw was thus brought into action. The procedure in testing was to measure the force at each $\frac{1}{32}$-in. slip up to a maximum slip of $\frac{3}{16}$ in. As this slip is more than would be allowed in practice, it was not thought necessary to carry the tests to the ultimate capacity of the joints.

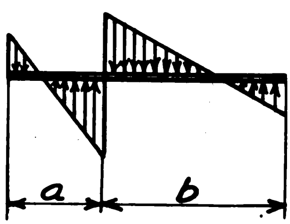

FIG. 26. FORCES ACTING ON SCREW IN JOINT.

a = thickness of side piece.
b = depth of penetration into centre piece.

Fig. 26 shows in a general way the forces acting on a screw in a joint. The screw in such a joint was but slightly deformed in cypress, the wood being so soft as to crush readily without bending the screw. In yellow pine and oak, however, the screws were bent in the characteristic reverse curve typical of nails and screws.

Curves in which the load per screw in pounds was platted against the slip in inches showed the following results: In cypress, while a joint having a thinner side-piece might be stronger than one having a thicker side-piece during the first increments of slip, it did not continue to be stronger during the last few increments of slip. In yellow pine and oak, however, the joint with the thinner side-piece, once being the stronger, continued so during the entire test.

Investigation relative to the determination of the proper proportion of the length of screw to the thickness of side-piece, showed that in general, the joint with the thin side-piece was the strongest. A ¾-in. side-piece and 2½-in. screw gives a ratio of 0.3 between side-piece and

TABLE IX

LATERAL STRENGTH OF SCREWS IN YELLOW PINE AND OAK CORRESPONDING TO A SLIP OF JOINT OF 1/32 INCH

Yellow Pine

Length of Screw, In.	Gauge of Screw	Thickness of Side Piece	Load in Lb. 1/32-in. slip	Load in Lb. 1/16-in. slip
3	20	1½	758	1002
		1	924	1094
3	16	1½	477	657
		1	557	750
3	12	1½	437	585
		1	586	696
2½	20	1½	590	810
		1	770	936
		¾	780	960
2½	16	1½	453	580
		1	563	702
		¾	526	631
2½	12	1½	414	530
		1	520	622
		¾	508	610
2	20	1	606	797
		¾	632	802

Length of Screw, in.	Gauge of Screw	Thickness of Side Piece	Load in Lb. $\frac{1}{32}$-in. slip	Load in Lb. $\frac{1}{16}$-in. slip
2	18	1	472	643
		¾	458	625
2	16	1	382	531
		¾	410	601
2	12	1	322	514
		¾	405	573
1½	18	1	346	432
		¾	460	537
		⅝	451	501
1½	16	1	320	365
		¾	394	504
		⅝	417	509
1½	12	1	317	360
		¾	394	473
		⅝	404	488
		Red Oak		
3	24	1½	707	1144
		1	933	1308
3	16	1½	543	754
		1	710	932
2½	24	1½	580	842
		1	750	1154
2½	16	1½	442	634
		1	557	809
		¾	627	874
2	20	¾	700	912
2	16	1	513	681
		¾	600	767
2	12	1	456	639
		¾	529	632
1½	18	1	514	601
		¾	603	802
1½	12	1	383	416
		¾	523	590

screw; a 1-in. side-piece and $2\frac{1}{2}$-in. screw has a ratio of 0.4. For a 2-in. screw, the $\frac{3}{4}$-in. side-piece gave the strongest joint, especially with the screws of the smaller gauges. This is a proportion of 0.375. For the $1\frac{1}{2}$-in. screw, the strongest joint was that with a side-piece $\frac{5}{8}$-in. thick, or a proportion of 0.4. Thus it would seem that a side piece of about 0.4 of the length of the screw will give the strongest joint. Or, conversely, to obtain the strongest joint, the screws should have a length of approximately $2\frac{1}{2}$ times the thickness of side-pieces. In the joints with yellow pine and oak timber, it was found that the strength of the joint varied as the square root of the penetration of the screws into the centre timber, while for the cypress joints the proportion varied as the cube root of the penetration into the centre timber.

Table IX gives the strength of the screws at a slip of joint of $\frac{1}{32}$ and $\frac{1}{16}$ in. for the joints framed with yellow pine and oak. This table has been condensed from the results published in the *Cornell Civil Engineer,* the values for slips above $\frac{1}{16}$ in. being omitted for the reason that this slip has been found to represent the elastic limit in the case of nailed joints, and the table may thus be compared with those of the nailed joints.

"In the case of yellow pine it was found that the strength of the joint varied with its weight, or specific gravity. The heavier joints invariably gave the larger results. In order to reduce all the values to a common standard, the weight of a joint of average specific gravity was computed for each size of side-piece. The results were compared with each other and corresponding differences in strength and weight noted. From the averages of these values the mean difference in weight of joint was found. Joints were reduced in this manner to the strength corresponding to the weight of a joint of average specific gravity. A difference of 0.1 lb. in the weight of a joint was found to make a difference of 10 lb. at the $\frac{1}{32}$-in. slip and 30 lb. at the $\frac{3}{16}$-in. slip.

Table X gives the average lateral resistance per screw for the yellow pine and oak joints at the assumed

TABLE X

AVERAGE LATERAL RESISTANCE OF SCREWS AT A SLIP OF JOINT OF $\frac{1}{32}$ AND $\frac{1}{16}$ IN. FOR YELLOW PINE AND OAK.

Gauge of Screw	Diameter of Screw d	d^2	Average Load Corresponding to a Given Slip of Joint of $\frac{1}{32}$ in.-Lb. Yellow Pine $\frac{1}{32}$-in. Slip	Yellow Pine $\frac{1}{16}$-in. Slip	Oak $\frac{1}{32}$-in. Slip	Oak $\frac{1}{16}$-in. Slip	Value for $S=8750d^2$	Value for $S=45G$
12	0.2157	0.0464	430	545	475	570	406	540
16	0.2684	0.0720	467	583	570	780	630	720
18	0.2947	0.0868	460	550	558	701	760	810
20	0.3210	0.1030	723	915	700	912	900	900
24	0.3738	0.1398	...	...	700	1112	1220	1080

elastic limit or for a slip of joint of $\frac{1}{16}$ in. The table also shows the relation of these loads to values obtained by multiplying the square of the diameter of the screw by an arbitrary factor of 8750, and also shows the relation of the loads to the values obtained by multiplying the gauge of the screw by an arbitrary factor of 45. With the exception of the No. 18 gauge screws, the table shows that the arbitrary formula, $S =$

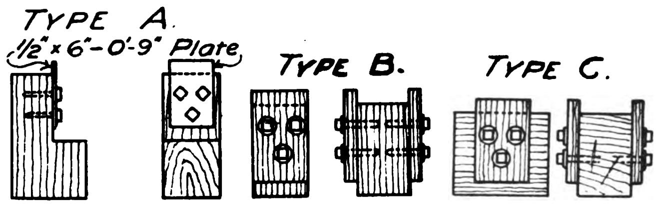

FIG. 27. TYPES OF LAG-SCREWED JOINTS.

Type *A* is an 8 by 10-in. or 12-in. timber, 1 ft. 5½ in. long.

Type *B* is composed of two 1¼ to 2 in. by 8 in. pieces, 1 ft. 1 in. long, and an 8 by 8-in. timber, 1 ft. 1 in. long.

Type *C* is composed of two 1⅝ to 2 in. by 8-in. pieces, 1 ft. long, and an 8 by 12-in. timber, 1 ft. 2 in. long.

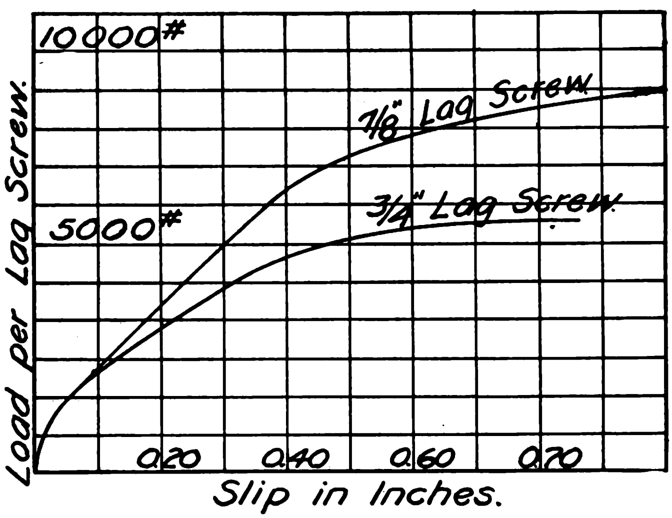

FIG. 28. LOAD CURVES FOR LAG-SCREWED JOINTS WITH STEEL PLATES. CURVES SHOW AVERAGE FOR ONE LAG SCREW.

8750 d^2, where $S =$ the resistance of the screw at a slip of joint of $\frac{1}{16}$ in., and $d =$ the diameter of the screw, holds fairly well, and may be adopted for determining working loads. The difference between the actual resistances as shown for yellow pine and oak is small, and in conformity with the relative properties of the two timbers.

For determining the diameters of the screws, the standard rule

$d = 0.0578 + 0.01316\ G$

was used, where $d =$ the diameter of the screw, and $G =$ the gauge of the screw.

The working loads for Douglas fir may be taken as shown in Table XI, whose values have been computed from the formula

$$S = 4375\ d^2$$

TABLE XI

SAFE LATERAL RESISTANCE OF COMMON WOOD-SCREWS WITH DOUGLAS FIR

Gauge of Screw	Safe Lateral Resistance	Gauge of Screw	Safe Lateral Resistance
12	205 lb.	20	450 "
14	256 "	22	529 "
16	315 "	24	615 "
18	380 "		

Lateral Resistance of Lag Screws. The action of a lag screw when subjected to lateral shear and flexure is similar to that of a nail, but more like that of a common wood-screw. Since the diameter of a lag screw in proportion to its length is considerably greater than that of a nail, the bending is less marked, and the resistance is dependent almost wholly upon the bearing strength of the timber.

The only data on the resistance of lag screws is given in Kidder's 'Architects and Builders Pocket Book,' and in Thayer's 'Structural Design.' In the latter volume, the lag screw is treated as if it were a bolt, when computing its resistance. Kidder shows a detail of the end joint of a scissors truss, in which a thin steel plate is lagged to the truss chord. When used in this manner with Douglas fir the value of a $\frac{3}{4}$ by $4\frac{1}{2}$-in. lag screw with a steel plate of $\frac{5}{16}$ in. minimum thickness is given at 2100 lb., similarly, the value of a $\frac{7}{8}$ by 5 in. lag screw is given at 2800 lb. There is no statement made as to the basis on which these values are selected. Thayer's figures for the same conditions are 1125 lb. and 1575 lb., respectively.

In an article, already mentioned,* I published the results of some tests on lag-screwed joints made at the University of California in 1913. In these tests, $\frac{3}{4}$ by $4\frac{1}{2}$ in. lag screws were used to fasten 2 in. planking to 12 in. blocks of timber, with the screws bearing against the fibres of the planking, and across the fibres of the blocks. The results are given in Table XII. The test joints were made by lagging two 2 by 8-in. planks to the sides of a 12 by 12-in. timber block.

The failure in joints *a, c, e,* and *f* was due to the screws splitting the side of the main timber. In joints *b* and *d,* the attached piece was sheared by the screws.

No detail measurement of slip was made, but it was noted that the first large movement appreciable to the eye occurred at a load of approximately 2400 lb., and was between $\frac{1}{16}$ and $\frac{1}{8}$ inch.

More recently, I made a series of tests on lag-screwed and bolted joints at the Panama-Pacific International

TABLE XII

1913 TESTS OF LAG-SCREWED JOINTS

Joint	First Slip Total Load, Lb.	First Slip Load per Screw, Lb.	Ultimate Load Total Lb.	Ultimate Load Per Screw, Lb.
a(6 screws)	11,000	1,833	23,000	3,833
b(6 screws)	16,000	2,667	30,000	5,000
c(8 screws)	21,000	2,625	30,000	3,750
d(6 screws)	16,000	2,667	31,000	5,167
e(8 screws)	20,000	2,500	30,000	3,750
f(8 screws)	18,000	2,250	29,000	3,625
Average value of one screw		2,423		4,187

**Engineering News,* Vol. 71, No. 13.

TABLE XIII

1915 TESTS OF LAG-SCREWED JOINTS

Type	Number and Size of Lag Screws	Thickness of Side Pieces	Thickness of Centre Pieces	Load at Apparent Yield Point, Lb.	Load at Slip of $\frac{1}{16}$ in., Lb.	Final Load, Lb.	Final Slip, In.	Remarks
A	Three $\frac{3}{4}$-in.	$\frac{1}{2}$-in. steel plate		6,500	7,700	20,000	0.92	Autograph record
A	Three $\frac{3}{4}$-in.	$\frac{1}{2}$-in. steel plate		5,500	6,700	19,700	0.75	
A	Three $\frac{7}{8}$-in.	$\frac{1}{2}$-in. steel plate		5,000	6,700	30,000	1.14	
A	Three $\frac{7}{8}$-in.	$\frac{1}{2}$-in. steel plate		8,000	7,500	30,000	1.15	
B	Six $\frac{7}{8}$-in.	2-in.	8-in.	29,000	12,500	49,100	1.40	Ultimate load
B	Six $\frac{7}{8}$-in.	2-in.	8-in.	22,000	13,800	44,700	1.44	Ultimate load
B	Six $\frac{7}{8}$-in.	2-in.	8-in.	25,000	10,700	44,400	1.69	
B	Six $\frac{3}{4}$-in.	$1\frac{3}{8}$-in.	8-in.	16,000	9,400	28,885	1.30	
B	Six $\frac{3}{4}$-in.	$1\frac{1}{2}$-in.	8-in.	21,000	12,000	33,400	1.14	
B	Six $\frac{3}{4}$-in.	$1\frac{1}{4}$-in.	8-in.	20,000	12,400	34,000	1.49	Ultimate load
C	Six $\frac{7}{8}$-in.	2-in.	8-in.	21,000	14,400	34,000	1.61	
C	Six $\frac{7}{8}$-in.	2-in.	8-in.	20,000	10,400	33,000	1.70	
C	Six $\frac{3}{4}$-in.	$1\frac{3}{8}$-in.	8-in.	.....	8,500	20,000	0.54	Load removed
C	Six $\frac{3}{4}$-in.	2-in.	8-in.	.....	10,500	14,000	0.13	Load removed

Exposition, through the courtesy of the Tinius Olsen Testing Machine Co. of Philadelphia. These tests have been described in a recent article.* In these tests, careful measurements of the slips of the joints were made, corresponding to constant increments of load, in addition to making autographic records of the load-deformation curves through the medium of the autograph attachment on the machine.

The lag-screwed joints consisted of four joints in which a ½-in. steel plate was fastened to a timber block, and ten joints in which wooden plates varying from 1¼ to 2 in. thick were lagged to 8 by 8-in. and 8 by 12-in. blocks. The detail makeup of these joints is shown in Fig. 27, *a*, *b*, and *c*. Fig. 28 shows the curves for the joints with steel plates, while Fig. 29 shows the typical curves for the all-timber joints. The results of the tests are given in Table XIII.

Reference to Fig. 28 indicates that the working values for lag screws given by Kidder correspond to slips of joint of 0.08 in. for the ¾ by 4½-in. screws and 0.12 in. for the ⅞ by 5-in. screws.

With respect to the safe working values to be adopted, there may be some difference of opinion. In the case of the joints with steel plates, the first break in the load-slip curve occurs at a total load on the joint of about 6000 lb. With the all-timber joints, there is also a slight break in the curves at about 8000 lb., but a marked break at a total load on the joint of approximately 23,000 lb. for the ⅞-in. screws and 18,000 lb. for the ¾-in. screws. It was found for the steel-plate joints that when the load was removed at about 8000 lb, the joint did not fully recover its slip, although the remaining set was probably due to the initial adjustment of the joint.

In conformity with the tests on nailed joints, it is recommended that the working values for lag screws be taken at one-half the loads producing a slip of $\frac{1}{16}$ in. The safe lateral resistances would then be as follows:

**Engineering News*, July 20, 1916.

	Lb. per Screw
Metal plate lagged to timber, ¾ by 4½-in. lag screw	1030
Metal plate lagged to timber, ⅞ by 5-in. lag screw	1200
Timber planking lagged to timber, ¾ by 4½-in. lag screw	900
Timber planking lagged to timber, ⅞ by 5-in. lag screw	1050

There appeared to be no reduction in stiffness for those joints in which the lag screws bore across the fibres of the timber in the centre block during the first portion of the tests.

In the Proceedings of the American Railway and Maintenance of Way Association, Vol. 10, 1909, there is published a method for determining the safe lateral re-

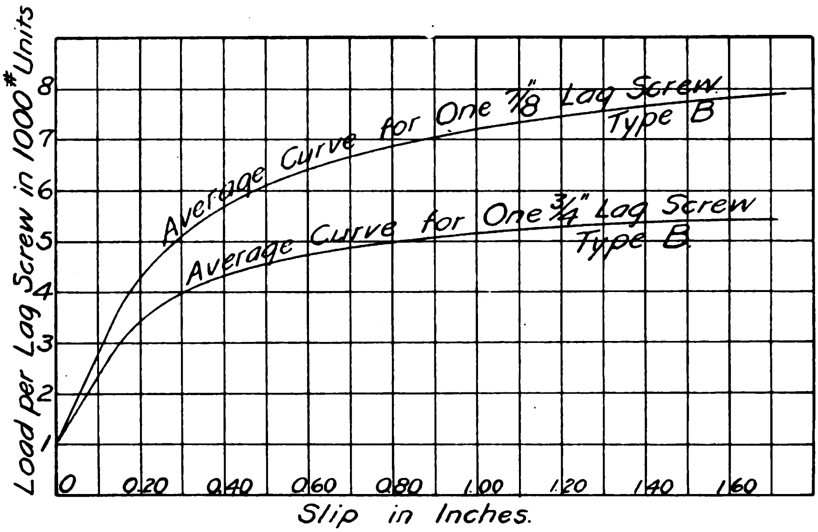

FIG. 29. LOAD CURVES FOR LAG-SCREWS IN ALL-TIMBER JOINTS.

sistance of a track spike. The discussion is also given in Jacoby's 'Structural Details.' Referring to Fig. 30.

$$P = \frac{pbL^2}{4L + 6e} \text{ where}$$

P = safe lateral resistance of the spike.

p = maximum safe unit bearing stress of the timber on the spike.

b = diameter or side of the spike.

L and e, as shown in the figure.

Applying this formula to the joints of the tests having a steel plate lagged to the timber, assuming the diameter of the lag screw to be constant throughout its length and equal to its nominal diameter and using a limiting bearing-stress of 1300 lb. per sq. in. on the timber, there is found for the case of the $\frac{3}{4}$-in. lag screws with a $\frac{1}{2}$-in. plate, a safe lateral resistance of 770 lb. per lag screw. Similarly for a $\frac{7}{8}$ by 5-in. lag screw, the safe lateral resistance is found to be 1040 lb. The maximum flexural stress in the lag screw is 10,770 lb. per sq. in. for the $\frac{3}{4}$-in. lag screw and 13,400 lb. per sq. in. for the $\frac{7}{8}$-in. lag screw. These values do not differ greatly from those of the tests.

In Table XIV, the recommended values of Kidder and Thayer, the values as derived from the formula just given and those recommended by this article are given.

Lateral Resistance of Bolts. The values for the strength of bolted joints as computed by the methods and formulae given by the various text and hand books differ widely. Probably in no other instance in structural engineering is there a greater discrepancy between the results of the methods of design of the various authorities than in the case of the design of a bolted joint. Again, although this connection is one used constantly in ordinary construction, both temporary and permanent, apparently no tests have ever been made on the actual strength of bolted timber-joints from which working values might be selected with the assurance of reasonable accuracy. In fact, the only tests which are on record, as

TABLE XIV

COMPARISON OF SAFE LATERAL RESISTANCE OF LAG SCREWS WHEN USED TO FASTEN A METAL PLATE TO TIMBER AS GIVEN BY VARIOUS AUTHORITIES.

Authority	$\frac{3}{4}$ by $4\frac{1}{2}$-in. Lag Screw, Lb.	$\frac{7}{8}$ by 5-in. Lag Screw, Lb.
Kidder	2100	2800
Thayer	1125	1575
Theoretical formula	770	1040
Tests in present chapter	1030	1200

far as I have been able to determine, are those of E. E. Adams, first published in the California Journal of Technology of the University of California in 1904, and later discussed by me.*

The present discussion will review the common methods of designing bolted joints, and compare the results with those of the tests on 24 bolted joints which I made in 1915 in conjunction with the tests on lag-screwed joints previously described. For a detailed description of these tests, the reader is referred to *Engineering News* for July 20, 1916.

Current Methods of Design. Fig. 31 represents a splice in the tension chord of a truss. The thickness of

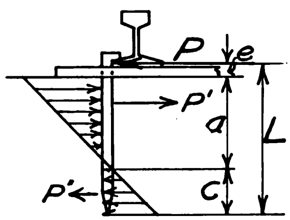

FIG. 30. DIAGRAM OF STRESS ON TRACK-SPIKE.

the chord is $2L$, while the thickness of either splice is L. The total stress in the chord is P, of which it is assumed that each splice-pad takes half.

Kidder in his 'Architects and Builders Pocket Book' states with regard to the design of bolted timber-joints, "When the pieces joined together are not more than two inches thick, so that they can be tightly drawn together, thereby producing a good deal of resistance from friction, the bolts may be considered as rivets, and proportioned for shearing and bearing only, the bending

*Engineering News, Vol. 71, No. 13. Since writing this chapter, I am informed that Harold A. Thomas of Rose Polytechnic, Terre Haute, Indiana, has made a series of tests on bolted timber joints which are to be published in the technical press.

moment being neglected. When the pieces of wood are more than two inches thick, the bolts should be proportioned for shearing, bearing, and flexure.'' Where bending is considered, Kidder recommends that the bending moment, M, be taken as 1/12 P times the distance between the centres of the splice pads. In Fig. 32, M would

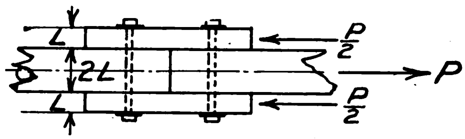

FIG. 31. SPLICE IN TENSION CHORD OF TRUSS.

then be equal to 1/12 P times $3L$ or $\frac{1}{4}$ PL. H. S. Jacoby in his 'Structural Details' shows the bending moment to be equal to $\frac{1}{2}$ PL when the pressure against the bolt is considered to be uniform over its entire length. M. A. Howe in 'A Treatise on Wood Trusses' uses the same formula as Jacoby, or $M=\frac{1}{2}\ PL$. W. J. Douglas in Merriman's 'American Civil Engineers Handbook' disregards the bending in bolts, where the pieces joined are less than three inches thick, otherwise the bolt is considered as a restrained beam, and computed as recommended by Kidder. The working stress used for bending in the bolts in the preceding formulae is 22,500 lb. per sq. in. Horace R. Thayer* in 'Structural Design,' Vol. 1, recognizes the necessity of a method of computation for bolt sizes more consistent with practice. He recommends that the pressure on the bolt be considered as concentrated on the portions of the bolt immediately adjacent to the contact faces of the timbers; the pressure to be considered uniform over the length on which it exists. This length a is to be such, that the resultant bending in the bolt will just equal the flexural or shearing strength of the bolt.

Thus referring to Fig. 32, which shows the assumed distribution of load on the bolt, where

$S=$ maximum allowable flexural unit stress in bolt.

*See also *Engineering News*, Vol. 71, No. 17, p. 923.

B = allowable unit bearing stress against ends of fibres of timbers.

d = diameter of bolt.

Assuming that the shear on the bolt need not be considered, and that the bending on the bolt alone need be considered, $a = d\left(\frac{\pi S}{32B}\right)^{\frac{1}{2}}$

For Douglas fir with iron bolts, Mr. Thayer uses S = 12,000 lb. per sq. in., and B = 1250 lb. per sq. in.

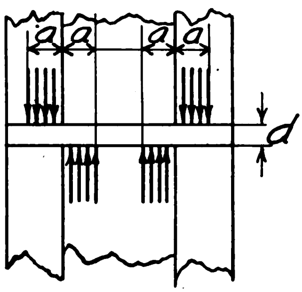

FIG. 32. DIAGRAM OF STRESS ON BOLT.

For steel bolts, S is increased to 16,000 lb. per square inch.

In Table XV is shown the working strength of such joints, as determined by the methods of Jacoby, Howe, Kidder, Merriman, and Thayer, as well as from curves taken from actual tests and from a formula derived by myself. In the case of all these authorities, the lowest and highest values are shown; namely, the values for joints with 2 in. and 2⅝-in. splice-pads and those with 6-in. splice-pads. This table emphasizes the wide variation in the prescribed practice of designing bolts.

There is this radical difference between Thayer's method and the others quoted, namely, using Thayer's analysis the value of L in the joint may be in excess of a without decreasing the strength of the joint. In other words, the joints with a 12-in. centre timber and 6-in. side-timbers will have the same strength as the joints with side-timbers of a width equal to the distance a.

TABLE XV

COMPARISON OF VALUES FOR WORKING STRENGTH OF BOLTED JOINTS, DETERMINED BY VARIOUS METHODS.

Authority.	2 Inch Side Timbers. 4 Inch Centre Timbers				2 7/8 Inch Side Timbers 5 3/4" Inch Centre Timbers.				6 Inch Side Timbers. 12 Inch Centre Timbers.			
	3-5/8 In. Bolts.	3-3/4 In. Bolts.	3-7/8 In. Bolts.	3-1 In. Bolts.	3-5/8 In. Bolts.	3-3/4 In. Bolts	3-7/8 In. Bolts.	3-1 In Bolts.	3-5/8 In. Bolts.	3-3/4 In. Bolts.	3-7/8 In. Bolts.	3-1 In. Bolts.
Merriman (a) (W. J. Douglas.)	9,000	10,800	12,600	14,400	12,937	15,675	18,112	20,200	1,080	1,860	2,960	4,410
Howe. (b)	1,793	3,100	4,920	7,350	1,248	2,160	3,420	5,110	598	1,030	1,640	2,450
Kidder. (c)	10,125	12,144	14,196	16,200	2,590	4,460	7,100	10,600	1,080	1,860	2,960	4,410
Thayer. (d)	3,330	4,800	6,540	8,850	3,330	4,800	6,340	8,850	3,330	4,800	6,540	8,850
Jacoby. (e)	1,620	2,740	4,400	6,615	1,295	2,230	3,555	5,300	540	930	1,480	2,205
Test Curves. (f)	4,250	5,000			4,250	5,000			4,250	5,000	6,750	9,500
Test Curves. (g)	5,500	6,500			5,500	6,500			5,500	6,500	9,000	12,500
Dewell Formula.	(h) 3,171	(h) 4,350	(h) 5,700	(h) 7,380	(h) 3,747	(h) 4,920	(h) 6,255	(h) 7,890	4,388	6,300	8,550	11,115

a- Bearing Pressure = 1200 lb. per Sqr. In., Stress In Bolts = 22,500 lb per Sqr. In.
b- " " 650 " " " " " " " = 25,000 " " " "
c- " " 1350 " " " " " " " = 22500 " " " "
d- " " 1250 " " " " " " " = 16000 " " " "
e- " " 1080 " " " " " " " = 22500 " " " "
f- Values = Half Loads, when Δ = 0.0625 In.
g- " = Loads; when Δ = 0.025 In.
h- " = For M = RPl ~~where~~ R ~~varies from 1/3~~ to 2/27

$$l = d\left(\frac{\pi S 27}{32 B}\right)^{1/2}$$

Values of l, for S = 16,000 lb. Sqr. In.
B = 1,300 " " "

d.	l.	Joint Strength.
5/8 In.	3.6	4,388
3/4 In.	4.3	6,289
7/8 In.	5.0	8,531
1 In.	5.7	11,115

Description of Tests of Bolted Joints. As noted previously, the 1915 series of tests embraced twenty-four bolted joints, the bolts varying in size from $\frac{5}{8}$ to 1 in. diam. Fig. 33a and b shows the details of all the joints tested. The tests were compressive tests. The washers were standard cut-steel pressed washers throughout. Before commencing the tests, all nuts were loosened so that friction would not affect the results. This was to approximate the condition of a joint after shrinkage has taken place. Careful measurements were made of the slip of the joints, in addition to recording the load-deformation curves by means of the autographic attachment of the testing machine.

Table XVI gives a summary of the results of the tests. A careful study of the curves representing the relation of load to slip has led to the conclusion that for the test joints having the same number and diameter of bolts, except for those joints in which the bolts bear across the

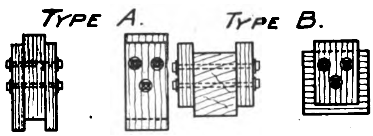

FIG. 33. DETAILS OF JOINTS TESTED IN 1915. SIZES OF TIMBERS AND BOLTS AS LISTED IN TABLE XIV.

fibres of the centre timber, the load-slip or load-deformation curve is practically a constant. In other words, for the limits of the tests, it appears that the strength of a bolted joint depends upon the number and size of the bolts, and is nearly independent of the thickness of the timbers forming the joint. Fig. 34 shows the curves representing the relation between slips and loads for each set of joints having bolts of the same diameter. These curves are drawn as an average of the curves of the individual tests.

For those joints where the bolts have bearing across the fibres of the centre timber, the load-slip curves appear to coincide with those of the end-bearing joints up to a total load of approximately 30,000 lb., or up to loads of 5000 lb. per bolt. For working loads therefore, the resistance of these joints is apparently as great as those having all end-bearing. Fig. 35 shows a photograph of some of the joints which were cut open after testing.

From a study of the condition of the bolts after testing, the following theory of the distribution of loads on the bolts, and their consequent bending is proposed.

Let

S = maximum allowable flexural unit-stress in the bolt.

B = maximum allowable unit bearing-stress against the ends of fibres of timber.

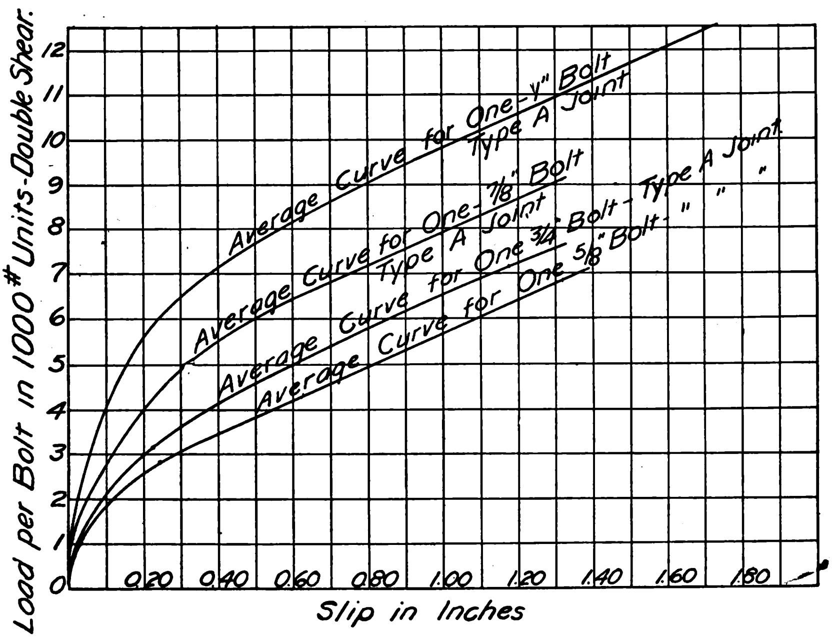

FIG. 34. CURVE OF RELATION BETWEEN LOADS AND SLIP OF JOINTS TESTED.

t = width of splice-pad.

a, b, B^1, P, P_2 as shown = one-half width of main timber.

It will be assumed that the distribution of load on the

TABLE XVI

1915 TESTS OF BOLTED JOINTS

Joint Type	Number and and Size of Bolts	Thickness of Side Pieces	Thickness of Centre Pieces	Load at Apparent Yield Point, Lb.	Load at Slip of $\frac{1}{16}$ in., Lb.	Final Load, Lb.	Final Slip, In.	Remarks
A	Three $\frac{3}{4}$-in.	2-in.	4-in.	.....	*	42,000†	...	
A	Three $\frac{5}{8}$-in.	2-in.	4-in.	16,000	6,500	41,000	1.85	
A	Three $\frac{3}{4}$-in.	2-in.	4-in.	12,500	7,000	45,000	1.95	
A	Three $\frac{3}{4}$-in.	$2\frac{3}{4}$-in.	$5\frac{3}{4}$-in.	11,000	8,000	45,000	1.15	
A	Three $\frac{5}{8}$-in.	2-in.	4-in.	13,000	5,000	43,500	1.68	
A	Three $\frac{3}{4}$-in.	$2\frac{7}{8}$-in.	6-in.	10,000	6,500	36,250	0.64	
A	Three $\frac{3}{4}$-in.	4-in.	8-in.	15,000	6,000	55,000	1.39	
A	Three $\frac{5}{8}$-in.	$3\frac{3}{4}$-in.	8-in.	13,000	6,500	27,000	0.72	
A	Three $\frac{5}{8}$-in.	$3\frac{3}{4}$-in.	8-in.	13,000	6,500	20,000	0.47	
A	Three $\frac{3}{4}$-in.	$3\frac{7}{8}$-in.	8-in.	15,000	4,000	28,000	0.58‡	
A	Three $\frac{3}{4}$-in.	6-in.	12-in.	13,000	7,000	60,000	1.64	
A	Three $\frac{5}{8}$-in.	6-in.	12-in.	14,000	7,500	30,000	0.83	
A	Three $\frac{5}{8}$-in.	6-in.	12-in.	15,000	9,500	46,000	1.64	
A	Three $\frac{7}{8}$-in.	6-in.	12-in.	18,000	3,500	63,000	1.78‡	

Joint Type	Number and and Size of Bolts	Thickness of Side Pieces	Thickness of Centre Pieces	Load at Apparent Yield Point, Lb.	Load at Slip of $\frac{1}{16}$ in., Lb.	Final Load, Lb.	Final Slip, In.	Remarks
A	Three 1-in.	6-in.	12-in.	26,000	20,000	70,000	1.53	
A	Three 1-in.	6-in.	12-in.	27,000	16,500	75,000	1.78	
A	Three $\frac{3}{4}$-in.	6-in.	12-in.	16,000	10,500	45,000	1.28	
A	Three $\frac{7}{8}$-in.	6-in.	12-in.	23,000	2,000	60,000	1.46‡	
A	Three $\frac{7}{8}$-in.	$5\frac{7}{8}$-in.	12-in.	22,000	12,500	60,000	1.54	
A	Three $\frac{7}{8}$-in.	$5\frac{3}{4}$-in.	12-in.	21,000	13,500	60,000	1.26	
B	Three $\frac{3}{4}$-in.	$2\frac{3}{4}$-in.	6-in.	11,000	7,500	35,000	0.73	Ultimate
B	Three $\frac{3}{4}$-in.	4-in.	8-in.	14,000	10,500	23,000	0.53	
B	Three $\frac{3}{4}$-in.	4-in.	8-in.	15,000	10,500	29,000	0.52	Ultimate
B	Three $\frac{3}{4}$-in.	$2\frac{7}{8}$-in.	6-in.	15,000	8,000	34,000	0.63	Ultimate

NOTES.

*No slips except final slip made on this joint.

†The final loads are not necessarily the ultimate loads, as no effort was made to find the ultimate load.

‡These joints had a large initial slip which was probably due to a loose fit of the bolts in the holes.

The average slip of the joints under the heading "Load at Apparent Yield Point" was 0.12 in. Using the formula devel oped in the text, the flexural unit-stress in the bolts for this load is approximately 35,000 lb. per square inch, which is not an excessive value for the elastic limit in flexure of the bolt.

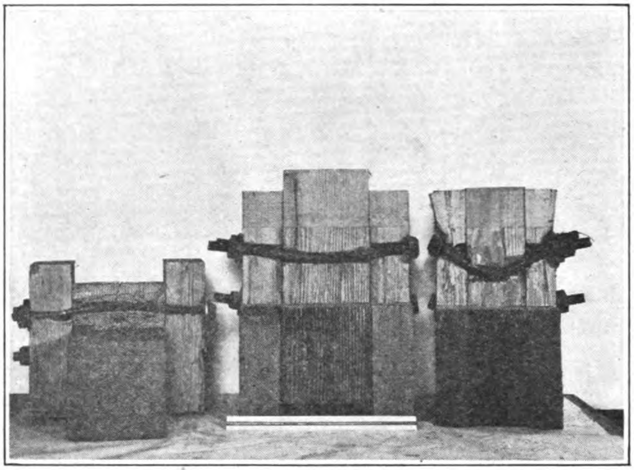

FIG. 35. PHOTOGRAPH OF JOINTS AFTER TESTING.

bolt is as shown in Fig. 36, namely, tringular in shape, concentrated near the contact surfaces and of such arrangement as to produce the condition of a restrained beam with some point of contraflexure at each contact surface.

In order to conform to these conditions b must equal $2a$. Then $B^1 = \frac{1}{2} B$.

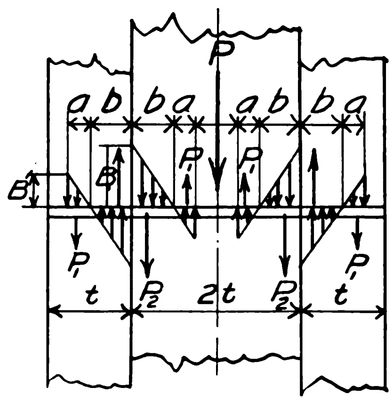

FIG. 36. ASSUMED DISTRIBUTION OF LOAD ON BOLT.

The distribution of bending moment will be as shown, there being two equal maximum moments of amount,

$M = 2/27\ Pl.$ where $l = a + b$ and must be less or equal to t.

Under the assumption that beyond a certain minimum value of t, or width of side piece, the strength of the joint is independent of the length of bolt required, the length l may be found, such that the moment resulting from the load on this length of bolt will just equal the flexural strength of the bolt. This method is similar to that of Thayer, the difference being in the assumed distribution of load, the resulting moment and the computed strength of bolt.

The maximum moment will occur at a point of zero shear, or at a point distant $\frac{1}{2}\ b = a$ either side of the contact face.

Let the moment be computed at point X

$$M = P_1\left(\frac{l}{3} + \frac{2}{9}l - \frac{1}{9}l\right) = \frac{4}{9}P_1 l$$

$$P_1 = \frac{1}{2}\left(\frac{B}{2} \times \frac{l}{3}\right)d = \frac{Bld}{12}$$

$$M = \frac{4}{9} \times \frac{Bld}{12} \times l = \frac{1}{27}dBl^2 = \frac{\pi d^3 S}{32}$$

$$l^2 = \frac{\pi d^3 S.27}{32Bd}$$

$$l = d\left(\frac{\pi S.27}{32B}\right)^{\frac{1}{2}}$$

With the safe stresses for flexure of bolts and bearing for the timber, the strength of the bolts is easily determined; and for all thicknesses of side pieces in excess of the limiting value l, the strength will be constant. For thicknesses of side pieces under l, the safe strength of the joint would be a question of bearing of the bolts against the timber, if the pressure distribution be assumed to remain constant, resulting in a decrease in the strength of the joint. The results of the tests do not bear out this assumption. It is believed that in joints where the thickness of side timbers is less than the limiting value l, the pressure-distribution diagram, while holding to the general triangular shape, changes in its relative dimensions a and b, within the limits, where $a = O$ and $a = \frac{1}{3}t$. Further, it is held that the ratio of a to t is always such, that the resulting bending moment on the bolt bears the same relation to the capacity of the bolt in bending as the maximum intensity of pressure on the timber bears to the resistance of the timber in bearing.

Following out these assumptions, the strength of the test joints with 2-in., $2\frac{5}{8}$-in., and 6-in. side timbers have been calculated, using 16,000 lb. per sq. in. as the flexural stress in the bolts, and 1300 lb. per sq. in. as the limiting unit-bearing pressure on the timber. These computations were made by the use of the diagram shown in Fig. 37. The results are entered in the bottom line of Table XV, and are seen to coincide approximately with the values taken from the test curves.

The preceding discussion of the theory of bolted joints tacitly assumes the case of comparatively thick timbers with bolts of small diameters. As the ratio of the diameter of bolt to the thickness of splice-pad is increased, the pressure-distribution on the bolt will change from the triangular shape to a trapezoidal shape, and finally, for the case of short thick bolts of great stiffness, the pressure-distribution will become uniform along the length of the bolt. In other words, the limit in this direction will be the case where the strength of the joint is determined by crushing of the timber. Obviously, the joints tested do not fall within this class. The strength of bolted joints where the pressure distribution is trapezoidal may be found by diagrams constructed along the lines of Fig. 37.

From the results of the studies on bolted joints, the safe working loads for bolts in double shear, having all end bearing, may be taken as shown in Table XV. For bolts having side bearing on the timber the safe loads may be taken as two-thirds the values of the table. While the tests did not consider bolts in single shear alone, working loads for this condition may be taken at one-half the values of the table.

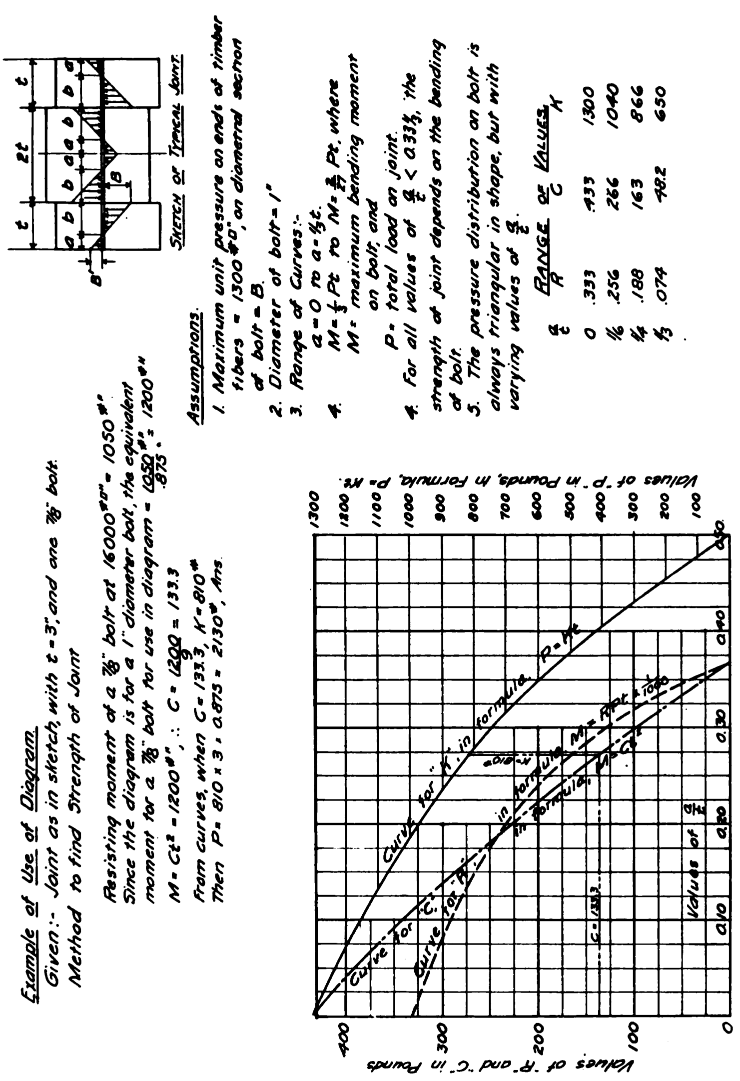

FIG. 37. DIAGRAMS FOR COMPUTING STRENGTH OF BOLTED JOINTS.

It is hardly necessary to state that further tests are required to confirm the theory presented, but in the absence of such tests, the method advocated is believed to be reasonable, and to give results that are safe.

CHAPTER VI

End Joints of Trusses

The timber truss, with its details of joints, forms perhaps, the most important and interesting subject of timber design. A roof truss of the Howe type is the simplest form of truss, as far as the calculation of stresses is concerned. Consequently we find in practice that the main sections of such roof trusses are usually of the proper size, but that the details, particularly the end joints, are often quite deficient in strength to develop the calculated stresses. Thus the truss that the designer imagined confidently had a large safety factor, may actually be not far from failure. The live load used in the design is in many instances the saving factor of the truss.

It is not the intention in what follows to treat of the solution of stress diagrams in roof trusses. In such mathematical discussion of primary or secondary stresses as is given, the reader's knowledge of the derivation and the proper application of the different formulae used, is assumed. In other words, it is taken for granted that he has a working knowledge of structural mechanics. It is proposed to discuss the different details of a typical roof truss from the standpoint of both theoretical and practical efficiency.

The number of types of end joints that may be used might almost be said to be legion, and no attempt will be made to list or describe them all. No one type can be specified that will be applicable to all trusses. The individual case must govern the selection. For example, a roof truss that rests upon a masonry wall will usually require an altogether different type of end joint from a truss which frames into a post. The consideration of clearance may decide whether a shoe is necessary, or whether the batter post of the truss may simply dap into the lower chord and be bolted thereto. Certain simple end-details can be used only with trusses of small chord-stresses. Again, the consideration of wind action in a truss which is a unit of a portal frame may involve the necessity of an end bolster, with carefully detailed connection to post, truss-chord and knee-brace.

There are, however, a few types of end joints which have marked superiority, both from directness of action of the stresses, and from simplicity and ease of fabrication and framing. The cardinal principle may be set down as a basis for design of all end joints, that the complete thrust of the batter post should be taken by one line of action alone. To illustrate, a joint should never be framed so that the thrust is taken partly by lugs dapped into the lower chord, and partly by an inclined bolt. Tests* have demonstrated conclusively that the two systems will not act together, and that either the lugs or the bolt will take the whole stress up to the commencement of failure, before the other system will come into action.

In the following paragraphs, eight types of end joints are detailed and cost estimates of six are given. In order that a comparison of these different details may be made upon the same basis, a typical truss known as the English roof truss has been chosen. The span is 70 ft., the distance between trusses is 24 ft., and the load has been assumed at 40 lb. per sq. ft. of horizontal projection. Of this total load, 13.5 lb. per sq. ft. has been considered as acting at the lower-chord panel-points, being made up of one-half the weight of the truss plus the weight of the ceiling. The skeleton diagram of the truss, together with the stress diagram

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**Engineering Record*, Vol. 42, November 17, 1900. This article by F. E. Kidder shows full size end joints after failure, and discusses the tests. The article is summarized and the illustrations reproduced in 'Jacoby's Structural Details,' pages 274-276. Jacoby also discusses other tests of end joints.

is shown in Fig. 38. The stresses in the various members are indicated on the left half of the truss, while the required sizes are marked on the right half.

The following working stresses have been used in designing the details.

Timber	Lb. per sq. in.
Tension on net section	1,500
Compression, end-bearing	1,600
Compression, cross-bearing	285
Shear, longitudinal	150
Steel	
Tension	16,000
Shear	10,000
Bearing	20,000
	Lb. each
$\frac{3}{4}$-in lag screw in steel plate	1,030
$\frac{7}{8}$-in. lag screw in steel plate	1,200

All computations incident to the design are given, so that the method may be followed by the reader. For the sake of simplicity, it is assumed that the truss in all cases rests upon a masonry wall, although no details of support are shown. It is also assumed that the centre line of support passes through the intersection of the centre lines of the chords.

Types A and B. Where the clearances, relative inclination of upper chord to lower chord, and the magnitude of the stresses will permit, the simplest and cheapest details for end joints are those represented by Types A and B, shown in Fig. 39 and 40.

In these two details, the thrust of the upper chord is taken completely by bearing and shear on the lower chord. As shown by the detail computations, the required length of the uncut portion of the chord for shear is 41 in. The inclined bolts in both details take no calculable stress. In Type A, the length of the cut *d* is determined from the curves of Fig. 19 and 20, of Chapter IV, which give the allowable pressures on surfaces inclined to the direction of the fibres. On examining these curves, it is evident that if the pressures shown by the curve of Fig. 20 be used, this type of detail must be abandoned, unless the sections of chords are materially increased, as the required cuts in the chord will be too deep. If, however, the safe bearing pressure be taken in accordance with the curve of Fig. 19, the required depth of toe-cut is 5 in. measured on the normal face of the strut. This leaves an uncut portion of the chord with an effective area of 4¼ in. × 10 in. =

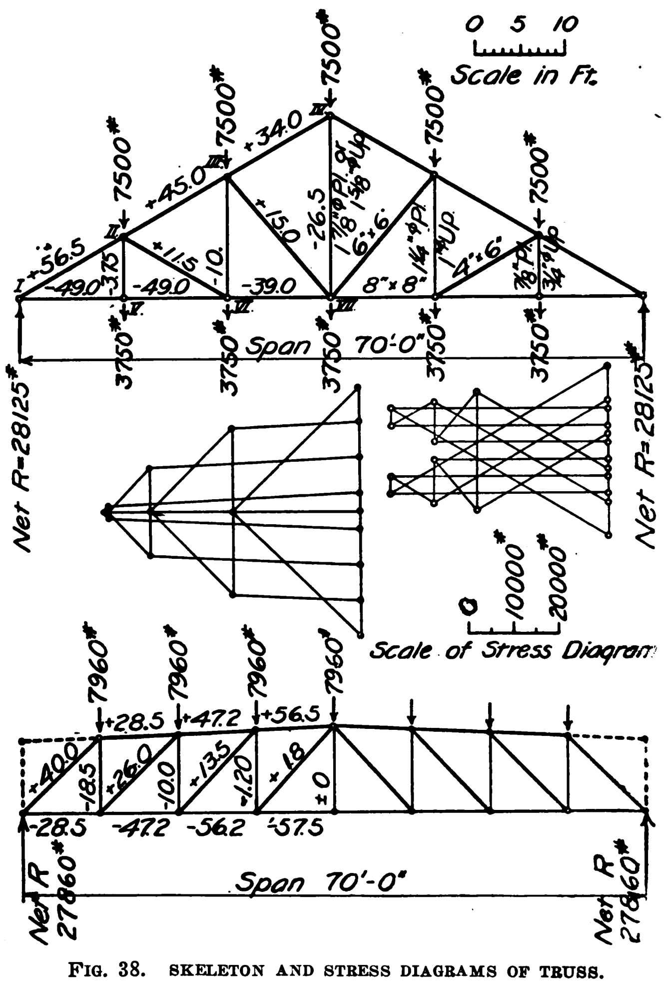

FIG. 38. SKELETON AND STRESS DIAGRAMS OF TRUSS.

42.5 sq. in. The average tensile stress on this section is then $\frac{49,000}{42.5} = 1200$ lb. per sq. in. With the centre lines of truss members and end support intersecting in a point, as assumed in finding the stresses in the truss members, there will be excessive secondary bending due to the couple of the horizontal component of the thrust of the upper chord and the resultant tension in the lower chord, the latter acting at one-half the uncut depth of the lower chord. The moment of such couple in Fig. 39 is 49,000 lb. $\times$ 4 in. $=$ 196,000 in. lb. To overcome this moment, which would stress the chord to failure, the line of action of the reaction of the truss, or the centre line of support, must be placed at such a distance to the left of the intersection of the centre lines of the chords, that the couple formed by the reaction of the truss and the resultant of the vertical thrust of the upper chord on the lower chord will equal the moment of the first couple. It has been assumed that the thrust of the upper chord is uniformly distributed over its toe. Consequently the vertical component of this thrust will also be uniformly distributed over the same area* In this case the vertical component of the thrust happens to coincide in position with the line of the truss reaction. Therefore the line of action of the truss reaction must be placed $\frac{196,000}{28,125}$

*It is obvious that with a normal cut on the upper chord, as in this detail, the whole thrust of the upper chord must be taken on the toe of the post, and that the inclined cut can take no pressure. The assumption of a uniformly distributed pressure over the toe of the upper chord requires that there exist another component of the stress in the chord which is normal to its centre line, otherwise the joint cannot be in equilibrium. This component is small, and will be resisted either by friction of the toe on the cut in the lower chord, or by tension in the bolts. In effect, such assumed distribution of forces assumes that the direction of stress in the upper chord is inclined slightly to the centre line of the chord, and is coincident with a line drawn between the centres of the normal cuts at the ends.

= 7 in. to the left of the centre of the toe, or 7 in. to the left of the intersection of the truss members.

In Type B, the cuts of the upper chord are so modified that neither of the two surfaces are normal to the centre line of the chord. Consequently, each surface will take a component of the total stress. The angle between the two bearing surfaces may of course vary; in this case it has been made 90°. Computations made to determine the depth of cut in the lower chord, which are shown in detail in Fig. 40, indicate that this depth must be $4\frac{1}{8}$ in. This distance determines the length of the other surface, which is found to be more than sufficient for its component of pressure. There then remains to be determined the position of the centre line of support, so that no secondary bending will exist in the lower chord. For this condition, the moment of all

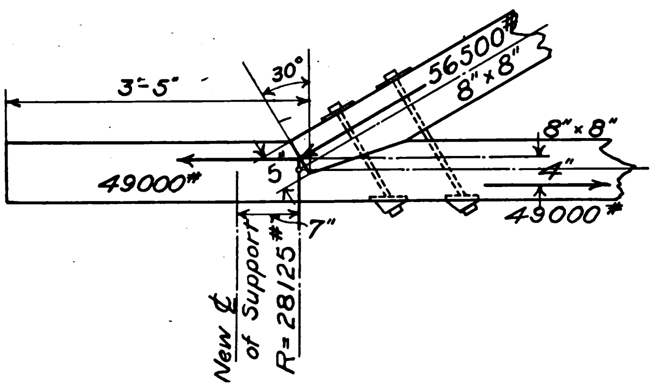

FIG. 39. END JOINT, TYPE A.

COMPUTATIONS

Depth of toe:

$\Theta = 60°$, therefore $n = 1400$ lb. per sq. in., from Fig. 19, Chapter IV.

Required area in bearing $= \frac{56,500}{1400} = 40$ sq. in.

Required depth $= \frac{40}{8} = 5$ in.

Length of chord projection for shear:

$\frac{49,000}{150 \times 8} = 41$ in. $= 3$ ft. 5 in.

forces acting on the lower chord must equal zero. Each normal component of stress on the two bearing surfaces of the upper chord is resolved into its horizontal and vertical components. The product of the resultant of these horizontal forces by the distance between such resultant and the centre line of the uncut portion of the lower chord is 49,000 lb. $\times$ $3\frac{3}{4}$ in. $=$ 183,500 in. lb. The resultant of the two vertical components of the normal pressures is found to lie at a distance of $3\frac{1}{2}$ in. to the left of the reaction. The new centre line of support must therefore be placed at a distance to the left of this point such that the product of the vertical re-

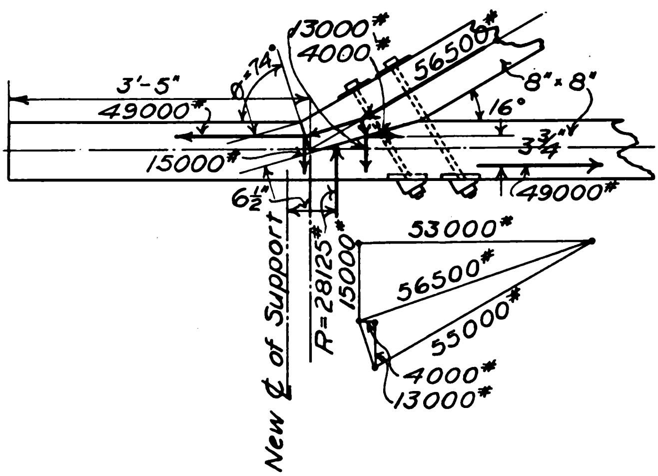

FIG. 40. END JOINT, TYPE B.

COMPUTATIONS

Depth of toe:

$\theta = 74°$, therefore $n = 1680$ lb. per sq. in., from Fig. 19 Chapter IV.

Required area in bearing $= \frac{56,500}{1680} = 32.8$ sq. in.

Required depth $= \frac{32.8}{8} = 4.1$ in.

Pressure on inclined bed:

$\theta = 16°$, therefore $n = 400$ lb. per sq. in.

Required area in bearing $= \frac{13,000}{400} = 32.1$ sq. in.

Actual area $=$ 16 in. $\times$ 8 in. $=$ 120 sq. in.

action by this distance will equal the moment of 183,500 in. lb. found above. This length is $6\frac{1}{2}$ in., so the new centre line of support must lie 3 in. to the left of the intersection of the centre lines of the upper and lower chords. The long projection of the lower chord beyond the point of intersection of its centre line with that of the upper chord makes it extremely improbable that the Types A and B end joints could be used in an actual case. Undoubtedly some form of shoe would be found necessary.

Types C and D. Types C and D, shown in Fig. 41 and 42, are comparable to the common end-detail of a steel truss, in which the two chords deliver their stresses into a common gusset plate. This is particularly true for Type D. In Type C, each stress is completely taken in bearing by the steel tables riveted to

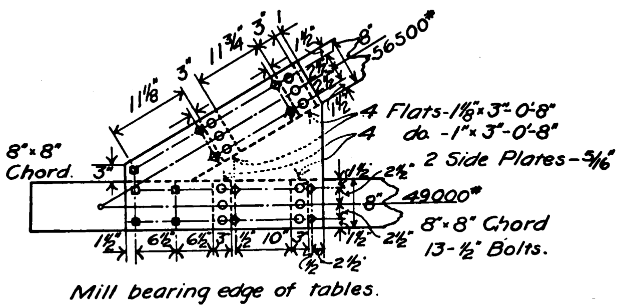

FIG. 41. END JOINT, TYPE C.

COMPUTATIONS

Bearing area of tables for upper chord $= \dfrac{56{,}500}{1600} = 35.3$ sq. in.

Assuming two tables each side of chord, depth of table $= \dfrac{35.30}{4 \times 8}$

$= 1.103$ in. $= 1\frac{1}{8}$ in.

Rivets required in each table $= \dfrac{56{,}500}{4 \times 4420} =$ three $\frac{3}{4}$-in. rivets.

Thickness of side plates for bearing against rivets $= \frac{5}{16}$ in.

Shearing area required for each table $= \dfrac{56{,}500}{4 \times 150} = 94.2$ sq. in.

Distance required between tables $= \dfrac{94.2}{8} = 11.75$ in. $= 11\frac{3}{4}$ in.

Treating side plates as columns, $r = 0.0903$. $L/r = \frac{11.75}{0.0903} = 130$.

Moment of rotation on each table $= \frac{56{,}500}{4} \times \frac{1}{2}(1.125 + 0.3125) = 10{,}130$ pound-inches.

Stress in bolts $= \frac{10{,}130}{2 \times 3.5} = 1450$ lb. Use two ½-in. bolts for each table.

Bearing area of tables for lower chord $= \frac{49{,}000}{1600} = 30.6$ sq. in.

Assuming two tables each side of chord, depth of tables $= \frac{30.6}{4 \times 8} = 0.955$. Use 1 in.

Rivets required in each table $= \frac{49{,}000}{4 \times 4420} =$ three ¾-in. rivets.

Shearing area required for each table $= \frac{49{,}000}{4 \times 150} = 81.6$ sq. in.

Distance required between tables $= \frac{81.6}{8} = 10.2$ in. Use 10-in. bolts as for upper chord.

Area between last table in upper chord and end of upper chord greater than minimum area of 94.2 sq. in.

Net section of lower chord $= (8-2)\ (8-1) = 42$ sq. in.

Unit stress in lower chord $= \frac{49{,}000}{42} = 1170$ lb. per sq. in.

Bill of Material for One End-Connection

	Pounds
Two $\frac{5}{16}$-in plates with area of 9.2 sq. ft., at 12.72 lb.	117.4
Four flats, 1⅛ by 3 by 8 in., at 7.65 lb.	30.6
Four flats, 1 by 3 by 8 in., at 6.80 lb.	27.2
Twenty-four rivet heads at 0.136 lb.	3.3
Thirteen bolts, ½ by 9 in., at 0.65 lb.	8.5
Total weight of steel	187.0

Cost of One End-Connection

187 lb. steel at $0.04	$7.48

the side plates. As will be seen by the detailed computations, the thickness of the tables is a factor of the capacity of the timber in end bearing. A sufficient distance between tables must be given to provide the necessary shearing area; bolts must be provided to hold the tables in their notches; the side plates must be thick enough, acting as columns between the lines of bolts, to take the largest stress; and last, each table must have enough rivets to hold its individual portion of the total stress.

In principle, this type of end joint is perfect. There is no eccentricity of the principal stresses, and consequently no secondary stresses. As a type of end detail from the standpoint of field work, the joint is not so good. It will be found practically impossible to cut and fit the notches for the tables with sufficient accuracy so that each table will take equal and uniform bearing. Moreover, any deficiency in bearing will be extremely difficult, if not impossible, to remedy by

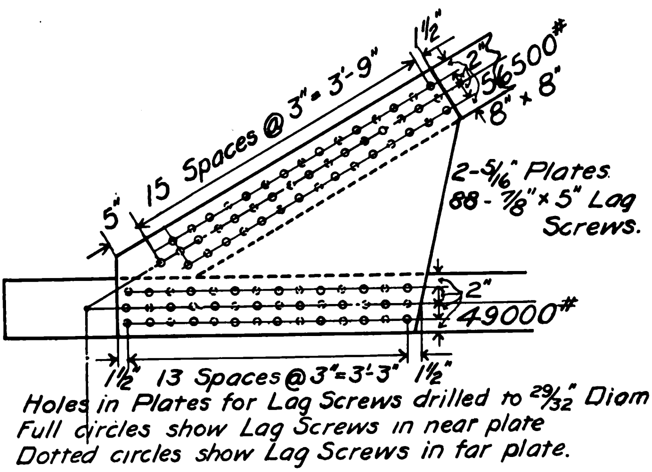

FIG. 42. END JOINT, TYPE D.

COMPUTATIONS

Number of lag screws in upper chord $= \dfrac{56{,}500}{1200} = 47.$

Number of lag screws in lower chord $= \dfrac{49{,}000}{1200} = 41.$

Thickness of plate $= \frac{5}{16}$ in.

BILL OF MATERIAL

	Pounds
Eighty-eight $\frac{7}{8}$ by 5-in. lag screws at 0.95 lb.	82.6
Two $\frac{5}{16}$-in. plates with area of 15.4 sq. ft. at 12.75 lb.	196.5
Total weight of steel	279.1

COST OF ONE END-CONNECTION

279.1 lb. steel at $0.04 .... $11.16

shimming or similar methods, on account of inaccessibility after the side plates are bolted to the timber. The joint is not susceptible to field examination for defects, and this is perhaps its worst feature.

Fig. 42 (Type D) shows a modification of Type C, in which the steel tables are abandoned, and the chord and batter-post stresses are transmitted to the gusset plates by means of lag screws acting in shear. Except for the consideration of economy, this joint has all the advantages of Type C, and none of its disadvantages. All stresses are concentric, and with good inspection during construction, one may rely on a close fit of the lag screws in the timber. The holes in the steel plates for the lag screws are drilled to a diameter of $\frac{1}{32}$ in.

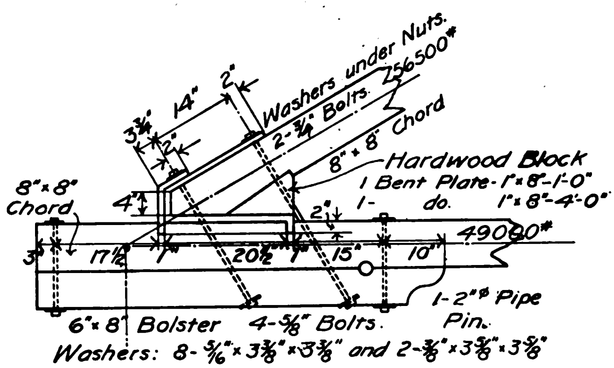

FIG. 43. END JOINT, TYPE E.

COMPUTATIONS

Area required for bearing between upper and lower chord $= \dfrac{28{,}125}{285} = 99$ sq. in.

Depth of lugs $= \dfrac{49{,}000}{2 \times 1600 \times 8} = 1.915$ in. Use 2-in. lugs.

Thickness of lugs for bending:

Bending moment one lug $= 24{,}500$ lb. $\times 1\frac{1}{2} = 36{,}750$ pound-inches.

Thickness of lug assumed to be 1 in.

Required section modulus $= \dfrac{36{,}750}{25{,}000} = 1.465.$

Required thickness of lug $= 1.05$ in. Use 1-in plate as assumed.

Length required for shear between lugs $= \dfrac{49{,}000}{2 \times 150 \times 8} = 20.4$ in. Use 1 ft. 8½ in.

Depth of toe $= \dfrac{1600 \times 8}{49{,}000} = 3.84$ in. Use 4 in.

Bearing stress of 1600 lb. per sq. in. is used, as timber fibres are confined and therefore capable of taking full end compression.

Stress in bolster = horizontal component of stress in two ¾-in. bolts, $= 4830 \text{ lb.} \times 2 \times 0.5 = 4830$ lb.

Number of shear pins required $= \dfrac{4830}{800 \times 8} = 0.75$. Use one 2-in. pin.

BILL OF MATERIAL FOR ONE END-CONNECTION

One plate, 1 by 8 by 12 in., at 27.20 lb	27.20
One plate, 1 by 8 in. by 4 ft., at 27.20 lb	109.00
One bolt, ¾ by 25 in., at 3.52 lb	3.52
One bolt, ¾ by 33 in., at 4.54 lb	4.54
Four bolts, ⅝ by 15 in., at 1.54 lb	6.16
Two washers, ⅜ by 3⅝ by 3⅝ in., at 1.40 lb	2.80
Eight washers, $\frac{5}{16}$ by 3⅝ by 3⅝ in., at 1.02 lb	8.16
Total weight of steel	161.38

One 2-in. extra heavy steel pipe-pin.
One bolster 6 by 8 in. by 6 ft., 24 ft. B.M.
One hardwood block.

COST OF ONE END-CONNECTION

Steel, 161.38 lb., at $0.04	$6.46
Pipe pins, 1, at $0.25	0.25
Bolster, 24 ft. B.M., at $0.04	0.96
Hardwood block, 1, at $0.50	0.50
Total cost end connection	$8.17

greater than the diameter of the shank of the lag screw; the holes in the timber are to be bored in accordance with the specifications to be given in the concluding article of this series. The lag screws are to be screwed and not driven into place. Lag screws are better fitted for this type of joint than are bolts, as it would be practically impossible to bore a hole from one plate as a template and strike the corresponding hole in the opposite plate exactly. Consequently, were bolts to be specified, it would undoubtedly be found after the fabrication of the joint that the bolts were sprung or bent into place, and their value in shear would be questionable.

This type of end detail is well suited to trusses of an A-shape, resting upon posts. The side plates in such cases may be extended to engage the top of the post, and thus to give considerable stiffness to the building-frame.

Type E. In Fig. 43 is shown a detail of end joint using a steel shoe made of two plates. This detail is similar to that shown in Jacoby's 'Structural Details, p. 262, Fig. 63b. The points to be noted in a design of this type are (1) the depth of lugs cut into the chord to give the required bearing area, (2) the thickness of the projecting lug to resist bending, (3) the distance between lugs in order that there may be sufficient shearing area to take the increment of stress, (4) the depth of the vertical end-cut of the upper chord so that the necessary area be provided for bearing against the fibres of the timber, (5) the length of horizontal cut on the upper chord for distributing the vertical component of its thrust across the fibres of the lower chord, and finally (6) the size of the inclined bolts for holding the joint together.

Attention should be called to the fact that in this detail, the centre line of the upper chord intersects the base of the shoe practically at the toe of the shoe. In consequence, there will be a tendency for the shoe to rotate in a counter-clockwise direction, bringing an uneven distribution of bearing over the base of the shoe, with a possible crushing of the fibres of the wood at the top of the lower chord under the toe of the shoe. This tendency to uneven bearing pressure will be counteracted by the action of the inclined bolts, which, if always tight, will come into play with the application of the load to the truss, and will cause a readjustment of the joint stresses and a consequent approximately uniform distribution of the vertical bearing pressure.

From the standpoint of field work, this type of shoe is an excellent one, though extravagant of steel. It is simple in its action and comparatively easy to frame into the timber. Care must be taken, of course, to see that both lugs have an even bearing against the chord. As has been noted already, this is a hard thing to secure, and is a fault of all shoes having more than one bearing surface. However, the shoe has only two lugs to fit, as against eight in Type C. Moreover, the inspection for uniformity of bearing is easy to make, and if necessary shimming can be done readily and effectively.

This shoe can only be used for stresses requiring not more than two lugs, hence its field of application is limited. Another defect is that the forge work is difficult with the thickness of plate used. Especially is this true of the bending of the end of the inner plate to form the inner lug. Incidentally, this detail forms a good example of the consideration of actual unit working stresses as compared to purely theoretical values, as mentioned in the first article of this series. With a 2-in. depth of lug, the bearing pressure against the ends of the fibres is assumed to be 1600 lb. per sq. in. On account of the fillet formed in bending the plate, the actual bearing area will be decreased and the actual unit working stress will probably be found to be around 1800 lb. per square inch.

Type F. Type F, illustrated in Fig. 44, is a modification of Type E, in which steel tables riveted to the shoe plate are substituted for the lugs of Type E. Its advantages are (1) the main plate may be reduced to the minimum thickness as required by consideration of shear and tension alone, (2) any number of tables may be used, and (3) the forge work is less than in the previous type. A point to be considered in a shoe of this type is that no table should be placed under the end of the batter post. The notch for the table will invariably be made deeper than the table itself, so that the vertical bearing of the steel table on the timber chord cannot be counted upon to distribute load.

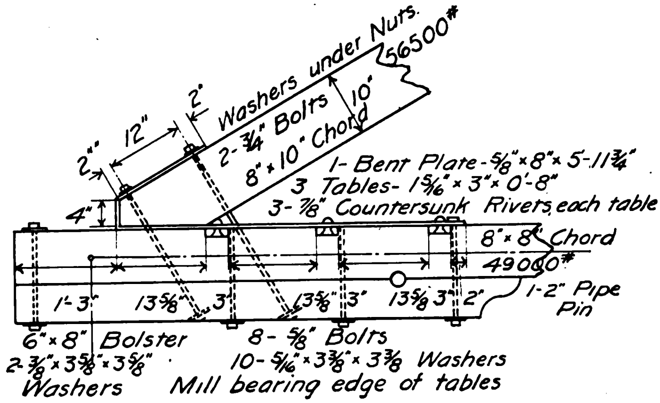

FIG. 44. END JOINT, TYPE F.

COMPUTATIONS

Depth of toe as in type C, 4 in.

Area required for bearing between upper and lower chord $= \frac{28{,}125}{285} = 99$ sq. in.

A 10-in. depth will therefore be required for the upper chord, giving an area of 8 by 13 in. $= 104$ sq. in.

Depth of tables (assuming three used) $= \frac{49{,}000}{3 \times 8 \times 1600} = 1.275$ in. Use $1\frac{5}{16}$ by 3 in.

Assuming three rivets in each table, stress in each rivet $= \frac{49{,}000}{9} = 5450$ lb. Use three $\frac{7}{8}$-in. rivets in each table.

Thickness of plate for bearing against rivets $= \frac{3}{8}$ in.

Thickness of plate for shear $= \frac{49{,}000}{10{,}000 \times 8} = 0.614$ in.

Thickness of plate for tension $= \frac{49{,}000}{16{,}000 \times (8 \text{ in.} - 2.80 \text{ in.})} =$ 0.59 in.

Make plate $\frac{5}{8}$ in. thick.

Moment of rotation of tables $= \frac{1.3125 \text{ in.} + 0.625 \text{ in.}}{3} \times \frac{49{,}000}{3}$ $= 15{,}800$ pound-inches.

Stress in bolts $= \frac{15{,}800}{3\frac{1}{2}} = 4520$ lb.

Add stress due to pin in bolster $= \frac{1}{4} \times \frac{1}{2} \times 800 \text{ lb.} \times 8 \text{ in.} =$ 800 lb.

Total stress in two bolts $= 5320$ lb. Use two $\frac{3}{4}$-in. bolts.

Using two $\frac{3}{4}$-in. diagonal bolts, the horizontal component in the bolster will be as in Type C, requiring one pin.

Distance required between tables for shear $= \dfrac{49{,}000}{3 \times 8 \times 150} =$ 13.6 in. Use $13\frac{5}{8}$ in.

BILL OF MATERIAL FOR ONE END-CONNECTION

	Pounds
One plate, $\frac{5}{8}$ by 8 in. by 5 ft. $11\frac{3}{4}$ in., at 17 lb.	101.50
Three plates, $1\frac{5}{16}$ by 3 by 8 in., at 8.95 lb.	26.85
One bolt, $\frac{3}{4}$ by 23 in., at 3.28 lb.	3.28
One bolt, $\frac{3}{4}$ by 30 in., at 4.13 lb.	4.13
Eight bolts, $\frac{5}{8}$ by 16 in., at 1.62 lb.	12.96
Ten washers, $\frac{5}{16}$ by $3\frac{3}{8}$ by $3\frac{3}{8}$ in., at 1.02 lb.	10.20
Two washers, $\frac{3}{8}$ by $3\frac{5}{8}$ by $3\frac{5}{8}$ in., at 1.40	2.80
Nine $\frac{7}{8}$-in. rivet heads, at 0.24 lb.	2.16
Total weight of steel	163.88
One 2-in. extra heavy steel pipe-pin.	
	Ft. B.M.
One bolster, 6 by 8 in. by 7 ft.	28.00
One 2 by 8 in. by 40-ft. extra length upper chord	26.50
	54.50

(Total B.M. = 53; use one-half only as labor will be practically the same.)

COST OF ONE END CONNECTION

Steel, 163.88 lb. at, $0.04	$6.55
Pipe-pin, one, at $0.25	0.25
Lumber, 54.50 ft. B.M., at $0.04	2.18
	$8.98

The size of diagonal bolts in both Types E and F are not susceptible of computation, but are determined by judgment and experience. In the present instance, two $\frac{3}{4}$-in. bolts have been used for the diagonals, and two $\frac{5}{8}$-in. bolts for holding the lugs or tables in their notches. The sizes of the vertical bolts are found as shown in the detailed computations.

Type G. Fig. 45 illustrates a detail of end joint in which a cast-iron shoe is used to transmit the thrust of the batter post to the lower chord. The details of such a shoe may be arranged in several ways, but the form shown represents a rather common type. As there are no diagonal bolts, the vertical pressure of the shoe on the lower chord is not uniform, and hence a toe has been provided, extending beyond the end of the batter post. The depth of the lugs are determined by limitations of end bearing on the timber, and their spacing by considerations of shearing of the timber. As the area of the base of the shoe is large, the first lug may be placed underneath the batter post. The thickness of the lugs is found by treating them as projecting cantilevers taking shear and bending, using the stresses shown in the detail computations. The number, size, and arrangement of ribs is largely a question of judgment, remembering that cast iron is rather brittle, and that the shoe must consequently be well stiffened to resist tension and bending. The thickness of the metal is determined by the requirements of tensile and flexural stresses and by general considerations of a minimum thickness for castings to resist the unknown stresses of shrinkage and the probability of unseen blowholes.

Type H. Type H, illustrated in Fig. 46, is a cheap and, with well-seasoned timber, effective type of end joint, where the circumstances of clearance will permit of its being used. The principles involved in its design are simple; the pins take the whole thrust, the bolts being assumed to resist a tension equal to one-half the total stress on the lower chord. The bolster must be investigated for shear on the uncut portion, and tension on the net section back of the surface of application of the upper chord. It must be emphasized that the whole effectiveness of the joint depends on the question of whether any shrinkage of the timber, subsequent to the fabrication of the joint, is to be apprehended. If unseasoned timber is likely to be used in the framing, the detail should not be used, as the cross-shrinkage of the bolster and chord will allow the pins to become loose, and an undue strain will come upon the bolts with a consequent slip of the joint. This detail of end joint may be further modified by omitting the shear pins, and notching the bolster into the lower chord,

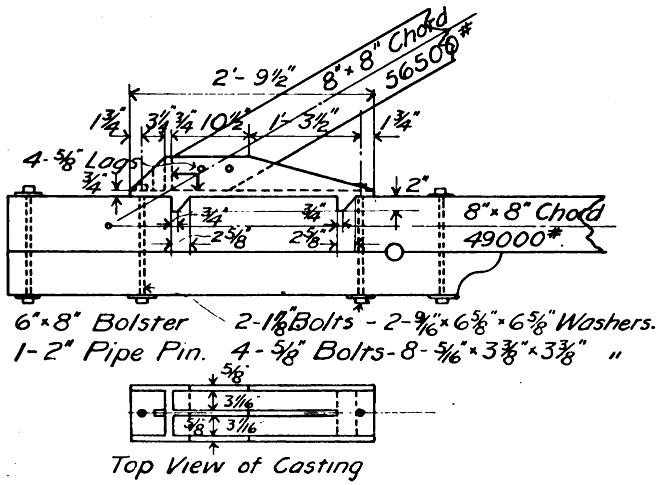

FIG. 45. END JOINT, TYPE G.

COMPUTATIONS

Depth of toe:

$\theta = 60°$, $n = 1400$ lb. per sq. in., from Fig. 19, Chapter IV.

Required area in bearing $= \dfrac{49{,}000}{1400} = 35$ sq. in.

Required depth of vertical cut $= \dfrac{35}{8} = 4.4$ sq. in.

Required area of horizontal cut:

$\theta = 30°$, $n = 650$ lb. per sq. in., $A = \dfrac{28{,}125}{650'} = 43.2$ sq. in.

Length of horizontal cut $= \dfrac{43.2}{8 - (3 \times \frac{5}{8})} = 7.1$ in.

Maximum unit bearing-pressure of shoe on lower chord:

As designed, toe of shoe is 9 in. beyond point where the vertical component of the thrust of upper chord intersects base of shoe. Call this distance $u = 9$ in. Let length of shoe be $L = 2$ ft. $9\frac{1}{2}$ in. $= 33\frac{1}{2}$ in. Let $p =$ maximum unit stress desired. Then $p = 2\left(2 - \dfrac{3u}{L}\right) \times \dfrac{P}{L}$, where $P = \dfrac{\text{Vertical reaction}}{\text{Width of chord}} = 3520$ lb.

Therefore $p = 250$ lb. per sq. in.

Depth of lugs as in Type E, 2 in.

Thickness of lug; as in Type E, bending moment on lug $= 24{,}500$ lb. $\times 1\frac{3}{8}$ in. $= 33{,}700$ pound-inches.

Required section modulus $= \dfrac{33{,}700 \text{ in. lb.}}{4000} = 8.45 = \frac{1}{6} \times 8 \text{ in.} \times t^2$.

Required thickness of lug $= t = 2.62$ in., make $2\frac{5}{8}$ in.

Distance required between lugs as in Type E $= 20\frac{1}{2}$ in.

Moment of rotation of lugs $= 24{,}500$ lb. $\times \frac{1}{2}$ (2 in. $+ \frac{3}{4}$ in.) $= 33{,}800$ in. lb.

$$\text{Stress in bolt back of lug} = \frac{33{,}800}{(2\frac{5}{8}\text{ in.} + \frac{3}{4}\text{ in.})} = 10{,}000 \text{ lb.}$$

Use one $1\frac{1}{8}$ in. bolt.

BILL OF MATERIAL FOR ONE END CONNECTION

One casting, weight 118 lb.	118 lb. cast iron
	Lb. steel
Two $1\frac{1}{8}$ by $16\frac{1}{2}$ in. bolts, at 6.71 lb.	13.4
Four $\frac{5}{8}$ by 16 in. bolts, at 1.63 lb.	6.5
Two washers, $\frac{1}{2}$ by $6\frac{5}{8}$ by $6\frac{5}{8}$ in., at 6.2 lb.	12.4
Four washers, $\frac{5}{16}$ by $3\frac{3}{8}$ by $3\frac{3}{8}$ in., at 1.01 lb.	4.0
	36.3
One 2 in. extra-heavy pipe-pin.	
One bolster, 6 by 8 in. by 6 ft.	24 ft. B.M.

COST OF ONE END CONNECTION

118 lb. cast iron, at \$0.0325	\$3.84
36.3 lb. steel, at \$0.04	1.45
One 2-in. pipe-pin, at \$0.25	0.25
One bolster, 24 ft. B.M. lumber, at \$0.04	0.96
Total cost of one end connection	\$6.50

in which case a small shrinkage will not cause the joint to slip. This modified Type H is shown in the detailed roof truss of Fig. 71.

General Summary of End Joints. For the details of end joints using lag screws, the question may arise as to whether or not all of the lag screws may be counted upon as acting together. I believe that this condition will be realized approximately; certainly to the same extent that the lugs of the other types of shoes will act together. As has been noted before, the holes for the lag screws should be either punched or drilled, preferably the latter, to a diameter not greater than $\frac{1}{32}$ in. larger than the nominal diameter of the lag screw. It is to be emphasized that all lag screws are to be screwed, and not driven into place, in holes of the proper diameter. First, a hole should be bored with a length and diameter equal to the length and diameter of the unthreaded shank of the lag screw, continuing with a

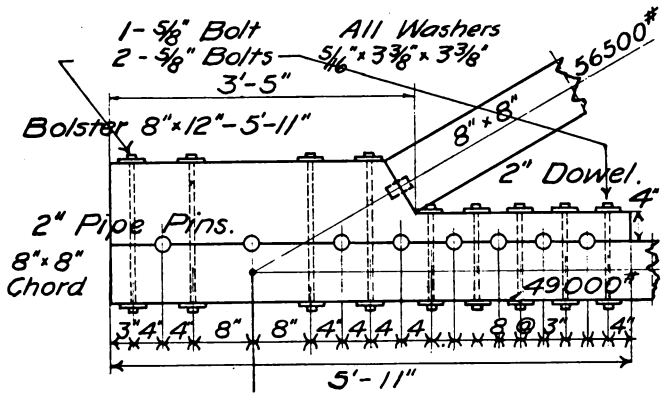

FIG. 46. END JOINT, TYPE H.

COMPUTATIONS

Required area of bolster for shear $= \frac{49,000}{150} = 327$ sq. in.

Required length of uncut portion of bolster for shear $= \frac{327}{8} = 40.5$ in.

Required number of 2-in. pins $= \frac{49,000}{8 \times 800 \text{ lb.}} = 7.7$. Use 8 pins.

Required net area of bolster for tension:

Maximum stress on cut portion of bolster $= 4 \times 6400$ lb. $=$ 25,600 lb.

Net section of bolster as detailed $= 3$ by 8 in. $= 24$ sq. in.

Unit stress in tension $= \frac{25,600}{24} = 1070$ lb. per sq. in.

Total stress in bolts $= \frac{1}{2} \times 49,000$ lb. $= 24,500$ lb.

As detailed, have eleven ⅝-in. bolts, 35,500 lb.

BILL OF MATERIAL FOR ONE END CONNECTION	Lb.
Five ⅝ by 21½-in. bolts, at 2.17 lb.	10.9
Six ⅝ by 13½-in. bolts, at 1.4 lb.	8.4
Twenty-two washers, $\frac{5}{16}$ by 3⅜ by 3⅜ in., at 1.01 lb.	22.2
One dowel, 2 by 4 in., at 3.6 lb.	3.6
Total weight of steel	45.1
Eight 2-in. extra-heavy steel pipe-pins.	
One bolster, 8 by 12 in. by 6 ft.	48.0 ft. B.M.

COST OF ONE END CONNECTION	
Steel, 45.1 lb., at \$0.04	\$1.80
Eight pipe-pins, at \$0.25	2.00
One bolster, 48 ft. B.M. lumber, at \$0.04	1.92
Total cost of one end connection	\$5.72

second hole of a length and diameter of the threaded portion of the shank at the base of thread. Careful and insistent inspection is necessary to secure the condition of lag screws screwed into place, as the carpenter will almost invariably drive the screws into place, if not watched.

In all the shoes with riveted lugs, special care must be exercised to see that good riveting is secured. This statement may seem so self-evident as to be foolish to mention. It must be remembered, however, that the steel and iron work on a timber-framed structure is usually let to a small iron-shop, if not to a blacksmith, and careless work is to be anticipated. Attention must also be paid to the milling of the bearing faces of the lugs. It is a curious fact that the average iron-worker regards any piece of steel or iron as being so superior to timber in strength, that he does not consider defective forge-work or riveting as of much importance. In his opinion, a failure of a steel shoe on a timber truss would be an impossibility. Again, in fabricating the truss, the steel shoes themselves should always be used as templates in cutting the notches for the tables, and boring the holes. The use of a well-made shoe, neatly finished, will result in more careful work on the part of the carpenter when framing the truss, than if the shoe is roughly made. To secure the best results, the steel shoes, as well as all the iron work, should be detailed and marked carefully and plainly. All the work should be carefully inspected before it is allowed to leave the shop. This inspection should preferably be done before painting the iron, if painting is to be done.

In the details shown in Fig. 43 and 44, the hole for the diagonal bolts in the upper chord should be specified as $\frac{1}{8}$ in. larger than the diameter of bolts. This is to allow the upper chord or batter post to slip easily into the toe of the shoe when the load is brought to bear upon the truss, and to thus take care of a possible untrue fit of the batter post into the shoe. With this arrangement, no bending of the diagonal bolt will result, when the batter post wedges into the toe of the shoe. I have examined many roof trusses before erection, where shoes with tables or lugs have been used, and have found in a number of instances that the toe of the batter post did not touch the shoe. Obviously in such a case, when the truss was erected and the load applied, this bolt must have been badly overstrained, both in tension and bending, if it had a driving fit in the upper chord.

The preceding investigations show that the cast-iron shoe has the advantage in economy over the other metal shoes investigated for the case under discussion, while the end joint using 2-in. shear pins is the cheapest of all. It should not be assumed that this same relation as regards economy, holds for all trusses. In general, it has been my experience that joints of Types F and G, and especially Type F, are the most suitable and reliable for trusses with end stresses of some magnitude. Other joints of different types may be used for special cases, but it is believed that the types here shown will cover all the cases the engineer is likely to meet, where single-stick chords are used. The unit costs used may be questioned. It is not contended that the relative costs of joints as given represent the actual costs of each detail. Many factors that would influence the price cannot here be considered, and hence the net costs of the different types of joints are only roughly approximate.

CHAPTER VII

Intermediate Joints of Trusses

An intermediate joint in a truss differs from the end joint only in the smaller stresses to be considered, and in the existence of a length of adjacent chord sufficient to take the component of the diagonal stress in the web by longitudinal shear in the timber. The discussion of the end joints of Types A and B will therefore apply in principle to all other joints of the typical truss shown, if the tension of the rod at the panel point be substituted for the end reaction. Details of intermediate joints in roof trusses furnish a good indication of the extent of the designer's knowledge of structural mechanics. Frequently, and especially where the truss is counterbraced in the central bays, the opposing diagonals are merely butted against one another with no provision for transmitting the component of the diagonal stress to the chord.

Fig. 47, 48, and 49 illustrate three methods of detailing the joints at panel points No. II and IV of the English roof truss of Fig. 38 of Chapter VI. Of these details, the two shown in Fig. 47 and 48 are the most common. The details of Fig. 49 is seldom used; nevertheless it is the most consistent and logical in principle, and the simplest of construction of the three types shown. This statement can best be brought out by a detailed discussion of the three types.

Type A, Fig. 47. Since neither of the two bearing surfaces of the indent is normal to the longitudinal axis of the strut, both surfaces exert a pressure on the chord, which can be determined by resolving the stress in the member, 11,500 lb., into two components, perpendicular, respectively, to the planes of the two bearing surfaces. The angle which each bearing plane makes with the direction of fibres of the timber determines at once the allowable unit bearing-pressure, as discussed in Chapter IV. This unit pressure requires a certain minimum bearing-area, and the necessary depth of indent is thereby determined. Each of the two components must be investigated in this manner.

It is evident that the shape of the chord indent and the length of the two bearing surfaces can be found only by a 'cut and try' method. For example, in the present instance, the web stress, 11,500 lb., is resolved into the two components 8200 lb. and 6000 lb. The 8200 lb. component has an inclination of 30° with the direction of fibres, or the bearing plane makes an angle of 60° with the fibres. By reference to Fig. 18, Chapter IV, the safe unit pressure for this angle is seen to be 1400 lb. per sq. in. The required bearing area is then $\frac{8200}{1400} =$ 5.86 sq. in., which necessitates a depth of indent of $\frac{5.86}{6} = 1$ in. Similarly the limiting inclination of the 6000-lb. component is $12\frac{1}{2}°$, which is the angle which the bearing surface makes with the direction of fibers. The safe unit pressure for this angle is 410 lb. per sq. in. The required area for bearing for this surface is then $\frac{6000}{410} = 14.6$ sq. in., which corresponds to a length of cut of $\frac{14.6}{6} = 2.5$ in. In a similar manner, the depth of indent for panel point No. VI is found to be $1\frac{1}{8}$ in., corresponding to $\Theta = 76°$, $n = 1700$ lb. per sq. in., giving required area of 6.4 square inches.

The required angles for the cuts must be noted carefully on the plans in order to secure the conditions in the field that are assumed in the design of the joint. As each diagonal of the truss may have a different slope for its end cuts, the most careful and accurate workmanship will be necessary to secure the desired results. This type of joint therefore violates the principle that all carpenter work should be made as simple as possible.

Type B, Fig. 48. In this detail, the end cuts of the struts are normal cuts. The length of the chord indents can be determined at once, since the total stress of the strut must act on the normal face of the strut alone. The conditions in this detail are exactly as discussed for the case of the Type A, end joint, in the pre-

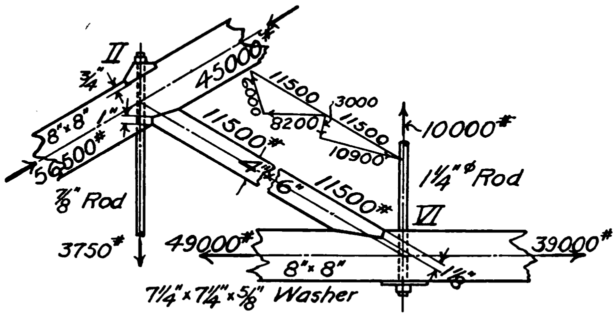

FIG. 47. INTERMEDIATE JOINT, TYPE A.

ceding chapter. For panel point No. II, the angle Θ is $30°$, $n = 670$ lb. per sq. in., the required area in normal cut is $\frac{11500}{670} = 17.2$ sq. in., and the required length of cut is $\frac{17.2}{6} = 2.87$ in. For panel point No. VI, $\Theta = 60°$, $n = 1400$ lb. per sq. in., the required area of cut is $\frac{11500}{1400}$ $= 8.2$ sq. in., and the required length of cut is $\frac{8.2}{6} =$ 1.37 in., or $1\frac{3}{8}$ in. The force necessary to hold the strut in equilibrium, and which must be developed by friction along the normal cuts of the strut will now be found. With the assumption that the thrust in the strut acts uniformly over the area of the normal cuts, the moment developed is the thrust in pounds multiplied by the eccentricity in inches, which latter is the distance between the centres of the normal bearing areas at the ends of the strut. The moment is therefore 11,500 lb. $\times$ $3\frac{3}{8}$ in. $=$ 38,800 in.-lb. The length of the strut is approximately 153 in., therefore the force to be developed in friction is $\frac{38800}{153} = 254$ lb. This frictional force will act parallel to the normal cut of the strut. Assuming that the coefficient of friction of wood on wood is 0.20, the effective resistance may be counted upon as amounting to $0.20 \times 11{,}500$ lb. $= 2300$ lb. In addition to this frictional force, such joints should be always well toe-nailed. Two 16D nails will give a resistance of 256 pounds.*

Both of the details of Fig. 47 and 48 involve a considerable depth of cut into the chord. In the upper chord, theoretically, the indent is of no consequence, since the chord is in compression, and a tight joint is assumed; actually, however, there is a considerable loss in efficiency. In the lower, or tension chord, the depth

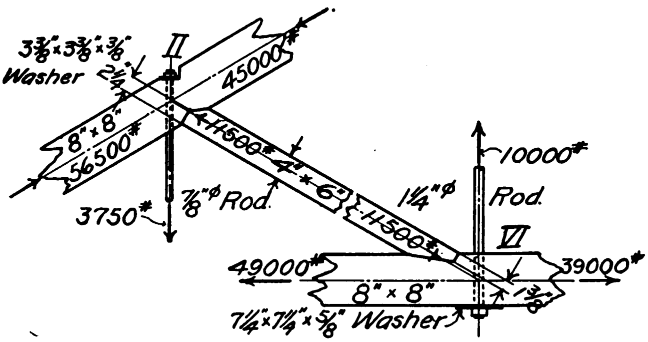

FIG. 48. INTERMEDIATE JOINT, TYPE B.

of cut is important, so that any detail reducing the depth of indent is to be favored.

There is also some eccentricity in the action of the various forces around the panel points in both details. For example, in Fig. 48, panel point No. VI, the horizontal component of the thrust of the strut acts at the centre of the toe, while the resultant tension in the lower chord acts at the centre of the uncut depth of chord. The moment of this couple is therefore (49,000 lb. – 39,000 lb. $=$ 10,000 lb.) $\times\ 3\frac{3}{8}$ in. $=$ 33,375 in.-lb. The vertical component of the stress in the strut is 6000 lb., and also may be taken as concentrated at the centre of the toe. This force forms a couple with the tension in the vertical rod (10,000 lb.) less the concentration at the panel point (4000 lb.), or a resultant tension of 6000 lb. The amount of the couple is therefore 6000 lb. times the horizontal

*See Chapter V.

distance between the centre of the rod and the centre of the normal cut on the strut, or 6000 lb. $\times$ $3\frac{3}{4}$ in. $=$ 22,500 in.-lb. These two moments are in opposite direction of rotation, therefore the resultant moment is 33,375 in-lb. $-$ 22,500 in.-lb. $=$ 10,875 in.-lb. The net section of the chord, taking out the hole for the rod and the dap in the chord, is 6 in. wide by $6\frac{5}{8}$ in. deep $=$ 39.7 sq. in. The net section modulus is $\frac{1}{6} \times 6$ in. $\times$ $(6\frac{5}{8}$ in.$)^2 = 43.8$. The resultant tension in the chord is therefore $\frac{49000}{39.7} + \frac{10875}{43.8} = 12{,}350$ lb. $+$ 248 lb. $=$ 12,598 lb. per sq. in. While the secondary stress is negligible in this case, it does not follow that it can always be ignored, and any truss designed for high unit working-stresses should

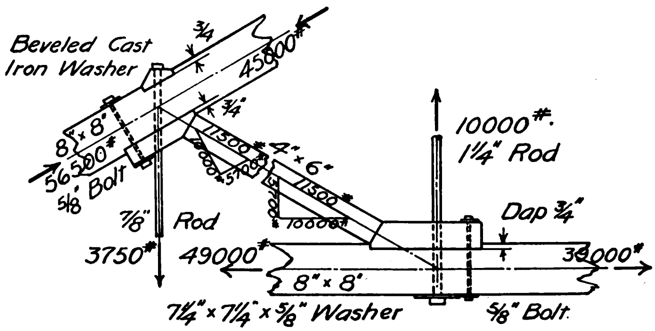

FIG. 49. INTERMEDIATE JOINT, TYPE C.

have its joints investigated for secondary stresses. It should also be borne in mind that any variation in the relation of the web members meeting in a panel point, resulting from careless detailing or framing may increase these secondary stresses to a considerable amount.

In roof trusses employing details of intermediate joints of Types A and B, it will often be found that, with the condition of the centre lines of all members meeting at a common point satisfied, the toes of the struts will either bear against the rod, or the hole for the rod will cut away part of the strut. Sometimes this condition cannot be avoided if the strut is to be dapped into the chord. If it so happens that the rod has not a driving fit in the chord, which condition will usually exist, especially with an upset rod and a deep chord, the toe of the strut will have bearing against the chord for only a part of its width. The result of this condition will be that the actual bearing area may not be over one-half of what was assumed in design, and the unit bearing stress may consequently be double the allowable.

Type C, Fig. 49. The disadvantages of details of intermediate joints of Types A and B, as shown in the preceding paragraphs are lacking in the detail of Type C, illustrated in Fig. 49. In this joint, the strut has a full bearing on the butt block, and the butt block, in turn, utilizes the total width of the chord for bearing. Also, the detail takes advantage of the full bearing pressure in end compression of the butt block on the chord, resulting in a minimum depth of cut into the chord. Nearly all the cuts are normal, and the others are simple. All the cuts can be easily and accurately laid out and made by the carpenter. The length of the butt block can be adjusted to fit all conditions of possible interference with other connections. Its minimum length is determined by longitudinal shear. The bolt through the end of the butt block holds the block securely in its socket. Whether there is any actual tension in the bolt depends upon the length of the butt block. This can be determined at once by inspection. If the line of the thrust of the strut falls within the base of the block, there can be no tension in the joint. However, it is well to provide at least a $\frac{5}{8}$-in. bolt to bind the joint together thoroughly. I have used this joint in many trusses of all types, and have found it to be an extremely satisfactory detail in all cases.

In Fig. 50, alternate details of intermediate joints are illustrated. These may be used for panel points No. II and VI, using the unit bearing pressures on inclined planes in accordance with the curve recommended by Howe, and shown in Chapter IV, Fig. 20. The lower values for bearing-pressures result in deeper chord indents than for the details of Fig. 47 and 48, and also necessitate increasing the strut from a 4 by 6-in. to a 6 by 6-in. timber, in the case of Type B, in order to provide sufficient bearing area against the upper chord.

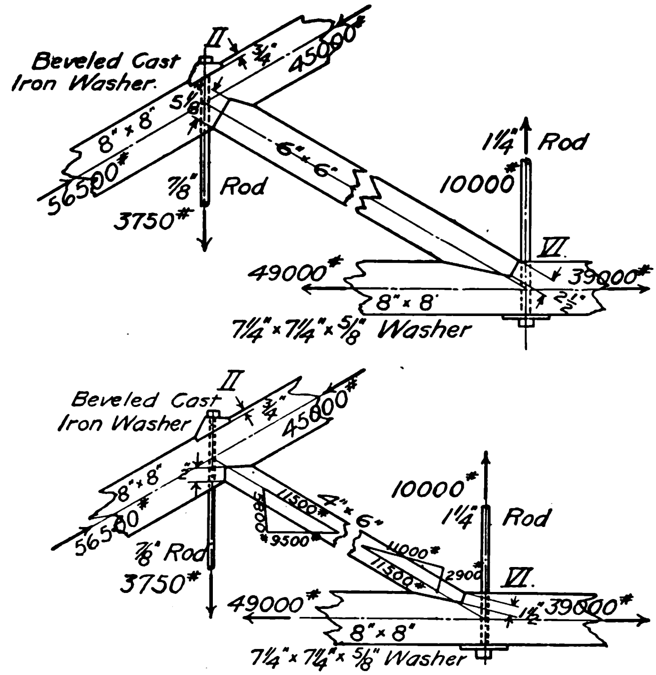

FIG. 50. INTERMEDIATE JOINTS DESIGNED BY HOWE'S FORMULA.

The detail calculations need not be repeated, as they are similar to those made for Fig. 47 and 48.

Applying the lower bearing pressures to the butt block detail, or Type C, it will be found that the 4 by 6-in. strut must be replaced by a 6 by 6-in. strut, in order to provide sufficient bearing area for the inclined cut of the butt block. The alternative would be to use a hardwood timber, such as oak, for the butt block.

This type of truss, with its small inclination of web struts to upper chord, will usually require attention in order that the joints may have sufficient bearing in accordance with the allowable unit working-stresses adopted.

CHAPTER VIII

Tension and Compression Splices

In Fig. 51 to 56 are presented details of six types of tension splices, the details shown being for the centre panel of the lower chord of the English roof truss of Fig. 38, Chapter VI. As in the case of the end-joint details for the same truss, the calculations for the design are fully shown in the figures. In all cases, the splice is designed for only the computed stress in the chord. This fact will influence any deductions that may be made regarding the comparative economy of the different types. It will be seen that in some cases it would be impossible to increase the capacity of the splice without weakening the main member beyond the allowable limit. Most specifications provide that all splices be made of sufficient strength to develop the main section of the member spliced, regardless of the possible smaller computed stress existing in the member.

The details here shown are not presented as covering the whole field of tension splices. They do show, however, some of the most efficient forms. Various modifications are possible, as for example, the substitution of square or rectangular keys of hardwood or metal for the shear pins for Fig. 56, the omission of the wooden splice plates of Fig. 53, and the tabling of the main member itself. For a description of the various forms of splicing timbers, including the lapped and scarfed splice, the reader is referred to the texts of Jacoby, Howe, Thayer, Kidder, and others.

The detailing of tension splices is a problem to be decided for the individual case, in conformity with the circumstances of importance of the connection, cost of materials, quality of workmanship to be expected, possibility of occasional inspection after completion, and the particular requirements of the splice. It will usually be found, however, as in the case of other truss connections, that certain details stand out as superior to the many that may be used, and that such type or types may be employed successfully for almost all the cases that will arise, with minor modifications.

For convenience of reference the details shown may be listed as follows: Fig. 51, the Bolted Fish-plate Type, Fig. 52, the Modified Bolted Fish-plate Type, Fig. 53, the Tabled Fish-plate Type, Fig. 54, the Steel-Tabled Fish-plate Type, Fig. 55, the Tenon-Bar Type, and Fig. 56, the Shear-Pin Type. The advantages and disadvantages of each type will be discussed briefly, in order that an intelligent selection may be made for any actual case.

Bolted Fish-Plate Type. The size of the bolts in this detail are computed in accordance with the formula $M = \frac{1}{2}P \times \left(\frac{t'}{2} + \frac{t''}{4}\right)$, where t' = the thickness of splice pad, or fish-plate, and t'' = the thickness of the main

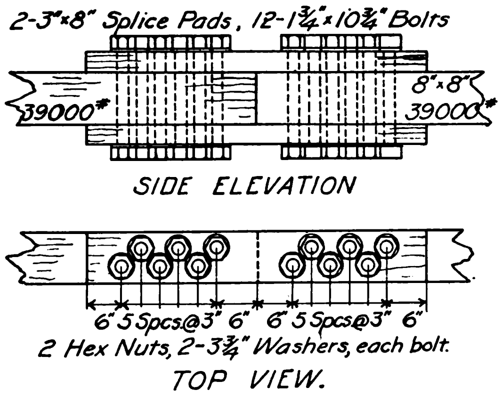

FIG. 51. BOLTED FISH-PLATE SPLICE.

COMPUTATIONS

$$\text{Net area required} = \frac{39{,}000 \text{ lb.}}{1500} = 26 \text{ sq. in.}$$

Since the end detail required an 8 by 8-in. chord, the splice pads or fish-plates will be made 8 in. deep. Plates 2 by 8 in. are not sufficient, so use 3 by 8-in. Assuming the bolts to be

1¾ in. diameter, the net area of fish-plates will be 6 in. × [8 in. − (2 × 1¾ in.)] = 27.0 sq. in., and the unit tension will be $\frac{39{,}000 \text{ lb.}}{27} = 1445$ lb. per sq. in. This calculation assumes the bolts spaced in pairs, and not staggered. However, with the large diameter of bolts used, the net section of chord should be figured as if the bolts occurred in pairs.

Six bolts are arbitrarily selected for each side of joint.

Bending moment on one bolt $= \frac{19500}{6} \times [(\frac{1}{2} \times 3 \text{ in.}) - (\frac{1}{4} \times 8 \text{ in.})] = 11{,}380$ in.-lb.

Required section modulus $= \frac{11380}{24000} = 0.474$ in.

$d^3 = \frac{0.474}{0.098} = 4.82$ in. and $d = 1.67$ in. Use 1¾-in. bolts.

The unit bearing pressure on the diametral section of the bolts $= \frac{39000}{1.75 \times 6 \times 6} = 619$ lb. per sq. in., which is less than one-half the allowable.

Distance Required Between Bolts	Inches per bolt
Total shearing area required $= \frac{39000}{150} = 260$ sq. in., or	3.61
Area required for transverse tension $= \frac{39{,}000 \text{ lb.} \times 0.1}{150 \times 6 \times 6}$	= 0.72
Adding diameter of bolt....................................	1.75
Required spacing of bolts....................................	6.08

Use 6-in.

Bill of Material for One Splice	Pounds
Twelve 1¾ by 18½-in. bolts at 12.6 lb....................	151.5
Twenty-four 1¾-in. nuts at 3.2 lb........................	76.8
Twenty-four 3¾-in. circular washers at 0.4 lb............	10.0
Total weight of steel	238.3

Two 3 by 8-in. pieces, 4 ft. 6. in. long = 18.0 ft. B.M.

Cost of One Splice as Detailed	
Steel, 238.3 lb. at $0.04..................................	$9.55
Timber, 18 ft. B.M. at $0.04..............................	0.72
	$10.27

Bill of Material for One Splice, Using 1¾-in. lateral pins.	Pounds
Twelve 1¾ by 15⅞-in. lateral pins, at 11.37 lb. (including nuts)	137.0

	Pounds
Twenty-four 3¾-in. washers at 0.4 lb	10.0
Total weight of steel	147.0
Timber as before.	

COST OF ONE SPLICE

Steel, 147.0 lb. at $0.04	$5.88
Timber as before	0.72
	$6.60

timber. As explained in Chapter V, this formula is that used by Jacoby and Howe, and is based on the assumption of uniform bearing of the timber along the length of the bolt. The use of this formula results in an excessive diameter of bolts being required, not only adding to the cost of the splice, but decreasing the capacity of the main timber for tension.

Besides the bending in the bolts, the net section of main timber and fish-plates must be investigated for sufficient area to resist the computed stress; the bearing pressure of the timber against the bolts must not exceed the allowable unit working-stress, the distance between bolts, and also the distance between any bolt and the end of the timber, must be sufficient for longitudinal shear on the timber, and also for transverse tension. In proportioning the splice for the latter stress, it may be assumed that the transverse tension tending to split the timber along the centre line of bolts is equal to one-tenth of the longitudinal stress in the chord. The working stress for transverse tension is taken at 150 lb. per sq. in.

For determining the necessary spacing of bolts, the net distance as required by longitudinal shear in the timber is found, and to this distance is added the net length required for resisting transverse tension. To the sum of these two is added the diameter of the bolt. This combined distance is the minimum that should be used. The distance of the last bolt from the end of the timber should theoretically be one-half that of the computed bolt spacing. On account of the tendency of timber to check at the ends, those bolts in the details acting in shear, as in the splice now under discussion, have been placed a distance of six inches from the end of the timbers.

In the figure, the bolts are shown as $1\frac{3}{4}$ in. diam., and full-size nuts are indicated. If they can be obtained at a reasonable cost, standard lateral bridge-pins will be cheaper, and an alternate cost estimate has been prepared on this basis. The actual diameter of the lateral pin is $1\frac{11}{16}$ in. instead of $1\frac{3}{4}$ in. The reduction in size is not important, since it is still within the computed necessary diameter.

The disadvantages of this detail are the large sizes of bolts, with a corresponding loss in efficiency of the total splice, as has been noted. As was discussed in Chapter V, the general theory on which the sizes of bolts has been computed is believed to be incorrect, and such a joint would seldom be used in an actual case.*

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*This statement needs some explanation. Where, as is the case under discussion, the pressure distribution on the bolt is assumed to be uniform, and the diameter of the bolt is then made of such a dimension that the bolt will have a resistance to bending sufficient to withstand the bending moment resulting from such uniform pressure distribution, the design cannot be said to be inconsistent, and it is believed that the action will be as assumed. Further, it may be said that no bolt of lesser diameter will give as high a total resistance per bolt of the joint. It is obvious that if the bearing of the timber on the bolt is uniform along the length of the bolt, and if the bolt is large enough to resist the resultant bending, the capacity of the joint is limited by the safe unit bearing-pressure of timber on a cylindrical metal-pin. If at the same time, the flexural strength of the bolt is attained, the inference might be drawn that the design was the most economical that could be made. Such an inference would be correct, were the price of metal the same for all sizes of bolts, or for stock bolts or lateral pins. The criticism that I make of the design under discussion is that bolts of a smaller diameter are not given credit for the resistance that they can develop. A joint framed with the bolts nearly two inches in diameter has the appearance of a monstrosity when actually viewed in the field, and always excites the ridicule of the carpenter. It is granted that the carpenter's opinion has no bearing on the case, if the design is correct. However, the carpenter in this

Modified Fish-Plate Type. The principles of design of this detail are the same as in the previous type, except that the sizes of bolts are proportioned from the values given in the previous article of this series. One-inch bolts have been chosen; there is no necessity for using this size as against either a smaller or larger diameter. In general, the fewer bolts there are to place, the less will be the cost of labor, and the more certain will be the combined action. Against these considerations must be weighed the amount of metal in the bolts, and the availability of the chosen size. Stock bolts are of course, cheaper than special sizes.

As the working values used here were taken from the results of tests in which the action of washers did not play a part, the splice is detailed with standard malleable washers, which will allow the joint to be drawn together fairly tightly without crushing the timbers. An estimate of an alternate detail, in which the bolts have been spaced at the minimum distance allowable, and in which standard pressed-steel washers are used has also been prepared.

The modified fish-plate splice is easily framed, and for many joints is the most economical, when all factors are considered. All bolts are to have a driving fit in the timber. This is a condition that can easily be obtained with good inspection. The simplest method of assuring a driving fit with bolts is to examine the size of the bit which the carpenter uses, and to see that all holes are bored from one side only. In case a bolt has not a driving fit, it should be withdrawn, and another bolt of the next larger size be used. For this reason, it is well to detail such joints with a slightly larger spacing of bolts than is actually required.

Tabled Fish-Plate Type. The tabled fish-plate joint for the case under consideration is simple and effective.

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instance knows what is undoubtedly true, that the designer did not realize that bolts of a smaller diameter are capable of developing much more resistance to lateral shear than is stated in the textbooks.

The stress of the chord and fish-plates is taken in tension, shear, and end-compression of the timber, with comparatively small secondary tension in the bolts. The bolts thus act in their most efficient manner, not being

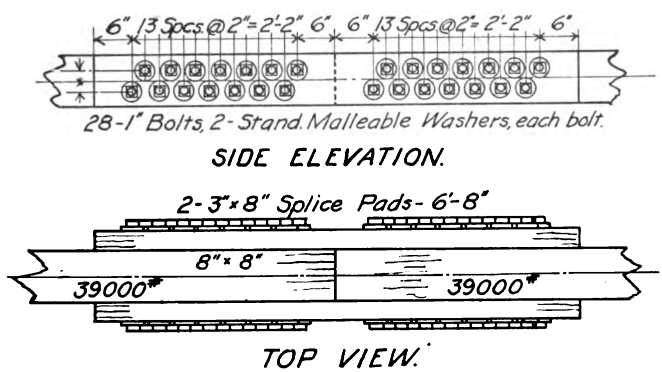

FIG. 52. MODIFIED FISH-PLATE SPLICE.

COMPUTATIONS

Bolts of 1 in. diameter will be used. The strength of one bolt in double shear with a thickness of fish-plate of 3 in. is, from Chapter V, 2664 lb.

Number of bolts required $= \frac{39000}{2664} = 14.6$. Use fourteen 1-in. bolts.

Distance required between bolts (total shearing area required, as before, 260 sq. in.):

	Inches
Spacing of bolts for shear $= \frac{260}{14 \times 6 \times 2} =$	1.55
Spacing required for transverse tension $= \frac{39000 \times 0.1}{150 \times 14 \times 6} =$	0.31
Adding diameter of bolts	1.00
Required spacing of bolts	2.86

Bolts will be spaced 2 in. staggered.

Required area of chord and plates for tension need not be investigated.

BILL OF MATERIAL FOR ONE SPLICE

	Pounds
Twenty-eight 1 by 16½-in. bolts at 4.81 lb...............	135.00
Fifty-six 1-in. standard malleable washers at 0.75 lb.....	42.00
Total weight of steel.............................	177.00

Two 3 by 8-in. pieces 6 ft. 8 in. long = 28.00 ft. B.M. timber.

Cost of One Splice

Steel, 177 lb. at $0.04	$7.08
Timber, 28 ft. B.M. at $0.04	1.12
	$8.20

For a rigid comparison with the previous type, the spacing of the bolts would be decreased to 3 in., and circular steel pressed washers used. The bill of material and cost would then be as follows:

Bill of Material

	Pounds
Twenty-eight 1 by 15½ in. at 4.60 lb	128.6
Fifty-six 1-in. circular washers at 0.16 lb	9.1
Two 3 by 8-in. pieces 5 ft. 3 in. long = 21.0 ft. B.M.	137.7

Cost of One Splice

137.7 lb. steel at $0.04	$5.55
21 ft. B.M. timber at $0.04	0.84
	$6.39

subjected to lateral forces. All cuts of the timber are square, and where the amount of the stress to be transferred across the joint in the chord can be taken by not more than two tables on either side of the chord joint, the detail may be regarded as reasonably certain in its action. Washers of generous size must be provided, in order that the joint may be well pulled together at the time of framing and the bolts be able to hold the tables in place when the stress comes into the splice. The calculations for Fig. 53 show a moment of 39,000 in.-lb. to be counteracted by the tension of the bolts in each table, acting about the vertical cut in the chord, or the bearing end of each table. It is obvious that if the bolts should fail to hold the fish-plates in place, this moment would have to be taken by the plates acting as beams in flexure. The net section modulus of the plate at the plane of the cut for the table is $\frac{1}{6} \times 8\text{ in.} \times (2\frac{1}{2}\text{ in.})^2 =$ 8.35. The flexural stress would therefore be $\frac{39,000\text{ in.}}{8.35} =$ 4680 lb. per sq. in., and the maximum stress on the fish-plate would be 4680 lb. $+ \frac{19,500\text{ lb.}}{2\frac{1}{2} \times 8} =$5655 lb. per sq. in.,

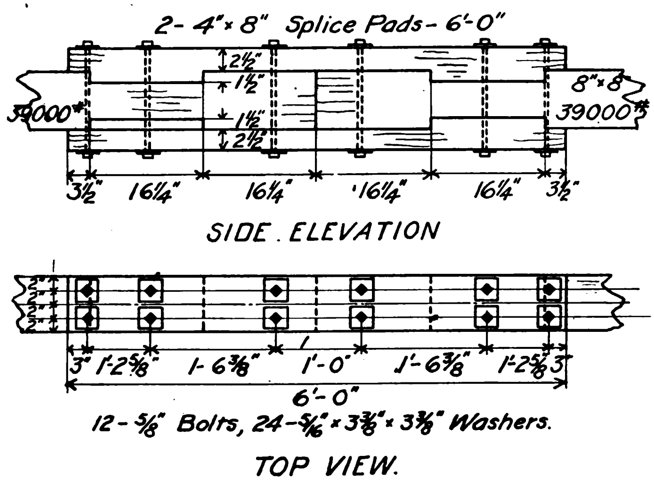

FIG. 53. TABLED FISH-PLATE SPLICE.

COMPUTATIONS

Depth of cut for table and chord: Area required for cut $= \dfrac{39000}{1600 \times 2 \times 8} = 1.52$ in. Make $1\frac{1}{2}$ in.

Length of table for shear: Area required $= \dfrac{39000}{8 \times 2 \times 150} =$ 16.25 in. Make $16\frac{1}{4}$ in.

Size of bolts required: The resultant stress in the fish-plate acts at the centre line of the uncut portion, while the resultant of the pressure of the table on the chord acts at half the depth of the cut. The total thickness of fish-plate should be 4 in., since a 3-in. piece of timber would not give sufficient area for tension. There is thus a couple acting on the fish-plate equal to one-half the stress in the chord multiplied by one-half the thickness of the fish-plate, or 19500 lb. $\times$ 2 in. $=$ 39,000 in.-lb. This moment must be resisted by tension in the bolts acting about the bearing face of the tables. The bolts should be placed at the centre of the tables. Their lever arm is therefore 8 in., and their stress $\dfrac{39000}{8} = 4875$ lb. Two ⅝-in. bolts will be provided. In addition, for binding the joint together, eight ⅝-in. bolts will be placed as shown in the detail.

BILL OF MATERIAL FOR ONE SPLICE

	Pounds
Twelve ⅝ by 14½-in. bolts at 1.50 lb	18.00
Twenty-four washers 5/16 by 3⅜ by 3⅜ in. at 1.02	24.50
Total weight of steel	42.50

Two 4 by 8-in. pieces 6 ft. long = 32.00 ft. B.M. timber.

COST OF ONE SPLICE

Steel, 42.50 lb. at $0.04	$1.70
Timber, 32 ft. B.M. at $0.04	1.28
	$2.98

a value far beyond the allowable safe stress. The joint is therefore dependent to a large degree on the tightness with which the timbers are held in place by the bolts, and excessive shrinkage in the timber would allow the fish-plates to be overstrained. In such a joint, if thoroughly seasoned timber is not certain to be employed, the fish-plates should be given a generous section, and additional bolts over those required by the computations should be provided. Spiking in the form of toe-nailing will also assist in holding the fish-plates in place. The bolts resisting the tension due to the incipient bending should be placed at the centre of the tables, in order that the fibres of the fish-plates will receive equal bearing under the washers.

Steel-Tabled Fish-Plate Type. The calculations necessary in the design of this type of splice are similar to those of the Type C end joint. The net area of steel in the plates, the bearing area of tables, number of rivets in the tables, their spacing for longitudinal shear on the timber, the number of bolts to hold the tables in position, and the net area of timber must all be sufficient to hold their respective stresses.

The splice is an effective one, and is fairly economical where good work in the fabrication of the metal can be obtained. The detail works well for joints carrying heavy stresses. The objections that may be offered to the splice, outside those of cost of materials, are the number of tables that may be required, necessitating careful fitting into the timber, in order that snug and uniform bearing may be assured between steel and timber. For this reason the detail may be listed in the class that especially requires good and careful inspection on the part of the engineer.

Tenon-Bar Type. The bar and tenon splice is one of

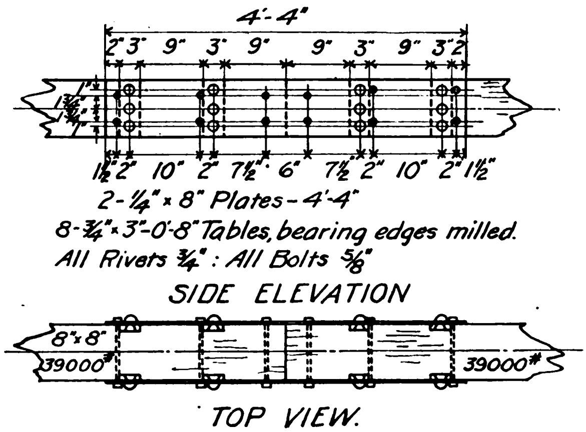

FIG. 54. STEEL-TABLED FISH-PLATE SPLICE.

COMPUTATIONS

Bearing area required for tables $= \dfrac{39000}{1600} = 24.4$ sq. in.

Total combined depth of tables $= \dfrac{24.4}{2 \times 8} = 1.53$ in. Make $1\frac{1}{2}$ in.

Will use tables $\frac{3}{4}$ by 8 in., requiring eight tables in all.

Each table transmits $\dfrac{39000}{4} = 9750$ lb., and requires three $\frac{3}{4}$-in. rivets, as determined by bearing on a $\frac{1}{4}$-in. plate.

Net section of $\frac{1}{4}$ by 8-in. steel plate $= \frac{1}{4} \times [8 \text{ in.} - (3 \times \frac{7}{8} \text{ in.})] = 1.34$ sq. in.

Net section of one plate required $= \dfrac{39000}{2 \times 16000} = 1.22$ sq. in.

Size of bolts required to resist moment on tables: Moment $= 9750$ lb. $\times \frac{1}{2}$ in. $= 4857$ in.-lb. Tension in bolts $= \dfrac{4875}{3\frac{1}{2}} =$ 1400 lb. Will use two $\frac{5}{8}$-in. bolts.

BILL OF MATERIALS FOR ONE SPLICE

	Pounds
Two $\frac{1}{4}$ by 8 by 4 ft. 4 in. plates at 29.4 ft.	58.8
Eight $\frac{3}{4}$ by 3 by 8-in. tables at 5.1 lb.	40.7
Twenty-four $\frac{3}{4}$-in. rivet-heads at 0.14	3.4
Twelve $\frac{5}{8}$ by $9\frac{1}{2}$-in. bolts at 1.14	13.7
Total weight of steel	116.6

COST OF ONE SPLICE

Steel, 116.6 lb. at $0.04	$4.67

the older types of timber splices, and was formerly used to considerable extent in bridge work, but is not often seen at the present time. It is distinguished from all other splices by its simplicity and directness. There is but one bearing surface, consequently the area taken out by the bar is a large part of the gross area of the chord. As the bar is rectangular in shape, the full end-bearing value of the timber can be taken advantage of, and there is no cross-tension on the timber tending to split the chord. The detail computations consider the size of bar for bearing against the ends of the fibres, and for bending in the bar, the required distance between the bar and the end of the timber for shear, the net section of chord, and the area of tension bolts, using, of course, the area at the root of threads. It should be observed that the length of the bar is determined by the long diameter of the hexagonal nut of the bolts, so that sufficient distance may be obtained for tightening the nuts. As the bar is a short beam in bending, the high unit flexural-stress of 24,000 lb. per sq. in. is permissible. For holding the splice firmly together, two 2 by 8-in. pads have been provided, bolted through the chord. This will be necessary wherever a single stick is to be spliced. In the case of a built-up chord, such as is usual with a railroad or highway bridge of long span, the entire chord is never spliced at one point, the splices in the individual timbers of the chord being staggered. Packing blocks are provided between the sticks, through-bolts being used to bind the whole together thoroughly.

Shear-Pin Type. In this detail, the tension is transmitted by shear in the pipe or hardwood pins, and a consequent secondary tension in the bolts. The working values are in accordance with the results of tests, as has already been described in Chapter IV. With thoroughly seasoned timber, the detail is a reliable one. It should be remembered that this detail should not be employed with very green timber, as the ability of the pins to transmit shear is a function not alone of the end-bearing value of the timber, but also of its strength in

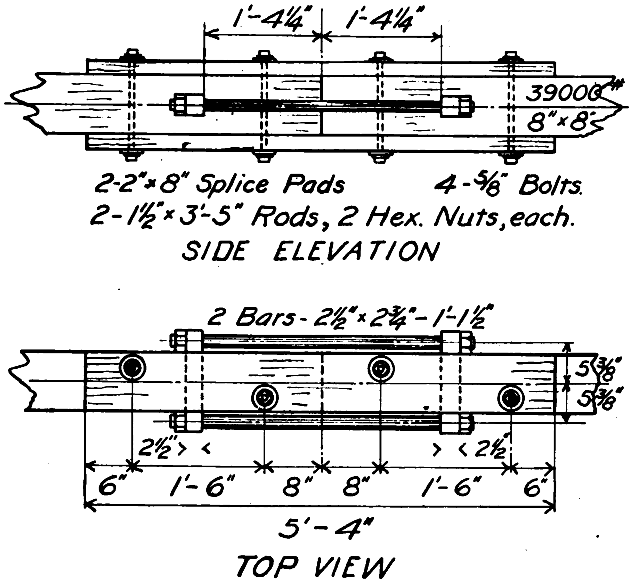

FIG. 55. TENON-BAR SPLICE.

COMPUTATIONS

Size of rod: area required $= \frac{19{,}500 \text{ lb.}}{16{,}000} = 1.22$ sq. in.

A $1\frac{1}{2}$-in. rod has an area at root of thread of 1.295 sq. in. Use this size.

The long diameter of a $1\frac{1}{2}$-in. hexagonal nut is $2\frac{3}{4}$ in., hence the distance from the side of the timber to the centre line of the bolt must be slighty more than $1\frac{3}{8}$ in. Will make this distance $1\frac{7}{16}$ in.

Size of bar required: The bearing of the timber against the bar will be assumed to be uniform per unit area of bearing. Hence the bending moment on the bar will be 19,500 lb. $\times$ [$1\frac{7}{16}$ in. + ($\frac{1}{4} \times 8$ in.)] = 19,500 lb. $\times$ 3.4375 in. = 67,000 in.-lb.

The required bearing area is $\frac{39{,}000 \text{ lb.}}{1{,}600} = 24.4$ sq. in.

The required width of bar is therefore $\frac{24.4}{8} = 3.07$ in. Use 3 in.

Use a fiber stress of 24,000 lb. per sq. in., since the case is that of a short beam restrained at the ends to some extent.

The necessary section modulus is $\frac{67000}{24000} = 2.79$ in.

$\frac{bh^2}{6} = 2.79 = \frac{3 \times h^2}{6}$. Therefore $h = 2.37$ in.

Will use bar $2\frac{3}{8}$ by 3 in.

The shearing length required, or the distance between the edge of bar and the end of the timber, is $\frac{39000}{150 \times 2 \times 8} = 16.23$ in. To this distance will be added one-half the width of the bar, making the distance from the centre of bar to the centre of splice, say, 1 ft. $5\frac{1}{2}$ in.

Bill of Material for One Splice

	Pounds
Two steel bars, $2\frac{3}{8}$ by 3 in. by 1 ft. $5\frac{5}{8}$ in. at 35.6 lb........	71.2
Two $1\frac{1}{2}$-in. rods, 3 ft. $4\frac{1}{2}$ in. long at 20.2 lb...............	40.4
Four hexagonal nuts at 2.0 lb...........................	8.0
Four $\frac{5}{8}$ by $13\frac{1}{2}$ in. at 1.4 lb.............................	5.6
Eight $\frac{5}{8}$-in. standard malleable washers at 0.23 lb.........	1.8
Total weight of steel................................	127.0

Two 2 by 8-in. pieces 5 ft. 6 in. long = 15 ft. B.M. timber.

Cost of One Splice

Steel, 127 lb at $0.04....................................	$5.08
Timber, 15 ft. B.M. at $0.04............................	0.60
	$5.68

cross bearing. In the action of the splice, there is a couple on the pin, tending to spring it out of its hole. Any shrinkage in the timber will allow some slip of the joint, because of the action described.

General Summary of Tension Splices. As was stated in the case of the estimates of costs of the different types of end joints, the figures representing the costs can be regarded as only approximate. The actual amount of labor required for each type of detail, whether end joint or tension splice, is difficult to estimate accurately. In the costs given herein, all timber in place has been figured at the same rate, and the same statement applies to the steel, whether such steel is forged, riveted, or is in the form of bolts. This assumption would be justified only in the case of a very large job. On a small job comprising only a few roof trusses, this method of estimating costs would probably be seriously in error. Again, prices of material fluctuate, not alone in relation to time, but also with the situation of the job. It will be recognized, therefore, that not alone must the prices of the different types of tension splices be taken as only approximate, but that their relative costs must be regarded as only comparatively accurate.

While the relative cost of any one type of tension splice

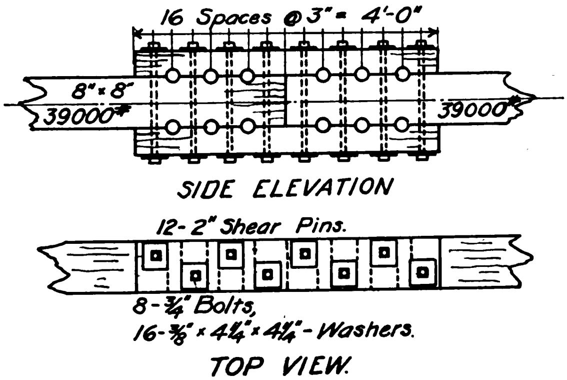

FIG. 56. SHEAR-PIN SPLICE.

COMPUTATIONS

Using fish-plates of 3 by 8-in. timbers, the net section of plates will be 4 by 8 in. = 32 sq. in. The unit tensile stress in the plates will then be $\frac{39000}{32} = 1216$ lb. per sq. in.

The number of 2-in. pins required will be $\frac{39000}{8 \times 800} = 6.1$. Use six pins.

The tension to be taken in the bolts will be 19,500 lb., requiring $\frac{19500}{4830} = 4.03$ or four ¾-in. bolts.

For developing the bolts, plate washers, ⅝ by 4¼ by 4¼ in. will be used.

BILL OF MATERIAL FOR ONE SPLICE

	Pounds
Eight ¾ by 15½-in. bolts, at 2.32 lb.	18.56
Sixteen washers, ⅝ by 4¼ by 4¼ in., at 1.92 lb.	30.70
Total weight of steel	49.26

Twelve 2-in. pipe-pins or hardwood pins.

Two 3 by 8-in. pieces 4 ft. long = 16 ft. B.M. timber.

COST OF ONE SPLICE

Steel, 49.3 lb. at $0.04	$1.97
Twelve pipe-pins at $0.10	1.20
Timber, 16 ft. B.M. at $0.04	0.64
	$3.81

is a vital factor to be considered, other considerations than cost alone will generally decide the detail to be used. For example, where the roof truss is to be exposed, wooden splice-pads may be objectionable from the standpoint of appearance. In such a case, steel plates are a necessity, unless a laminated chord be used, and the problem will then resolve itself into a question of using a bolted-steel fish-plate splice, or a tabled-steel fish-plate.

In the case of joints in which the stresses to be resisted are comparatively small, the modified bolted fish-plate splice will be found satisfactory. In the case of the truss illustrated here, the number of bolts required is too large from a practical standpoint, and the tabled fish-plate detail is, perhaps, the most satisfactory of the types shown. Where the stresses are still larger, the tabled-steel fish-plate will be found to offer an economical solution. Of the various types of splices illustrated, the bar-tenon type is the only one that is practically free from the effects of shrinkage of the timber. Next may be classed the bolted fish-plate, followed by the steel-tabled fish-plate, the wooden-tabled fish-plate, and lastly, the shear-pin splice.

Shrinkage is a factor that is almost impossible to avoid. For this reason, in all timber design, it is well to use conservative stresses. In the case of a roof truss, there usually exists a considerable safety factor in the live load assumed in the design. Where the timber work is protected from the weather, and at the same time is accessible for inspection, the joints should be carefully watched, and the bolts tightened as the timber shrinks. If the structure is a building that is heated, the full shrinkage may occur in a few weeks. Green timber may take one or two years to season completely. It should be emphasized, therefore, that thoroughly seasoned lumber only should be used for construction which will not be accessible for inspection and maintenance.

In addition to the requirements of computed stresses in tension splices, the unknown stresses of erection must be provided for. A splice joint should always, therefore, have some general stiffness in addition to its capacity to resist the known stresses. An illustration of this statement is seen in the detail of the bar-tenon type, in which two 2 by 8 in. splice-pads are used, although not required theoretically. The general statement may be made that, where it is possible to secure chords of full length so that splicing may be eliminated, it is better to use the long sticks, even at a considerable increase in the unit cost of the timber.

Compression Splices

Compression splices may be divided into two classes, those which take compression only, and those which may be called upon at some time to take either flexure or tension, or a combination of each.

As in the case of tension splices, it is not the purpose to discuss here the many types of joints which are employed in timber construction. Each type has its advantages and disadvantages. The reader who is interested in the subject will find a very complete description and discussion of timber joints in Chapter II of Jacoby's 'Structural Details.' Three of the most common types of compression splices are shown in Fig. 57. These may be termed the butt joint, the half lap, and the oblique scarf, respectively. The figure illustrates the fundamental difference between the butt type and all others, namely, that the former has only one surface of contact, and the others two. In accordance with the principle already mentioned, that all timber joints should be made as simple of fabrication as possible, the butt joint is superior to the others, whose efficiency is largely dependent upon the experience and the care of the carpenter in framing. It is self evident that one bearing surface is always better than two.

In the butt joint shown, the thickness of the splice-pads and the number and size of the bolts may be varied to accommodate the conditions existent in the member, whether the splice-pads are required only to hold the main timbers firmly in position, or must transmit tension or compression across the joint. In cases where the main member must resist considerable flexure, it may be necessary to use metal or hardwood shear pins in addition to the bolts.

The oblique scarf has more flexural strength than the half-lap; otherwise I consider it much inferior to the half-lap. There is much less timber in straight end-bearing in the oblique scarf than in the half-lap, and the

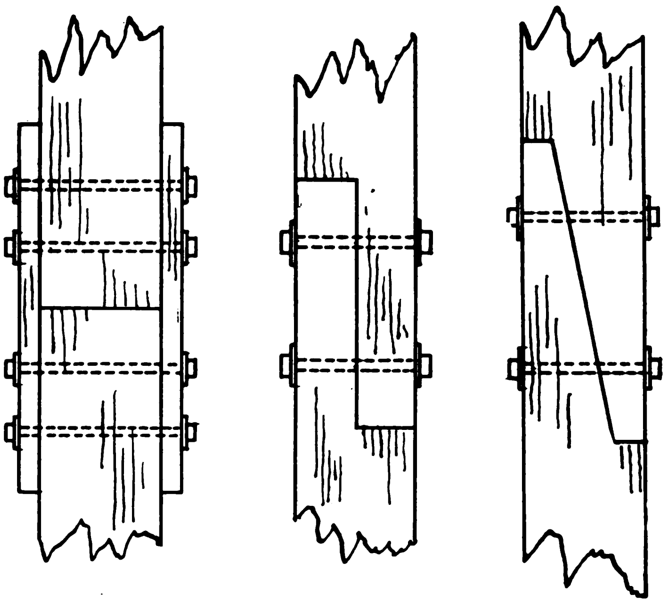

FIG. 57. TYPES OF COMPRESSION SPLICES.

oblique cut, if unresisted by the normal cuts would tend to separate the two timbers.

In erecting a structure, in which the member to be spliced is vertical in position, the bolts through one end of the splice pads should be placed in the field, so that the joint may come to a full bearing before the bolt holes are bored.

Fig. 58 and 59 show two details of the upper-chord joint of a small timber highway bridge, in which the batter-post frames into the upper chord. Here again the comparison may be made between a somewhat complicated detail, as illustrated by Fig. 58, and a joint in which but one straight cut is required, as in Fig. 59. The latter detail fulfills all the functions of the former, and there seems to be no need for the double cut of the first detail. If any further fastening between the chord and batter post is required than is given by the detail of Fig. 59, it is best obtained by splice pads across the joint, either of steel or timber.

General

All of the joints of the present and previous chapters are of the class that may be termed the open type, as

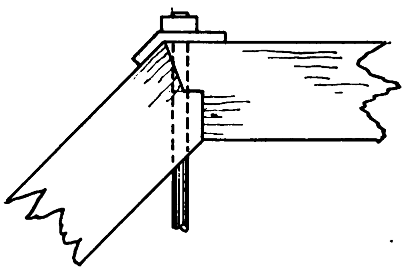

FIG. 58. COMPLICATED UPPER-CHORD DETAIL.

opposed to the closed, or housed class, the latter embracing the simple housed, the cogged, the halved, the dovetailed and other joints. The housed joints, with the exception of the halved joints, are seldom seen in building construction. The housed joints are used in mine tim-

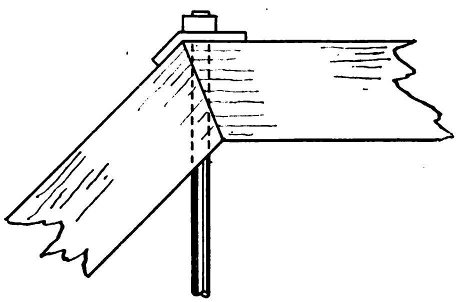

FIG. 59. SIMPLE UPPER-CHORD DETAIL.

bering, both above and underground, and also in cribbing. Aside from the fact that they are more difficult to construct, they offer more chance for decay than the simple open joints, and their efficiency is but a small proportion of the total strength of the timber. For example, the main timber is badly cut by an entrant tenon, compression across the fibres is introduced, and the effect of cross shrinkage of the timber is a maximum. The use of such joints requires large timbers working at a low unit-stress, or, in other words, at a low efficiency.

CHAPTER IX

Main Members of Trusses

Compression Chords and Struts. The importance which is attached to the details of a truss may be inferred from the fact that their design has been discussed before that of the main members. A case where the details of a truss are amply sufficient for the stresses that may come upon them, and the main members are insufficient, seldom if ever occurs.

The present treatment of compression members may be divided into two sections; first, a discussion of solid timber struts subject either to concentric compression alone, or to a combination of concentric compression and cross-bending, and second, a discussion of built-up timber struts, straight or curved, and taking either simple compression or compression combined with bending. The bending in either case may be due to an eccentricity of the primal stress or may be produced by transverse loading.

The calculations for the design of intermediate truss-joints, as given in the previous article, indicated that the struts of a timber truss are usually dependent on the required area for end-bearing, rather than upon the allowable working-stresses from the standpoint of column action. This condition will prevail in the average roof-truss encountered in practice, where the length of the struts is short.

If, in a building truss, the roof joists rest directly upon the upper chord and are rigidly fastened thereto, the latter may be considered to be supported by the joists, and column action will not enter into the determination of its section. When, on the other hand, the roof joists frame parallel to the truss, or when the joists do not rest directly upon the upper chord, the chord must be considered as a column, and its allowable unit stress determined by the application of a column formula.

Formulas giving the safe working-stresses for timber columns are to be found in almost every text-book on structural engineering, and in most specifications for steel structures. The two formulas most commonly used are those of the American Railway Engineering Association, and the U. S. Department of Agriculture, Forestry Division. The first mentioned is expressed as follows:

$$p = C\left(1 - \frac{1}{60}\,\frac{L}{d}\right), \text{ where}$$

$p =$ working unit stress, in pounds per square inch.

$C =$ safe fibre stress in end compression, in pounds per square inch.

$L =$ length of column in inches.

$d =$ least diameter or dimension of column in inches.

The U. S. Department of Agriculture formula is more complicated. It is

$$p = C\left[\left(\frac{700 + 15c}{700 + 15c + c}\right)\right]$$

In this expression, p and C represent the same quantities as in the formula above. In addition,

$c = \frac{L}{d}$, where

$L =$ length of column in inches, and

$d =$ least diameter or dimension of column in inches.

Two other formulas may be quoted, that of Milo S. Ketchum for mill buildings, and that of the Seattle Building Ordinance, 1914. Using the same nomenclature as before, these two formulas are

$$p = C\left(1 - \frac{1}{100}\,\frac{L}{d}\right), \text{ (Ketchum)}$$

$$p = C\left(1 - \frac{1}{70}\,\frac{L}{d}\right) \text{ (Seattle)}$$

The value of C recommended by the American Railway Engineering Association for railroad bridges and trestles is 1200 lb. per sq. in. for Douglas fir. For high-

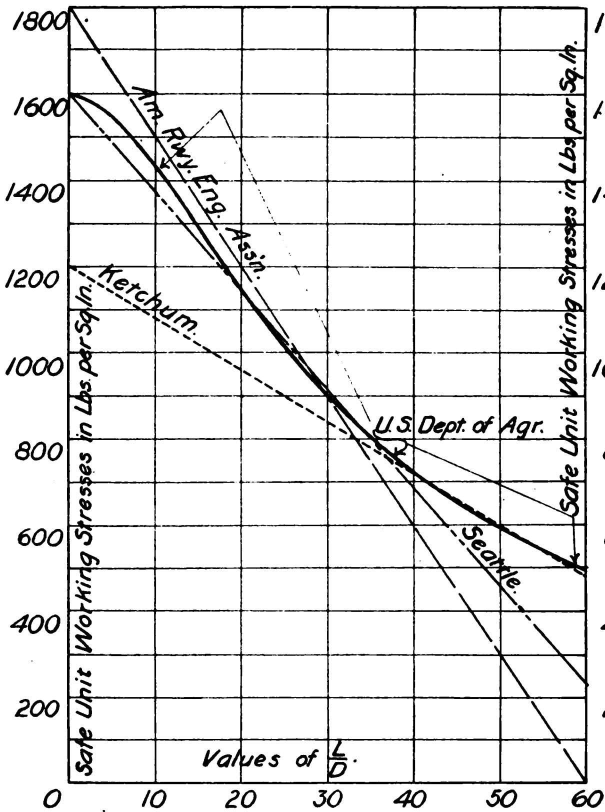

Fig. 60. COMPARISON OF COLUMN FORMULAS.

way bridges and trestles, C may be increased to 1500 lb. per sq. in., and for buildings a value of 1800 lb. per sq. in. is allowed. Ketchum's recommended value of C for Douglas fir is 1200 lb. per sq. in., while that of the Seattle Ordinance is 1600 lb. per square inch.

For purposes of comparison, these various formulas have been platted in Fig. 60, for the case of use in a building structure, protected from the weather. With the exception of Ketchum's formula, the values as given by the various formulas are not far different between the limits of 15 $\frac{L}{d}$ and 35 $\frac{L}{d}$. Sixteen hundred pounds per square inch is the value recommended in this article for use with Douglas fir in building construction with a grade of lumber of No. 1 Common. The selection of any one of these formulas as against the others for use in designing will usually not materially affect the section of the strut or column. The value of $\frac{L}{d}$ should not exceed 60 for any column or strut.

When computing the necessary size of the upper chord of a truss, the area of sections taken out by rods, bolts, washers, etc., must not be forgotten, and the proper allowance must be made for these losses of effective area. The fit of the web compression-members or the butt-blocks, if used, may be assumed to be such that the joint is 100% efficient, if an allowance for their cuts would mean a serious increase in cost. Otherwise, it will be better to make some allowance for poor fitting. The relation of actual load on the truss to probable load, as well as other considerations make this point one to be decided by the engineer for the particular case in hand.

Composite or Laminated Compression Members. In beginning this discussion, the general statement may be made that composite or laminated columns and struts, that is, columns built up of dimension stock, and spiked or bolted together, should be avoided whenever possible. In designing roof trusses for armories, skating-rinks, etc., it is often necessary to use trusses with arched chords. In such instances, laminated chords acting in compression and in tension may be a necessity. The requirement of bending the individual members of the chords to a curve of a comparatively short radius decrees that these members be either built of boards or of comparatively thin planking. The peculiar complications of such a truss will be discussed a little later.

For the general case of built-up columns, it may be argued that the average quality of timber in such a composite column is higher than that of a solid stick. On the other hand, and by far counter-balancing this small advantage, is the practical impossibility of making the built-up column act as a single stick. Composite columns may be separated into two classes, the first class comprising those constructed of a number of boards or planks, laid face to face, and bolted or nailed together, and the second class, consisting of those columns built of several laminations, with their edges tied together by cover plates. These two classes are illustrated by Fig. 61, *a* and *b*. Tests* have shown conclusively that the first class, when bolted together at the ends and the middle, will act as individual sticks. Expressed in another way, it may be said that the strength of a composite

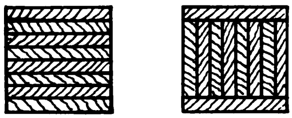

FIG. 61, *a* AND *b*. TYPES OF LAMINATED COLUMNS.

column, without cover-plates, and bolted together, is the sum of the strengths of the individual boards or planks, acting as separate columns with a length equal to that of the whole column. When, in place of, or in addition to, the bolting, such laminated sticks are spiked together thoroughly, the total strength of the column is in excess of the sum of the individual sticks.

In an endeavor to throw some light on this subject, I made a few tests in 1915 on some small composite columns. The results were published in *Engineering News,* Vol. 75, No. 7, February 17, 1916. Five built-up columns, constructed in three different ways, with a 3 by 4-in. section, and 23 in. long, were tested in compression to failure, and for comparison, two solid timbers, of the same cross-section and length, were also tested. The details of the test columns are shown in Fig. 62, while the load-deformation curves are shown in Fig. 63. The lumber was No. 1 common Douglas fir, with ends true and square, and surfaced. The individual boards were sur-

*See 'The Elasticity and Resistance of the Materials of Engineering,' by Wm. H. Burr, 1905 edition, pp. 539-541; 'The Materials of Engineering,' by J. B. Johnson, pp. 682-683; also 'Structural Details,' by H. S. Jacoby, pp. 210 and 217.

faced on one side. Fig. 64 shows the columns after failure.

The column of Type *b* was found to be as strong as the single stick. At the ultimate load, however, the individual pieces separated somewhat. The failure in this

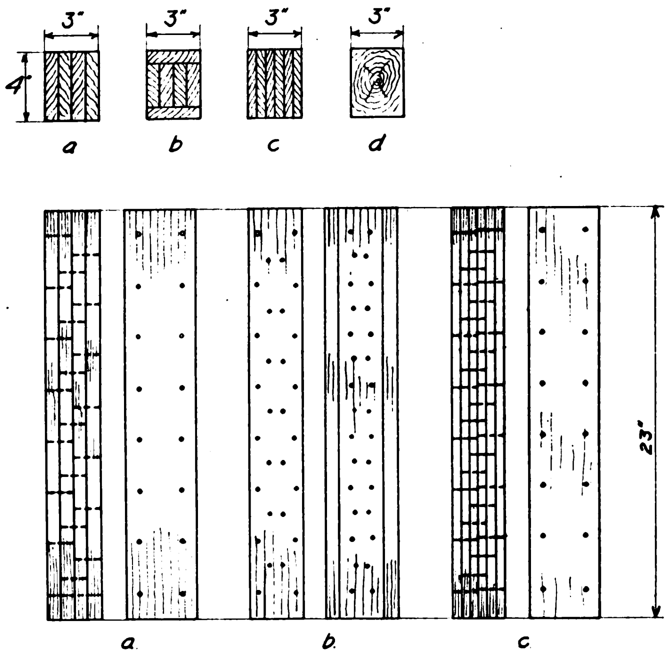

FIG. 62. DETAILS OF TEST COLUMNS.

column was a combination of crushing, resulting from straight compression, and of tension, due to the bending of the individual sticks. The columns of Type *c* were far deficient in strength as compared to the other types. The ratio of length to least diameter of the individual boards of the *c* columns was 46, while the corresponding quantity for the *a* columns was 31. The $\frac{L}{d}$ for the solid sticks was $7\frac{2}{3}$. The ultimate strengths of these three types of columns as computed by the formula of the U. S. Department of Agriculture, assuming the ultimate strength of the timber in end-compression to have been 4500 lb. per sq. in., and assuming the sticks to have acted as individual columns, would be as follows:

TABLE XVII

	Type *a*	Type *b*	Type *c*
$\frac{L}{d}$	31	31	46
Ultimate strength (computed)	29,600 lb.	29,600 lb.	21,400 lb.
Ultimate strength (actual)	49,000 lb.	50,000 lb.	38,000 lb.
Efficiency	98%	100%	76%
Average of values of line (2) and strength computed as solid sticks	39,900 lb.	....	35,800 lb.

While the small number of the tests, and the diminutive size of the specimens does not warrant forming too

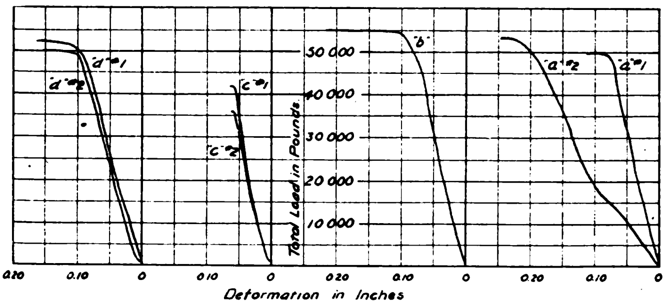

FIG. 63. LOAD-DEFORMATION CURVES OF LAMINATED COLUMNS.

definite conclusions to be extended to large-size columns, it is evident that the columns of Type *b* or the 'cover-plate' type are much superior in strength to the plain laminated type. This is what might be expected from a theoretical standpoint. An inspection of the values of the ultimate strengths as given in Table XVII indicates that the actual strengths of the columns of Types *a* and *c* are not far from the mean of the strengths computed first as a solid stick and then as a summation of individual sticks. In this connection, it is well to note that the spiking of these composite columns was exceedingly thorough, and such effective spiking could not be expected in actual construction. Even though equivalent spiking were to be specified and shown on the drawings of actual columns, it would be a Herculean task for the inspector to secure this result in the field. When, in addition to the practical impossibility of obtaining sufficient nailing, it is remembered that in the case of a laminated chord of a truss, and also in the case of many composite columns, the individual boards or

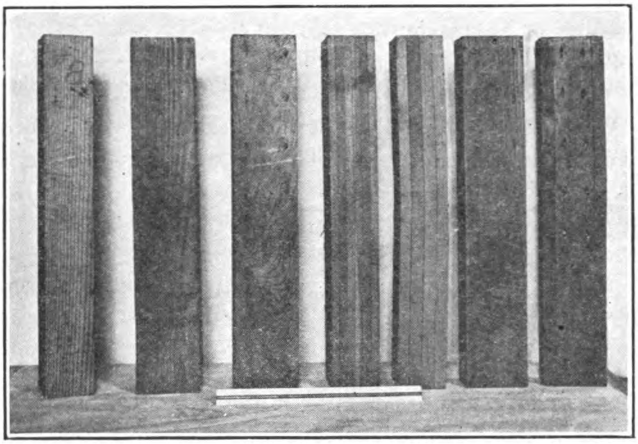

FIG. 64. TEST-COLUMNS AFTER FAILURE.

planks splice at various points throughout the length of the chord or column, it will be realized that another element of weakness is introduced, namely, the failure of the carpenter invariably to secure perfect butt-joints in the splices. Any such imperfect splice will, of course, put an additional stress upon the spikes, which must then transmit the load of the spliced timber to the adjoining boards.

From a consideration of the above factors, and until further tests prove otherwise, I recommend that the strength of a composite column of the type of Fig. 61*a* be taken at 80% of the mean of the strengths computed (1) as a solid stick, and (2) as a summation of the strengths of the individual sticks considered as individual columns. For columns of the type as illustrated in Fig. 61*b*, or the 'cover-plate' type, I recommend that the strength be taken as 80% of that of a solid stick of equal cross-section and length.

Curved Laminated Truss-Chords in Compression and Tension. The preceding discussion has considered only straight columns or struts. The much more complicated case of a curved laminated truss-chord must now be treated. The subject is one that is generally avoided in the few text-books on timber-framing, or at best is dismissed with brief mention. However, as has been stated in a former paragraph, the case of a laminated truss-chord, acting in compression or in tension, is one that occurs frequently, and it therefore becomes of vital importance to establish, if possible, average safe working-stresses for such chords. In addition to the average unit-stress on the section, there is introduced the complication of secondary stresses. Due to the fact that in the solution for the stresses of such a truss, the chords are assumed to be straight between panel-points, when actually they are curved, there is produced a bending in the chords, equal to the total main stress in the chord multiplied by the maximum eccentricity of the centre line of the chord measured from the straight line connecting the two adjacent panel-points, except as this bending moment may be modified by conditions of continuity and fixedness of the chord. Further, as has been mentioned previously, there exists a considerable initial stress in each lamination of the chord, resulting from springing the boards to the required curve during construction. The amount of the bending due to the assumption, in the stress analysis, of a chord of straight segments may be computed; also the initial stress in each board due to framing to a curve may be found. The difficulty arises in determining the actual efficiency of such a composite beam in resisting the bending due to the eccentricity of the main stress,* and in deciding for

*Reference is made to 'Graphical Analysis of Roof Trusses,' by Charles E. Greene, 1905 edition, p. 82, and the following statement is quoted from the paragraph entitled 'Curved what length of time the modulus of elasticity of the timber remains constant. With regard to the first consideration, the efficiency of the chord-section to withstand bending depends on the number and position of the splices of the boards, and the ability of the nails to resist, without slip, the longitudinal shear between the laminations. Remembering that the nails are also called upon to resist the shear due to column action, and, to a large extent, that due to the initial bending of the boards, and further, as shown in a previous article, that nailed joints slip at a comparatively small load, it will be realized that the laminated chord should not be credited with a high efficiency.†

The complications and the uncertainties of the problem can best be appreciated by a practical example. For this purpose, Fig. 65 shows a skeleton diagram of one of the three-hinged roof arches of the main group of buildings of the Panama-Pacific International Exposition. In this figure the sizes of the various members of the arch are indicated. Fig. 66 gives a detail of a portion of the lower chord of the arch. The maximum compressive

Beams:' "If the planks are bent to the curve and laid upon one another, this combination is not nearly so effective as the former (scarfed boards side by side, the plane of the boards being parallel to the plane of the loading—H. D. D.), but it can be more cheaply made. The lack of efficiency arises from the unsatisfactory resistance offered to shear between the layers by the bolts or spikes. The strength to resist bending moment will be intermediate between that of a solid timber and that of the several planks of which it is composed, with a deduction of one for a probable joint. If the curved member has a direct force acting upon it and a moment arising from its curvature, the treatment will follow the same lines; but the joints, if there are any, will be more detrimental in case there is tension at any section. Such curved pieces are sometimes used in open timber trusses for effect, but their efficiency is low on account of the large moment due to curvature."

†In framing a curved laminated truss chord, such as the one under discussion, the required curve is marked out on the floor of the fabricating platform, and blocks are then nailed to the floor along the curve. These blocks hold the boards in position. One after the other, the boards are then bent to the

stress is 42,400 lb., while the maximum tension is 45,400 lb. These stresses are due to the following loads:

	Lb. per sq. ft.
(1) Dead load plus live load	35
(2) Dead load plus wind load	

The wind on the side walls, or the vertical portion of

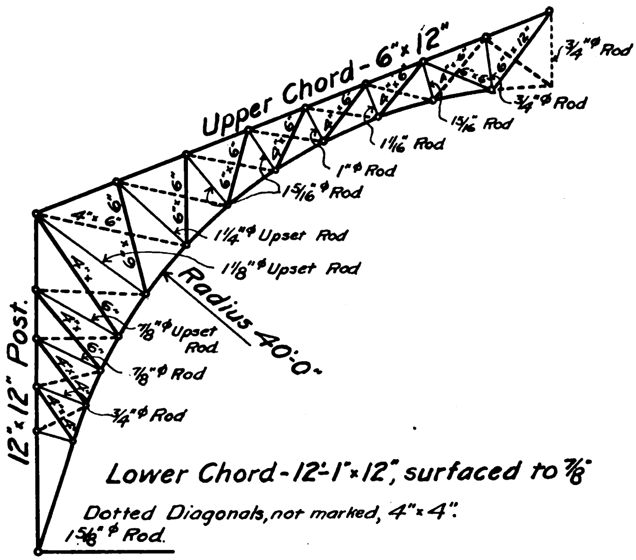

FIG. 65. DIAGRAM OF 3-HINGED ARCHED TRUSS FOR MAIN EXPOSITION BUILDINGS.

the truss, was taken at 20 lb. per sq. ft., and the wind on the roof in accordance with Duchemin's formula, with $P = 30$ lb. per sq. ft. For the condition of dead

curve, each successive board being nailed to the preceding one. Both chords are usually framed on the floor in their correct relative position, and the web members, struts and rods, are then placed in the truss. The truss is often fabricated completely before the blocks are released. The chords are thus to some extent maintained in their correct shape by the action of the web members against the butt-blocks, and the butt-blocks against the first boards. The nails and the bolts binding the board or boards between the butt-blocks are in shear. The statement above, of which this note is an explanation, is believed, then, to be a reasonable one.

load alone, the lower chord is in compression, with a stress of approximately 24,200 pounds.

Referring to Fig. 66, the eccentricity of the centre line of the chord from a straight line connecting the

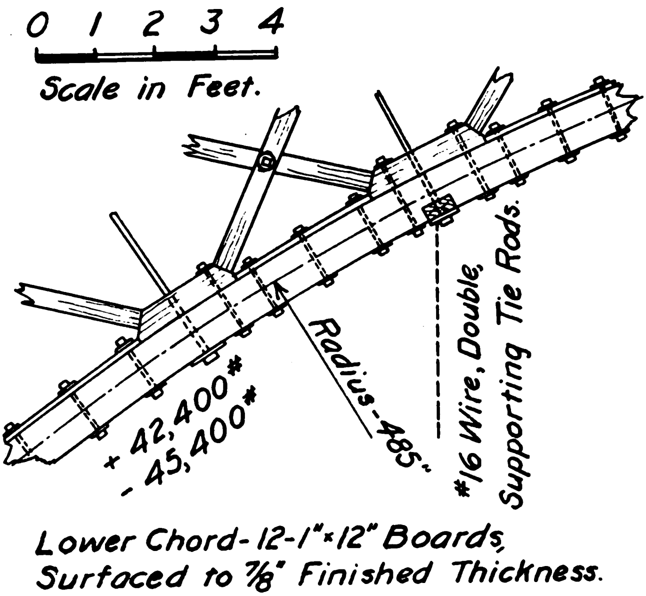

FIG. 66. DETAIL OF PART OF LOWER CHORD.

adjacent panel points is 1 in. The bending moment due to this eccentricity, disregarding the effect of continuity of the chord, is therefore 42,400 in.-lb. As the chord is continuous, and held rigidly at the panel-points by the long butt-blocks, the apparent bending moment may be reduced by the factor $\frac{3}{4}$, making the effective bending moment 31,800 in.-lb. If the chord were a solid stick, the section modulus would be $\frac{1}{6} \times 12 \times (10.5)^2 = 220.5$, and the maximum fiber stress would correspondingly be $\frac{31800}{220.5} = 144$ lb. per sq. in. The actual efficiency will be taken in accordance with the recommendations of Mr. Greene, namely, as the average between the section modulus of the solid stick of equivalent cross-section and the sum of the section moduli of the separate boards, minus one. In other words, the efficiency of the chord will be taken at $\frac{118.6}{220.5} = 0.54$, or say one-half. The actual maximum fibre-stress due to the bending will therefore be twice 144 or 288 lb. per square inch.

For finding the initial stress due to the springing of the boards to the curve of the chord, the formula

$$M = \frac{EI}{R}$$

will be used. This formula is one of the forms of expressing the bending moment in any beam according to the Common Theory of Flexure.

$M =$ bending moment in inch-pounds.
$E =$ modulus of elasticity.
$I =$ moment of inertia in inches.
$R =$ radius of curvature in inches.

We may also write the equation,

$$M = K \tfrac{1}{6} bd^2, \text{ where}$$

$K =$ maximum unit fiber-stress in any board,
$b =$ width of any one board, and
$d =$ depth of any one board, both in inches.

Equating the two expressions, we have

$$\frac{1}{6} Kbd^2 = \frac{EI}{R} = E \frac{1}{12} bd^3 \frac{1}{R}, \text{ whence}$$

$$K = \frac{1}{2} \frac{dE}{R}$$

The radius of curvature may be assumed, for practical purposes, to be constant for all boards of the chord, and its value will be taken at 485 in. Then,

$$K = \tfrac{1}{2} \times \tfrac{7}{8} \times \frac{1}{485} \times 1{,}500{,}000 = 1350 \text{ lb. per sq. in.}$$

The average gross unit compression in the chord is $\frac{42400}{12 \times 10.5} = 337$ lb. per sq. in. To find the maximum unit compressive stress in the chord, the three values found above must be added. Thus, adding $288 + 1350 + 337$, the maximum compressive stress is seen to be 1975 lb. per square inch.

The unsupported length of the boards may be taken at 33 in., or slightly more than the distance between the ends of the butt-blocks. Due to the continuous chord, and the long butt-blocks bolted through the chord, which produce to a great degree the effect of 'fixedness,' the length of the column may be reduced to $\frac{1}{2} \times 33 = 16\frac{1}{2}$ in. The ratio of length to least width is therefore $16\frac{1}{2} \times \frac{8}{7} = 19$. The safe unit stress is then 1170 lb. per sq. in., using the U. S. Department of Agriculture formula, with $C = 1600$. If, on the other hand, the allowable unit-stress be determined on the basis of column action of the chord as a whole, the unit-stress will be, using the same formula, 1600 lb. per sq. in. Using the recommendations set forth previously, the allowable unit fibre stress would be 80% of the half sum of 1170 + 1600, or 1100 lb. per sq. in. The computed maximum stress is therefore considerably in excess of the allowable. It must be stated, however, that the lumber in these laminated chords was clear and straight grained, it being so specified and furnished. Consequently, its ultimate strength was considerably in excess of the average grade to which the column formula applies. Further, the computed fibre stresses are for the condition of dead load and wind. With dead load alone acting on the truss, the maximum fibre stress would be 1650 lb. per sq. in. Considering the fact that this truss was for a temporary building, the unit stresses, while high, were considered as safe. The strength of the long butt blocks, dapped into the chords, and bolted thereto is an important factor in stiffening the laminated chord in compression, and unless such construction exists, the effective length of column should be taken as the panel length of the truss.

In a similar manner, the maximum tensile stress may be found. The stress due to springing the boards to position is the same as before; the stress due to secondary bending is greater in the proportion of the principal stresses, or $\frac{45400}{42400} \times 288 = 308$ lb. per sq. in. The average unit stress in tension on the whole chord is $\frac{45400}{126}$ $= 360$ lb. per sq. in. An allowance must be made in this case for splicing of the boards; it will be assumed that the chord has an efficiency of 75%. The average stress of 360 lb. per sq. in. must therefore be increased by the factor $\frac{4}{3}$, resulting in an actual unit stress of 480 lb. per sq. in. Finally, combining all the unit stresses we have a total unit stress of $1350 + 308 + 480 = 2138$ lb. per sq. in. For the grade of timber used, this is not an excessive unit stress.

The change in the modulus of elasticity of the timber has been mentioned. It is a fact, established by tests, that if a load be left on a timber beam for some length of time, the modulus of elasticity of the timber will drop to approximately one-half its value for temporary loads. This phenomenon is generally expressed by the recommendation that, in computing the deflection of timber beams, the modulus of elasticity for a 'dead' or constant load be taken at one-half the value used for 'live' or temporary loads. It is believed reasonable, therefore, to state that while the stresses due to springing the boards of the truss-chord illustrated above are actual stresses at the time of framing, a change in the properties of the timber eventually takes place, resulting in a decrease in the modulus of elasticity, and consequently, a diminution in the stress due to shaping the boards to the curve. Just how long a time is required for this change to take place it is difficult to say, the time being dependent to some extent on the original moisture-content, the amount of bending introduced in the chords, and the protection from the weather in the structure of which it is a part.* It is believed the initial

*For the purpose of obtaining some definite measurement of the amount of this initial stress remaining in such laminated chords, I conducted some tests on the boards of the chords of one of the Trusses 'A' of the Panama-Pacific International Exposition (one of the same trusses just discussed) through the kindness of C. H. Munson, assistant to the Director of stresses are reduced within a few months nearly one-half.

The complicated conditions existing in a curved, laminated truss-chord, will now be appreciated; also the force of the statement that such a section should be avoided whenever possible. The calculations and the reasoning of the above discussion may seem to be both doubtful in accuracy and cumbersome. I am frank to admit that the result reached in the illustration chosen rests upon a number of assumptions whose validity cannot, perhaps, be definitely proved. However, the main tenets are true: the initial stress due to springing the boards to a curve does exist, and approximately to the amount computed, when the boards are first bent; afterward, this stress undoubtedly decreases; also there does

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Works. On September 29, 1916, three laminations were removed from a chord which was built approximately two and one-half years previous. The length of the chord, and the middle ordinate of the arc of the approximate circle to which the boards sprang back on being released were measured, and from these measurements the radii of the circles to which the boards returned have been computed. Of the three boards measured, the radii of their respective circles were 95, 81, and 96 ft., or an average of 91 ft. The same boards were again measured on October 14, 1916, and the respective radii were found to have increased to 128 ft., 129.5 ft., and 132 ft., or an average of 130 ft. As the fibre stress due to the curvature is in direct proportion to the radius of curvature, it may be stated that the measurements indicated that $\frac{40}{(130-40)}$ of the initial stress remained in the boards up to the last date mentioned, or, in other words, that approximately 45% of the initial stress still remained in the chords. This calculation is on the assumption that the modulus of elasticity of the timber had not changed. The boards were again inspected after about three weeks, when they had nearly straightened out. The results of these experiments were not such as to justify any definite conclusions. The boards did not form a true circular arc after being removed from the truss, so that accurate measurements were impossible. They were stored inside a warehouse, and lay on their edges. They were, therefore, free to take their natural shape, except as the friction of the floor held them to the curved shape.

exist a secondary stress of bending due to the curve of the chord. The actual amount of the reduction of the initial bending stresses is somewhat uncertain, but the assumptions made herein are believed to be fair.

Curved laminated chords are more efficient in tension than in compression, and a truss with the compression chord of solid members, even if broken and spliced at every panel-point, is in many cases to be preferred over one in which both compression and tension chords are curved laminated sections. This statement is made advisedly. I have seen instances of laminated curved compression-chords in a badly buckled condition.

Timber Tension-Members. The tension chord of a truss, when framed in timber, needs no further discussion. It has been shown above, and also in the treatment of end-details, that secondary stresses very often add considerably to the primary stresses. Timber will seldom fail in straight tension; the details will give first. For this reason, it might seem reasonable to use a much higher unit stress in tension than has been recommended in these articles. However, because of the uncertainties of the actual amount of secondary stresses, and the variation in the structure of the material, it is recommended that from 1500 to 1800 lb. per sq. in. be taken as the extreme limit for tension in the case of live and dead loads for permanent structures.

Tension-Rods. The selection of the proper size of tension rods is not merely the problem of dividing the maximum stress by the allowable unit stress. Certain other factors enter into the problem from the practical standpoint, and these will be discussed briefly.

To the computed stress in a tension rod of a truss, as found from the stress analysis, it is well to add an initial tension. In fabricating a truss, camber is usually introduced, and largely by means of springing the chords, cutting the web compression members to fit, and holding the truss in this strained position by tension in the rods. The amount of this initial tension that should be added may be taken at from 1500 lb. for the smaller roof trusses to 3000 lb. for the larger trusses, the values given being for each rod of the truss.

Either plain or upset rods may be used, the former being cheaper for the shorter rods, and the latter economical for the longer rods. The dividing line for any case can be determined easily from local prices. In using plain rods, it must not be forgotten to take the area at the base of the threads as the net section in determining the size of rod to be used. If upset rods are specified in designing the truss, great care must be exercised to see that no welded rods are furnished. It is the custom in some small shops, when rods with upset ends are specified, to weld 'upsets' to the body of the rods. This practice results largely from the fact that such shops have no upsetting machines. Welds in plain rods are, of course, a possibility, and the inspector must needs watch for them, especially in long rods, but their occurrence is not so probable as it is in the case of upset rods.

Another factor to be considered in the design of tension steel is the quality of steel to be expected. This consideration will affect the working stress to be used. On the Pacific Coast at least, re-rolled steel is used almost exclusively for the stock sizes of rods. The principal objection to re-rolled steel is its variable composition, as shown by its fibrous, laminated fracture. Again, although medium steel may be specified, wrought iron may be furnished. The quality of the material in the tension rods of a truss can only be determined by tests, and these are not always convenient or possible to make. It is of interest to the designer of such trusses, therefore, to be familiar with the limitations of the market, and his design may be modified accordingly. For the purpose of indicating the nature of the metal commonly furnished under specifications calling for medium steel corresponding to standard specifications, there is given below the results of some tests on various sizes of rods taken from material submitted by contractors under specifications calling for medium steel to correspond to

Table XVIII

RESULTS OF TESTS ON ROUND STEEL TRUSS RODS

Nominal dimensions, inches	$1\frac{1}{2}$	$1\frac{1}{4}$	$1\frac{1}{8}$
Actual dimensions, in	1.498	1.238	1.129
Actual area, sq. in	1.7624	1.203	1.001
Yield point, actual load, lb	53,070	37,780	33,150
Yield point, lb. per sq. in	30,112	31,404	33,117
Ultimate strength, actual load, lb	88,130	61,620	51,960
Ultimate strength, lb. per sq. in	50,005	51,221	51,908
Elongation in 8 inches, in	2.00	2.30	2.35
Elongation, per cent	25.00	28.75	29.25
Dimensions, reduced section	1.240	0.960	0.824
Area, reduced section	1.207	0.7238	0.5332
Reduction of area, per cent	31.5	39.83	46.73

Character of fracture: $1\frac{1}{2}$-in. round steel truss rods, characteristic, badly laminated, small distinct bars in mass; $1\frac{1}{4}$-in., part cup, characteristic, badly laminated; $1\frac{1}{8}$-in., characteristic, slightly laminated.

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the standard specifications adopted by the Association of American Steel Manufacturers.

Comparing these results with the specifications of the Association of American Steel Manufacturers, the ultimate strength is below the limit for structural steel (60,000 lb. per sq. in.), and somewhat below the limit set by the specifications of the American Society for Testing Materials (55,000–65,000 lb. per sq. in.); the elastic limit is satisfactory; the percentage of elongation is satisfactory, the requirement being a minimum percentage on an 8-in. length of $\frac{1400000}{\text{Ult. Strength}} = \frac{1400000}{51000} = 27.5\%$ for material not over $\frac{3}{4}$ in. in thickness, with an allowable deduction of 1% for each $\frac{1}{8}$-in. increase in thickness, except that the minimum elongation shall not be less than 18%. For the $1\frac{1}{8}$-in. rod, the minimum elongation required is, according to the rule above, 24.5%; and for the $1\frac{1}{2}$-in. rod, the minimum is 21.5%.

The testing company reporting the above tests classified the material as wrought iron. In my opinion, this classification was erroneous; the fracture had somewhat of the fibrous texture characteristic of wrought iron, as opposed to the silky or granular fracture of steel, but

TABLE XIX

TENSION TESTS ON TRUSS RODS AND BOLTS

Material	¾-in. bolt	¾-in. bolt	Truss rod	Truss rod	Truss rod	Truss rod	Truss rod
Mark or number	No. 1	No. 2	*A**	*B*†	*C*†	*D**	*E*†
Dimensions of cross-section, in.	0.736	0.60	0.858	0.855‡	0.865‡	0.983	0.988
Area of cross-section, sq. in.	0.425	0.283	0.578	0.574	0.588	0.759	0.767
Load at yield point, lb.	14,300	13,820	18,700	.....	.....	24,000	.....
Strength at yield point, lb. per sq. in.	33,600	48,900	32,400	.....	.....	31,700	.....
Maximum load, lb.	20,890	17,830	27,500	28,850	28,950	36,000	36,400
Maximum strength, lb. per sq. in.	49,000	63,000	47,500	50,400	49,300	47,400	47,500
Elongation in 8 inches	.....	.....	1.4**	.....	.....	1.93	.....
Per cent elongation in 8 inches	.....	.....	17.5	.....	.....	24.1	.....
Dimensions of reduced cross-section, in.	.....	.....	0.60	.....	.....	0.65	.....
Area of reduced cross-section, sq. in.	.....	.....	0.283	.....	.....	0.331	.....
Per cent reduction of area	.....	.....	51	.....	.....	56.5	.....
Character of fracture	.....	.....	fibrous	fibrous	fibrous	fibrous	fibrous
Speed of machine, inches per minute	0.04	0.04	0.04	0.04	0.04	0.04	0.04

Remarks:

Bolt No. 1—Test of strength of head of bolt. Body of bolt failed.

Bolt No. 2—Test of strength of thread of bolt. Bolt failed in thread.

Dimensions at root of thread.

*This test on centre portion of a truss-rod.

†Test on upset rod.

‡Diameter near centre of rod.

**Failed outside of gauge points.

Truss Rod A—End near upset badly twisted; 8 threads per inch; diameter root of thread, 1.04 in.; failed in body of rod.

Truss Rod B—Diameter near upset, 0.834 in.; failed in body of rod.

Truss Rod E—Diameter near centre of rod, 0.988 in.; diameter near upset, 0.983 in; diameter of root of thread, 1.07 in.; failed in body of rod.

was actually what may be described as a mild 'mongrel' steel. The laminated composition is characteristic of the material, and shows to better, or worse, advantage in the case of plates. Fig. 67 shows a typical specimen.

Table XIX gives the results of some other truss rods and also of bolts. These results are introduced for the

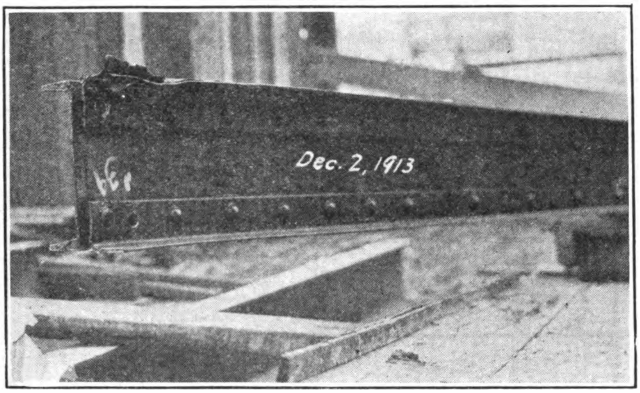

FIG. 67. SPECIMEN OF DEFECTIVE RE-ROLLED STEEL.

purpose of giving a general idea of the fairly uniform characteristics of the steel of this class. It will be noticed that the elastic limit is quite high.

For use in rods, or plate connections taking tension alone, it is believed that the material may be used with confidence, employing a stress of 16,000 lb. per sq. in. for dead and live load. For important work, such as where the full live load is a certainty, this steel should not be used, and rigid adherence to the standard specifications should be required.

CHAPTER X

Bracing-Trusses—Details of Howe-Type Roof Truss—Lattice Trusses—Truss Connections to Posts

Bracing-trusses in building construction may serve one or all of three purposes, first, that of stiffening the top or compression chords of the main roof-trusses, second, providing general stiffness to the building against wind, and third, supporting the roof joists

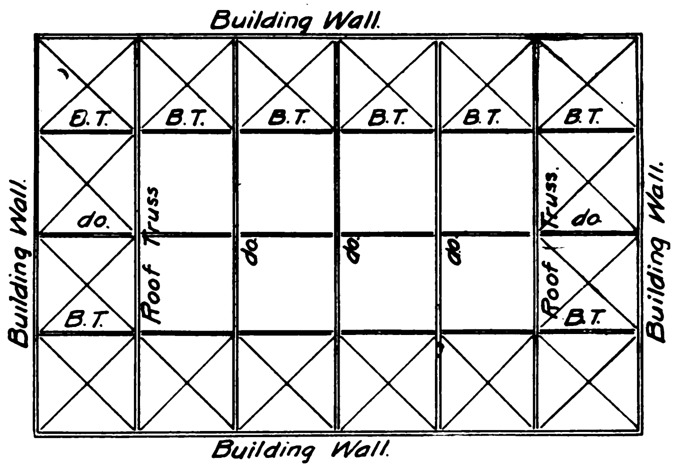

FIG. 68. GENERAL PLAN OF BRACING-TRUSSES.

directly, and transferring this load to the main roof-trusses. In the latter event, the bracing-trusses are generally referred to as purlin trusses. It might seem at first thought that the most economical arrangement of framing would be secured by using purlin trusses, and utilizing them as bracing-trusses. This is not necessarily true, however. While, by this scheme, the upper chords of the main trusses are relieved of cross-bending from the roof joists, the additional material necessary in the bracing-trusses to enable them to carry the roof is usually considerable. In addition, more steel is required in the main roof-trusses, because the shear in these trusses is a constant from the supporting columns to the point of attachment of the purlin trusses,

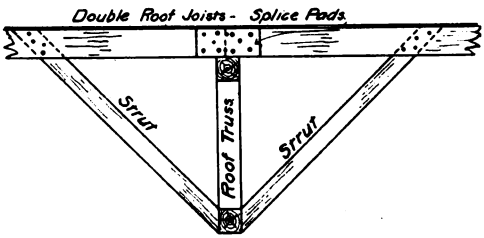

FIG. 69. METHOD OF TRUSSING ROOF JOISTS.

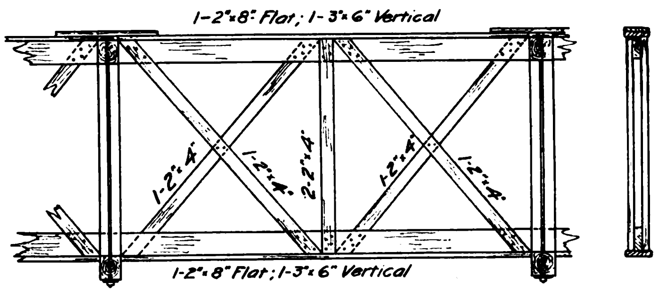

FIG. 70. DETAIL OF BRACING-TRUSS.

and for requirements of general stiffness, the rods of each main truss between the purlin trusses cannot be altogether omitted, even though the shear due to roof covering and joists is zero. Because of the varying factors of span and spacing of the main roof-trusses, direction of slope of the roof, and possible limitations of clearances and ceilings, no hard and fast rule as to the most economical arrangement of framing can be stated. I have found, however, that in actual cost of construction, there is little difference between a roof framed with the joists resting directly upon the chords of the main trusses, and one in which purlin trusses are employed.

In the case of a bracing-truss which carries no roof load, the principal points to be observed are that the chords have a section capable of taking compression, and that the bracing-truss has a good and rigid attachment to the chords of the main trusses. Theoretically, the lower or tension chord of a roof-truss needs no stiffening. Practically, however, it is well to support it laterally, not alone to keep it from warping out of shape, but also for the purpose of adding general stiffness to the building frame.

The actual stress which may come upon a bracing-truss is usually indeterminate. In many cases, however, a definite scheme of wind bracing may be provided, in which the bracing trusses play an important part. For example, the roof may be stiffened to act as a horizontal beam against wind pressure, transferring the wind loads to the end or side-walls, or to columns and walls. In such cases, diagonal rods are usually introduced in the plane of the roof-joists, the upper chords of the roof-trusses and the bracing-trusses acting as the chords of the horizontal wind-trusses. This is an effective way in which to stiffen a building against wind, provided that the connections are carefully studied, and made strong enough properly to fulfill their respective functions and provided that the walls are well braced. Fig. 68 illustrates the general scheme.

For buildings of small height and truss spans, sufficient stiffness may be obtained by trussing the roof joists, similar to the detail shown in Fig. 69. A detail of a bracing-truss which is easily framed and is efficient is shown in Fig. 70.

The requirements of bracing in timber-framed buildings are no different from those of steel-framed buildings. The general conditions of provision for wind pressure, arrangement of main trusses and bracing-trusses are the same in either type of building, with due allowance for the nature of the roof to be supported. In this connection, the excellent texts of Ketchum and Tyrell on the subject may be profitably studied by the reader interested in the construction of mill buildings, and other buildings having large open spaces, necessitating long columns and roof trusses. As was remarked in the introductory chapter, steel-framed buildings of these types are generally designed by a competent engineer. The same building, however, if framed in timber, is often planned by an architect unacquainted with the fundamental principles of structural engineering. That the consideration of roof bracing is vital will be appreciated by reading the account of two recent failures of timber buildings, one in Salt Lake and the other in Atlanta, Georgia*. The last named failure resulted in loss of life. It is significant that in both cases definite information as to the plans of the buildings, as well as accounts of the failure, were hard to obtain, there being an evident desire on the part of both the architect and the municipal authorities to hush up the matter.

Details of Howe-Type Roof Truss

In Chapters IV, V, and VI the design of the details of timber trusses has been discussed and illustrated by typical joints of open-panel trusses. In this chapter, it is desired to show a complete truss, designed in accordance with the principles set forth in the preceding chapters. Another type of roof-truss, the lattice-truss, as distinguished from the open-panel type, will also be described, and illustrated with a typical case. The design of the supporting columns is so closely interwoven with the subject of roof-trusses, as to require simultaneous treatment. In general, the illustrations of roof-trusses given in the text-books consider trusses supported on masonry walls. While this is a common case, the engineer or architect is confronted frequently with the problem of a timber-framed building, that is, a building with timber roof-trusses and timber columns, forming a structural frame supporting the walls and roof, which may be of wooden sheathing or of corrugated iron. Indeed, this case has been the most common in my experience. Here,

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*_Engineering News_, Vol. 75, No. 25 and Vol. 76, No. 2.

many of the details which might be used in connection with masonry walls have had to be either discarded or modified. The timber columns do not merely support the dead weight of the roof-trusses; they become a part, with the trusses, of a definite structural frame, technically termed a 'transverse bent,' which resists the lateral forces of the wind and stiffens the building. This subject was mentioned above, in speaking of the function of bracing-trusses. The designer of such a 'transverse bent' must consider carefully wind forces, and the means of providing for them. In this chapter it is not the intention to treat of wind forces in any detail, but the connections of trusses to columns will be discussed.

In Chapter VI, Fig. 38, is shown a diagrammatic elevation and stress-diagram of a 70-ft. span Howe-type timber roof-truss. Fig. 71 of the present chapter gives the truss completely detailed. The spacing of the trusses is assumed at 24 ft., and the loading at 38 lb. per sq. ft. of horizontal projection of roof surface. For simplicity, all loads are assumed to act at the upper chord of the truss. This assumption is somewhat in error, as the dead weight of truss and bracing trusses should be taken as distributed between the upper and lower chords. The resultant error is, however, small and can be neglected in this case.

In the design, it has been further assumed that the roof-trusses are for a building of the 'mill-building' type, that is, having a definite structural frame of timber trusses, columns, wall-girts, etc., and that some stiffness against lateral forces is desired, although no definite length of columns has been established, nor have any wind stresses been computed.

This detail presents what, in my opinion, is the most economical and efficient truss for such buildings. It is designed with conservative unit-stresses in all its details; it is simple of construction, and direct in its action. The rods are slightly larger than is required by the stresses indicated on the stress diagram in order to allow for initial tension when fabricating the truss. Washers of ample size are provided so that the rods can work to their full capacity. The butt-block type of intermediate joints has been used; hence the diagonal struts have full bearing at their ends. Attention is directed to the detail of the end-joint. This detail, where circumstances will permit of its use, is believed to be the most efficient and at the same time, the cheapest, that can be found. (If the lower chord could extend beyond the post. the end details, Types *A* and *B* of Chapter VI could be used). The roof-joists rest directly upon the upper chord. While this introduces the secondary stresses of bending into the chord, the joists at the same time support the chord laterally in its weakest dimension, acting as a column, and so permit rather high unit fibre-stresses. To give sufficient lateral support, it is required that the roof-joists be well spiked to the chord, and that they also be well spiked at their laps. In the end-panels, bolsters have been provided for the lower chord. These bolsters, being well bolted to the chord, not only take care of any secondary stresses due to the action of the end-detail, but also provide additional bolting space for the attachment of the lower chord to the column.

The truss rests concentrically on the posts, hence there is no bending in the post due to eccentric loads. To accomplish this result, it is necessary that the knee-braces be cut and framed into post and bolster after the truss has been erected and all of the dead load of the roof is in place. Otherwise, the slight deflection of the truss, when the roof load is placed, will cause the knee-braces to transfer a horizontal thrust to the post, with consequent bending in the post. Possibly this may seem an unnecessary refinement in timber-framing; however, the cost of cutting and fitting the knee-braces after the truss is erected and the roof loads are in place is not excessive, and I believe that the slight additional expense is justified.

Note the bolting of knee-braces to post and truss

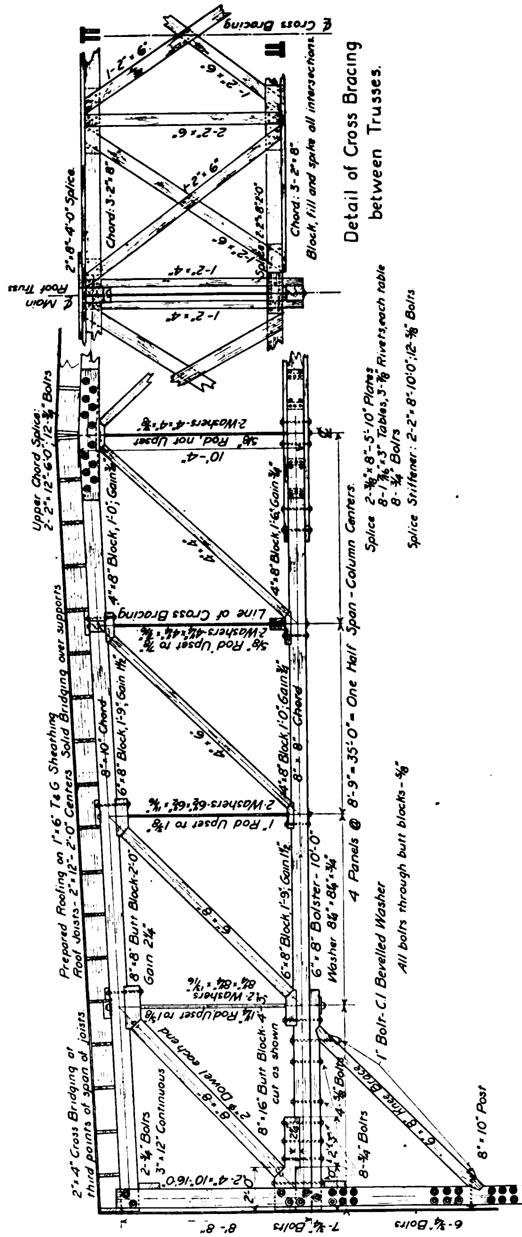

FIG. 71. DETAIL OF HOWE ROOF-TRUSS.

chord with cast-iron bevelled washers. The use of such washers, set into the knee-brace, insures that the brace is capable of withstanding some tension. If computations for wind show that the knee-braces must take considerable tension, metal side-plates, bolted or lagged to the knee-brace and to the post or truss-chord and bolster may be substituted for the bolts shown. Without going into the theory of wind stresses in transverse framed-bents at this time, it may be said that the exact distribution of the wind moment in the column in this case is somewhat indeterminate, not only from the fact that the condition of the base of the post is unknown, that is, whether 'hinged' or 'fixed,' but also from the conditions here present of three intersections of the truss with the post, namely, the connection of the knee-brace to the post, and the connections of the upper and lower chords to the post. However, the statement may be made that the maximum bending-moment in the column occurs either at the foot of the knee-brace or at the intersection of the lower chord with the post. For this reason, the detail here shown, with the post spliced over the truss, the splice-pads with a total capacity in bending equal to that of the post, and extending well below the foot of the knee-brace, and bolted thoroughly to the post and truss, provides a condition of maximum stiffness consistent with simplicity of fabrication and ease of erection. If the knee-brace is omitted, as is common in many instances, the computation of the bending-moment due to wind, and the forces acting on the bolted joints, is a simple matter. For the purpose of presenting the conditions of wind bending in such a case, diagrammatic representations of these moments have been prepared and are shown in Fig. 72, *a, b,* and *c,* which illustrate the influence of the end conditions of the columns. Fig. 72*a* represents the bending for a column with 'free' or 'hinged' ends; Fig. 72*b* shows the bending for a column with 'fixed' ends; while Fig. 72*c* illustrates the bending for an intermediate condition, namely, for a post half-way between hinged or fixed at the base.

Fig. 71 also shows the detail of the bracing-trusses between the main roof-trusses. Such a truss, as was explained in the preceding article, may play an important part in resisting the wind on the building; its chords may, in such a case, take stresses, either of compression or tension, that can be calculated with a reasonable amount of accuracy. Its connection with the main truss must then be carefully studied, particularly the splices, in order that it may fulfil its part properly in the general scheme of bracing the building. On the other hand, the bracing trusses may take no stresses that can be computed. Nevertheless, no roof-truss should be constructed without due attention to the need of bracing trusses. The detail shown here is simple and cheap, and at the same time is effective.

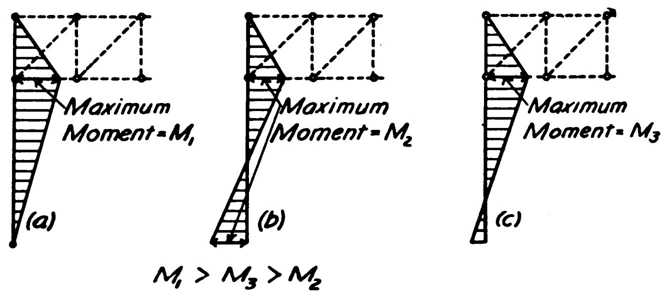

FIG. 72. BENDING-MOMENT DIAGRAM FOR WIND-STRESSES IN COLUMNS.

Note the 'T' section of the chords, enabling them to take compression, as well as to tie the main trusses together.

The splice of the lower chord merits mention possibly. Besides the steel-tabled fish-plate splice, which has a capacity of the computed stress in the chord in this panel, two wooden stiffening splice-pads are bolted through the chord to give the truss additional stiffness during erection. Some criticism may perhaps be made of placing a tension-splice at the point of greatest chord-stress. While such a position for the splice is not desirable and should in general be avoided, the lengths of available timbers will sometimes require placing the splice at the centre of the truss. If due attention is given to the detail, and conservative unit-stresses are used, there need be no apprehension of the strength of the truss.

It should be noted that Fig. 71 does not represent a working detail, and is not intended as such. To make this detail a 'working' drawing, for shop and field, sundry additional information should be given, such as a shop drawing of the steel splices, spacing of all bolts, etc.

Lattice-Trusses

The lattice-truss, diagrammatically illustrated in Fig. 73, is often employed in roof construction for moderate spans. This type of truss, as distinguished

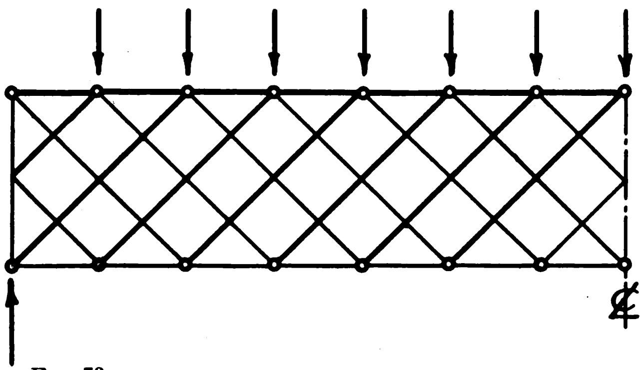

FIG. 73. OUTLINE OF HALF-ELEVATION OF LATTICE-TRUSS.

from the open-panel type, was probably first used for bridges. Burr and Falk, in their 'Design and Construction of Metallic Bridges,' Edition 1905, note "A later type of timber bridge which was most extensively used in this country was invented by Ithiel Towne in January, 1820, which was known as the Towne lattice-bridge. This timber bridge was among those used for railroad structures. It was composed of a close timber-lattice, heavy planking being used for the lattice members, and they were all joined by wooden pins at their intersections. This type of timber structure was comparatively common not longer than twenty-five years ago, and probably some structures of its kind are still in use. The close lattice work with its many pinned intersections made a safe and strong framework, and it enjoyed deserved popularity. It was the forerunner in timber of the modern all-riveted iron and steel lattice-truss. It is worthy of statement, in connection with the Towne lattice, that its inventor claimed that his trusses could be made of wrought or cast-iron as well as timber. In many cases timber arches were combined with them.''

In the case of a railway bridge of the latticed type, the chords of these trusses are firmly supported laterally, the top chords by the upper lateral-bracing, and the lower chords by the floor system, and also by the lower lateral-bracing. In using the lattice-truss as a roof-truss, especial care must be taken to see that the unit compression-stress in the upper chord does not exceed the safe unit-stress for the chord treated as a long column. Due to the necessity of making the chords of a lattice-truss deep for bolting, and the use of thin material for the web members, the truss as a whole is rather thin and deep, as compared with a truss of the Howe type, for example. It will therefore have a tendency to twist, and must be braced accordingly.

When the roof-surface slopes from the centre of the truss to the ends, the upper chords may be given the slope of the roof, or the truss may be constructed with horizontal chords, and the roof-surface furred, either by means of a low studded-wall or short post and roof girder. The truss with a sloping upper-chord is somewhat more difficult to construct, as the diagonals have different lengths, and the intersections of diagonals and upper chord are not uniform. For this reason, I prefer in general to build such trusses with parallel chords, and then to construct a studded-wall on the upper chord. In such an event, the trusses must be tied together by bracing trusses or struts, with possibly the additional precaution of stiffening the upper chord laterally by means of a 2-in. plank nailed to the upper edges, forming a 'T' section.

It is hardly necessary to say that the lattice-truss is an indeterminate structure, and that the exact stresses cannot be found by the ordinary methods for solving the stresses in a roof-truss. It is customary to consider the truss as a combination of a number of Warren trusses, each taking its proportion of the total load. The chord-stresses are, of course, the sum of the chord stresses in the individual Warren trusses. Or, the lattice-truss may be computed as a plate girder in determining the approximate chord-stresses. For finding the stresses in the diagonal web-members, the shear at any section may be divided by the number of web-systems, and the quotient resolved into the line of the diagonal. For finding the required number of bolts or nails fastening the webs to the chords, the stress in the diagonal must be resolved into the two components parallel to and perpendicular to the chords, respectively. The component perpendicular to the chords, or the shear in the section, acts through the bolts across the grain or fibre of the chord-timbers, and hence may be the feature determining the size of the bolts.*

To illustrate the design of a typical latticed roof-truss, there is here given the computations for a truss of this type, shown in Fig. 74. The span is 40 ft., the spacing 24 ft., and the total loading, including the weight of the truss, 35 lb. per sq. ft. of projected horizontal roof-surface. The figure gives a half elevation of the truss. For finding the maximum stresses in the diagonal web-members, the stress diagram shown in Fig. 75*b* has been constructed.

All intersections of web-members are spiked, and

*This is strictly true of the end-diagonals only; the size of the bolts in the intermediate diagonals are determined from consideration of pin action, as explained in the detail computations.

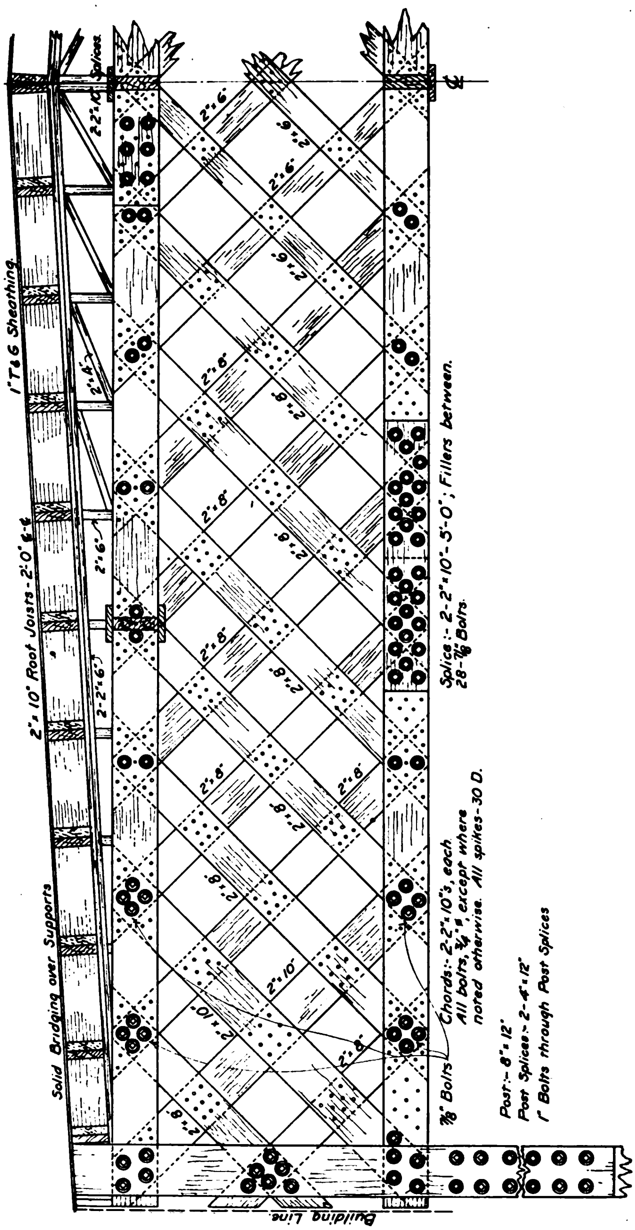

FIG. 74. HALF-ELEVATION OF LATTICE-TRUSS.

the intersections of webs and chords are spiked in addition to the bolting. Fig. 74 shows the attachment of the lateral bracing which consists of a bracing truss at the centre of the span, and a strut between trusses at the quarter-points of the span. For a truss of this span and loading, it is not necessary to use fillers between webs and chords, but for trusses of greater span or loading such fillers may be required.

The lattice-truss is not an efficient truss from the standpoint of material, but where timber is comparatively cheap, and steel in the form of rods, plates, etc., is either expensive or difficult to procure, this type of truss may be the most economical to use. The lumber required is all dimension stock, and may be had in any lumber-yard, and the bolts are stock bolts. The skill required in framing is small, and the ordinary house-carpenter may do a satisfactory job. The weak feature of the latticed roof-truss is usually found to be the tension-splice of the lower chord. This splice should be carefully designed, and an ample number of bolts be provided to take the chord stress at that point. For determining the chord stress at any point, a simple method is to construct a bending-moment diagram similar to that shown for this truss, in Fig. 75*a*.

Computations for Lattice-Truss. Span, 40 ft. Spacing of trusses, 22 ft. Depth of trusses: The depth of a truss with horizontal chords, namely, the vertical distance between the centre lines of the chords, should be between $\frac{1}{6}$ to $\frac{1}{10}$ of the span, $\frac{1}{8}$ being an economical ratio. In a lattice-truss with both chords horizontal, the depth should be a simple proportion of the span, in order to secure good intersections of diagonals with the chords. The depth in this case will be taken at 5 ft., which is a ratio of depth to span of $\frac{1}{8}$.

Loading of horizontal projection of roof, 35 lb. per sq. ft.

Total load on truss $= 22$ by 35 by 40 ft. $= 30{,}800$ lb.

Gross reaction $= \frac{1}{2}$ total load $= 15{,}400$ lb.

Maximum bending moment $= \frac{1}{8} \times 30{,}800 \times 40 = 154{,}000$ lb.-ft.

Maximum chord stress $= \dfrac{154{,}000}{5} = 30{,}800$ lb.

Required net area of tension chord $= \dfrac{30800}{1500} =$ **20.4 sq. in.**

The chords will be composed of two 2 by 10-in. timbers, giving a gross area of 40 sq. in., or twice the net area required. This may appear excessive, but 2 in. is the minimum thickness that should be used, and the

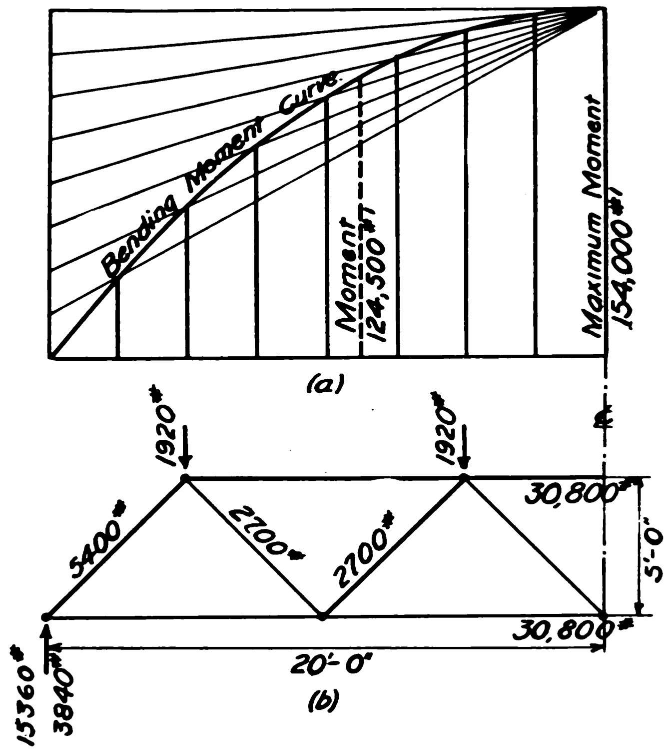

FIG. 75. DIAGRAM OF BENDING-MOMENT AND MAXIMUM STRESSES IN MEMBERS OF LATTICE-TRUSS.

width of 10 in. will give ample bolting and spiking space, which is a vital requirement. The strength of the truss depends upon the bolting and spiking, and for bolting to be effective, the bolts must not be placed too close together, nor too close to the ends or the edges of the timbers.

Stresses in Diagonal Members. (See Fig. 75*b*.) This figure represents one of the four web-systems, each forming a Warren truss. The panel loading for one such system is $2\frac{1}{2} \times 22 \times 35$ lb. $= 1925$ lb. The maximum diagonal stress is 5400 lb. The same result is reached by working from the end reaction. Thus, dividing the total reaction by the number of web-systems, and resolving such shear into the line of the diagonal, there results,

$$\frac{15400}{4} \times 1.407 = 5400 \text{ lb.}$$

Bolting and Spiking of Diagonals. If spiking alone were to be counted upon for fastening the web-mem-

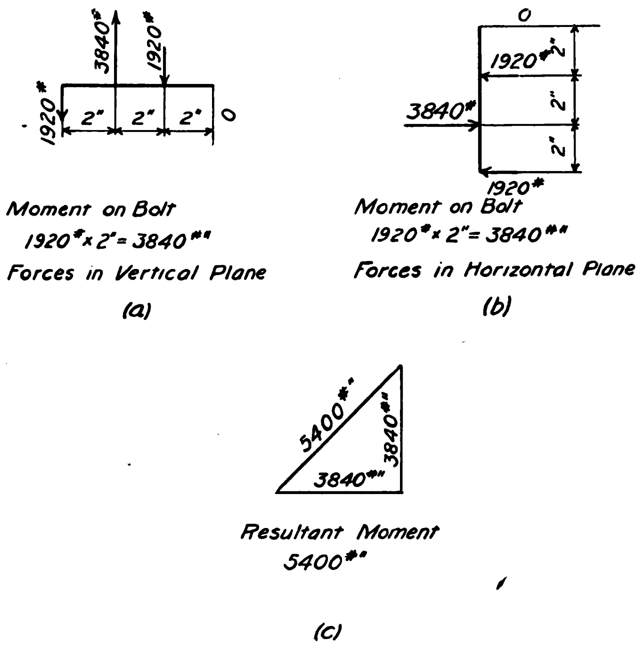

FIG. 76. MOMENT-DIAGRAMS FOR DIAGONALS OF LATTICE-TRUSS.

bers to the chords, each web-member would have to have sufficient spikes fastening it to the chord to transmit the stress in such diagonal. A 30D spike is $4\frac{1}{2}$ in. long; this size of spike is about the largest that should be used. The safe resistance of a 30D spike to lateral shear is 194 lb.; eleven spikes is about the maximum number that may be used without danger of splitting the timber. The maximum resistance of spiking is therefore 11×194 lb. $= 2134$ lb. In order that both chords may act together, they should be bolted through, in addition to the spiking. For the web-members with a stress of 2700 lb. (see Fig. 75 *b*), two $\frac{3}{4}$-in. bolts and nine 30D spikes will be used. This will take care of all panel-points except the first four from the ends.

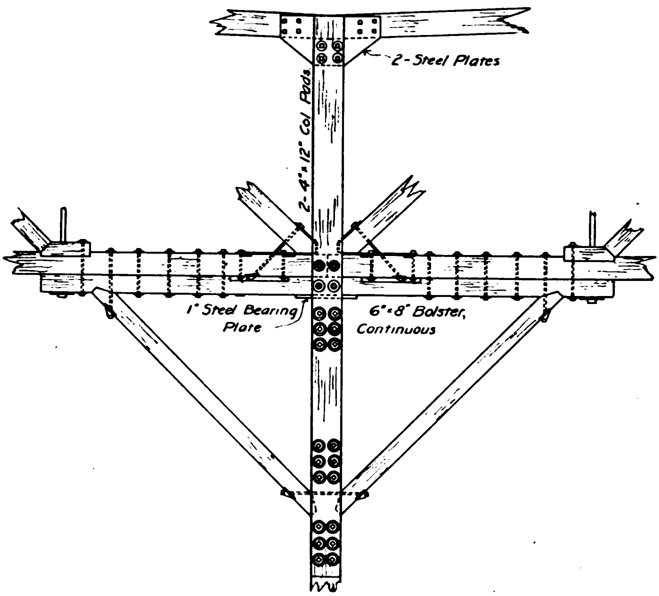

FIG. 77. CONNECTION OF TRUSS AND POST.

For the first four panels from the end, the bolts will act as pins, with forces as shown in Fig. 76, *a*, *b*, and *c*. In Fig. 76*a* and *b*, the web-stresses are resolved into their components in horizontal and vertical planes, the reactions found, and the moments computed. Fig. 57*c* shows the resultant moment to be 5400 in.-lb. Four $\frac{7}{8}$-in. bolts will give a resisting moment of 4200 in.-lb. at a flexural stress of 16,000 lb. per sq. in. Using four $\frac{7}{8}$-in. bolts, then, there is required in addition $\frac{5400 - 4200}{5400}$ $\times$ $5400 = 1200$ lb. to be taken by spikes directly into the chords from one diagonal. The detail shows six 30D spikes, which are good for 1164 pounds.

Where the end diagonals intersect the chords and posts, it is essential to secure a strong connection. To accomplish this purpose, two of the end diagonals have been made 2 by 10 in., and 1-in. bolts are used through the post splice-pads. The actual bending on these bolts is difficult, if not impossible to determine, depending somewhat upon the efficiency of the filler between the

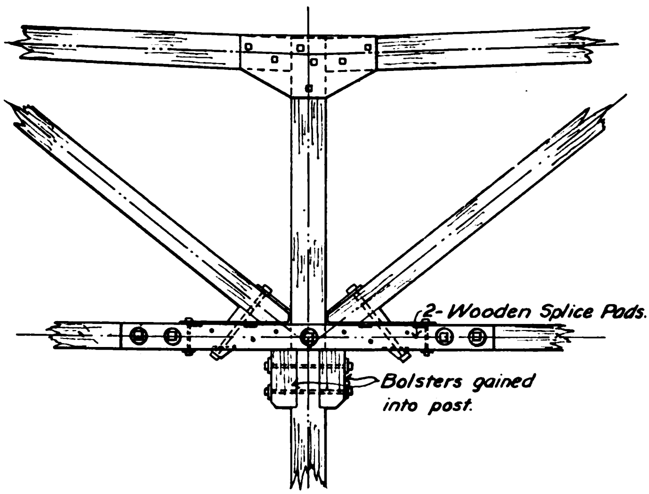

FIG. 78. TRUSS AND POST CONNECTION.

web-member and the chord. The bolting shown will be ample, however, to take care of the stresses indicated by Fig. 75*b*. Theoretically, the centre web-members take no stress for uniform loading. For this reason, they have been reduced in size to 2 by 6 inches.

Lower-Chord Splice. For determining the chord stress at the point of splice in the lower chord, the bending-moment diagram of Fig. 75*a* has been constructed. On a base of one-half the span of the truss, a rectangle is drawn with a height proportional to the maximum bending-moment at the centre. The base and the left side of the rectangle are then divided into the same number of equal parts, in this case, eight. From the upper right-hand corner of the rectangle, radiating lines are drawn to the division lines of the left side. The intersections of these radiating lines with the verticals erected on the base line at the points of division determine the parabola representing the bending-moment. From this bending-moment curve, the moment at the point of the splice is seen to be 124,500 lb.-ft., and the chord stress is therefore $\frac{124500}{5} = 25{,}000$ lb. The detail shows fourteen $\frac{7}{8}$-in. bolts, which from the

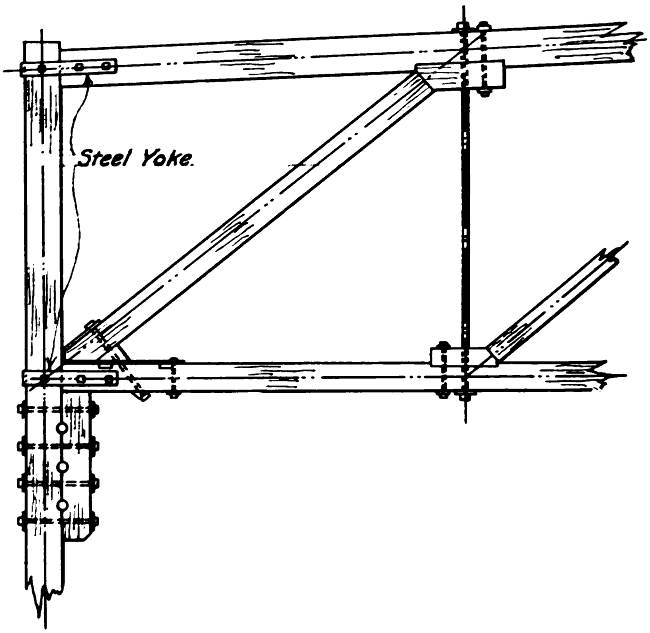

FIG. 79. POORLY DESIGNED TRUSS AND POST CONNECTION.

values given in Chapter V, have a resistance of 26,600 pounds.

Upper-Chord Splice. As the upper chord is in compression, a true butt-joint will be assumed, and the splice-pads designed merely for holding the joint together, and supplying some tensile resistance for unknown erection stresses. The post has been assumed as an 8 by 12-in. timber. As detailed, the truss rests directly upon the post, and for stiffness against lateral forces, two 4 by 12-in. splice-pads are provided, well-bolted to the post and to the truss.

The preceding discussion illustrates the method of design of a lattice-truss. While, as was noted previously, the stresses are indeterminate, the approximate stresses can be found, and a reasonably rational design made. In some instances, particularly where there is a ceiling-load to support, it may be advisable, and even necessary, to make the chords of four pieces, instead of two. Kidder's handbook gives the sizes for lattice-trusses of various spans and spacing, and recommends in all cases chords constructed of four timbers. This practice, in my opinion, is not advisable, since the deeper the chords, the more space there is available for bolting and spiking. It might be mentioned that the use of four planks to each chord results in what are termed 'cumulative stresses.' In other words, the two chord-timbers next to the web-members must not only take their own proportion of the total chord-stress, but must also transmit the part of the total stress borne by the two outside chord-timbers. This construction, therefore, results in an overstraining of portions of the inner chord-timbers.*

Truss Connection to Post

The method of connection of truss to post, illustrated in Fig. 71 and 74, furnishes what in my opinion, is the most efficient detail that can be devised. If there are

*In this connection, note the following statement from Kidder's 'Architects and Builders Pocket Book,' edition 1905, page 898, "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 the butt joint." This statement is misleading; in a lattice-truss with each chord built of four sticks, the upper chord needs through bolting to the same extent as the bottom chord. For splicing the tension member, special bolting and spiking is required. In general, the bolts between the joints will have to be spaced closer than 2 ft. centre to centre.

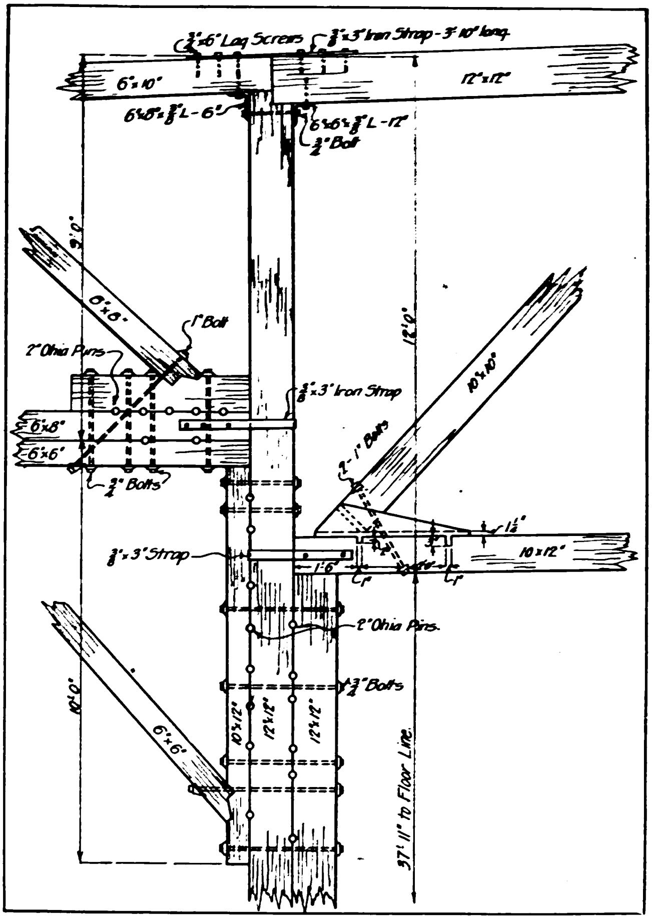

FIG. 80. TRUSS AND POST CONNECTION THAT IS WEAK AND YET WASTEFUL OF MATERIAL.

interior posts, that is, if there are several trusses across the width of the building, this detail can only be used for the outside, or wall posts, and the connection to the interior post or posts, will have to be modified. Fig. 77 and 78 illustrate two methods that may be used, Fig. 77 being for the Howe truss of Fig. 71. For this case, the end-detail of Fig. 71 must be abandoned, on account of lack of room, and a steel shoe substituted.

As between the two details of connections to post shown in Fig. 77 and 78, the particular circumstances of the building to be framed must determine the type of connection. The wind shear to be transferred across the post may require special treatment with a special detail. The post in the detail of Fig. 78 is considerably weakened by 'gaining' the bolsters into the post, and this reduced section is at the critical point for resistance to bending stresses from wind.

Fig. 79 shows a detail of connection of truss to a wall post which is not good, but which I have seen used to a considerable extent. In this detail, there is eccentricity of loading, and a consequent bending in the post, notwithstanding the fact that the centre lines of the post, lower-chord and batter-post intersect in a common point. Were the end of the chord to bear snugly against the post, and were the iron tie-strap to provide sufficient tensile connection between truss and post, the joint would not produce bending in the post. In actual construction, however, the truss will invariably be cut slightly short to facilitate erection. Even with an initial snug fit, shrinkage of the post will soon destroy this tight fit. Similarly, the post will shrink away from the yoke, and the value of the tie-strap be largely lost. The detail thus gives a false impression of stiffness. It is true that the joint may be tightened after shrinkage has taken place by shimming and wedging, but the chance of this extra work being done after the building is completed is small, and any connection which minimizes the effect of shrinkage is to be preferred.

It is sometimes instructive to learn how not to do things. Fig. 80 is a detail of a truss and post connection used in one of the concession buildings at the Panama-Pacific International Exposition. It is reproduced here to illustrate how it is possible to use a great quantity of material without obtaining great strength, and especially without gaining an appreciable amount of stiffness. Expensive construction does not necessarily mean strength. The principal defect of the construction shown is its lack of stiffness. The only ties between truss-chords and posts are the small steel tie-straps or yokes, fastened with lag-screws. As these yokes bear across the fibre of the posts, their maximum resistance limited by this stress is $36 \times 300 = 10,800$ lb. As a matter of fact, the pressure across the face of the post would never be uniform, as the strap is not stiff enough to so transmit the pull. The pressure would all be concentrated near the sides of the post, and would crush the corners of the post, should any amount of pull come upon the strap. As shown above, shrinkage of the timber would soon destroy the efficiency of these yokes.

Both end-details are objectionable. The double cut on the end connection of the left truss, with a small shearing area is inefficient. Double cuts similar to this introduce cumulative stresses, as the total horizontal component of the thrust of the batter-post must eventually come upon the shearing area between the inner or lower end-cut of the batter-post and the end of the bolster. The cast-iron shoe of the truss on the right is poorly designed. Here again the two different depths of lugs introduces cumulative stresses in shear. The thickness of 1 in. for the first lug with a depth of 2 in. is altogether too small. If the full stress ever came upon this lug, it would fail through flexure. For a unit-bearing pressure of 1600 lb. per sq. in. this lug would be stressed in flexure to 31,200 lb. per sq. in., acting as a cantilever. No bolts are provided to hold the inner lug in its cut in the timber.

While, because of the large live load figured on the truss, and the safety factor, it was in no danger of failure, the designs, of which this is an example, are not only uneconomical, but the owner of such a building is not getting security in proportion to money expended. A stronger and stiffer structure could have been secured at a less expense. When a competent engineer checks such a design, and points out, for example, the weakness of the tie-straps or the end-details, he is sometimes accused of attempting to add material unnecessarily, and to prove the claim, the sizes of the different members are pointed out, as sufficient evidence of the safety of the structure. Sometimes, one of the most difficult things to make an owner realize, is that heavy members of trusses and posts do not necessarily indicate a strong construction.

CHAPTER XI

Theory of Column-Action—Tests of Timber Columns

Considered from the standpoint of safety of construction alone, the design of a solid timber post to support a concentric vertical load is merely a question of selecting the column formula to be used, and providing the required area as determined by this formula. In this respect, a column is no different from a strut of a timber truss. Column formulas were discussed to some extent in Chapter X, and working values may be selected from those formulas.

Theory of Column Action

The phenomenon of column action is best established by the Rankin or Gordon formula, and without attempting to go extensively into the mathematics of this 'theoretical' formula, it will be of interest to examine briefly the history and derivation of the component parts of the expression. A full discussion of column action may be found in any standard work on structural mechanics, for example, Merriman's 'Mechanics of Materials,' Church's 'Mechanics of Engineering,' Burr's 'The Elasticity and Resistance of the Materials of Engineering,' and others. The following discussion is presented for the purpose of emphasizing the importance of the effect of eccentric loading, by calling attention to what many engineers are inclined to forget, namely, that bending is an important part of long-column action even with concentric loading. I cannot do better than quote from the text of Burr mentioned above, as follows: "There is a class of members in structures which is subjected to compressive stress, and yet whose members do not fail entirely by compression. The axes of these pieces coincide, as nearly as possible, with the line of action of the resultant of the external forces, yet their lengths are so great compared with their lateral dimensions, that they deflect laterally, and failure finally takes place by combined compression and bending. Such pieces are called 'long columns,' and the application to them, of the common theory of flexure, has been made in Article 24." And from Article 24, "A 'long column' is a piece of material whose length is a number of times its breadth or width, and which is subjected to a compressive force exerted in the direction of its length. Such a piece of material will not be strained or compressed directly back into itself, but will yield laterally as a whole, thus causing flexure. If the length of a long column is many times the width or breadth, the failure in consequence of flexure will take place while the pure compression is very small." Mr. Burr then develops Euler's formula for long columns, which is

$P = \frac{4\pi EI}{L^2}$, where E is the modulus of elasticity and I is the moment of inertia.

"It is to be observed that P is wholly independent of the deflection, that is, it remains the same, whatever the deflection, after the column begins to bend. Consequently, if the elasticity of the material were perfect, the weight P would hold the column in any position in which it might be placed, after bending begins." The above formula is the basis of 'Hodgkinson's formula,' for the resistance of long columns.

"Two different formulas were first established for use in estimating the resistance of long columns; they are known as 'Gordon's formula' and 'Hodgkinson's formula.' Neither Gordon nor Hodgkinson, however, gave the original demonstration of either formula. The form known as Gordon's was originally demonstrated and established by Thomas Tredgold—while that known as 'Hodgkinson's formula' was first given by Euler. In 1840, however, Eaton Hodgkinson, F.R.S., published the results of some most valuable experiments made by himself on cast and wrought-iron columns—and from these experiments he determined empirical coefficients applicable to Euler's formula, on which account it has since been called Hodgkinson's formula. Mr. Lewis

Gordon deduced from the same experiments some empirical coefficients for Tredgold's formula, since which time it has been known as Gordon's formula."

With this brief history of the origin of these famous column formulas, we may go at once to the derivation of Gordon's formula for long columns. "Since flexure takes place if a long column is subjected to a thrust in the direction of its length, the greatest intensity of the stress in a normal section of the column may be considered as composed of two parts. In fact the condition of stress in any normal section of a long column is that of a uniformly varying system composed of a uniform stress and a stress-couple. In order to determine these two parts, let S represent the area of the normal cross section; I, its moment of inertia about an axis normal to the plane in which flexure takes place; r, its radius of gyration in reference to the same axis; P, the magnitude of the imposed thrust; f, the greatest intensity of stress allowable in the column; and D, the deflection corresponding to f. Let p' be that part of f caused by the direct effect of P, and p'' that part due to flexure alone. Then, if h is the greatest normal distance of any element of the column from the axis about which the moment of inertia is taken, by the common theory of flexure,

$$c'PD = \frac{p''I}{h}; \text{ therefore } p'' = \frac{c'PDh}{I}$$

Also,

$$p' = \frac{P}{S}, \text{ therefore } p' + p'' = f = \frac{P}{S}\left(1 + \frac{c'SDh}{I}\right)$$

Hence, $$P = \frac{fs}{1 + \frac{c'SDh}{I}}$$

This equation may be considered one form of Gordon's formula.

Burr then shows that $D = a\frac{L^2}{h}$, in which expression a is a constant. Making this substitution, and expressing I in terms of S and r, there results the formula,

$$P = \frac{fs}{1 + a\frac{L^2}{r^2}}$$

The preceding treatment illustrates clearly that column action, for long columns, or, as has been stated, for timber columns whose length is greater than twenty times the least cross dimension, consists of a uniform compression plus a cross-bending. Since there is a flexural stress on such columns for concentric loads, it is obvious that the addition of bending moment due to eccentric loading decreases the strength of the column, and that the proportion of flexural stress to the total stress for eccentrically loaded columns is greater the longer the column in proportion to its least width. Further, it may be said that eccentrically loaded columns are in the realm in which the fewest tests for strength have been made. It follows, then, almost as an axiom, that eccentrically loaded columns should be avoided wherever possible, and that where they must be used, careful consideration of the maximum combined unit-stress must be made in order that such maximum unit-stress shall not exceed the safe unit-stress for the column.

Uneven ends on columns, or ends not exactly normal to the axis of the stick will produce eccentricity. In fabricating steel columns, care is always taken to mill the ends of the column to a true and even bearing, and the bearing or base plates are usually planed to an even surface and a uniform thickness. On the other hand, the timber column even though it may carry heavy loads, as in warehouse or heavy mill-construction, is at best trued by a carpenter's square.

The ultimate strength of timber columns is not a matter of definite knowledge, because of the lack of sufficient tests on full-sized columns. This is especially true of long columns; for example, columns whose length is from 40 to 60 times the least width. It is interesting to note that the formulas of the American Railway Engineering Association give a value of zero for the safe working stress for a column whose $\frac{L}{d}$ is 60. On the other hand Ketchum would allow a working stress of 480 lb. per sq. in. for this column, and the formula of the U. S.

Department of Agriculture, Forestry Division, gives a value of approximately 500 lb. per sq. in. for the same column, when C is taken at 1600 lb. per sq. in. This wide variation in recommended working values is unfortunate, and might cause the layman to believe that the engineer's formulas were worthless. The practical meaning of this variation is that the actual strength of columns with a large $\frac{L}{d}$ is uncertain. Columns with a greater $\frac{L}{d}$ than 20 will generally fail by lateral buckling, a fact which has been definitely proved by tests on full size specimens.

Tests of Timber

The published data on the strength of full-sized timber columns is meagre; practically all of the recorded tests were made by the United States Government at the Watertown arsenal. Undoubtedly some other tests on small columns have been made in technical schools and colleges, but the results of these are not generally known.

In Fig. 81 the ultimate strengths of the timber col-

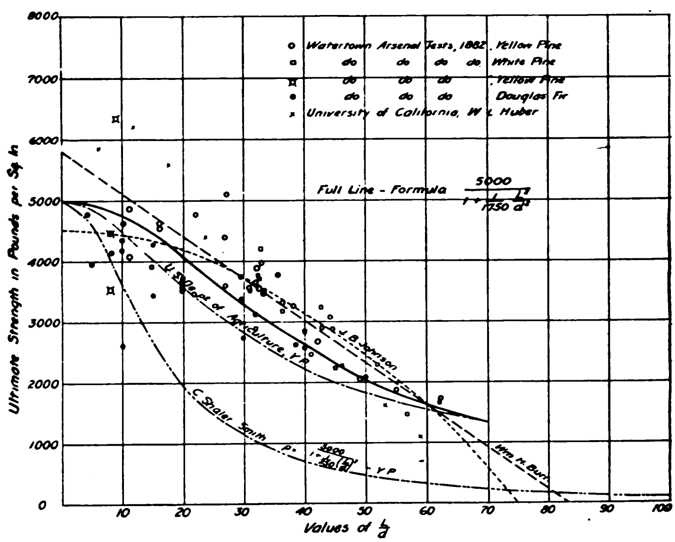

FIG. 81. RESULTS OF TESTS OF WOODEN COLUMNS.

umns tested at the Watertown arsenal are plotted. These values were taken from the digest of the tests made by J. B. Johnson and W. H. Burr. As these tests were published in 1882, I have added the results of some subsequent tests made at the same laboratory, also a few tests made by W. L. Huber on small Douglas fir columns, but with a wide variation in the ratio of length to least width. These sticks were 1.7 in. square, and the tests were a part of the regular course in the testing laboratory of the University of California. The Watertown arsenal tests of 1882 were on yellow pine and white pine. The size of these columns varied from 5.3 by 5.3 in. by 27 ft. 6 in. to 8.25 by 8.25 in. by 15 ft. For the case of the white-pine tests, I have arbitrarily increased the recorded values by the ratio 1.52 in order to give more data on the variation of strength with the change in $\frac{L}{d}$. This procedure is in error to some extent; it would be correct only if the ratio of the compressive strengths of the two timbers was the same as the ratio of the respective moduli of elasticity in bending, and if the moduli of elasticity bore a constant relation to the respective compressive strength throughout the range of the tests. It will be seen, by referring to Fig. 14 of a preceding chapter that the ratio of the compressive strengths of long-leaf yellow pine to white pine is $\frac{6500}{4400} =$ 1.475, while the ratio of the respective moduli of elasticity is $\frac{1610000}{1130000} = 1.425$; the average is 1.45. The Watertown arsenal tests on short columns of the same timbers, that is, columns which failed by compression alone with no lateral deflection, showed the average ultimate strength of yellow pine to be 4442 lb. per sq. in., while the same quantity for the white pine was 2414 lb. per sq. in. These figures give a ratio of 1.84. The ratio used (1.52) is the average of the ultimate strengths of the columns with an $\frac{L}{d}$ of 22 and over. The difference between the two figures shows the influence of bending.

Table XVIII gives the results of some tests on Douglas fir columns published in 'Tests of Metals,' 1896. These results are also incorporated in Fig. 81.

TABLE XVIII

TESTS ON DOUGLAS FIR COLUMNS*

Least width, in.	Length		$\frac{L}{d}$	Ultimate strength, lb. per sq. in.	Modulus of elasticity
8.12	26 ft.	0.125 in.	38.4	2600	1,651,000
10.12	25 "		29.6	3700	1,875,000
10.04	25 "		29.9	2700	1,785,000
8.18	20 "		29.3	3371	1,704,000
10.10	16 "	8.00 in.	19.7	3500	1,875,000
10.06	16 "	8.00 "	19.9	3700	1,639,000
8.21	13 "	3.75 "	19.5	3600	1,756,000
10.12	12 "	6.00 "	14.8	3900	1,854,000
10.08	12 "	6.00 "	14.9	3400	1,393,000
8.08	9 "	11.8 "	14.9	4249	1,791,000
10.12	8 "	3.90 "	9.9	4312	1,743,000
9.98	8 "	4.05 "	10.0	4138	1,792,000
10.07	6 "	8.13 "	8.0	4100	1,963,000
7.92	6 "	7.98 "	9.9	2600	1,904,000
10.07	4 "	1.94 "	5.1	4626	1,675,000
8.13	3 "	4.00 "	5.0	3988	....

*Tests of metals, Watertown Arsenal.

Various Formulas for Ultimate Strength. In Fig. 81 are shown some of the various formulas for ultimate strength of yellow-pine timber columns. W. H. Burr, from the results of the Watertown arsenal tests advocates the following straight line formula

$p = 5800 - 70\,\frac{L}{d}$, p being the ultimate strength in pounds per square inch, this formula to be used only between the limits 20 $\frac{L}{d}$ and 60 $\frac{L}{d}$.

On the basis of the same tests, J. B. Johnson proposed the parabolic formula

$p = 4500 - 1.0\,\frac{L^2}{d^2}$, this formula to be used between the limits $\frac{L}{d} = 1$ and $\frac{L}{d} = 50$. At the latter limit the parabola is tangent to the curve of Euler's formula $p = \frac{4\pi EI}{L^2}$ when $E = 1{,}620{,}000$ lb. per sq. in. This formula is for partially seasoned yellow-pine columns. For dry long-leaf yellow-pine columns, he proposed the formula $p = 6000 - 1.5 \frac{L^2}{d^2}$.

The U. S. Department of Agriculture formula, is also shown, with $C = 4500$ lb. per sq. in.

W. H. Burr in his text already quoted, states that some 1200 tests on full sized specimens of square and rectangular yellow-pine columns were made by C. Shaler Smith for the Ordnance Department of the Confederate Government, and that the results indicated that the following formulas represented the ultimate strengths of the columns.

1. For green, half-seasoned sticks answering to the description, 'good merchantable lumber'

$$p = \frac{5400}{1 + \frac{1}{250} \frac{L^2}{d^2}}$$

2. For selected sticks, reasonably straight and air-seasoned under cover for two years and over

$$p = \frac{8200}{1 + \frac{1}{300} \frac{L^2}{d^2}}$$

3. For average sticks cut from lumber which had been in open-air service for four years and over

$$p = \frac{5000}{1 + \frac{1}{250} \frac{L^2}{d^2}}$$

The tables for strength of timber columns as given in Trautwine's 'Handbook' are based on the Shaler Smith formulas. These formulas are of the Rankin or Gordon form. It is of interest to note that the curves of the Shaler-Smith formulas do not fit any of the tests of the Watertown arsenal, as may be seen by reference to Fig. 81, where the last formula has been plotted.

In Fig. 81, I have plotted a curve of the Rankin-Gordon type which seems best to fit the results of the tests there shown, and find as noted in the figure, that the coefficient a has a value of about 1750 instead of 250, as found by Mr. Smith. As no numerical results of Mr.

Smith's tests are to be found, no comment can be made with regard to the difference between his proposed formulas and those of later engineers.

The elastic limit is high in proportion to the ultimate strength in a timber column. The average ratio as measured by the stress-deformation curves of Mr. Huber's tests is about 84%, while on some similar tests on 3½ by 3½ in. redwood columns, I found the proportion about 90%.

On the basis of the proposed column formula,

$$p = \frac{5000}{1 + \frac{1}{1750}\frac{L^2}{d^2}}$$

there is given in table XIX, the ultimate strength of Douglas fir timber columns, for the case of partially seasoned timber of the No. 1 common grade.

TABLE XIX

ULTIMATE AND WORKING STRENGTHS OF DOUGLAS FIR COLUMNS

Formula: $p = \frac{5000}{1 + \frac{1}{1750}\frac{L^2}{d^2}}$

$\frac{L}{d}$	Strength in lb. per sq. in. Ultimate	Working
10	4740	1355
12	4630	1325
14	4510	1290
16	4350	1245
18	4210	1205
20	4060	1161
22	3910	1120
24	3760	1075
26	3600	1030
28	3450	986
30	3310	946
32	3150	902
34	3020	864
36	2880	823
38	2740	785
40	2620	750
42	2490	712
44	2370	678
46	2260	646

$\frac{L}{d}$	Strength in lb. per sq. in. Ultimate	Working
48	2160	618
50	2060	589
52	1960	560
54	1870	535
56	1790	512
58	1715	490
60	1635	421

Working Strength of Timber Columns. Taking into consideration the adverse effect on the strength of timber of knots or oblique grain, the possibility of uneven end-bearing, eccentricity due to imperfect beam or girder connections, the effect of long-continued loads of large magnitude, and the relatively few tests on full-size sticks of a large ratio of length to least width, a safety factor of 3½ on the basis of ultimate strength as given by the tests quoted above would seem to be the lowest that should be used, and this only for buildings. The factor should be increased to five for unprotected structures, such as bridges or other outdoor construction.

Table XIX also gives the safe unit stresses for buildings based on the modified Rankin-Gordon formula with $a = 1750$, and a safety factor of 3½.

CHAPTER XII

Column Splices and Girder Connections—Floor Girders and Joists—Joist Hangers—Mill Construction

Column Connections. Other conditions than the allowable stress under column action often determine the size of a post in a timber-framed building, for example, the required cross-sectional area at the ends of the post to provide bearing area for the beams, girders, or trusses resting on the post, or the requirement of a general minimum size of column to give the proper stiffness to the building.

Except in the case of columns supporting floors or roof-bays of uniform size, the ideal condition of concentric loading will seldom be realized. Unless care is taken in the detailing of connections, wall columns will usually be loaded eccentrically, producing bending in the posts, the amount varying not only with the numerical value of the load and its eccentricity, but also with the nature of the connections. The case of truss connections to posts was discussed in the preceding chapter, where it was pointed out that many details involve considerable resultant bending.

Fig. 82, *a*, *b*, and *c* illustrate details sometimes seen in building designs. The defects in these three details are self-evident. In *a*, it is almost certain that the girders have not sufficient bearing area to prevent crushing of the fibres. If the upper post is working at an efficient stress, the fibres at the top and bottom of the girders must be stressed above their elastic limit. This condition will produce settlement of the upper floors, which, added to the shrinkage of the timbers, will crack plaster walls, or produce uneven floors.

In Fig. 82*b* sufficient area for bearing is given to the girders by the bolster, but both the top and the bottom of the bolster are probably over-stressed in cross-bearing. The shrinkage in this case will be that of the bolster only.

Fig. 82*c* shows the most defective details. Here the settlement because of shrinkage is the greatest.

With the use of a hardwood bolster, the crushing of the fibres of the bolster may be reduced, and possibly eliminated, although it must be remembered that even oak has an elastic limit across the grain of only approxi-

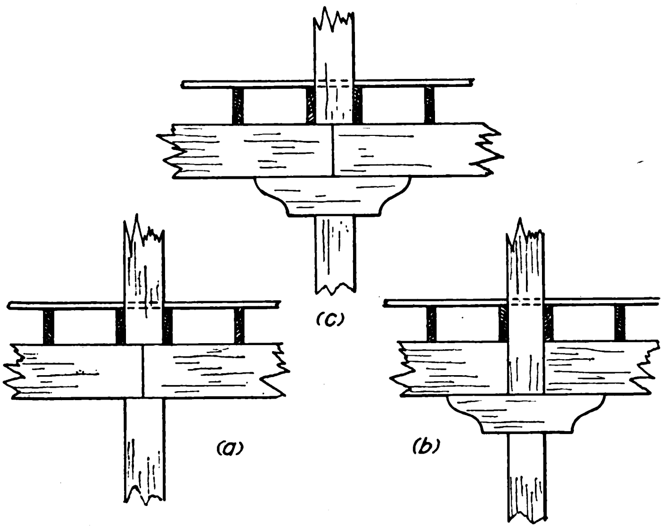

FIG. 82. EXAMPLES OF DEFECTIVE DETAILS.

mately 920 lb. per sq. in., for green timber, or about 50% greater than Douglas fir.*

To overcome the disadvantages of wooden bolsters, metal post-caps of cast-iron, wrought-iron or steel are commonly employed. Standard post-caps, usually of pressed steel, can be bought in the open market. Typical details of post-cap framing are shown in Fig. 83, the illustration being taken from 'Structural Timber,' Engineering Bulletin No. 2, published by the National

*These values are from the table of unit stresses adopted by the American Railway Engineering Association, as given in Table I, Chapter III.

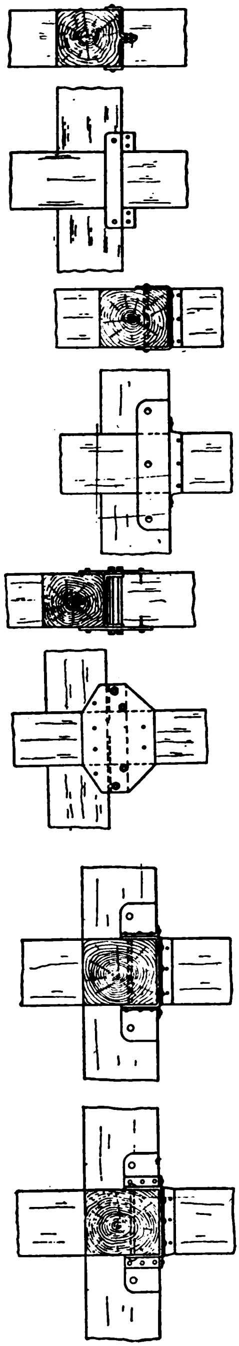

FIG. 83. POST-CAP DETAILS.

Lumber Manufacturers Association. Some of the more common of these post-caps are the Duplex, Goetz, Van Dorn, and on the Pacific Coast, Falls caps, and others. The prices of these vary considerably, and on a large job, it may be possible to build up structural post-caps that will meet the requirements and at the same time be cheaper. Four-way post-caps are open to the objection of resulting in unequal shrinkage, where wooden girders are used, since the joists supported by the girders will drop an amount equal to the shrinkage of the girder, while the joist or beam resting on the post-cap will not drop. This will occur even with the use of joist-hangers, except that when hangers of the Duplex type are used, the shrinkage will be only half of that of the type of hangers which fasten over the top of the girders, since the Duplex joist-hanger is secured to the girder by means of a circular nipple inserted into the girder at slightly above the centre of the depth of the girder.

Where the absence of ceiling will permit, the details of joints shown in Fig. 84, *a, b, c,* and *d,* will be found, on analysis, to be free from the defects of the connections shown in Fig. 82. The bolster-blocks are either dapped into the lower post, or bolted and keyed. In either case, they have end bearing, while their section may be large enough to provide ample bearing for the girders. Where the size of posts decreases with the succeeding stories, the trimming of the end of the lower post to the section of the upper post will ordinarily provide sufficient area for the bearing of the bolster blocks. The size of the bolts in Fig. 84, *a* and *b,* may be determined by taking moments about the centre of the lower bolster bearing. Thus, in Fig. 84*a,* neglecting the effect of the lower bolts, because of their short distance above the end of the bolster, the tension in the upper bolts may be found from the equation

$$T = \frac{Pa}{h},$$

where P is the reaction of the girder, a is the horizontal distance from the centre of the upper end of the bolster to the centre of the gain in the post, and h is the distance from the upper bolts to the lower end of the block. The working stress for the circular keys or pins may be taken from the tests mentioned in the preceding chapters. Pipe pins, 2 in. external diameter, and of extra-heavy section may be considered good for 800 lb. per lin. in. of pin. Oak, as has been shown, is practically worthless, and the same is true of gas-pipe. The

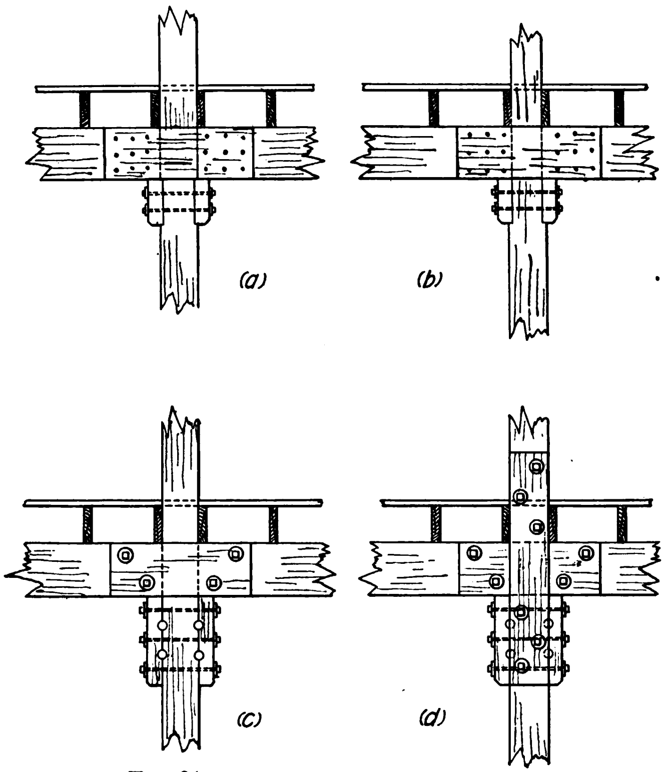

FIG. 84. RECOMMENDED JOINT DETAILS.

bolts should be designed to take a tension equal to the reaction of the girder.

It will be noted that in these details, the normal spacing of the joists has been modified at the post, to provide a joist at either side of the post. This is an inexpensive way to secure considerable stiffness in the building. The two joists are to be either spiked or bolted to the post, as the requirements for stiffness may warrant. In Fig. 84*d* the girders are shown tied together across the column by means of two wooden splice pads. The two sections of posts may be similarly tied together by the use of splice-pads, with fillers under them, of the thickness of the girder-splices. A one-inch thickness of girder-pad will usually give sufficient tie, if it is long enough to give the required spiking or bolting area.

Connection of Joists to Girders. The cheapest and most satisfactory manner of supporting floor joists is to rest them upon the girders. There is no device that is as satisfactory as putting the support directly under the load, without resorting to bending or shearing of metal. In buildings with wooden or corrugated-steel walls, the extra height of building and consequent expense resulting from this form of construction will be justified. On the other hand, in the case of a building with masonry walls, and several stories high, resting the floor joists on top of the girders in place of attaching them to the girders by means of metal hangers may add six or seven feet to the height of wall. From the standpoint of cost of construction alone, the cost of the extra walls may be considerably more than the cost of the necessary joist-hangers.

The danger of unequal shrinkage resulting from the use of hangers has been mentioned. On the other hand, when all the joists rest upon the girders directly, while the floors will settle uniformly through shrinkage, the floors will not remain level, since the wall-bays will drop at their inner ends the amount of the girder shrinkage plus the joist shrinkage, and the outer or wall ends will settle the amount of the joist shrinkage alone.

Where it is found necessary or advisable to employ joist-hangers, special hangers may be designed, or some of the standard makes on the market may be used. The standard makes may be divided into two classes, those of the duplex type which, as has been mentioned, are secured to the girders by means of an inserted nipple, and those which fasten by arms or straps which fit over the tops of the girders. The two types are illustrated by Fig. 85 and 86.

Joist-hangers should not be used indiscriminately, that is, without investigation as to their fitness for the particular case, and their ability to withstand the particular loads. *Engineering News* of November 20, 1902,

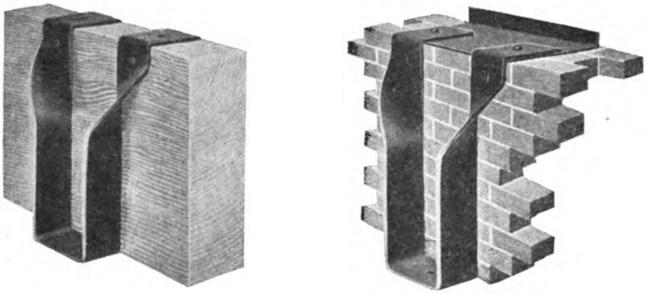

FIG. 85. TYPES OF JOIST HANGERS.

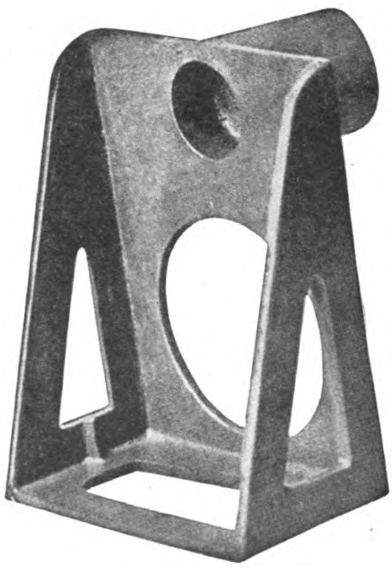

FIG. 86. DUPLEX JOIST-HANGER.

Vol. 48, page 420, describes the collapse of a building in Minneapolis through failure of joist-hangers, although these were of special design, and not standard hangers. The late F. E. Kidder discusses the general question of joist-hangers, in connection with this failure, in the subsequent issue of January 15 and February 5, 1903, Vol. 49, *Engineering News*. Kidder notes several tests as follows: (1) a standard hanger of the second class mentioned above made of $\frac{3}{8}$ by $2\frac{1}{2}$-in. wrought iron, which failed at 13,750 lb., or a unit stress in tension on the iron of 7333 lb. per sq. in.; (2) a Van Dorn hanger (type 2), where the arms began to straighten out at 13,300 lb., and failed at 18,750 lb.; (3) a double stirrup of $\frac{3}{8}$ by $2\frac{1}{2}$-in. wrought iron carrying two 8 by 12-in. timbers over one 12 by 14 in. failed at a load of 28,825 lb. on each side, or at a tensile stress of 15,273 lb. per sq. in.; (4) a duplex No. 35 hanger with a nipple $2\frac{7}{8}$ in. in diameter and $3\frac{1}{2}$ in. long, broke under a load of 39,950 lb. The bearing under the lower half of the nipple was 1977 lb. per sq. in., yet the compression on the wood and the effect on the girder was slight. The hanger failed by breaking of the iron directly under one of the nipples. Another duplex hanger of the same size failed at 38,000 pounds. These hangers are shown in Fig. 87.

Kidder points out that the first point of weakness in a joist-hanger of the stirrup type is the bending of the top strap, and the crushing of the fibres on the joist side of the top of the girder; the second point of weakness is the bending of the bottom of the stirrup supporting the joist, or the tendency to shear. The tests quoted above show that the metal of a joist-hanger does not fail by direct tension.

Referring to the first test noted by Kidder, the equivalent load on the 6 by 12-in. beam would be 26,000 lb. A 6 by 12-in. beam on a 10-ft. span is good for 17,100 lb. at a maximum fibre stress of 1800 lb. per sq. in.; the safety factor was therefore approximately $1\frac{1}{2}$. The double stirrup of $\frac{3}{8}$ by $2\frac{1}{2}$ in. failed at 28,825 lb. on each side, or at an equivalent load on the beam of 57,650 lb. An 8 by 12-in. beam on a 10-ft. span will carry 22,800 lb.; the safety factor was therefore $2\frac{1}{2}$.

In a catalogue of a standard joist-hanger there is published the result of some tests made for the company by

FIG. 87. JOIST-HANGERS AFTER TESTING.

TABLE XX

STRENGTH OF JOIST-HANGERS*

Size of hanger, in.	Size of material, in.	Initial load, lb.	Maximum load, lb.	Safe load on beam, lb.	Safety factor on initial failure	Tensile stress in steel at maximum load, lb. per sq. in.	Unit bearing pressure across grain on joist at maximum load, lb. per sq. in.
2 by 6	$\frac{1}{8}$ by $1\frac{3}{4}$	5,270	17,460	1,410	3.74	20,000	2,500
2 by 10	$\frac{3}{16}$ by $1\frac{3}{4}$	12,150	20,120	3,940	3.09	15,300	2,870
2 by 12	$\frac{3}{16}$ by 2	7,470	24,110	5,700	1.31	16,000	3,000
3 by 12	$\frac{3}{16}$ by $2\frac{1}{4}$	23,530	28,730	8,540	2.75	14,000	2,130
4 by 10	$\frac{1}{4}$ by $2\frac{1}{4}$	19,070	28,270	7,890	2.42	12,600	1,570
4 by 12	$\frac{1}{4}$ by $2\frac{1}{2}$	16,980	36,250	11,380	1.49	14,500	1,810
6 by 10	$\frac{1}{4}$ by 3	25,910	34,650	11,800	2.20	11,500	962
6 by 12	$\frac{3}{8}$ by 3	34,840	52,650	17,100	2.04	11,650	1,460
10 by 14	$\frac{3}{8}$ by $3\frac{1}{2}$	24,730	54,450	35,300	....	10,400	777

*The above relation between the safe load on the beam and the 'Initial load' is believed to be a correct interpretation of the results as given by the report. In both reports, the load for one hanger is not given directly. The loads given under 'Initial load' and 'Maximum load' are the total loads sustained by two single hangers. Without a careful reading of the description of the tests, the numerical results might be misleading.

The computed unit bearing pressures across the grain of the joists do not exist; they are calculated on an assumption of uniform bearing, to show that the practical capacity of the hanger, as would be limited by its use with Douglas fir, would be far below that of the maximum loads given.

a firm of testing engineers. The letter from the engineer is reproduced in the catalogue, and I quote the following significant statement: "The joist-hangers being tested on this occasion were taken from their regular stock, and of the following sizes, 2 by 12-in. and 4 by 12-in. They were mounted on heavy headers *with thin pieces of plate to prevent the arms from crushing the rough pine used and in order that the strength of the hanger could be tested, and not the lumber.*"*

In the same catalogue is published a similar letter of later date, embodying the results of further tests. These results are given in Table XX. in which I have also noted the equivalent load that the corresponding beam would stand with a 10-ft. span at a maximum fibre stress of 1800 lb. per sq. in., and the safety factor of the beam at the initial load (load at which the arm of the hangers began to rise). Certain extracts of the letter are also quoted as follows:

"All of the above hangers were mounted on eucalyptus timbers,† adhering as nearly as possible to the usual form of construction, and were spiked to the headers, after which the joists were dropped into place.

"Two readings were taken: The first one (initial load) at the moment the arm of the hanger began to rise, and the final (maximum load) when the arms straightened, and the timbers crushed so that further recording was

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*The italics are my own; this point is not emphasized in the catalogue.

†The strength of eucalyptus timber, grown in California, was investigated by E. L. Soulé and Thomas Williamson in 1904, as thesis work at the University of California in a series of 95 tests. The elastic limit of the timber in crushing, at right angles to the direction of the fibres, was found in 19 tests to be as follows: Maximum 1679 lb. per sq. in., minimum, 964 lb. per sq. in., and average 1368 lb. per sq. in. The corresponding moisture content was 49.6%, 36.4%, and 43.4%. The results of the tests on the hangers are not, therefore, directly applicable to Douglas fir, which, when green has an elastic limit across the fibres of 570 lb. per sq. in., as against 1368 lb. per sq. in. of eucalyptus. The strength of a joist-hanger is just as much a question of the capacity of the timber as of the hanger.

useless. Crushing of the timbers occurred when testing the 6 by 12-in. and 10 by 14-in. sizes."

The values in the table representing the unit bearing-pressures on the bottom of the joists are not necessarily the correct values; in fact, it is certain that they are incorrect. The values there given are computed on the basis of an even distribution of loading over the seat of the hanger. This condition probably existed in no case, but the values for the thin joist are more nearly correct than for the thicker joists, as the ratio of thickness of metal to thickness of joist (span of seat of hanger acting as a beam) is much greater than in the case of the large joists. In the latter instances, the pressure would all be concentrated at the sides of the joist, and the unit pressure may have been three or four times that given in the table.

One other point is of interest to mention, in connection with Kidder's report of the test of the duplex hanger. He states, as was noted, that the unit bearing-pressure of the $2\frac{7}{8}$ by $3\frac{1}{2}$-in. circular nipple was 1977 lb. per sq. in. at the time of failure, with but small effect in compression on the wood. H. S. Jacoby, in his 'Structural Details' notes with respect to the duplex or Goetz hanger, "it may be assumed, according to the results of tests, that the safe load is limited only by the safe bearing-value of the cylindrical bearing surfaces on the sides of the fibres of the supporting beam. As shown, the effective bearing area equals the horizontal projection of the cylindrical surface when the direction of pressure is perpendicular to the fibres." If the average unit pressure of a cylindrical metal pin be taken as the limiting pressure perpendicular to the grain, the nipple of the hanger (for longleaf pine) would have crushed the girder at a load of 10,500 lb. for green timber, and possibly 15,000 lb. for dry timber. This is additional evidence that such a consideration of cylindrical bearing is in error, as was already discussed in Chapter IV. On the basis of the theory there proposed, Kidder's computed unit bearing-pressure would represent approximately the elastic limit of the timber.

CALCULATIONS

Capacity of two circular nipples, 2⅞ by 3½ in., according to usual method $= 2 \times 2\frac{7}{8}$ in. $\times 3\frac{1}{2}$ in. $\times$ 520 lb. per sq. in. $\times$ 1.50. (Increase of 50% for probable condition of seasoned timber; value of 520 lb. per sq. in. is from Table I, Chapter III, for longleaf pine, green condition of timber, elastic limit in compression across the fibre) = 15,650 pounds.

Capacity of same nipples by method of Chapter IV $= 2 \times 2\frac{7}{8}$ in. $\times 3\frac{1}{2}$ in. $\times [(\frac{2}{3} \times 520) + (\frac{1}{3} \times 3500)] \times 1.50 = 45{,}600$ lb. (The figure 3500 is the elastic limit for green longleaf pine, as taken from Forest Service Bulletin 88.) The actual load on the hanger as tested was 39,950 pounds.

Mill Construction. The preceding discussion and the illustrations of details have not considered the question of fire risk, and some of the details are open to criticism from this standpoint. This subject is one that is treated to considerable extent in Kidder's 'Pocket Book,' and in other books. Engineering Bulletin No. 2, of the National Lumber Manufacturer's Association, entitled 'Structural Timber, Heavy Timber Mill Construction Building,' dealing with this construction, has been issued recently. Valuable information, including many tables for strength of timber structural members, is also given in the 'Structural Timber Hand Book on Pacific Coast Woods,' published by the West Coast Lumbermen's Association, with headquarters at Seattle.*

From the standpoint of fire-protection in buildings with a timber-framed interior construction, and brick walls, it is advisable to have all sections of beams, girders, and posts of as large section as practicable, even at the cost of economy in framing. Beams framing into

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*Both of these publications are of great value to those engaged in timber construction; the former for its presentation of the requirements of mill construction in accordance with the standards of the National Board of Fire Underwriters, and the latter for its many tables of strength of beams and columns.

walls should be self-releasing in case of fire, so that if the timber beams burn and fall, they will not pull the wall with them. Similarly, many standard post-caps are designed with the idea that the girders will pull out of their seats, if they burn and fall, without pulling down the post with them. It may be of interest to quote the definition of mill construction from the bulletin of the National Lumber Manufacturers' Association. This type of construction is divided there into three classes as follows:

"1. Floors of heavy plank laid flat upon large girders which are spaced 8 to 11 ft. on centres. These girders are supported by wood posts or columns spaced from 16 to 25 ft. apart. This type is often referred to as 'Standard Mill Construction.'

"2. Floors of heavy plank laid on edge and supported by girders which are spaced from 12 to 18 ft. on centres. These girders are supported by wood posts or columns spaced 16 ft. or over apart, depending upon the design of the structure. This type is called 'Mill Construction with Laminated Floors.'

"3. Floors of heavy plank laid flat upon large beams which are spaced from 4 to 10 ft. on centres, and supported by girders spaced as far apart as the loading will allow. These girders are carried by wood posts or columns located as far apart as consistent with the general design of the building. A spacing of from 20 to 25 ft. is not uncommon for columns in this class of framing where the load is not excessive. This type is more generally known as 'Semi-Mill Construction'."

Also, from the Building Code Recommended by the National Board of Fire Underwriters:

"Wooden girders or floor timbers shall be suitable for the load carried, but in no case less than 6 in., either dimension, and shall rest on iron plates on wall ledges, and where entering walls, shall be self-releasing. Walls may be corbelled out to support floor timbers where necessary. The corbelling shall not exceed 2 inches.

"So far as possible, girders or floor timbers shall be single sticks. Width of floor bays shall be between 6 and 11 feet.

"The practice in mill-construction of supporting the ends of beams on girders by means of metal stirrups or bracket hangers is objectionable. Experience has shown that such metal supports are likely to lose their strength and collapse when attacked by fire.*

"Floors shall not be of less than 3 in. ($2\frac{3}{4}$ in. dressed) flooring laid crossways or diagonally."

*No difference in insurance rate, however, will be made for this factor alone.

CHAPTER XIII

Foundations

The three cardinal principles of foundation design are met when (1) the safe bearing-pressure on the soil is not exceeded, (2) all footings exert the same pressure per unit area on the soil, and (3) the individual footings are each strong enough to withstand the loads coming upon them. The fulfilment of the preceding conditions involves the careful calculation of all loads coming upon the several piers or wall-footings, and the proportioning of the details so that such piers and footings may be strong enough in all their parts to distribute the individual loads with safety.

To an engineer, these principles are so self-evident that it seems redundant even to mention them. In structures of importance, such as bridges and steel-framed buildings, careful attention is paid to all these considerations. In timber-framed buildings, the second principle, that of providing equal bearing on all footings, is commonly neglected. The footings of a timber-framed building, unlike those of a steel-framed building, are usually the last details to be designed. The common practice is to compute approximately the maximum load on any one pier, design this pier accordingly, and either make all the others the same size, or else to establish their dimensions arbitrarily.

In a similar manner, the sills of walls are often made of the same size throughout the building, although the different walls will probably carry widely varying loads. Much of the cracking of plaster walls in dwelling-houses is on account of the unequal and often insufficient bearing of the foundation on the soil.

Foundations may be divided into two kinds for the purpose of this discussion, permanent and temporary.

It is not my intention to discuss the design of permanent footings. At the present time these are usually built of concrete, either plain or reinforced. Brick is also sometimes used. Discussion of the design of concrete footings and piers may be found in the numerous texts on concrete and reinforced concrete.

When a timber post rests on a concrete footing, it may be necessary to use a steel base-plate under the post. This will serve two purposes, first, to distribute the load of the post over the concrete, in order that the safe unit compressive stress be not exceeded, and second, that there may be an impervious surface between the concrete and the ends of the fibres of the timber. With regard to the first consideration, it must be remembered that timber can safely withstand a unit pressure of 1600 to 1800 lb. per sq. in., in end bearing, while concrete should not be stressed in compression over 350 to 450 lb. per sq. in. Standard steel base-plates, of several different makes, may be purchased, or a plain plate may be used. In either case, the plate should be well painted. Further, the bottom of the post should be treated with a good brand of wood-preservative. In no case should the end of the post be allowed to rest directly upon the concrete, as moisture will attack the post, and cause decay of the timber. The standard base-plates are fabricated with lugs fitting closely around the sides of the post. If a plain base-plate is used, it will be advisable to provide a dowel, embedded in the concrete base, and extending an equal distance into both the concrete and the post. The dowel may be a short piece of round steel rod, say $1\frac{1}{4}$ in. diam. by 6 in. long, or else a piece of heavy or extra-heavy steel pipe. In general no dowel should be used with a diameter of less than one inch.

Timber Foundations. Foundations made of timber are seldom used now except for temporary structures. Not many years ago, it was a common practice in California to use timber footings for dwelling-houses. For this purpose, redwood or cedar was employed, since both these varieties of timber resist decay to a considerable extent, even when embedded in the earth. Two kinds of redwood are found in California, the Coast redwood, or *Sequoia sempervirens,* and the Sierra redwood, or *Sequoia gigantea,* the latter being used principally in the San Joaquin valley. To my knowledge, the former is generally considered the better timber of the two for use in foundations, although I am by no means sure that such opinion is based upon anything but prejudice. Good sound cedar is practically as good as redwood, although I prefer redwood myself. Here, again, the preference may be based on prejudice, as I do not know of any tests establishing the length of time either redwood or cedar will resist decay, when buried in the earth. Indeed, there are so many factors, such as quality of timber, character of soil, amount of moisture, etc., affecting the life of a timber in contact with earth, that no single series of tests would establish a definite result. Fence-posts made of redwood or cedar have withstood the ravages of decay for many years. On the other hand, I have seen some redwood posts decayed after a few years' service.

Ordinary timber, such as Douglas fir, if in contact with the soil, or alternately wet and dry, will rot in a short time. It may be said that one to five years' service is all that can be expected from such timber, provided that it is untreated. For this reason, such timber is usually treated with some wood-preservative when placed in foundations. Some of these so-called wood-preservatives are, however, almost useless. Furthermore, even when using a good preservative, care should be taken to see that the timber is thoroughly dry, and that the preservative is well worked into the fibres of the timber, otherwise it will not be effective. Painting the timber lightly is a needless expense, since such treatment is of little value in adding to the life of wood exposed to underground conditions.

In the following discussion, I wish to consider briefly typical details of timber footings. Fig. 88, *a*, *b*, and *c*, show some types of timber footings that I have seen used in buildings. It is hardly necessary to point out the defects in these details. It is obvious that in Fig. 88*a*, the two planks, *m*, add no strength to the footing, and serve only to tie the sticks of the lower planking together. The plank *n* must distribute the whole load of the post to the lower layer of planks. With a soil-pressure of any appreciable amount, this plank must deflect to such an extent that the bearing of the soil is taken almost entirely by the middle plank of the lower layer,

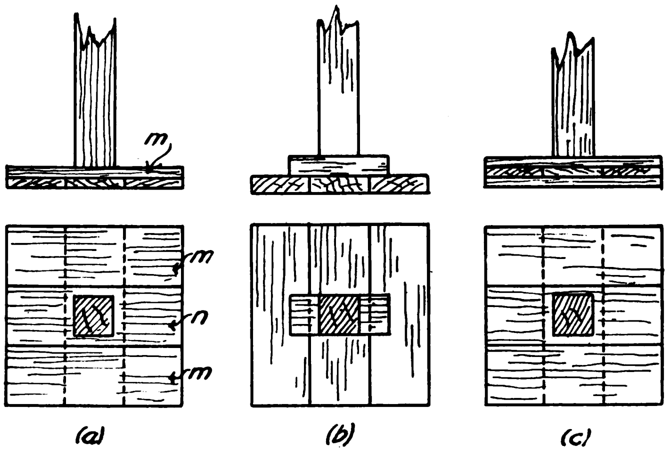

FIG. 88. TYPES OF DEFECTIVE FOOTINGS.

thus increasing the intensity of soil-pressure over that computed (assuming that computations were made).

In Fig. 88*b*, the distributing-cap is so short that again the middle plank of the lower layer is acting as the footing. The footing of Fig. 88*c* is somewhat stiffer, on account of the three layers of planking. Further layers of planking will, naturally, strengthen the footing, and in this way a detail can be constructed sufficiently stiff to distribute the load on the post uniformly over the area of the foundation, but this kind of foundation is neither efficient nor economical.

It may appear a waste of time and space to discuss such a detail, yet, as stated previously, I have seen it used extensively. Indeed, in checking the designs of the various foreign, State, and concession buildings, submitted to the Division of Works of the Panama-Pacific Exposition, I found such details quite common. The

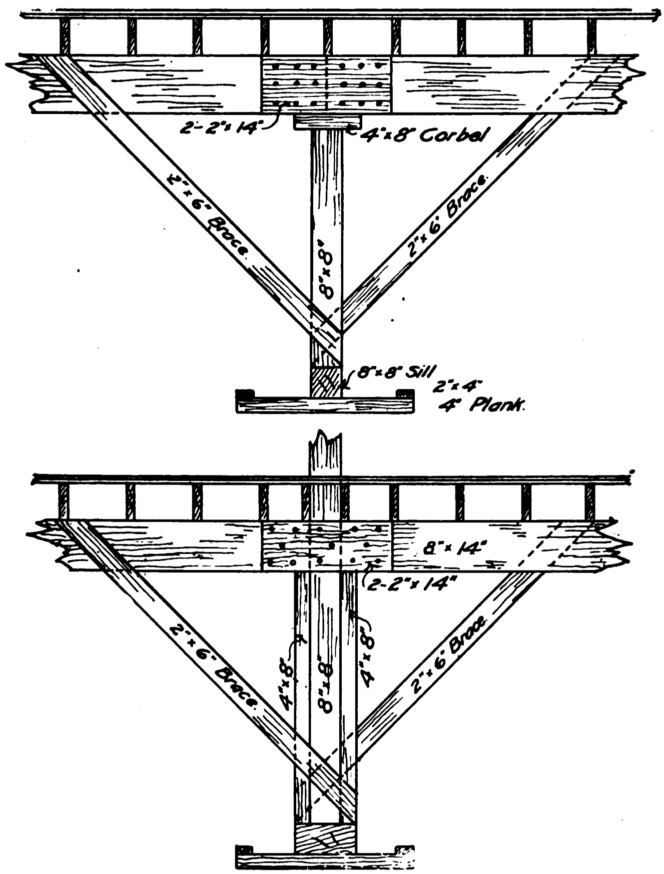

FIG. 89 AND 90. CORRECT DESIGN FOR TIMBER FOOTINGS.

fact that these structures were not designed in any one locality would seem to indicate that this type of footing is used extensively for temporary structures.

The best spread timber-footing for a small load is illustrated in Fig. 89. This detail is efficient and economical, and subject to rational analysis. Its design involves only the consideration of bearing pressure on soil and timber, and bending and longitudinal shear in timber. There is no tendency for the distributing-cap to be split by the punching effect of the post.

Fig. 90 shows a similar detail for a larger footing. The outer stringers are added to tie the bearing planks together. This figure also illustrates a typical detail for post and girder connection. The corbel shown is not for the purpose of reducing the unit bearing-pressure across the under side of the girders, but rather to allow for the possibility of the girders being cut too short to meet over the centre of the post. The 2 by 14-in. splice-pads not only tie the girders together, and so add general stiffness to the floor, but they also furnish a certain amount of end-restraint or continuity in bending to the girders, in case the actual centre lines of bearing of the girders are unsymmetrical with regard to the centre line of the post. The 2 by 6-in. braces may or may not be necessary, depending upon the height of the floor above the ground. Such braces are an effective means of stiffening a floor against vibration from machinery. Where such bracing is necessary, the post should be braced in all directions, and it will usually be sufficient to brace only every other floor bay. For the bracing in a plane normal to the plane of the girders, the joists immediately over the post may be spaced so as to allow the batter-braces to be spiked to the joists.

In Fig. 91 there is outlined a typical timber-footing for the case of a column extending through the floor. In this detail the girders are supported by short posts alongside and fastened to the main post. A modification of this detail is shown in Fig. 92, where the short posts are eliminated, and the main post is cut to receive the girders. Because of the expense of cutting the post and the weakening of the post resulting therefrom, the detail of Fig. 91 is to be preferred. The detail computations for the design of the timber footing of Fig. 92 is given, using the typical building of Fig. 93, al-

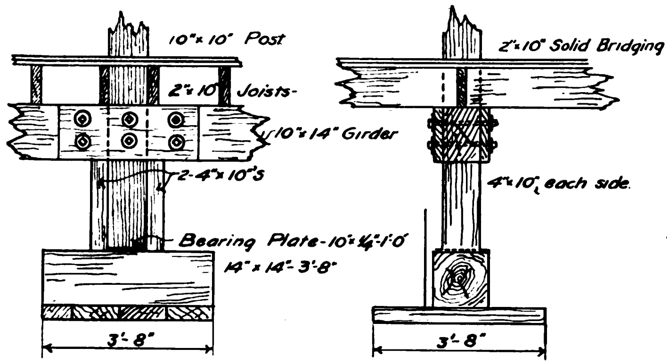

FIG. 91. TIMBER FOOTING FOR COLUMN EXTENDING THROUGH FLOOR.

though such a building, unless built for very temporary purposes, would have concrete piers.

Pile Foundations. Where the conditions of soil are such that piles are necessary, the details shown in Fig. 90 and 91 may be modified by resting the bottom of the post on the top of the pile. Iu such cases, however, a bolster, either of timber or of iron, should be placed between the ends of the post and the pile, in order to prevent moisture from attacking the post.

For piles of ordinary length, it will generally be found economical to arrange the spacing of floor-bays so that the full load-capacity of the pile may be utilized. The capacity should be determined either by test-piles or by

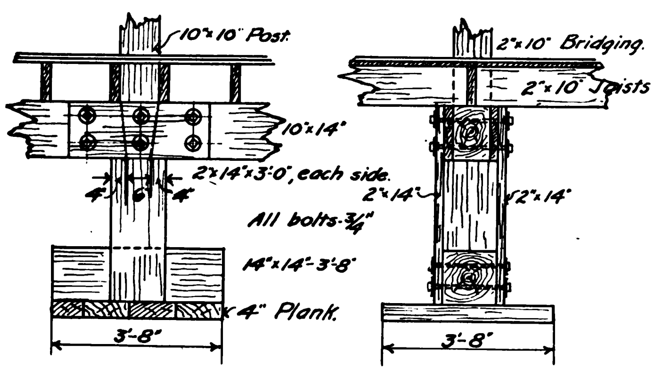

FIG. 92. MODIFICATION OF DESIGN SHOWN IN FIG. 91.

comparison with piles used under similar soil-conditions, supplemented by borings to determine the nature of the

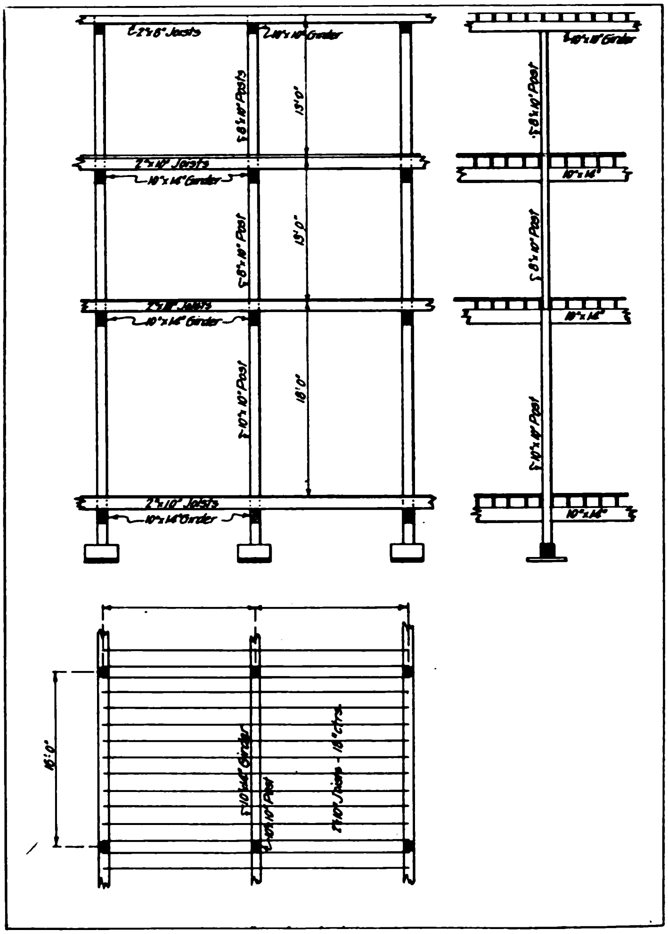

FIG. 93. TYPICAL TIMBER-FRAMED BUILDING.

COMPUTATIONS

Assuming tar and gravel roof-covering weighing 8 lb. per cu. ft. and Douglas fir weighing 3.5 lb. per ft. B.M.

Dead load of roof per sq. ft.	= 17 lb.
Dead load of floors per sq. ft.	= 15 lb.
Live load on roof per sq. ft.	= 28 lb.
Live load on floors per sq. ft.	= 85 lb.

Total dead load on footing:

Roof	= 3,800 lb.
Floors	= 10,100 lb.
Post and footing	= 1,100 lb.
Total	= 15,000 lb.

Total live load on footing, assuming that 60% reaches the footing:

Roof	= 3,760 lb.
Floors	= 34,300 lb.
Total	= 38,060 lb.
Total live load plus dead load..........	= 53,060 lb.

Assuming allowable pressure on soil of 2 tons per sq. ft., area required $= \frac{53060}{4000} = 13.25$ sq. ft.

Footing therefore will be 3 ft. 8 in. square.

Bearing area required under post $= \frac{53060}{285} = 186$ sq. in. Use 14 by 14-in. short post.

Sill will have overhang of $\frac{44 - 14}{2} = 15$ in.

Load on overhang $= 4000 \times 3.66 \times 1.25 = 18,300$ lb.

Bending moment $= 18,300 \times 7.5 = 137,500$ lb.-in.

Requires, for bending, an 8 by 10-in. timber laid flat.

Maximum shear $= 18,300$ lb.

Area required for longitudinal stress $= \frac{18300 \times 3}{2 \times 150} = 183$ sq.in.

Use a 14 by 14-in. sill.

Overhang of planking $= 15$ in. Load on overhang for 12-in. width $= 4000 \times 1.25 = 5000$ lb.

Bending moment $h = 5000 \times 7.5 = 37,500$ lb.-in. Requires a 4 by 12-in. plank for bending.

Maximum shear $= 5000$ lb.

Area required for longitudinal shear $= \frac{5000 \times 3}{2 \times 150} = 50$ sq. in., therefore a 4 by 12-in. plank is all right.

Bearing required for floor-beam.

Load $= \frac{(14 \times 16)\ (15 - [0.8 \times 85])}{2} = 9300$ lb. Depth of seat required $= \frac{9300}{285 \times 10} = 3.26$ in.

Therefore taper bottom of 10 by 10-in. column as shown.

underlying soil. Borings should be made in any event in order that full knowledge may be obtained of existing conditions. This statement applies not only to investi-

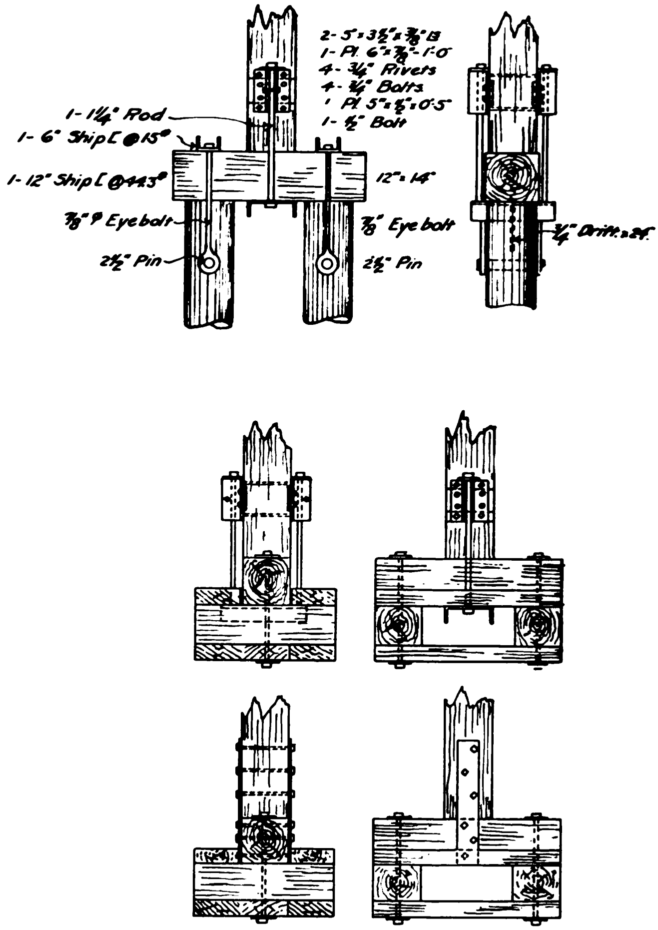

FIG. 94, 95, AND 96. TYPES OF COLUMN-ANCHORAGES.

gations to determine the capacity of piles and elevation of ground-water, but also to the study of foundation conditions in the case where spread footings are to be used. Generally a pile of an average length of 40 ft., and a butt of from 12 to 14-in. diameter driven properly to refusal may be expected to carry twenty tons without settlement.

Where conditions are such as to justify temporary construction, the piles may be cut off just above the ground-level, and posts used to carry the floor-girders, or the point of cut-off of the piles may be raised so that no posts are required. The question of relative cost will be the main factor in determining which method of the two will be used, and this must be computed for each

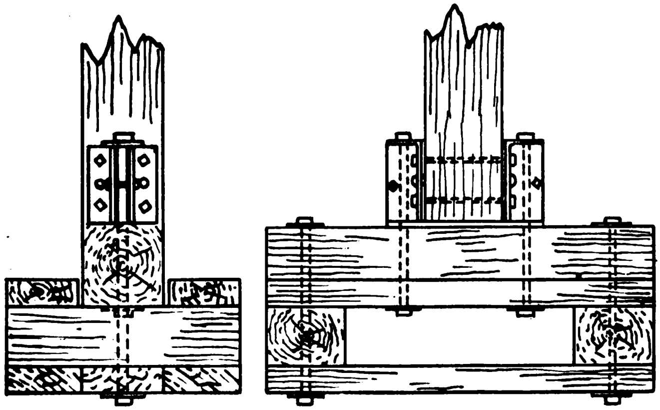

FIG. 97. TYPE OF COLUMN-ANCHORAGE.

individual case. Other things being equal, the first method is to be preferred, as the top of the piles, if they are cut off at any appreciable distance above the ground, are likely to be a considerable distance from their computed positions. This condition will disarrange the floor-system. For a permanent structure the timber piles must be cut off below the permanent ground-water level in order to prevent decay.

In this connection, it should be noted that 'one-pile' footings should only be used to support posts carrying no other floor than the first. For posts extending through the first story of the building and supporting some of the other floors, with the position of the posts determined by wall or girder-ties, provisions must be made for the foundation piles being at least six inches from their theoretical position. The use of a 'two-pile' footing with a wide cap will be necessary, even where the load coming upon the footing could be safely supported by one pile alone.

Anchorage for Columns. When uplift must be considered, which may occur in high, narrow structures, piles are important in providing anchorage. If the condition of the soil does not necessitate piles, the uplift must be handled either by burying the timber footing in the ground, or by using a large concrete footing to furnish the required weight. The latter condition is quite common. A timber-framed building is comparatively light. With a high narrow building, or a high building with only two or three posts in the direction of the width of the building, it may be desirable, or even necessary, to anchor the wall-columns against uplift, or else to secure the ends of the columns rigidly to the foundation. In this way the columns may be considered as practically 'fixed' in the computation of stresses, and the computed stresses in the columns resulting from wind reduced accordingly.

For anchoring columns securely to the foundations, various details may be employed, ranging from the simple expedient of using two or four thin straps, bolted or lag-screwed to the post (see Fig. 94), to the somewhat elaborate detail shown in Fig. 95, 96, and 97, which is made from plates, angle-bars, and anchor-rods.

The problem of anchoring a timber post to a concrete foundation is much simpler than where the footing is constructed of timber, either of the spread-foundation type, or a grillage resting on piles. A timber foundation is subject to shrinkage, and in such cases, where anchor-rods are used, the detail of connection should be arranged so that the nuts on the anchor-rods may be tightened after the shrinkage has taken place. Anchorages of timber columns to timber grillages are unsatisfactory at the best, since it is practically impossible to keep the connections tight. With concrete piers the case is different. Anchorages of the types shown in Fig. 94, 95, and 96, with the anchor-rods or plates embedded in the concrete can be expected to remain tight irrespective of conditions.

In Fig. 97, it will be noted that there is a bed-plate underneath the column and stiffener-angles. Attention is called to the fact that if the bolts through the column are loose, it will be impossible to draw these bolts tight by screwing the nuts on the anchor-rods, as the pull of the anchor-rods is not against the column, but directly against the foundation timbers.

Perhaps the most satisfactory anchorage that can be devised for a timber column, when the stresses in the anchor ties are of considerable magnitude, and when, especially, a rigid anchorage is desired, is the detail shown in Fig. 98, which is an adaptation of the tenon-bar splice. In designing such an anchorage, the width of the bar is determined either by the required area for bearing against the ends of the fibres of the timber in the post, or by the minimum width necessary for the size of anchor rod used. The height or thickness of the bar is found from considerations of bending, the total uplift in the post or pull in the anchor-rods being considered uniformly distributed along the bearing-length of the anchor-bar. As such bar is a short beam in bending, it is allowable to use a high unit fibre-stress in flexure. The usual unit bending-stress of 16,000 lb. per sq. in. may be increased to 24,000 pounds.

In order to keep the bending-moment in the anchor-bar at a minimum, it is advisable to use hexagonal nuts on the anchor-rods. This will allow the rods to be placed nearer the post than if the ordinary square nuts are employed.

This detail can be relied upon at all times. There is no initial slip in the anchorage, when the uplift comes upon the post, since the nuts on the anchor-rods can be drawn up tightly at the time of framing, and can always be maintained in this condition. The detail also

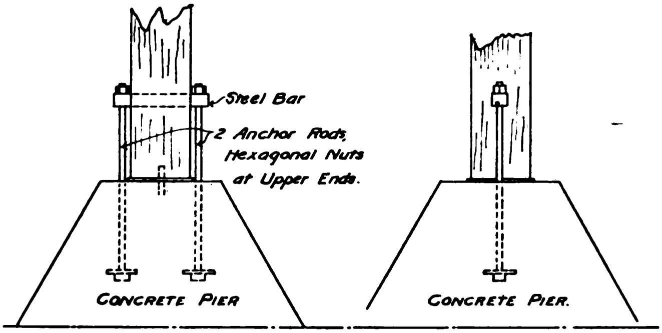

FIG. 98. APPROVED FORM OF FOUNDATION.

has the advantage over the others shown above, in that there is only one bearing-surface of metal upon timber.

CHAPTER XIV

Miscellaneous Structures

In the preceding chapters I have endeavored to present the underlying principles of structural mechanics as applied to timber framing, to show that timber construction is worthy of the same study as construction in steel or concrete, and to point out details of framing that best fulfil the requirements of the particular structure under consideration. This presentation has been in somewhat logical order, based upon the various structural features of a timber-framed building. Thus the text has considered in sequence: grading-rules, working-stresses, washers and pins, strength of nailed, screwed, and bolted timber-joints, the design of typical truss-joints, design of truss-members, bracing, columns, joist and girder connections, and foundations. While the discussion has been limited almost wholly to building design, it does not follow that the principles, methods of design, and details are applicable only to timber-framed buildings. The building has been chosen rather as an example, for the reason that the design and construction of a large timber building of the mill-building type includes practically all of the problems that will arise in timber-framing.

When other structures, such as bridges, trestles, towers, flumes, etc., are considered from a structural standpoint, the loads and their applications, and also the allowable working-stresses, may vary, but the same principles of design will hold. Many of the details of connections that are used in building construction may be employed in these other types of structures. As was stated in the opening chapter, timber railroad-structures, such as a combination timber and steel bridge, or a timber trestle, are well standardized. The same is true of the larger highway bridges. To a considerably less degree, flume-design follows certain standards.

In the present article, there is presented a somewhat superficial treatment of a few timber structures that may be classed as 'miscellaneous structures.' In the order of their discussion, these are (1) flumes, (2) head-frames, and (3) water-towers.

Flume-Design. To illustrate a typical problem in flume-design a timber flume will be assumed 9 ft. wide on the inside, and with a maximum depth of water of 5 ft. 3 in., as shown in Fig. 99. Further, it will be

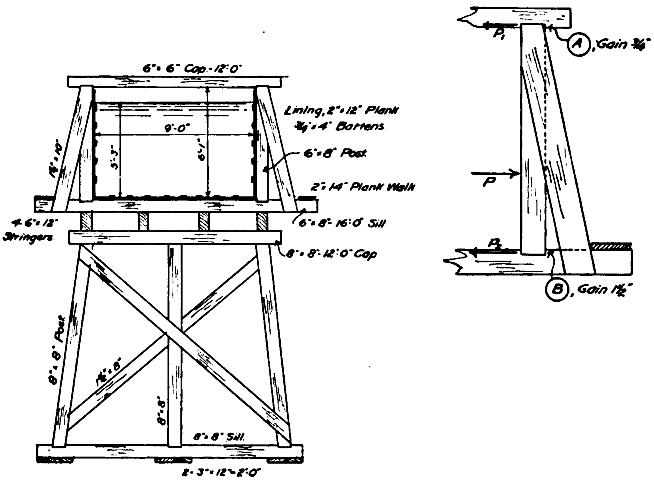

FIG. 99. CROSS-SECTION OF FLUME.

assumed that the timber is not Douglas fir, but Californian mountain pine. The following unit working-stresses will govern the design:

Tension and bending, 800 lb. per sq. in.

Bearing across fibres, 200 lb. per sq. in.

Bearing against the ends of the fibres, 700 lb. per sq. in.

Longitudinal shear, 100 lb. per sq. in.

The figure shows the typical section as framed, and the sizes of the different members. The details of connections are found as follows. The cross-ties will be placed 3 in. above the flow-line and the bents will be spaced 2 ft. 8 in. centre to centre.

Detail Calculations

(1) Vertical post.

Total pressure per lin. ft. $= P = \frac{1}{2} \times 62.5 \times (5.25)^2 = 860$ lb.

This pressure acts at $\frac{2}{3}$ of the depth of the water.

The reactions at A and B are then

$$\text{Pressure at } A = P_1 = \frac{1.75}{5.50} \times 860 \text{ lb.} \times 2.67 = 735 \text{ lb.}$$

$$\text{Pressure at } B = P_2 = \frac{3.75}{5.50} \times 860 \text{ lb.} \times 2.67 = 1565 \text{ lb.}$$

In determining the moment on the post, such moment may be found by considering the total pressure P as uniformly distributed along the length of post. This method is not exact, but is approximately correct.

$M = \frac{1}{8} \times 2300$ lb. $\times 5.50$ ft. $= 1580$ lb.-ft.

The resisting moment of a 6 by 6-in. timber is 2400 lb.-ft., while the maximum resisting moment of a 4 by 6-in. timber is 1600 lb.-ft., both computed at 800 lb. per sq. in., maximum unit fibre-stress in bending.

The post will be gained into the cross-tie at the top and into the sill at the bottom. The required bearing-areas at the top and bottom, are then

$$\text{Area at } A = \frac{735}{200} = 3.68 \text{ sq. in.}$$

$$\text{Area at } B = \frac{1565}{200} = 7.87 \text{ sq. in.}$$

If the posts, cross-ties, and sills are 6 in. wide, the gain at the top must be $\frac{5}{8}$ in., and the gain in the sill $1\frac{3}{8}$ inches.

(2) Stringers.

The trestle bents will be assumed to be spaced 8 ft. centre to centre.

The load on one stringer will then be

Water,	$3.25 \times 8 \times 5.25 \times 62.5 =$	8530 lb.
Flume, assume weight,		400
		8930 lb., say 9000 lb.

Moment on stringer $= \frac{1}{8} \times 9000 \times 8 = 9000$ lb.-ft.

Resisting moment of 6 by 10 in. timber $= 6650$ lb.-ft.

Resisting moment of 6 by 12 in. timber $= 9600$ lb.-ft.

The above calculations indicate that the vertical posts, cross-ties, and sills could be 4 by 6 in., the posts to be gained 1 in. into the cross-ties, and 2 in. into the sills. The cross-ties, as far as requirements of strength are concerned, could be made 4 by 4 in. This design, with the detail computations, has been taken from an actual case, a flume intended for a hydro-electric development. The sizes actually used were: cross-ties, 6 by 6 in.; posts, 6 by 8 in.; sills, 6 by 8 in.; and stringers, 6 by 12 in. See Fig. 99, which also shows the substructure. The engineers in this case used their calculations more as a general guide as to minimum requirements than to determine the actual sizes, and judgment and experience influenced the selection of practically every section. In this case, also, the decay of the timber through alternate wetting and drying was considered in employing sections larger than required by computations for actual strength. Standard practice has determined certain minimum sizes of timber for use in such cases: for larger flumes, requiring larger sections from a purely theoretical standpoint, the margin of safety to provide against decay could be reduced and the actual sections used would be nearer the sections determined from considerations of strength alone.

As an illustration of a larger flume, Fig. 100 shows one

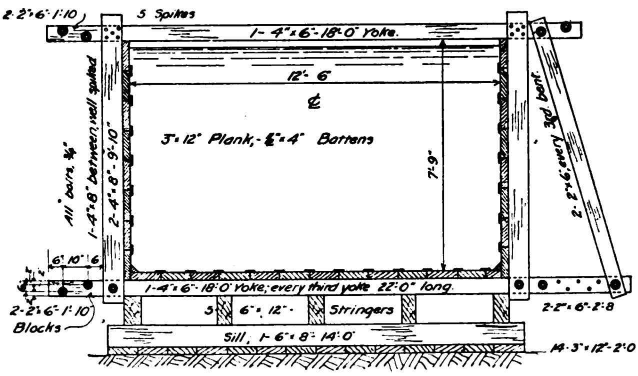

FIG. 100. CROSS-SECTION OF FLUME AT EXPOSITION.

flume used in the filtration-plant of the Panama-Pacific International Exposition. This flume is 7 ft. 9 in. high, and 12 ft. 6 in. wide. The side-posts are two 3 by 8 in. timbers, bolted and spiked to a 4 by 6 in. sill and a 4 by 6 in. cap, or yoke. Between the bottom and top yoke is a 3 by 8 in. filler, which stiffens the side-posts. In addition to the bolting and spiking of the posts to the top and bottom yokes, short wooden blocks were bolted and spiked to each yoke to receive the thrust of the posts. The side-posts were spaced 4 ft. centre to centre. Every third post was braced to the sills by two 2 by 6 in. braces. The flume was carried by five 6 by 12 in. stringers, having a span of 12 ft., the stringers being supported by 8 by 12 in. sills, resting upon 3 by 12 in. plank. All material was Douglas fir. As the plant was of a temporary nature, the sections were determined from considerations of strength and deflection alone. In flumes of this size, deflection of the stringers, and of the side-posts must be taken into consideration, as leakage may result if there is appreciable deflection in the side-posts or the sills under load.

Two types of joints for the flume-lining were used; one, using ½ by 4 in. battens with asphaltum in the joints, and the second an untreated spline-joint. The superintendent of the plant favored the results obtained from the spline-joint. I inspected the plant carefully several times, and as far as I was able to determine by observation, neither joint showed any material advantage over the other. Both were almost free from leakage.

In *Engineering News,* Vol. 76, No. 23, December 21, 1916, there occurs an article entitled 'Rectangular Wooden Flumes' by J. C. Stevens. Mr. Stevens treats of many practical details of timber-flume construction, but makes what I consider a radical statement to the effect that it is a waste of lumber to place battens on the inside of a flume. His recommendation is to place the battens on the outside of the sides, and on the underside of the floor, cutting them between the posts and sills. This is not the common practice on the Pacific Coast, where there are many large hydraulic enterprises using timber flumes. In triangular logging flumes,* the

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*See Bulletin No. 87, U. S. Department of Agriculture, 'Flumes and Fluming,' by Eugene S. Bruce.

battens are usually placed on the outside of the flume, and are often made continuous, the side-supports being cut accordingly. However, the two cases are not at all comparable.

Head-Frame for a Mine. Without going into an extended discussion of the design of head-frames, it is desired to give one example of the application to a timber head-frame of some of the principles and details advocated in preceding articles. For this purpose, there is shown in Fig. 101, a typical head-frame, taken from the *Engineering and Mining Journal* of October 11, 1913. The cut is from an article describing different types of timber head-frames. Sufficient data are not given by either the article or illustration to enable any of the stresses to be computed, and consequently no stress-analysis has been made. Indeed considerations of stiffness of the frame under working-conditions will require more bracing than might be computed from conditions of actual load and wind-forces. In a structure of this nature it is highly desirable not only that it shall be rigid under the racking received in the hoisting and dumping of ore, but also that the details are such that the joints may be kept tight.

Referring to the figure, the first criticism to be made is of the use of diagonal steel rods and horizontal struts of timber. Not only is this the most expensive system of framing under the usual conditions, where steel is expensive, and timber is comparatively cheap, but it results in bad details at the joints, making the whole structure difficult to tighten. With the rods horizontal, and the struts in an inclined position, the amount of steel would be the minimum, and the timber, while greater in quantity, would not add appreciably to the cost of the structure, and would be more than offset by the saving in other features. Consider one of the panel-points of the frame, where two rods intersect with the vertical post. Not only is the post cut to receive the horizontal strut, but in addition, two inclined holes of considerable size must be bored through the post at the same point, greatly weakening the post. It is safe to say, without knowing the size of all the rods, that 25% of the post is cut away. While this proportion may not seem to be large in itself, in this case it represents an area of some 60 sq. in. of timber that is useless. Expressed in other terms, an area equivalent to a 6 by 10-in. stick is not available. With such a detail, large sticks are a necessity.

Near the top of the head-frame, the inclination of the rods is considerable. It is evident that to pull the posts snugly against the horizontal struts, there must be exerted on the nut of the rod a force approximately twice that which is actually pulling the rear and front posts together. In other words, a large proportion of the stress in the rod, when tightening is in progress, is exerted in a vain attempt to lift the whole frame from its foundations. On the other hand, if the rods were horizontal, every pound of stress placed upon them when tightening the nuts would be exerted in pulling the posts tightly against the inclined struts. This statement is, needless to say, based on the assumption that the washers are large enough to prevent their being crushed into the timber of the posts.

A similar criticism may be made of the system of bracing below the loading-floor. Here would seem to be an excellent opportunity to use a truss of the Howe type, which would serve to carry the weight of the loaded floor to the supports, and could also be utilized to brace the whole frame against lateral forces. For this purpose, the bays of the truss should be counter-braced, and special attention should be paid to the end-connections of the truss, to see that they are capable of transferring both tension and compression to the posts.

With the design shown in Fig. 101, it is difficult to see how tension could ever exist in any but the main posts of the head-frame. It will be seen that the two intermediate posts are anchored to the concrete foundations. Apparently, however, there is no definite connection between these intermediate or minor posts and the main frame.

The final point of criticism in the design is the type of

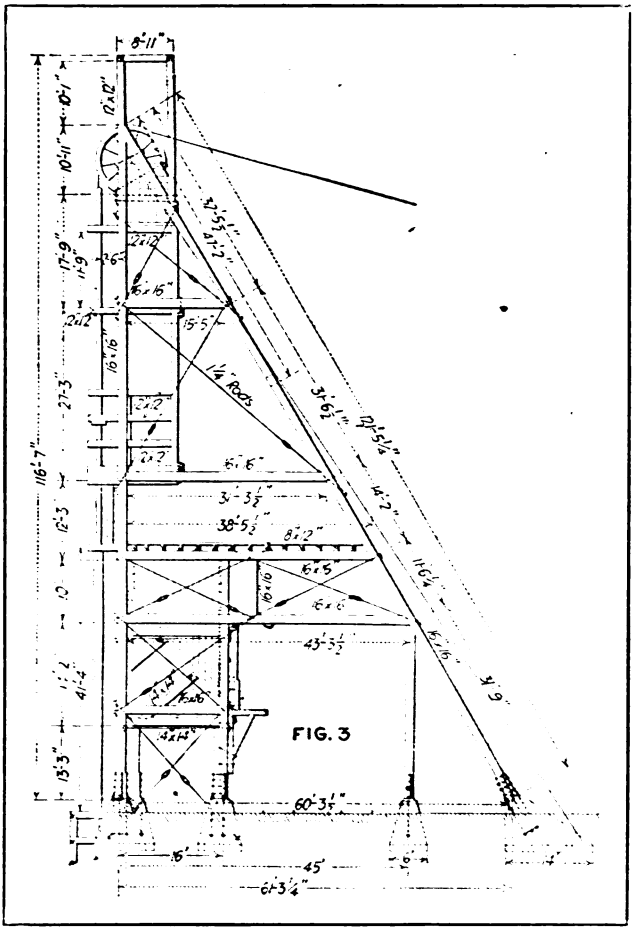

FIG. 101. POORLY DESIGNED HEAD-FRAME.

post-anchorage. Since the details are shown to a small scale, it is difficult to be sure just what such details are. It would appear that the anchorage consists of steel straps or anchor-rods flattened at one end to straps,

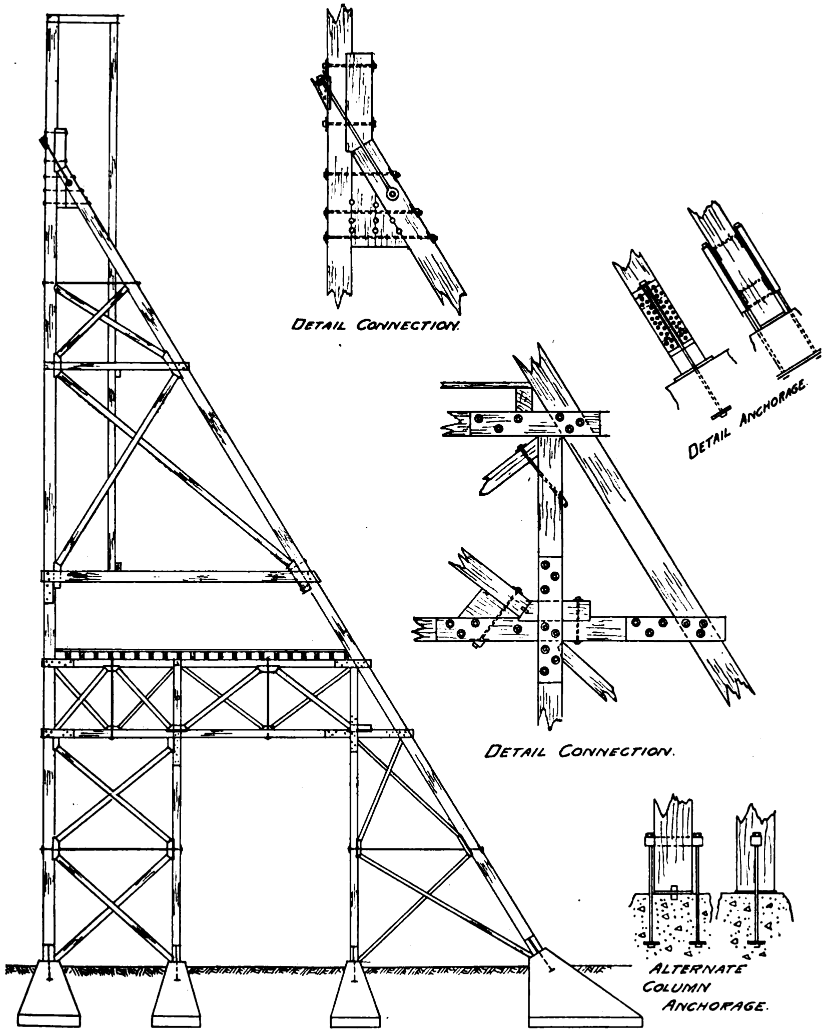

FIG. 102. REVISED DESIGN OF HEAD-FRAME.

buried in the concrete, and bolted through the posts. If the actual tension that may come into the posts is small, this detail may be entirely satisfactory; in fact, it is not my intention to imply that such a type of detail may not be strong enough in any case. There can be no chance of failure, provided that length of anchorage, size of strap-bolts, and number and size of bolts are sufficient for the stresses they will be called upon to carry. But, as was pointed out in Chapter XIII, such a detail, once in place can never be tightened. It would seem therefore, that a detail which could be adjusted at any time by tightening the nuts of the anchor-rods would be advisable, particularly as such a type of detail would not necessarily be any more expensive.

In order to emphasize the criticisms of Fig. 101 that have been made, there is shown in Fig. 102 a revised elevation of the head-frame, with some of the most important connections detailed. The two main posts have been reduced in size, since with the type of connections used, a much larger proportion of their section is available for use. No sizes have been noted on any of the rods or timbers; without knowing the loads there has been no attempt to compute stresses. The tie-rods have been placed in a horizontal position, and the diagonal members made compression timbers. Where two such diagonal struts intersect on the posts at a common point, butt-blocks have been used. For the support for the loading-floor a truss has been introduced, bolted at its ends to the main posts by means of splice-pads. The first vertical post to the left of the main inclined post of the frame, has been moved in to the left from its former position. This is to enable a more positive tie to be made at its upper end to the cross-truss, and also to allow another system of cross-bracing to be introduced. Clearance for cars would determine the position of this post.

Suitable washers of generous area should be provided for all bolts and rods, so that crushing of the fibres of the timber would not occur. This provision will add much to the life of the structure. It would also be ad-

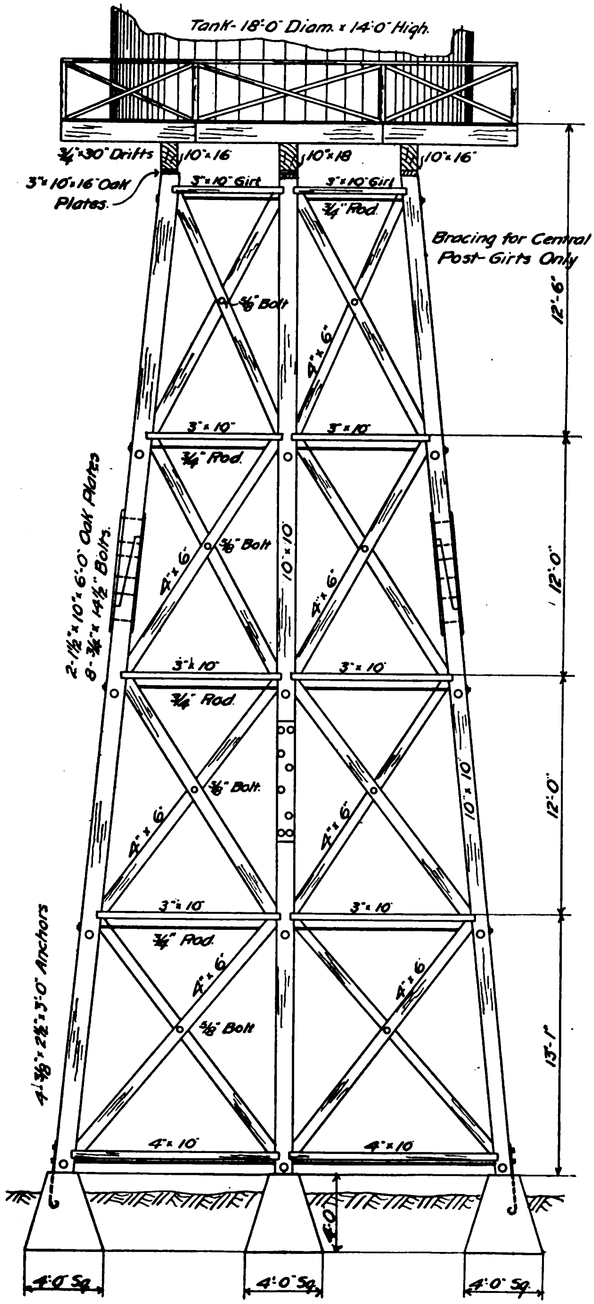

FIG. 103. ORIGINAL DESIGN OF TANK-TOWER.

visable to treat thoroughly the contact faces of all timbers, and the contact faces of all metal and timber with a good wood-preservative. The life of the head-frame will be lengthened by painting the inside of all bolt and rod-holes in the timber, the ends of struts, and the cut-faces of the timbers into which the ends of the struts and the butt-blocks fit. It might even be advisable to use castings at all panel-points, similar to the construc-

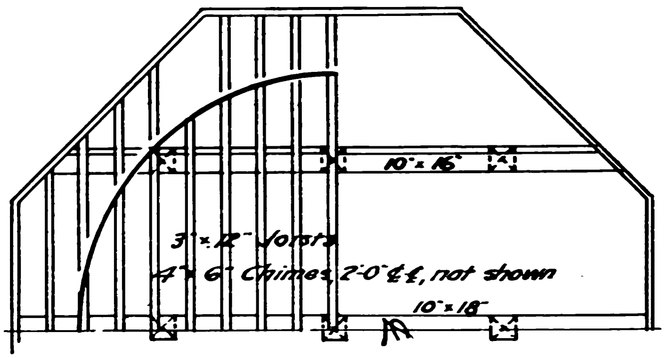

FIG. 104. FLOOR-PLAN FOR TANK.

tion used in standard timber railroad-bridges, even though the stresses might not, and probably will not, require the use of such castings. Metal base-plates under all posts are a necessity to prevent moisture from creeping up the timber.

Water-Tower. Another type of miscellaneous structures will be discussed in this chapter, namely, a water-tower. An example from actual practice will again be chosen, and an alternative design shown. In this case the first design (see Fig. 103), was submitted by a firm manufacturing and selling wooden tanks and pipes. Fig. 105 shows the tower as re-designed and built.

The principal points of difference between the design of Fig. 103 and 105 are (1) the omission of the column splices, in Fig. 105 (2) the change in details of the intersections of the diagonal struts with the posts, and (3) the revision of the post-anchorages.

Taking up these three points in the order mentioned, not only is a saving in cost made in the revised design by the omission of the column-splices, but a much stronger tower is secured. Attention is called to the splice proposed by the tank-manufacturers. The type of splice may be classed as the oblique scarfed joint with fish-plates. In a previous chapter objection has been made to this type of splice for timbers carrying heavy compression, because of the two surfaces of contact, requiring an accurate fit to be made. The fish-plates as required in the design were two $1\frac{1}{2}$ by 10 in. by 6 ft. oak plates, fastened with eight $\frac{3}{4}$ by $14\frac{1}{2}$-in. bolts. Why oak was specified is not clear. The strength of oak in end-bearing and in cross-bending is practically the same as Douglas fir, of which latter timber the posts are composed. The shearing strength of oak is not quite 25% greater than that of Douglas fir. The fear of splitting the splices if they were made of Douglas fir may have influenced the designer to substitute oak. If, however, splices were necessary, which in this case they were not, since the length of the posts is not abnormal, and long timbers were easy to obtain, a better detail would have been secured by using a straight normal cut for the post, and thicker splice-plates of Douglas fir.

The detail of the intersection of the inclined struts with the posts is, perhaps, the worst feature of the design of Fig. 103. Note that the horizontal 3 by 10-in. girts are set into the posts approximately one-fourth of the depth of the posts. A note appears on the drawings, 'Bracing for central post-girts only.' In this post over 50% of the cross-sectional area of the central post is in cross-bearing on the timber. A rough calculation indicates that the unit compression on the central post is 320 lb. per sq. in. for dead load with no wind. While this unit compression is not excessive, it is evidently more than the designer anticipated, since 3 by 10-in. oak plates are specified between the tops of the posts and the bottom of the stringers. The inclined braces bear directly across the fibres of the 3 by 10-in. girts. This is a poor detail, both on account of unit bearing-pressures, and of timber-shrinkage. The girts would shrink and leave a somewhat loose fit between post, girt, and braces.

The third point is the anchorage for the posts to the foundation. Each of the four corner-posts is tied into the concrete foundations by one $\frac{3}{8}$ by $2\frac{1}{2}$-in. tie-strap, fastened with two bolts, presumably $\frac{3}{4}$ in. diam. The strength of such anchorage, measured by the safe resistance of the bolts, is not over 3000 lb., measured by the tie-strap in tension, 10,000 lb., and by the weight of the concrete, 5472 pounds.

Turning to the revised design, as shown in Fig. 105, it will be seen that the post-splices have been omitted and the connection of inclined struts modified by introducing butt-blocks set into the posts; the latter change brings all post-timbers in end-bearing, and frees the joints from the effect of shrinkage of the timber, since the shrinkage of Douglas fir parallel to the fibres is small, and negligible for ordinary lengths of timber. The horizontal tie-rods now extend through the three outside posts. It is therefore necessary that horizontal struts be used in addition to the rods, in order that the increment of wind-shear may be transferred across the posts, from one system of bracing to the other. A more effective manner of handling this problem, although more expensive, would have been to have placed a turnbuckle on either side of the middle exterior posts. It is evident, that, with wind on the tower, the stress in any horizontal tie-rod will not be the same on both sides of the post in the revised design, and the difference of shear must be carried from one corner-post to the opposite one and back to the middle post by compression in the two 2 by 8-in. girts. The central post is tied to the exterior posts by two 2 by 8-in. girts, in two directions, the girts being bolted to the posts. These girts are therefore able to develop both compression and tension. Finally, the size of the column-footings has been increased, steel base-plates introduced, and the anchorages strengthened, each post having two anchor-straps lag-screwed into the posts.

In the re-design of the tower, a wind-load of 30 lb.

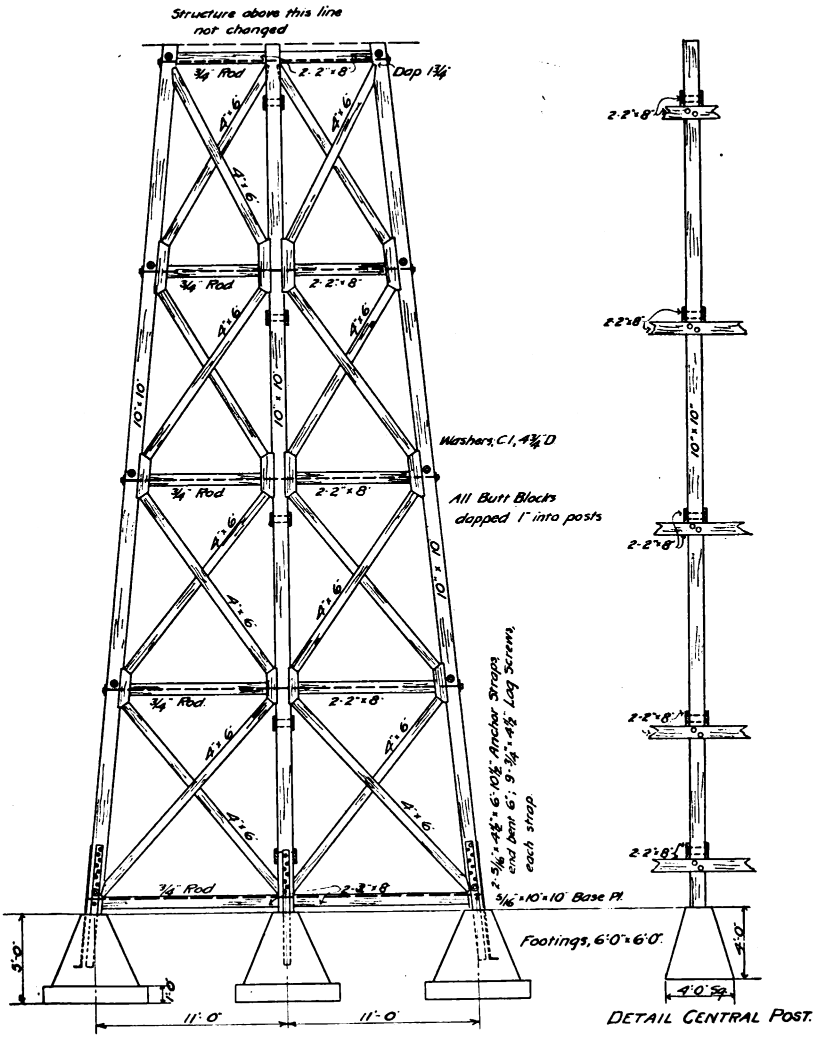

FIG. 105. REVISED DESIGN OF TANK-TOWER.

per sq. ft. of exposed surface was used. It should be stated that the tower was to be completely enclosed, and that it stood on the top of a hill, near the ocean shore, and exposed to the full force of the wind on the entire surface of structure enclosing the tower. The rather heavy anchorages are therefore justifiable.

The following data regarding the design of head-frames and ore-bins, contributed by Robert S. Lewis, professor of mining at the University of Utah, will be of value to those interested in mining structures.

Head-Frames. The small head-frames used in prospecting and development work are seldom designed by an engineer. Their construction and planning is generally left to a carpenter or contractor, and the excellece of the design depends upon the previous experience of the carpenter or contractor. In a mining district there is often a striking similarity in design of the different head-frames, either because of a common builder, or because the design of the first frame to be erected was copied by the builders of the other frames. The average life of a wooden head-frame may be taken at 10 years, hence, if the mine is likely to have a longer life, it is desirable that the frame be made of steel.

There are two general types of head-frames: the A-frame type and the four-post type, both shown in Fig.

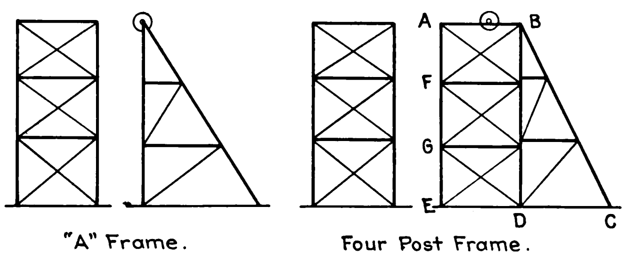

FIG. 106. TYPES OF HEAD-FRAMES.

106. The former is a simple type of frame. All stresses are determinate. The objection that the sheave is mounted by bolting the bearings to the front frame, thus bringing a pull on the bolts, can be met by special meth-

ods of mounting the sheave. However, to prevent the skip or cage from striking the front bracing, the front posts must be set back from the shaft. Consequently the sheave must be large enough to bring the hoisting-rope to the centre of the shaft. This may require the use of a very large sheave, entailing great inertia and wear on the rope on account of slippage when starting and stopping. This disadvantage is not serious unless a heavy load is handled at high speed.

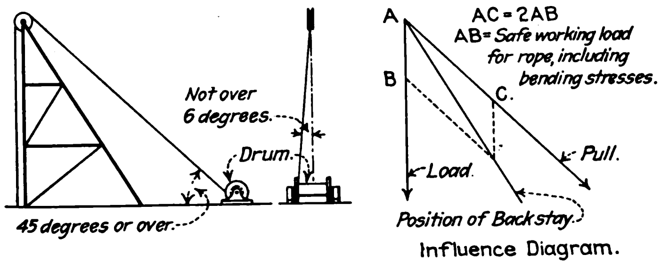

FIG. 107. HOISTING-DIAGRAM AND INFLUENCE DIAGRAM.

In the four-post type the sheaves are mounted on horizontal timbers placed on top of the structure. This frame lends itself to rigid construction in either wood or steel. The joint at *B*, is indeterminate, consequently the stresses cannot be computed exactly. The frame straddles the shaft, and there is no particular difficulty encountered in mounting the sheave, its diameter being computed after the size of the rope and the allowable bending stresses have been determined. However, the diameter of the sheave governs the width of the frame, as the horizontal timbers at the back of the frame must clear the sheave. When the loads to be handled are large, an extra post may be included between the other two, thus making a six-post frame. When a three or four compartment shaft requires a head-frame, a bent may be included between each compartment.

The height of the head-frame should be the sum of the following three quantities: the height of the landing-floor above the collar of shaft, which is determined by local conditions, the height of the skip or cage, and an allowance for overwinding. This allowance should be $\frac{1}{2}$ to $\frac{2}{3}$ of a revolution of the drum for direct-acting hoists.

The next step is to determine the position of the hoisting-drum. This should be placed so that the fleet angle of the rope, leading from a position on the extreme edge of the drum, is not greater than 6° to prevent a tendency of the rope to climb the grooves in the drum. Then the angle between the horizontal and the rope should not be less than 45° and should not be more than 55°. This is to avoid 'lashing' of the rope, and the necessity for employment of extra sheaves to support the rope.

The distance between panel-points is assumed, as there is no definite practice in this regard. It is generally between 12 and 20 feet.

When the position of the hoisting-drum is decided, the next point is to find the position of the back-stay of the frame. The following methods have been used:

1. Placing it parallel with the resultant of pull and loads.
2. Placing it parallel with the hoisting-ropes.
3. Placing it just outside the resultant of pull and load.
4. Empirical position, making the pull twice that of the load.
5. Placing it at 30° with the vertical.

Method No. 4 is satisfactory, and gives a safe position. It is not far from the position given by method No. 5.

The front-width of the frame at the bottom is calculated to prevent overturning by wind-pressure. The wind is assumed to blow around the first timbers and strike the rear-posts also, unless the entire structure is housed, when the total exposed area can be computed. The wind-pressure should be taken at 30 lb. per sq. ft. The front top-width must be such as to carry shaft-guides, and to give sufficient clearance between the cage or skip and the frame. The guides are from 4 by 6 up to 8 by 10 inches.

A method of computing the stresses is given in Ketchum's 'Design of Mine Structures.' For the four-post frame, a simpler approximate method is as follows:

In Fig. 106, consider that the structure *BCD* is rigid and takes most of the wind and live loads, and that *ABDE* serves merely to support the sheaves and transmit these loads to *BCD*. For the side-elevation of *BCD*, the resultant of the rope-pulls is resolved into horizontal and vertical components. All the horizontal forces are assumed to be applied at *B* and act on *BCD*. The reactions of the vertical components at *A* and *B* are computed and the structure *BCD* is assumed to carry those at *B*, while the front-posts carry the reactions at *A*. It may be assumed that the wind-loads at *A, F, G,* and *E* are carried by *BDC,* or they may be considered as producing stresses in the cross-bracing in *ABDE*. This method will give a structure which, if anything, errs on the side of safety. The cross-bracing in *ABDE* would be proportional to the size of the posts. Since part of the area of some of the sections is cut away for joints, the size of these members should be increased to provide the desired strength.

Ore-Bins. Ore-bins serve, in general, to regulate the movement of ore between the mine and railroad, or between mine and mill, so that a temporary cessation at one end does not stop operations at the other end of the system. In most cases, ore-bins are designed to hold from two to three days' supply of ore, but local conditions may call for modification of these figures. In a country where Sunday transportation of ore is forbidden by law, the capacity of an ore-bin must be such that it affords ample storage-space from Saturday to Monday, which means that the bin should hold practically a three days' supply of ore.

Timber, steel, and reinforced concrete are used for making ore-bins. The largest are usually built of steel. Medium and small-sized bins are generally built of timber, because of its availability, ease of working, and cheapness. At some mines, timber bins of large capacity have been built for the reason that the mine was beginning production, and it was impossible to obtain steel within a reasonable time.

Fig. 108-111 show side-elevations of the most common forms of bins. The triangular shape, shown by the dotted lines in Fig. 108 is used sometimes, but is not economical, on account of the large amount of timber re-

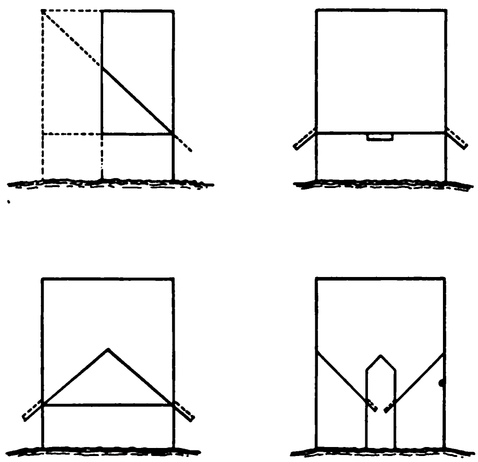

FIG. 108, 109, 110, AND 111. TYPES OF ORE-BINS.

quired. Flat-bottom bins, such as are shown in Fig. 109, may have discharge-gates on one side, both sides, or in the bottom. The bin shown in Fig. 110 is self-emptying. Fig. 111 shows a bin that requires only one railroad track because of its central discharge-gates.

Flat-bottom bins cannot be emptied without shoveling. However, this shape gives the maximum capacity for a given floor-space and height, the bottom is protected from wear, and the cost of the bin-bottom is from one-third to one-half that for an inclined bottom. A flat-bottom bin is also cheaper for a given storage-capacity. In order to be self-emptying, the bottom of a bin must slope at an angle of 45° or more. The bottom should be protected from wear by steel plates.

In figuring the capacity, the weight of ore is usually assumed at 100 lb. per cu. ft. Where the ore is dumped into the bin from several fixed points, the ore will stand in a series of cones and the full capacity of the bin cannot be obtained. An allowance for this condition should be made in computing the capacity of the bin.

In case the bin is to be part of a mill, its length should conform to the floor-plan of the mill. Since increasing the height of a bin increases its cost considerably, a long narrow bin is the most economical. The bin may be divided by partitions into pockets or compartments. The

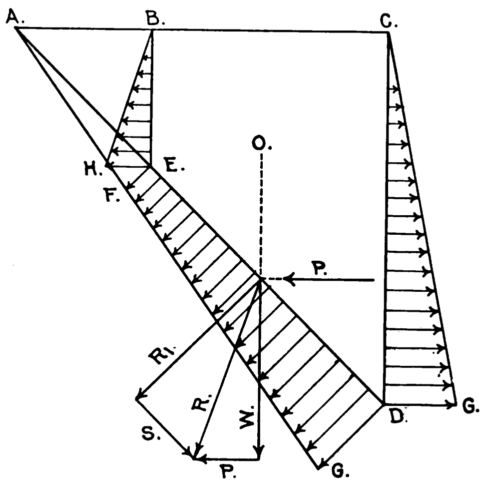

FIG. 112. INFLUENCE DIAGRAM FOR ORE-BIN.

number and size of these pockets depend mainly on the number of different classes of ore that must be kept separate. In a long bin, partitions add rigidity to the structure. Pockets are sometimes made 20 ft. long. As a rule, 20 ft. is also taken as the limiting width and height for a wooden bin. If it is made larger, the timbers must be of such a size that steel construction would be more economical.

When a bin has to support additional loads, such as crushers, or an engine and train of ore, the timbers must be designed for the extra load. The sudden stopping of a train results in severe stresses.

Foundations for bins must be high enough to permit the passage of wagons or cars under the loading-chutes. If the chutes are arranged to discharge at points one-third of the width of the car from the side next to the bin and at quarter-points along the length of the car, no shoveling will be needed to load the car to capacity.

For small or medium-sized bins, the vertical posts along the sides and ends of the bins are unsupported throughout their length. For large bins the load on these beams becomes so great that they must be reinforced by the addition of horizontal ties. A single beam may be used, in which case an I-beam or two channels, placed back to back, may be more economical than a large wooden beam. Tie-rods, from two to five vertical posts apart, run from the front to the back of the bin. To protect the tie-rods from falling ore, a beam with its upper part beveled and shod with a steel plate is sometimes placed just over the rod. Since the horizontal beams and the tie-rods are generally designed to carry all the loads, the vertical posts need be only large enough to carry their load from one horizontal beam to the other. With such construction, the ends of the bin may be a source of weakness. Longitudinal tie-rods in a bin are not desirable, so there is an unsupported span of the full width of the bin at the ends of the horizontal beams. The beams must be designed for this condition.

The stresses may be computed as shown in Fig. 112. The weight of ore being known, the weight of *ACD* for 1 ft. in width can be computed. This weight, *W*, acts as the centre of gravity of the triangle, *O*. The total pressure against *CD*, or $P = \frac{1}{2} wh^2 \frac{1 - \sin \Theta}{1 + \sin \Theta}$ where

Θ = angles of repose of the ore.

W = weight of ore per cubic foot.

h = total height, or *CD*.

The load *P* is assumed to be applied at a point $\frac{1}{3}$ *h* above the bottom. *R* is the resultant of *W* and *P*. Its normal component, *R*, is the total normal pressure against the bottom *AD*. To construct the graphic diagram, *R* (in pounds) = $\frac{1}{2}$ *AD* (in feet) times *GD* (in pounds), from which *GD* can be found and laid off on a suitable scale. Then the area *FEDG* = total normal load on bottom-beam *ED*. One-half of the horizontal component of the normal pressure against the bottom is assumed to be applied at *E* and causes bending in the rear-post. One-half of the component of the bottom-pressure parallel to *ED* is assumed to be applied at *E* to cause compression in beam *ED*. The spacing of the bottom-beams is now assumed and their size calculated. Supports may be used along the bottom-beams to keep them within a reasonable size. The front-pressure may be computed by either of the methods discussed above. Since a flat-bottom bin has only the weight of the ore to be carried by the bottom, the pressure against the sides is calculated by the formula for finding *P*. The total pressures found for the side and inclined bottom of a bin may be assumed to act uniformly over the length of these beams. This simplifies the design and introduces no serious error. The walls of the bin are assumed to be smooth, so that the angle of friction is taken as zero.

CHAPTER XV

Wind-Pressure and Wind-Stresses
Working Drawings

The subject of wind-pressure and wind-stresses is an unsatisfactory one to discuss. There is a wide variation in opinion as to the wind-pressure that should be adopted for different types of structures, and for different heights of the same structure. Again, the authorities differ as to the unit-stresses that should be allowed in designing for wind. Finally, several methods are in use for finding the stresses resulting from wind. The latter statement applies more particularly to the steel-framed office-building than to the mill-building type of structure.

In order to bring out these points more clearly, I quote from Milo S. Ketchum's 'Structural Engineers' Handbook.' This authority specifies in regard to mill-buildings, as follows:

"Wind Loads. The normal wind-pressures on trusses shall be computed by Duchemin's formula, with $P=30$ lb. per sq. ft., except for buildings in exposed locations, where $P=40$ lb. per sq. ft. shall be used.

"The sides and ends of buildings shall be computed for a normal wind-load of 20 lb. per sq. ft. of exposed surface for buildings 30 ft. and less to the eaves; 30 lb. per sq. ft. of exposed surface for buildings 60 ft. to the eaves, and in proportion for intermediate heights."

Also, after defining the unit working-stresses for dead and live loads,

"When combined direct and flexural stress due to wind is considered, 50% may be added to the allowable tensile and compressive stresses."

In the case of steel highway-bridges, Mr. Ketchum specifies as follows:

Wind Loads. The top lateral bracing in deck-bridges and the bottom lateral bracing in through-bridges, shall be designed to resist a lateral wind-load of 300 lb. for each foot of span; 150 lb. of this to be treated as a moving load.

"The bottom lateral bracing in deck-bridges, and the top lateral bracing in through-bridges, shall be designed to resist a lateral wind-force of 150 lb. for each foot of span. In bridges with sway-bracing, one-half of the wind-load may be assumed to pass to the lower chord through the sway-bracing. For spans exceeding 300 ft., add in each of the above cases 10 lb. additional for each additional 30 feet.

"In trestle-towers, the bracing and columns shall be designed to resist the following lateral forces, in addition to the stresses due to dead and live loads: The trusses loaded or unloaded, the lateral pressures specified above; and a lateral pressure of 100 lb. for each vertical linear foot of trestle-bent."

For direct wind-stresses, not combined with flexural wind-stresses, the above specifications allow an increase of 25% in the unit working-stresses; when direct and flexural wind-stresses are combined with dead and live load stresses, the unit working-stresses may be increased 50%. These specifications, while for steel structures, should also apply to timber structures, except possibly as regards the increase in unit working-stresses.

In the case of buildings of the mill type, a number of experiments have been made on small models, some of which would indicate that the ordinary assumptions as to the action of wind on buildings of this type do not hold. Albert Smith of Purdue University has found that in some instances there is tension in certain truss-members which by the commonly accepted method of design would take compression, and vice versa. In other words, he finds a suction on certain portions of the roof in such a building, instead of a pressure, or instead of neither suction nor pressure, as would be shown by the ordinary analysis.

Wind-pressure on a building produces bending in the columns, just how much bending is a disputed question.* There is no doubt that the specifications of Mr. Ketchum, if followed consistently, will result in a building of safe design. The question to be decided by the engineer is whether or not, such specifications, when applied to timber buildings, are too severe.

R. Fleming, of the American Bridge Company, has made a study of all available discussions in technical literature on this subject, and has published several articles on the subject in the *Engineering News*. Recently, in an endeavor to standardize the various conflicting specifications, he has proposed a set of Specifications for Structural Steel Work.†

On the subject of wind-pressure, Mr. Fleming proposes the following specifications:

"Wind-Pressure. All steel buildings shall be designed to carry wind-pressure to the ground by steel-framework.

"Buildings of Class No. 1 (mill buildings) not over 25 ft. to the eave-line shall be designed to resist a horizontal wind-pressure of 15 lb. per sq. ft. on the sides of the building, and the corresponding normal component on the roof according to the Duchemin formula for wind-pressure on inclined surfaces.

"Where buildings are more than 25 ft. to the eave-line, the horizontal pressure shall be taken at 20 lb. per sq. ft., and the corresponding normal component on the roof.

"Only the excess of the wind-stresses obtained by this paragraph over the wind-stresses according to Clause 13 (stresses due to dead and live load) need to be considered. In arriving at this excess the wind included in the total uniform loads designated in Clause 13 shall be assumed at 10 lb. per square foot.

*The difficult point to determine is the exact distribution of the reactions at the foot of the columns resulting from wind, and the amount of 'fixedness' existing in the column, the latter being dependent on the presence or absence of sufficient anchorage.

†*Engineering Record*, Vol. 74, No. 24, Dec. 9, 1916.

"For combined stresses due to wind and other loads, the above mentioned stresses (working stresses for dead and live loads) may be increased 50%, provided the section thus obtained is not less than that required if wind forces be neglected."

It is my experience in checking over many designs, and observing the sizes of members and connections of buildings which have stood for several years, that the greater number of buildings of the mill-building type which are supposed to be designed for a wind-pressure of 20 to 30 lb. per sq. ft. of exposed surface, would not stand over half this pressure if consistently figured according to the commonly accepted methods of design. The greatest weakness is found in the knee-brace connections to trusses and columns, and in the columns themselves. In the case of timber-framed buildings, this comment applies to the actual section of column as a whole; in the case of steel-framed buildings the inconsistence in design may lie in the relative strength of the column section as a whole and the details. For example, an analysis of a steel column built up of four angles laced together, will often show that, while the moment of inertia of the column-section as a whole is sufficient to take the bending due to a 20 or 30 lb. wind, the lacing-bars are far deficient to withstand the compression due to wind shear, as they are usually constructed of 'flats,' with a large ratio of length to radius of gyration.

In the case of a timber-framed building, of moderate dimensions, a rigid adherence to the standard specifications, such as Ketchum's quoted above, will often give results that are out of reason, when compared to buildings that have long given service, and whose strength no one would seriously question. This can best be brought out by a typical example. Consider a timber-framed building of the mill-construction type, with trusses 16 ft. centre to centre, 30-ft. span, and with a height to the eaves of 15 ft., height from floor to foot of knee-brace 11 ft., distance from floor to bottom of trusses 15 ft., and an over-all height of 24 ft. At 20 lb.

per sq. ft., the total wind-pressure on one bay is $16 \times 24 \times 20 = 7680$ lb. Assuming the wind-reactions to be equally divided between the windward and leeward columns (the usual assumption in design), the reaction on one post is 3840 lb., and the moment at the foot of the knee-brace is $3840 \times 11 \times 12 = 507{,}000$ lb.-in. Using a maximum fibre-stress of 1800 lb. per sq. in., the required section-modulus of the column is $\frac{507000}{1800} = 282$ in. (It is assumed that the dead load and the direct wind-load will not stress the post over 1000 lb. per sq. in. in addition to the 1800 lb. due to the wind). This section modulus corresponds to a 10 by 14-in. post. Yet a building of this size and type with posts 10 by 14 in., 16 ft. centre to centre carrying a corrugated-iron roof and walls, would be considered a monstrosity, and rightly so. It is true that a smaller post might be used, if the columns are fixed at the base. Assuming that the columns are rigidly fixed at their bases, the wind-pressure producing bending in the posts would then be $19\frac{1}{2} \times 16 \times 20 = 6240$ lb. The reaction producing bending would be 3120 lb., and the column-bending, $3120 \times 5\frac{1}{2} \times 12 = 206{,}000$ lb.-in. The required section-modulus of the timber is $\frac{206000}{1800} = 114$ in., which is furnished by an 8 by 10-in. timber. Even this size of post is too heavy. In addition, it will be found to be difficult to design an anchorage that will develop the required fixing-moment, and which can be depended upon to remain tight under all conditions.

In the first instance taken, the building with columns hinged at the base, the stress in the knee-brace is $\frac{15}{4} \times 3640$ lb. $\times 1.41 = 19{,}000$ lb. In the second case, columns fixed at the base, the knee-brace stress is $\frac{9.5}{4} \times 3120$ lb. $\times 1.41 = 10{,}400$ lb. Both these stresses will require a well designed connection of knee-brace to both column and post. A bolt or two, and a few spikes will not suffice. It must also be remembered that the connection must be designed so that it will be able to withstand both tension and compression.

For a building of the type and size just described, I believe that a wind-pressure of 10 to 15 lb. per sq. ft. of exposed surface is sufficient for the design of the trusses and posts. The girts should be designed for a load of 15 to 20 lb. per sq. ft. of tributary area. The posts should be tied into the foundations, since the dead load coming on the posts is small. I have often found it necessary to provide more concrete in the post-footings than is required from considerations of unit soil-pressure, in order to give rigidity to the building. In such cases, $\frac{5}{16}$ by 3-in., or $\frac{3}{8}$ by 3-in. strap-iron, bolted to the posts, and anchored in the concrete footing will give a certain 'fixedness' to the posts. The necessity of such anchorage can be determined easily: if the anchorage is merely to prevent overturning of the building, the direct wind-load in the column should not be allowed to exceed about 80% of the computed dead load in the column. If the weight of the concrete footing is utilized to give fixedness to the column and thus reduce the wind-bending, it may be found that a considerable mass of concrete is necessary.

In referring to overturning of the building, I have in mind incipient overturning, or a lifting of the windward-post off its base. It is almost inconceivable that the building could overturn as a whole. A 'mind's-eye' picture of the probable action of such a building under a terrific wind will emphasize the enormous strain that would come upon the knee-brace connections, and will bring home the fact that such connections are the most important in the whole structure in resisting lateral forces.

In making the above recommendations for a reduced wind-pressure to be figured on mill-construction buildings, I am considering localities not subject to cyclones or tornadoes. It is my belief, that a carefully designed timber-framed building, with connections intelligently studied, will be perfectly safe under all conditions that may arise on the Pacific Coast, at least, provided that the girts are designed for 20 lb. wind-pressure per sq. ft., and the frame for 15 lb. Indeed, in some places, and for some buildings, I would not hesitate to reduce the foregoing pressures to 15 and 10 lb. respectively. A 15-lb. wind-pressure would be produced by a gale of a velocity of 60 miles per hour, which seldom occurs even for a few minutes.

The corresponding unit-stresses to be employed in connection with the wind-pressures advocated above should not exceed 2000 lb. per sq. in. for combined dead and wind load, including flexural and direct wind-load stresses. Unless the building is of unusual length, the end-walls offer considerable resistance for transferring the wind to the ground, the roof acting as a horizontal or inclined truss delivering the wind-load to the end-walls. For this reason, such walls should be well braced, with diagonal bridging.

Working Drawings

Not only must the designer of timber-structures be able to compute the necessary sizes of the members, and the details of the connections; he must be able also to present his design clearly to the builder. This statement is not peculiar to timber-framing, yet it needs to be emphasized, especially in this connection. The engineer accustomed only to steel design, and even the engineer versed in reinforced concrete design, is prone to leave much to the detailer, knowing that standard practice will govern many details, and that he will check over such details after the design is completed, before fabrication of the structural steel or the steel reinforcing-bars is begun. As has been stated already in these pages, standard details, in timber design, do not exist. Left without working-details, and given sizes of main members, the carpenter will build a structure. Whether such structure will be safe, depends largely upon the carpenter's experience. A timber cut too short may be spliced with comparative ease, even if not with full safety, and by means of saw, hammer, and a few nails, a makeshift connection that may appear to be of sufficient strength can always be accomplished.

In the preparation of drawings for a timber-framed structure, two conditions present themselves, (1) when the structure is to be built by contract, and (2) when it is to be constructed by day labor or force-account. In the first case, the engineer may require detail drawings to be furnished, just as for a steel-framed structure, such details to be checked and approved by him before any material is bought. For the steel and iron-work, this method may be entirely satisfactory, provided that the contract drawings and specifications show clearly just what is wanted, since such detailing will in all probability be done by an experienced structural draftsman. This is providing that the job is of sufficient magnitude so that the steel and iron-work will be fabricated by a shop of some size. For the timber-work, it will be necessary for the designer practically to detail the job completely, as only in this manner can the desired connections be shown clearly. A case of an all-timber structure where the designer can show a diagrammatic plan, elevation, and sections, giving sizes of members, and main dimensions, and expect a draftsman to draw up satisfactory details is practically an impossibility.

In the second case, where the structure is to be built directly from the designer's plans, with no other details, particular care should be taken to see that every important member and connection is shown clearly. The steel should be detailed accurately and fully, the number and length of all rods, bolts, etc., listed, and all steel should be designated in accordance with a clear system of marking. In this work, one day in the office is worth at least two in the field. There is no better check that can be applied to drawings than to prepare an accurate list of every piece of material in the structure. In fact, I know of no better method to make one realize the convenience, not to say necessity, of fully-detailed drawings, than to be compelled to make a complete detailed estimate of cost. While in the case of small timber-structures, and for some larger ones, it is the custom of the contractor or carpenter to order bolts and other small steel material as he needs them in the course of construction, such a course will not be satisfactory on a large structure. Even on a small job, it is an inefficient and wasteful method.

The engineer will sometimes be called upon to furnish plans and specifications for timber structures in isolated localities, where all material needed for the job must be purchased beforehand, and shipped to the site, and where mistakes in ordering material or in showing details may cause serious delay and expense. For such a condition, I believe that it pays well to mark every bolt and rod, that is to say, all bolts of a certain length and diameter are to be given a special mark, as a letter or group of letters, or a combination of letters and figures, in accordance with some definite system. For example, all bolts in columns may be given the prefix *C*, as *C*-1, *C*-2, etc. Not only should these marks appear in the bolt-list after the particular bolt-size, but the marks should be placed on the bolts on the drawing in the elevation of the column. Further, the bolts should be shipped in bundles of one size and length, bound together, and tagged. This is, perhaps, going outside the domain of strict design, and into the field of detailing and construction, yet it should be a part of the designer's task, in the case under consideration, to detail the work, and to draw his specifications for the contractor furnishing the iron-work so that the field work will be a minimum. This suggestion as to marking applies to all steel of whatever shape. Rods should be tagged, and structural shapes plainly marked by painting, with the corresponding marks at the proper places on the drawings. It is highly desirable to mark the cutting lengths of the important timbers on the drawings; it is much simpler, and better, for the designer, who, at the time, has the structure well in mind, to note the lengths of timbers, than for the carpenter to compute the lengths. Objection might be made to this statement, on the ground that it puts the responsibility for accuracy on the engineer, rather than on the carpenter. For a contract drawing, such a contention may hold; for the detail drawing as required under our present assumption, the argument is unsound. Thorough checking is essential, and such checking should always be given, even at the expense of having the owner annoyed by an apparent needless delay in the completion of the drawings. After the structure is well under way, and the work is progressing smoothly and rapidly, the owner will forget any small delay in getting out the plans and specifications; on the other hand, he will seldom forget a mistake.

In the preparation of drawings for timber-framed structures, there should be a general plan, framing-plans, elevations, cross and longitudinal sections, and details. The exact number of drawings, it is hardly necessary to state, will depend altogether on the kind of structure, and its simplicity or complexity. In general, the plans as opposed to elevations, sections, and details, should be to the scale of eight feet to the inch, or, as commonly called, $\frac{1}{8}$-in. scale. In some cases, it may be advisable, for the sake of clearness, to use a larger scale, as $\frac{1}{4}$ in.; and certain small parts of the general plans may need to be re-drawn to a $\frac{1}{2}$-in. scale, in addition to the smaller scale. No matter how many parts of the building may be drawn to a large scale, as $\frac{1}{2}$ in., a complete plan to a $\frac{1}{8}$-in. or $\frac{1}{4}$-in. scale is needed, in order that the entire structure may be seen at a glance. The elevations can usually be shown to a $\frac{1}{8}$-in. scale, and the general cross and longitudinal sections to a $\frac{1}{8}$-in. or $\frac{1}{4}$-in. The details should be at a scale not less than $\frac{1}{2}$-in.

In the case of a frame building of the mill-construction type, taking a typical example, of a building 100 ft. long, and one bay wide, trusses say 40-ft. span, corrugated-iron sides and roof, and floor of timber construction, about 3 ft. off the ground, the following plans will show the work completely:

(1) One sheet, to a $\frac{1}{8}$-in. scale, showing the four elevations, with all window and door openings, the doors and

windows being lettered or numbered to correspond with details of same.

(2) Foundation-plan, to a $\frac{1}{8}$-in. scale, showing size and position of piers and wall-footings, with $\frac{1}{4}$-in. or $\frac{1}{2}$-in. details of the individual footings and piers.

(3) Floor-framing plan, to a $\frac{1}{8}$-in. scale, showing sizes of joists and girders and posts, with all dimensions of spacing of same, and centre lines of truss-posts, and first-floor posts.

(4) Roof-framing plan, showing main trusses, with their proper letters, bracing-trusses, bracing, roof-joists, roof-covering.

(5) Cross-section for the building to a $\frac{1}{2}$-in. scale, completely detailed as to roof-joists, trusses, columns, and floor-construction.

(6) Miscellaneous-timber details to a $\frac{1}{2}$-in. scale, as may be necessary.

(7) Details of all fabricated steel to a 1-in. scale.

In general, such scales as $\frac{3}{16}$-in., and $\frac{3}{8}$-in., should be avoided, although no hard and fast rule can be made. An architect employs a $\frac{3}{4}$-in. scale to show details on a contract drawing. It is often convenient, therefore, to use the same scale when preparing structural drawings for an architect; the architect's tracings may be superimposed on the structural drawings, and vice versa. Mistakes of clearances may sometimes be found in this manner. However, on the other hand, there is often an advantage in re-drawing the architect's outlines within which the engineer must confine his work; errors of scale are discovered in this manner. The converse is also true; the architect may find mistakes in the engineer's drawing when he lays it out on the architectural sheets.

It is unwise to furnish a drawing that is badly out of scale, even if it is fully and accurately dimensioned. This statement holds for construction in any material, but is especially true in timber framing, as the carpenter is almost sure to scale some timbers. For this reason, considerable erasing, and even re-drawing and re-tracing will be well worth the effort and expense, if, by such extra work, a drawing badly out of scale may be made to scale. Serious errors on the carpenter's or contractor's part may thus be avoided.

Finally, a general and comprehensive note should be placed on all structural drawings. This procedure may not be in accordance with the theory held by many, that written instructions are specifications, and as such, should not appear on the drawings. If this view is held, allow the specification writer to incorporate such instructions to the contractor in his specifications, but be verbose to the extent of repeating the more important points on the drawing in a general note. The specifications, bound separately from the plans, often become separated from them. Notes on a drawing cannot be detached from the details. Finally, it is a curious fact that a note on the drawings carries about twice as much weight with a carpenter as an obscure sentence in the specifications.

CHAPTER XVI

Specifications for Timber Framing

The following specifications are primarily for timber-framed mill buildings, to be constructed of Douglas fir. The unit stresses for timber, as given, are for partially air-seasoned timber, as distinguished from thoroughly seasoned material or from green timber. Further, the unit stresses are for the grade of timber known as No. 1 Common.

When the conditions are different from those just outlined, as, for example, green timber, structures exposed to the elements, or lumber containing No. 2 Common, lower unit stresses are to be used, and the proportional decrease in stresses must depend upon the judgment of the designer, in accordance with the particular conditions.

For the case of bridges, either railway or highway, the specifications of Milo S. Ketchum, as given in his 'Structural Engineers Handbook' shall be used.

Contract Plans

Unless specifically stated otherwise, the plans to be furnished are to be what are known as 'Contract Drawings.' That is, the drawings and specifications are to show the structure in such detail that the exact amount of all material may be determined without resorting to computations for strength of any member or detail of the structure, but subsequent shop and field details will be required, the same to be checked in a general way by the engineer for strength, but not for accuracy of detail-dimensions.

To this end, there shall be furnished, in general, a foundation-plan, framing-plan, sections, and elevations, and typical details of all connections, sufficiently dimensioned and noted, so that the detailer may understand fully the requirements of the design. The specifications shall state the kind and quality of all material entering into the structure and shall give all other information and requirements that the fabricator and erector of the structure will need in order to produce a workmanlike job in conformity with the requirements of the design.

In the case of building-plans, the scope of such plans may be more specifically stated as follows. There shall be furnished a general ground-plan, foundation-plan, floor-framing plan or plans, depending upon the number of floors, roof-framing plan, typical sections, cross or longitudinal, and details of all important connections.

Scale. The scale for framing-plans shall be $\frac{1}{8}$ in. or $\frac{1}{4}$ in. to 1 ft. The same scale shall be used for elevations and small sections. Larger sections in which it is desired to show connections of members in addition to the general arrangement of structural members, shall be on a scale of $\frac{1}{2}$ in. or $\frac{3}{4}$ in. to 1 ft., preferably the former. Details of steel and iron-work, as shoe-plates, washers, etc., shall be at a scale of not less than $\frac{3}{4}$ in. and preferably to a scale of $1\frac{1}{2}$ in. to 1 foot.

Detail Specifications—Structures of the Mill-Building Type

Under this class will come mill-buildings, power-houses, pump-houses, machine-shops, armories, skating-rinks, amusement pavilions, exposition buildings, etc.

Roof Loads. For localities where a snow-load cannot occur, the following minimum loads shall be used.

1. Dead Load. The dead load shall consist of the weight of the roof-covering, rafters, purlins, roof-bracing truss, and ceiling, where the latter occurs. The weight of the roof-covering, rafters, and purlins shall be taken as applied at the panel-points of the upper end of the truss. The weight of the roof-truss for light trusses may be considered as concentrated at the upper

chord. For roof-trusses, in which the dead weight of the roof-truss is over 15% of the total dead and live load supported by the truss, and including the weight of the truss itself, the weight of the truss shall be considered as applied equally at the upper and lower-chord panel-points. The weight of the ceiling, where such occurs, shall be considered as concentrated at the lower chord.

2. Live Load. The live load on the roof shall be taken at 20 lb. per sq. ft. of projected area for rafters and purlins, and the same figure shall be used in computing bending in the top chords of the trusses. The roof-covering shall be designed for a load of not less than 30 lb. per sq. ft. and computations shall be made both for strength and stiffness. The live load on the trusses shall be taken at not less than 15 lb. per sq. ft. of tributary area.

3. Wind Load. The wind load on the roof of buildings not over 25 ft. to the eaves, shall be considered as applied normal to the roof-surface, and the amount of such normal wind-load shall be computed by Duchemin's formula.

$$p = P \frac{2 \sin \Theta}{1 + \sin^2 \Theta}, \text{ where}$$

p = normal pressure on roof in lb. per sq. ft.

P = 15 lb. per sq. ft.

Θ = angle which the plane of the roof-surface makes with the horizontal.

For buildings over 25 ft. in height to the eaves, P in Duchemin's formula shall be taken at 20 lb. per sq. ft.

Walls. For buildings not over 25 ft. in height to the eaves, the wall-covering and girts or studs shall be designed for a horizontal wind-pressure of not less than 20 lb. per sq. ft., and the columns, when forming a transverse bent with the roof-trusses, shall be designed for a horizontal wind-pressure of 15 lb. per sq. ft. For buildings over 25 ft. in height to the eaves, the above pressures shall be increased 5 lb. per square foot.

All roof-trusses and columns shall be designed for the maximum of the two following conditions.

1. Dead load plus wind load.
2. Dead load plus live load.

Unit Working-Stresses

DOUGLAS FIR TIMBER (Grade No. 1 Common)

	Lb. per sq. in.
Tension with fibres	1,500
Compression, end-bearing	1,600
Compression across fibres	300
Bending, extreme fibre-stress	1,500
Modulus of elasticity:	
(*a*) For dead load only	1,200,000
(*b*) For live load only	1,600,000
Shearing with grain	150
Longitudinal shear in beam	175
Columns:	
For columns under 15 diameters	1,200
For columns over 15 diameters	$P = 1600\left(1 - \frac{1}{60}\frac{L}{d}\right)$

where P = unit working-stress in lb. per sq. in for centric loads
L = unsupported length of column in inches
d = least width of column in inches

Steel:	Lb. per sq. in.
Tension	16,000
Shear	10,000
Bearing	20,000
Cast Iron:	
Bending, extreme fibre stress	4,000
Tension	3,500

Pressures on Inclined Surface of Timber. The safe unit working-compression on timber on surfaces inclined to the fibres shall be taken in accordance with the formula:

$$n = p \sin^2 \Theta + q \cos^2 \Theta$$

Where n = allowable unit compression on inclined surface
p = allowable unit compression on ends of timber
q = allowable unit compression across fibres
Θ = angle which surface makes with the direction of the fibres.

Pressure of Circular Iron Pin on Timber. The safe average unit stress on the diametrical section of an iron pin bearing on timber in a close fitting hole, shall be taken as follows:

1. When the direction of loading is parallel to the length of the fibres,

$$p' = \tfrac{2}{3}p + \tfrac{1}{3}q$$

2. When the direction of loading is perpendicular to the length of the fibres,

$$p'' = \tfrac{1}{3}p + \tfrac{2}{3}q$$

In these formulas, p' and p'' are the safe average unit-stresses on the diametrical sections of the pin, parallel and perpendicular, respectively, to the direction of fibres.

Strength of Nails When Used with Douglas Fir. 1. Lateral strength of wire nails. The safe working-resistance of wire nails or spikes to lateral shear for static loads, bearing either against the ends of the fibres of the timber, or across the fibres, shall be taken as follows:

Size of nail	Safe lateral resistance, lb.
6D	48
8D	64
10D	80
12D	96
16D	128
20D	160
30D	240
40D	320
50D	400
60D	480
80D	640

2. Resistance of wire nails to withdrawal. The safe working-resistance of wire nails or spikes to withdrawal from timber, when the nail or spike is driven perpendicular to the fibres of the timber, shall be taken at 75 lb. per sq. in. of contact surface of wood and nail. For nails driven parallel to the fibres, the

safe loads shall be taken at 25 lb. per sq. in. of contact surface of wood and nail.

Strength of Common Wire Screws When Used with Douglas Fir: 1. Lateral resistance of screws. The safe working-resistance of wood screws to lateral shear, for static loads, bearing either against the ends of the fibres, or across the fibres, shall be taken as follows:

Gauge of screw	Safe lateral resistance, lb.
12	205
14	256
16	315
18	380
20	450
22	529
24	615

The length of the screw shall be approximately two and three-quarters times the thickness of the side-piece.

2. Resistance to withdrawal. The safe working resistance of wood screws to withdrawal from timbers, when the screw is inserted perpendicular to the direction of fibres shall be taken as follows:

Gauge of screw	Safe resistance to withdrawal per linear inch of insertion, lb.
4	75
8	100
12	125
16	140
20	150
22	170
28	185

For screws inserted parallel to the fibres, the safe working resistance to withdrawal shall be taken at 75% of the above values.

Strength of Lag-Screws When Used With Douglas Fir: 1. Lateral resistance when used in fastening planking to large timbers. The safe working-resistance of lag-screws to lateral shear when used in fastening planking to large timbers shall be as follows:

¾ by 4½ in. lag-screws	....	900 lb.
⅞ by 5 in. lag-screws	....	1050 lb.

The thickness of such planking shall not exceed ⅜ of the length of lag-screw.

2. Lateral resistance when used in fastening steel plates to timbers. The safe working-resistance of lag-screws to lateral shear when used in fastening metal plates to timbers, when such plates are not less than ¼ in. to ½ in. thick, shall be taken as follows:

½ by 4 in.	....	700 lb.
⅝ by 4 in.	....	860 "
¾ by 4½ in.	....	1030 "
⅞ by 5 in.	....	1200 "

3. Resistance of lag-screws to withdrawal: The safe working resistance of lag-screws to withdrawal, when inserted perpendicular to the fibres, shall be taken at 180 lb. per sq. in. of the surface obtained by multiplying the nominal diameter of screw by the length of the threaded portion of screw, excluding the tapering end.

Strength of Bolts

1. Bolts in Double Shear—All end-bearing on timbers. The strength of bolted timber joints should be computed by the methods explained in the text of Chapter V.*

For a given diameter of bolt, the length l shall be found, by use of the formula

$$l = d \left(\frac{\pi S 27}{32 B}\right)^{\frac{1}{2}},$$

when $l = a + b$ (See Fig. 36).

$d =$ diameter of bolt in inches.

$S =$ maximum allowable flexural unit stress in the bolt $= 16{,}000$.

$B =$ maximum allowable unit bearing-stress against the ends of fibres of timber.

*For a practical solution of the strength of bolted joints it will be necessary to construct diagrams similar to that of Fig. 37 of Chapter V. Tables can then be prepared that will cover the ordinary range of construction.

$t =$ thickness of splice-pad $=$ one-half thickness , of main timber.

a, b, B^1, P, P and $P_2 =$ as shown in Fig. 36.

The joint will be in one of the two following classes:

A. Thickness of splice-pad equal to or greater than computed value of l.

B. Thickness of splice-pad less than l.

If the point falls under class A, the strength of the joint for one bolt shall be found by means of the formula:

$$P = \tfrac{1}{2}Btd$$

where P = total safe load on joint for one bolt in double shear and bending.

If the joint falls under class B, it shall be still further classified as follows:

a. Pressure uniform along length of bolt.

b. Pressure distribution along length of bolt trapezoidal in shape, with a maximum intensity B at contact faces of main timber and splice-pads. The lowest unit pressure will be B', at the centre of main timber, and at the outside-faces of the splice-pads. The value of B' will vary between the limits $B' = 0$ and $B' = B$.

c. Pressure distribution along length of bolt triangular, but with varying values of a and b, the limits being $a = O$ and $a = \tfrac{1}{3}$.

2. Bolts in Double Shear—Centre or main timber with bearing across the fibres, splice-pads or outside timbers with end-bearing.

Use values two-thirds those of Class 1.†

Drift-Pins: The safe resistance of round drift-pins to withdrawal, when such drift-pins have been driven perpendicular to the fibres of the timber and in holes of a diameter not greater than $\tfrac{7}{8}$ of the diameter of the drift, shall be taken at 180 lb. per sq. in. of contact-surface of wood and metal.

When such drift-pins are driven parallel to the fibres,

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†The case of metal plates bolted to timbers is purposely omitted, as I prefer to await the publication of tests which have been made.

and in holes of a diameter not to exceed $\frac{7}{8}$ of the diameter of the pins, the safe resistance to withdrawal shall be taken at 90 lb. per sq. in. of contact-surface of wood and metal.

The safe resistance of such drift-pins to pulling through the timber in the direction of driving shall be taken at not to exceed 60% of the above values.

Shear-Pins: The safe working-resistance of 2-in. circular shear-pins of solid steel, extra heavy steel pipe, Hawaiian ohia, or Australian iron bark shall be taken at 800 lb. per linear inch of pin. Bolts shall be provided with a total capacity in tension equal to one-half the total load on the joint. Pins shall be spaced not closer than six inches centre to centre.

The shear-pin joint shall be used with seasoned timber only.

General Conditions of Framing

Special attention shall be paid to laying out column centres, and the general arrangement of trusses, posts, girders, and joists, in order that a stiff structural frame may be secured. To this end, the roof-trusses shall be well braced, both upper and lower chords, by means of bracing-trusses, or their equivalent. The wind-stress on the building shall be carried to the foundations through the structural frame, and all parts thereof shall be consistently designed to accomplish this purpose.

Roof-joists shall be lapped over truss-chords not less than 12 in., and spiked well to each other and to the truss-chord, or if such roof-joists abut over the chord, splice-pads not less than 2 ft. long shall be provided on both sides of each joist.

Knee-braces to trusses, when used, shall be attached rigidly to the truss and post, and shall meet the truss at a panel-point only.

Interior floor-columns, when possible, shall be in line with the wall-posts, and these cross-lines of posts shall be well tied together by means of girders or joists. For this purpose, the joists shall, when possible, be so spaced at the posts, that two joists shall tie the lines of columns together. When the girders frame into the posts, the girders shall be well tied to the posts by means of splice-pads.

Girders framing into the sides of posts shall be supported, when possible, on side-bolsters, dapped into the posts and bolted to them, and such bolsters shall have all end-bearing.

In buildings of a height of 20 ft. or over to the eaves, diagonal bracing-rods shall in general be provided in at least every other one of the outside-bays, in the plane of the upper or lower chords of the roof-trusses. Special cases may occur where this requirement is not necessary, and this will depend upon the judgment of the engineer.

When timber posts rest upon concrete foundations, a steel or iron base plate shall be provided between the concrete and the bottom of post.

For all structures exposed to the weather, special attention shall be paid to the detailing of joints, in order that such finished joints shall shed rather than hold the water that would tend to collect from rain. In the case of such structures, all bearing-surfaces of timber to timber, and timber to iron, and also all ends of timber, shall be treated with one good coat of wood-preservative. When the importance of the structure will permit, steel or iron bearing-surfaces shall be provided at the ends of timber that is bearing against the side of timber.

Roof Covering

Corrugated Steel: When used for permanent buildings, corrugated steel shall never be less than No. 24 gauge. When this weight of steel is used, the maximum spacing of purlins shall not exceed 4½ ft. The end laps shall be not less than 6 in. and side laps not less than two corrugations.

Timber Sheathing: Timber sheathing shall be dressed and matched lumber, free from loose knots, and of a width not to exceed 6 in. Purlins or rafters shall be spaced so that the deflection for a live load of 30 lb. per sq. ft. shall not exceed $\frac{1}{360}$ of the span of the sheathing, using the formula:

$$\Delta = \frac{3}{128} \frac{wl^4}{EI}$$

where Δ = centre deflection in inches for uniform loading.
w = load per square foot.
l = length of clear span in inches.
E = modulus of elasticity.
I = moment of inertia.

For sheathing of a nominal thickness of one inch, and covered with prepared roofing, the spacing of rafters for permanent buildings, shall preferably not exceed two feet.

Details of Roof Trusses

When the roof joists rest directly upon the upper chords of trusses, the bending-stresses in the chords resulting from such condition of loading shall be computed, and the sum of the direct compression and compression due to bending shall not exceed 1600 lb. per sq in. nor shall the direct compression exceed the safe unit-stress considering the chord as a column.

In computing bending in the chords, such chords may be regarded as continuous beams, supported at the panel joints.

The area of holes for bolts and rods through both compression and tension members of timbers shall be deducted, in order to obtain the net section to resist compression, tension, and bending, the diameter of holes for rods being assumed as $\frac{1}{8}$ in. larger than the nominal diameter of rod or upset end of rod.

In general, full deduction shall be made for notches cut in truss chords for butt-blocks and web-members. In special cases, where such provision will add considerably to the cost, and where the designer is to have full control of framing, such deduction need not be made in the case of the compression-chord, provided that such notches occur only on the compression-side of the chord regarded as a continuous beam for transverse bending, and provided that the butt-block detail is employed.

All tension-rods shall be of steel, conforming to the Manufacturers' Standard Specifications.

If upset ends are used, such upsetting shall be done by machine. No welding will be allowed.

All rods on trusses shall be given an initial tension of at least 1500 lb. and allowance for such tension shall be made in the design.

All joints of end or batter posts of trusses with the lower chord shall be provided with a proper detail, capable of developing the computed stresses in the truss members. Such detail of end-joint shall provide definite lines of action, and such joint shall be, as far as possible, a simple joint, depending for its strength upon one type of detail.

When inclined bolts are used to connect the main members of an end-joint, such bolts shall not have a greater slope than 60° with the centre line of lower chord.

In details of end-shoes employing lugs or tables set into the lower chord, the spacing of such lugs or tables shall be arranged so that no lug or table occurs directly under the end of the upper chord or the batter-post.

The holes in the timbers for inclined bolts in details employing end-shoe plates shall be $\frac{1}{4}$ in. larger than the nominal diameter of bolt.

No daps in chords for butt-blocks shall be less than $\frac{3}{4}$ in. deep.

The minimum thickness of metal in shoe-plates shall be $\frac{3}{8}$ inch.

Steel Lugs and Tables: (Applies particularly to end-shoe plates and tension-splice plates).

The bearing faces of lugs or tables shall have a smooth even surface. If rolled bars are used for tables, they shall be milled on the bearing edges.

The bolts holding the lugs or tables in the notches in the timber shall be placed as near to the lugs or tables as possible.

When rivets are countersunk on one side, in plates less than $\frac{5}{8}$ in. thickness, the values shall be taken at 7500 lb. per sq. in. for shear, and 15,000 lb. per sq. in. for bearing.

All holes in metal over $\frac{3}{4}$-in. diam. shall be drilled, not punched.

No steel lug or table shall have a thickness of less than $\frac{5}{8}$ inch.

Details of Columns

No column shall have a greater ratio of length to least width than 60.

Columns may be considered as fixed at the ends, where provision can be made for obtaining the condition of fixedness assumed.

When bending resulting from wind occurs in columns, the combined stress because of dead load, direct wind-compression, and wind-bending shall not exceed the safe unit-stress as given by the column formula, increased by 25%, considering the width of the column in the plane of bending.

When the column is rigidly supported laterally at the point of maximum combined stress, such maximum combined unit-stress may equal but shall not exceed 2000 lb. per sq. in., and the total combined unit-stress at the centre of the section of column that is unsupported laterally, shall conform to the stress allowed by the column formula, as outlined above.

Built-up columns shall be avoided whenever possible. The strength of built-up columns, composed of two or more sticks bolted together, either with or without packing-blocks, shall be considered as equal to the combined strength of the single sticks, each considered as an independent column.

When it is necessary to employ columns built of planking, such columns shall preferably be of the 'cover-plate' type, in which the edges of the interior planks are tied together by cover-plates. The strength of such a built-up column shall be considered as 80% of a solid stick of the equivalent cross-sectional area. The strength of a built-up column composed of planks laid face-to-face and spiked together thoroughly shall be considered as 80% of the mean of the strengths computed (1), as a solid stick, and (2), as a summation of the strengths of the individual sticks considered as separate columns.

When columns are built of large timbers placed at a considerable distance from each other, such timbers shall be tied together by means of 2 by 12-in. lacing-plates, inclined at an angle of approximately 60° with the axis of the column, and fastened to the column by means of lag-screws, not less than $\frac{3}{4}$ by 6 inches.

When such columns are built of two timbers, laced on both sides, the effective moment of inertia of such built-up columns shall be taken as 80% of the theoretical moment of inertia of the column.

Curved laminated compression members shall be avoided when possible. Where it is necessary to employ such members, the strength shall be computed in accordance with the principles of Chapter IX, taking into account average unit-compression, flexural stress resulting from bending and from eccentricity of loading, and initial flexural stress resulting from springing the boards to a curved shape.

Bolts. All bolts shall be provided with cast-iron or steel plate washers, of a size such that the unit-stress in cross-bearing on the timber under the washer shall not exceed the safe unit-stress for cross-bearing when the bolt is stressed in tension to 16,000 lb. per square inch.

Bolts shall preferably be spaced not closer than 6 in. centre to centre, and not less than 6 in. from the end of any timber, nor less than $2\frac{1}{2}$ in. from the sides of any timber. This rule shall apply to bolts of sizes not to exceed 1 in. diam. For larger sizes the minimum distances given above should be increased accordingly.

All bolts except as otherwise specified, shall be driven in holes of a driving fit.

Inclined bolts through timber shall preferably be provided with beveled cast-iron washers, instead of using standard washers and cutting inclined daps in the timber.

Lag-Screws. All lag-screws shall be screwed, not driven into place.

All lag-screws fastening timber to timber, shall be provided with standard circular pressed-steel washers under their heads.

Holes for lag-screws in steel plates shall be drilled to a diameter of $\frac{1}{32}$ in. larger than the nominal diameter of the lag-screw. In placing lag-screws, a hole shall first be bored of the same diameter and depth as the shank, and the hole then continued with a diameter equal to the diameter of the screw at the root of the thread.

Drift-Pins. Drift-pins shall preferably be round, with or without heads, and shall be driven in holes of a diameter of approximately 80% of the diameter of the pin, and of a length somewhat larger than the length of the drift-pin.

Tension-Splices. Tension-splices shall be of such a type that the effects of cross-shrinkage of the timber will be a minimum. Neither the tabled-steel fish-plate, nor the shear-pin splice shall be used on timbers over 8 in. thick, since the cross-shrinkage of the timber will allow the splice-plates or pads to separate.

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