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LOADING TESTS ON CONVENTIONAL AND TRUSSED ROOF CONSTRUCTIONS

by

A.T. Hansen

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PREFACE

Development of economies in house design and construction is a continuing responsibility of the Division of Building Research. In such studies the Division has been considering critically each of the principal components of the standard house with a view to improvement in design and economy.

This has led to an investigation of roof design in which the Division has followed the lead of American research workers in considering the possible use of prefabricated trusses for house roofs in place of the conventional built-in roof design.

Report No.77 of the Division, "Structural Testing of Two W-trusses" recorded results of load tests on trusses using a design developed in the United States but subjected to the more severe loadings typical of Canadian conditions. This present report deals with loading tests on trussed roofs and several conventional roof constructions. The types of trussed roofs tested were typical of those suggested for use in the United States.

Ottawa,
May 1956

N.B. Hutcheon,
Assistant Director.

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LOADING TESTS ON CONVENTIONAL AND TRUSSED ROOF CONSTRUCTIONS

by

A.T. Hansen

Conventional wood-frame construction as used in house building has evolved to its present status largely through experience in use. The construction system as a whole is difficult to analyse by a standard engineering approach although parts of the system have, in recent times, been designed and specified on the basis of accepted loading requirements. Roof rafters and floor joists are typical of elements where allowable working stresses and design loads are used to select members.

When a change in the conventional system is contemplated it is not always simple to evaluate the effect or merit of the change since there has not been established any generally accepted criteria upon which to base an over-all comparison. In spite of this, new types of construction have been tested and their performance and acceptance based on the knowledge gained through similar tests on conventional types of construction.

In the course of a preliminary study of roof construction, the author attempted to determine the nailing requirements for conventional roof framing under design loads as specified in the National Building Code. From this study there was evidence that conventional roof framing may not be capable of supporting design loads with the normal margin of safety. From theory it can be shown that the nailed joints which are commonly used are inadequate to develop the strength of the rafters. Loading tests on conventional roof structures might then provide a basis for more balanced and economical design in conventional roof framing.

Trussed roof construction is a relatively new development in house construction practice but the possibilities of this form of roof construction have not been explored widely in Canadian practice. One of the difficulties in the way of general acceptance is lack of knowledge of the performance characteristics and cost of trussed constructions. A truss designed according to recognized engineering practice for house construction cannot compete with the conventional roof construction on an economic basis.

In view of the apparent inconsistency of a trussed roof construction as compared with conventional roof construction, both acceptable under similar conditions, it was decided to load-test a number of constructions typical of both systems. Since trusses designed for a location representative of Canadian conditions had been tested previously*, a number of truss designs typical of those suggested for use in the United States were investigated together with several representative conventional constructions.

A. CONVENTIONAL ROOF FRAMING

(1) Description of Test Structures

a. General Description

In order to determine representative types of roof construction, visits were made to several housing developments in the Ottawa area. From these visits it was decided to test three basic types of conventional roof construction. All tests were carried out on a 24-foot span using a roof slope of 5/12. Yard-run eastern spruce was used in all assemblies.

In order for the test structures to resist lateral buckling due to the applied load, the test specimens were tested in pairs placed at 16 inches on centre with 1- by 6-inch sheathing applied to the rafters.

The resistance to lateral thrust that may be provided by walls will vary and may be quite small. Tests were conducted, therefore, both with rollers under one end of the assembly to simulate the condition where walls offered little lateral resistance, and also with both supports of the structure restrained to duplicate the other extreme where the walls are assumed not to move.

A total of three tests were done on most assemblies and the results were averaged.

Three basic types of roof framing techniques were tested (Figs. 36, 37, and 38), and will be referred to as Types I, II, and III.

b. Collar Ties and Rafters

The size and position of the collar ties for the tests was decided upon after an examination of the National Building Code and CMHC Building Standards. The National Building Code requires that a minimum of 2- by 8-inch eastern spruce rafters

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* Structural testing of two W-trusses by D.B. Dorey. Report No. 77, Division of Building Research, National Research Council, Canada. December 1955.

16 inches on centre be used to span a 12-foot horizontal distance, assuming a snow load of 40 pounds per square foot, if the rafters have no intermediate vertical supports. There is no indication in the Code that the collar tie is interpreted to act as an intermediate rafter support. The Code also states that collar ties must be used when the ends of the rafters are not tied together by ceiling joists. It does not state, however, what constitutes a suitable tie. It is questionable if a suitable tie exists when Types II and III constructions are used. It is conceivable that even in Type I construction the rafters and joists may not lap but may be fastened independently to the rafter plate by toe-nails. In fact this was observed often in the course of the field investigation. There is an indirect tie to be sure but it is obviously not as good as in the case where the joists and rafters are joined directly by nails as well as being toe-nailed to the top plate.

The Code also requires that if a minimum of 1 by 5 inches is used as collar ties then these collar ties must be laterally supported at their centres by a 1- by 4-inch strip at right angles to the collar ties.

CMHC Building Standards have practically the same requirements with one important exception. Although CMHC Building Standards do not state specifically that collar ties may be acceptable as intermediate rafter supports this is the interpretation which is commonly acknowledged in practice. Thus, since an unsupported length of 2- by 4-inch eastern spruce rafters, 16 inches on centre, is permitted to span a 6-foot-11-inch horizontal distance, it is considered acceptable to CMHC to use 2- by 4-inch rafters provided that the collar tie divides the rafter so that the horizontal projections of each of the two rafter segments do not exceed 6 feet 11 inches. For a horizonal span a 12-foot 2- by 4-inch rafter may be used if the collar tie is placed near the mid-span of the rafter. The Standards also state that a minimum of 2- by 4-inch collar ties may be used when the collar ties support a ceiling, or in unfinished attics 1- by 5-inch collar ties may be used provided the collar ties are laterally supported at their centres by a continuous 1- by 4-inch strip. The Standards do not state, however, in what cases collar ties must be used.

In both the National Building Code and CMHC Building Standards the position of the collar ties is limited to the middle third of the rafters.

In order that a series of tests should represent a fair cross-section of current building practices, it was decided that both interpretations of the function of the collar tie be acknowledged. It was therefore decided that both 2- by 4-inch and 2- by 8-inch rafters be tested.

the rafters and joists or the pitch of the roof, the type of wood used and the accepted snow load, all of which seriously affect the number of nails required.

In the tests conducted here all joists were lapped at the splice over the bearing partition and fastened with three $3\frac{1}{2}$-inch nails through the two joists as well as two $3\frac{1}{2}$-inch toe-nails from the joists to the top plate of the bearing partition as shown in Fig. 39.

In Type I construction the rafters and joists were tied together with three $3\frac{1}{2}$-inch nails through both members as well as three $3\frac{1}{2}$-inch toe-nails from the rafter to the top plate and two $3\frac{1}{2}$-inch toe-nails from the joists to the top plate. This was observed to be seldom done in practice but since this nailing schedule satisfies all the requirements of the codes, this scheme of nailing was decided upon (Fig. 36).

In Types II and III constructions the nailing requirements for connecting the rafter plate to the joists and headers as well as connecting the headers to the joists were not covered by the nailing schedules. Field observations of these types of construction showed that practices varied considerably. It was decided, therefore, to arrive at nailing schemes which would represent the best practices in these constructions. The nailing schemes are shown in Figs. 37 and 38.

It was also noted in field observations that collar ties were fastened to the rafters by either two or three $3\frac{1}{2}$-inch nails. In the test structures three $3\frac{1}{2}$-inch nails were used.

(2) Testing of Test Structures

a. Test Equipment

Loads were applied to the rafters by means of eight hydraulic tension jacks anchored to the floor and connected to a hydraulic pump by a common oil line. The pump was fitted with a pressure gauge to measure the oil pressure in the system. The force exerted by the jacks was calibrated against the pressure gauge on the pump by placing the jacks in a hydraulic testing machine and recording the pressure on the pump pressure gauge with the corresponding readings on the testing machine. Corresponding readings were taken for a range of loads so that the necessary calibration curves could be obtained.

The jacks were placed midway between the pairs of test specimens and positioned so that the applied load would simulate the same bending moments in the rafters and the same end reactions as for a uniform load.

The forces exerted by the jacks were transmitted to the rafters by means of $\frac{3}{4}$-inch steel rods from the jacks to steel channel sections lying across the rafters as shown in Figs. 16 and 40.

The ceiling load was applied directly to the ceiling joists by means of lead-filled bags placed in such a way as to simulate a uniform ceiling load.

Figure 41 shows the positions and magnitudes of the applied loads.

The test structures were supported on concrete blocks, as shown in Fig. 16, with braces supporting the end blocks against movement (Fig. 13).

Short peices of $\frac{3}{4}$-inch diameter steel bars sandwiched between two steel plates were used as supports under one end of the structure in those tests simulating the condition of no lateral resistance to thrust by the exterior walls (Fig. 13).

For those tests in which the walls are assumed not to move, the double 2- by 4-inch bearing plates at each end of the structure were securely bolted to the floor beam to restrain movement (Figs. 6 and 8).

b. Instrumentation

Dial gauges were placed under each end of the supporting plates to measure the settlement of the structure under the applied loads. Dial gauges were also placed at the ends of the rafters to measure any horizontal movement of the rafters under load (Fig. 13).

Dial gauges were also placed on the rafters at the ends of the collar ties to measure the relative movement between rafters and collar ties to determine when the collar tie acts in tension and when it acts in compression. Only one collar tie was instrumented in each test (Fig. 14).

Vertical deflections of the rafters were measured in the following manner. Fine piano wire was strung from the peak of the assembly and at 4 and 6 feet on either side of the peak (measured horizontally). The wires were fastened to the sheathing midway between the two sets of rafters and passed over pulleys fastened to the floor directly below. From the pulleys below the rafters the wires were led by other pulleys to a recording board. At the ends of the wires were fastened 1-pound weights, the vertical position of which could be noted on the recording board (Fig. 15). The average vertical movement of the two sets of rafters registered the same movement on the recording board.

c. Application of Load and Testing Procedure

At the beginning of the test a 10-pound-per-square-foot ceiling load was applied to the ceiling joists and allowed to remain for the duration of the test.

The weight of the rafter loading equipment hanging from the rafters, plus the weight of the applied sheathing, was practically equal to the weight of roofing and sheathing which would be applied in practice so that the dead weight of the roofing did not have to be allowed for in the hydraulic load application.

The hydraulic loads were applied in increments corresponding to 10-pound-per-square-foot uniform snow loads. After each increment of loading was reached, the load was held for 5 minutes and all dial gauge readings and rafter deflections recorded. The loads were increased to a total of 40 pounds per square foot at which point the loads were reduced to zero. Loading was again increased in 10-pound-per-square-foot increments until failure occurred.

Three tests on both roller supports and fixed end supports were not carried out on all types of conventional construction. In Types II and III constructions some failures were such that failure was independent of the type of supports used. In these cases the results of the failures were included in both the average failure with roller supports and fixed end supports, even though the actual tests in some cases were on roller supports only (Table II).

Photographs were taken of the failures of most of the structures tested and the moisture contents of the structures recorded by means of a resistance-type moisture meter.

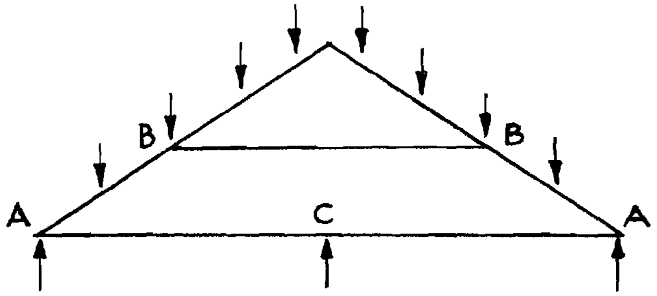
d. Recording of Results

All dial gauge readings were taken to the nearest .001 inch and rafter deflection readings to the nearest .01 inch. The readings of the dial gauges placed under the double 2- by 4-inch plates at either end of the test structures were deducted proportionately from the rafter deflection readings to determine the true vertical rafter deflections. From these corrected rafter deflections the deflections normal to the rafters were calculated for the 40-pound-per-square-foot loads (Table II).

(3) Results of Tests

a. Deflection Characteristics

As can be seen in Table VI, there is a considerable variation in the deflection ratios of similar size rafters in both the 2- by 4-inch and 2- by 8-inch rafter constructions, and the deflection ratio varied with the type of construction and with the type of end support.



An explanation of the various rafter deflection ratios is as follows: If the ends of the rafters (A) are assumed not to move under load, the deflection of the rafters at B will cause the collar tie BB to act in compression. On the other hand, if we consider that the rafter does not deflect under load and the rafter ends can move outward at A, then the collar tie must act in tension. In practice, both actions occur simultaneously. The stiffer the rafter, the less will be the tendency to compress the collar tie, and the more rigid the joints are at A and C the less will be the tendency to stretch the collar tie. The net effect of the two actions may cause either a tension or a compression of the collar tie depending on the relative movements produced by the loading.

The greater the compressive force in the collar tie, the greater will be the action to resist the deflection of the rafters under applied load. Conversely, the greater the tension in the collar tie the greater will be the tendency of the collar tie to increase the rafter deflection. Therefore, it follows, that the deflection ratio of the rafters will depend upon the degree of compression or tension in the collar tie for any given size and span of rafter.

This explanation will show why the rafter deflection ratios were greater for tests on roller supports than for tests on fixed end supports, and would explain why the rafter deflection ratios of the structures with the strongest heel joint detail (Type I) were less than those for the weakest type of heel joint (Type III).

As a generalization based on observations of dial gauges at the ends of the collar tie, it may be said that when 2- by 8-inch rafters were used the collar ties acted in compression at the lower loads, and at the higher loads the stress usually reversed and the collar ties acted in tension. With the 2- by 4-inch rafters, the collar ties usually act in compression throughout the test due to the flexibility of the smaller rafters. The loads causing a reversal of stress in the collar tie may be seen in Table II.

b. Load-carrying Capacities

The ultimate short-term load-carrying capacity for Type I construction built with 2- by 4-inch rafters was found to be 56 pounds per square foot for roller supports and 72 pounds per square foot for fixed supports. For Type II construction the average failure loads were between 40 and 43 pounds per square foot and for Type III construction only 18 pounds per square foot (Table VI).

The test results for the 2- by 8-inch rafter constructions showed these structures to possess considerably greater strength than 2- by 4-inch rafter constructions (Table VI). The failure load for Type I construction was 89 pounds per square foot when the structures were tested on roller supports and 125 pounds per square foot if the ends of the structures were fixed. For Type II construction the average ultimate load was from 82 to 84 pounds per square foot and only 46 pounds per square foot for Type III construction.

It is important to keep in mind that collar ties were also used with all 2- by 8-inch rafter construction and were placed at mid-span of the rafters. If no collar ties were used, which is allowed under the National Building Code for Type I construction and possibly for Types II and III constructions (depending on how the Building Code is interpreted), then the structures would probably be weaker than those 2- by 8-inch structures tested. Again if the collar ties were placed at the upper third of the rafters which is the limiting position, their effect on preventing the rafter ends from spreading would be only about two-thirds as great than if the collar ties were placed at mid-span of the rafters, and the over-all strength of the structures would in all probability be less.

A comparison of the results of 2- by 4-inch and 2- by 8-inch rafter constructions reveals some interesting facts. As can be seen from the average results in Table VI, the failure loads of the 2- by 8-inch rafter constructions are substantially higher than the failure loads for the 2- by 4-inch rafter constructions. In many cases the types of failures were identical for both 2- by 4-inch and 2- by 8-inch rafter constructions and occurred at the nailed joints, at the joist lap or heel joint. The nailing for both 2- by 4-inch and 2- by 8-inch rafters, however, was identical. It must be assumed then that the outward thrust exerted by the 2- by 4-inch rafters was considerably more than the thrust for the 2- by 8-inch rafters with similar loads. One possible explanation is as follows: The outward thrust of a rafter supporting W pounds per lineal foot and spanning L feet on a 5/12 slope is $\frac{1}{2} \times \frac{12}{5}$ WL if the action of the collar tie is ignored. The outward thrust is $\frac{3}{4} \times \frac{12}{5}$ WL if the collar tie is assumed to be an adequately designed pin connected member. The 2- by 8-inch rafters did not deflect sufficiently for the collar ties to contribute very much as a compression member, and in many cases the collar ties actually acted in tension. In this instance the rafters probably act much like simple beams and the outward thrust approaches a value less than $\frac{1}{2} \times \frac{12}{5}$ WL pounds. There is considerably more flexibility in the 2- by 4-inch rafters and the compressive force exerted by the collar ties is substantially greater than if 2- by 8-inch rafters were used. In this case the horizontal rafter thrust will approach $\frac{3}{4} \times \frac{12}{5}$ WL pounds.

If, on the basis of the assumption that the horizontal thrust of 2- by 8-inch rafters equals $\frac{1}{2}$ x 5/12 WL and 2- by 4-inch rafters equals $\frac{3}{4}$ x 5/12 WL, the failure loads of the nailed joints are calculated for each type of construction, the results show that the ultimate strengths of the nailed joints are similar regardless of rafter size. The dead weight of the roof structure must naturally be taken into account in these calculations, and in Types II and III constructions the frictional resistance in the heel joint must be allowed for. This would seem to indicate that the explanation for the difference in the horizontal thrust with the different size of rafters is substantially correct.

B. TRUSSED ROOF CONSTRUCTION

(1) Description of Test Structures

a. General Description

All tests were carried out on 24-foot span trusses using a roof slope of 5/12. Yard-run eastern spruce was used on all assemblies for the structural members and $\frac{1}{2}$-inch Douglas fir plywood used for gusset plates.

The trusses were tested in pairs placed 16 inches on centre and sheathed with 1- by 6-inch sheathing. As in the tests on conventional construction, the tests were conducted using both roller supports and fixed end supports. Three tests were conducted on most assemblies to obtain an average value for results.

The connections used in fastening the test trusses to the supporting 2- by 4-inch plates were purely arbitrary and consisted of fastening by toe-nailing rather than by using patented connectors. The number and size of nails were arbitrarily adjusted to suit the type of heel joint encountered. These nailing schemes are shown in Figs. 42, 43, 44, 45 and 46.

The types of trusses tested are shown in Figs. 42, 43, 44, 45 and 46 and will be referred to as Types A, B, C, D, E, and F.

b. Type A Truss

Type A truss is similar to the University of Illinois Small Homes Council split ring W-truss with some modifications. Two-by-four's were used rather than 1 x 4's for the diagonals and $3\frac{1}{2}$-inch nails were used throughout. Whereas 1100 p.s.i. stress grade Douglas fir was suggested by the Small Homes Council for the structural members, yard run 2- by 4-inch eastern spruce was used in these tests (Fig. 42).

c. Type B Truss

Type B truss is similar to the nailed truss design developed by E.G. Stern of the Virginia Polytechnic Institute with some modifications also. Common nails were used throughout rather than helically threaded nails as suggested in Mr. Stern's design. It was also decided to use $\frac{1}{2}$-inch plywood gusset plates and splice plates in the test structures rather than 1- by 6-inch dressed boards. Although Mr. Stern's design called for the use of 1450 p.s.i. graded lumber, 2- by 4-inch eastern spruce was used. Some revisions were also made in the nailed joints (Fig. 43).

d. Type C Truss

Type C truss was modelled after the University of Illinois glued truss design for a 4/12 slope. The slope was revised to 5/12 and the gusset plate sizes at the heel joints changed accordingly. The positions of the diagonals were changed to correspond to the positions of the diagonals in Types A and B trusses, that is, the short diagonals connected to the centres of the top chord to the third points of the lower chord (Fig. 44). Here, as in the other trusses, eastern spruce was used instead of 1450 p.s.i. graded Douglas fir as called for in the plans.

e. Types D, E and F Trusses

Types D, E and F trusses were basically similar to each other with minor differences in the heel joints. These trusses were developed as the result of tests on other roof frames with the hope of obtaining a truss which would be both adequate and economical. More will be said about these trusses later (Figs. 45 and 46).

(2) Testing of Test Structures

a. Test Equipment

Loads were applied in much the same way as for tests on conventional constructions. The positions and magnitude of the top chord loads were the same as shown in Fig. 41. The 10-pound-per-square-foot ceiling load was applied by means of lead-filled bags laid directly on the lower chords of the truss at the end quarter points of the three bottom panels of each truss.

The test structures were supported on concrete blocks under each end of the trusses and the blocks were braced against movement in the same manner as for tests on conventional construction.

b. Instrumentation

Dial gauges were placed at each end of the double 2- by 4-inch plates at the ends of the trusses to measure the settlement of the trusses under the applied loads. Dial gauges were also placed against the outside ends of each of the double plates to measure any horizontal movement of these plates (Figs. 33 and 34).

Deflection measurements of the trusses were taken at all panel points and at the mid-spans of the lower chords of the trusses by means of wires and pulleys in much the same way as the rafter deflections were taken in tests on conventional constructions. These measurements were taken midway between the pairs of trusses so that the average deflections of the two trusses being tested were measured.

c. Application of Load and Testing Procedure

All dial gauge readings and truss deflections were noted before the ceiling loads and hydraulic loads were applied. The ceiling load was then applied and allowed to remain for the duration of the test. Five minutes after the ceiling load was applied all the readings were again noted.

The hydraulic loads were then applied in increments simulating 10-pound - per-square-foot snow loadings. Five minutes after each loading increment the readings were again taken. The loading was continued until a total of 40-pound - per-square-foot load was applied, at which point the loads were reduced to zero. Loading was again increased in increments of 10 pounds per square foot until failure occurred.

As for conventional construction tests, photographs were taken of the failures and the moisture contents of the trusses were recorded.

d. Recording of Results

All dial gauge readings were recorded to the nearest .001 inch and truss deflections to the nearest .01 inch. The dial gauge readings under the end bearing plates were deducted proportionately from the truss deflections to obtain the true vertical deflections.

Although the trusses were tested at 16 inches on centre the results were adjusted to apply to trusses spaced at 24 inches on centre as well.

(3) Results of Tests

a. Type A Trusses

It may be seen in Table V that the average failure loads for this type of truss spaced 24 inches on centre were 65 pounds per square foot for trusses tested on roller supports and 68 pounds per square foot for trusses on fixed end supports. The corresponding average maximum deflections of the lower chords were .86 inch and .77 inch respectively for a 40-pound - per-square-foot snow load. The maximum deflections divided by the total span is 1/335 for trusses on roller supports and 1/370 for trusses on fixed end supports. The commonly accepted limiting ratio deflection to prevent plaster cracking is 1/360. The results show that this type of truss is very close to this value. However, for long-term loading of 40 pounds per square foot the deflections would in all probability be greater and the deflection ratio might substantially exceed the 1/360 limit.

The most common cause of failure in these trusses was in the lower chord near or at the split rings, with other failures being due to the top chord breaking in bending or failure in the nailed joints of the long diagonal (Figs. 17, 18, 19, 20 and 21).

Due to the nature of the top chords of these trusses in which the lower halves of the top chords were offset from the upper halves, the top chords all had a very definite tendency to bend laterally under loading. The design of these trusses was such that this bending was in the same direction regardless of how the trusses were orientated. That is, it was not possible to counteract this bending effect by turning one truss at 180° to the other. The overall effect in a roof, therefore, would be that the entire roof would tend to warp under loading. Just how much warping any roof system could withstand without showing any visible signs is not known. This tendency, however, might be overcome if the design of the trusses was altered for half of the trusses so that the upper chords of each truss tended to bend in opposite directions to each adjacent truss.

b. Type B Trusses

This type was the second strongest and the second most rigid type tested. Failures in most cases were due to nail joint failures at either end of the long diagonals while other failures were due to joint failure in the short diagonal or to tension failure in the bottom chord (Figs. 22, 23, 24, 25, 26 and 27). In two cases (Tests Nos. 36 and 40), the trusses failed when the top chord bowed laterally under the higher loads.

The failure load for this type of truss at 24 inches on centre averaged 99 pounds per square foot with roller end supports and 110 pounds per square foot with fixed end supports.

The deflection ratio for the bottom chord of these trusses at 40-pound-per-square-foot snow load and spaced 24 inches on centre is 1/600 for roller supported trusses and 1/650 for trusses on fixed end supports. This is considerably less than the 1/360 limiting ratio. The fact that this was under short-term loading, however, must be taken into consideration.

c. Type C Trusses

This type of truss was the strongest and most rigid of all types tested. The average failure load for trusses at 24 inches on centre was 106 pounds per square foot on roller supports and 119 pounds per square foot on fixed end supports.

The average deflection ratio of the bottom chord for this type of truss at 40-pound-per-square-foot snow loading spaced 2 feet on centre is 1/1440 for trusses on roller supports and 1/1310 for trusses on fixed end supports. Both these values are considerably less than the 1/360 limit even when considering the fact that the trusses were loaded by short-term loading.

The most common type of failure in this truss was at the glue joint of the gusset plate at the peak with the only other true failures occurring at the glue joint of the gusset plate connecting the bottom chord to the diagonals. The principal cause of failure seemed to be due to the fact that it was difficult to get a good glued joint at the peak where the members were connected by a glue bond to a rigid gusset plate on one side and a 1- by 4-inch strip of wood on the other. Unless all of the members were of the same thickness then a good glue bond was not possible. Also if the members were cupped due to drying shrinkage, which was often observed during the course of the tests, the glued bond was effective only over part of the required bonding area. When the long diagonals were slightly warped it was very difficult to get a good bond at both peak gusset plate and the plate connecting the long diagonal to the bottom chord. It should be added here for the sake of interest that casein glue was used and was applied by a stiff brush to one surface of each glued joint.

d. Types D, E and F Trusses

All of these types were quite similar in design. The only difference in each was at the heel joint. The difference between Type D and E trusses was that in Type E truss $\frac{1}{2}$-inch common washers $1\frac{1}{4}$ inch in diameter were used at the heel joint and in Type D truss 2- by 2- by 1/8-inch washers with $\frac{1}{2}$-inch diameter holes were used. (Fig. 45). Type F differed from Type D only in the small notch cut in the joist of Type F which was added with the hope that its addition would stiffen the truss (Fig. 46). It was not effective, however, and it was decided that further tests would be done on Type D only. Therefore, the discussion will be limited to Type D. Only four tests were conducted in Type D - two on roller supports and two on fixed end supports.

This type of truss was the second weakest of those types tested having an average failure load, with the trusses 2 feet on centre, of 88 pounds per square foot on roller supports and 75 pounds per square foot on fixed end supports.

The deflection ratio of the bottom chord was 1/480 for roller supports and 1/370 for fixed end supports with a 40-pound-per-square-foot snow load. Although these values are less than the limiting 1/360 ratio it should be stated that this type of truss design showed a considerable tendency to creep under the higher loads. This characteristic was quite marked with this type of truss as compared with the other types tested. This would lead to the possibility that this type of truss might deflect considerably more under a sustained 40-pound-per-square-foot load and have an ultimate deflection considerably greater than 1/480 and 1/370. This possibility should be investigated by long-term loading tests.

Another feature of this truss which should be examined closely is the connection at the heel joint. If the members are bolted together when the wood is at a high moisture content and then allowed to dry, it is probable that the joint may loosen. Just how much effect this would have on the strength and rigidity of the trusses is not known and should also be investigated.

C. COST STUDY

It was decided that a comparative cost study of the various types of truss constructions would be very useful in evaluating the different types of trusses against each other and with conventional construction. A rough time study was therefore carried out to obtain sufficient data to make such comparisons.

In the cost study the wage rates and materials cost used were the prevailing rates in the Ottawa district. From the information supplied, however, it should be possible to convert the various costs to apply to any district. In the time study, the time was observed to the nearest minute and is a rough study only. In connection with the time study it should be mentioned that two men assembled and cut the lumber and plates and full use was made of a power hand saw and a bench saw. Only one man did the nailing in all cases and the helper, who was not a carpenter, merely helped to put the pieces of the trusses in place and generally assisted the carpenter. The time study, however, does not include the time or material used in making the assembly jig or in cutting the pattern pieces. A summary of the results of the time study is shown in Table III.

In order to compare the cost of trusses to the cost of conventional construction on an approximate basis, the cost of material that was used in conventional roof framing was calculated. The summary is shown in Table IV. Labour costs were not included since no time study was made on conventional construction.

The cost does not include other possible economical advantages which may be obtained by the use of trusses.

These include more freedom of design, unobstructed work area, elimination of bearing partitions, better opportunity to plaster the ceiling and outside walls, as well as laying the finished floor without partitions obstructing the work, and the speed with which a building may be enclosed. It should also be added that under mass production methods, trusses could be made faster and cheaper than they were made in these tests since there was little opportunity to perfect fabrication techniques and to make the most efficient use of both men employed in building the trusses.

D. GENERAL DISCUSSION OF RESULTS

The failure loads for the trusses were equal to or greater than the strongest type of conventional construction with equal spacings for each.

With the failure loads of trusses calculated for 24-inch — on-centre spacing, the results compare quite favourably with conventional construction spaced at 16 inches on centre. Types B and C trusses at 24 inches on centre have failure loads of the same order as the strongest type of conventional construction at 16 inches on centre. Although Type A truss had failure loads considerably greater than Type III construction with 2- by 4-inch and 2- by 8-inch rafters, and Type II construction with 2- by 4-inch rafters, it was not as strong as Types I or II with 2- by 8-inch rafters and had approximately the same strength as Type I with 2- by 4-inch rafters. Type D truss was also stronger than Type III construction with 2- by 8-inch and 2- by 4-inch rafters and Types I and II constructions with 2- by 4-inch rafters. Although it was not as strong as Type I with 2- by 8-inch rafters it had approximately the same strength as Type II with 2- by 8-inch rafters.

The cost of trusses also compares quite favourably with the cost of conventional construction even when the labour cost for conventional construction is not allowed for. With this cost of labour for conventional construction included the trusses would appear in an even more favourable light. The cost of trusses at 24 inches on centre is impressive when compared with conventional construction using 2- by 8-inch rafters at 16 inches on centre with the average saving being in the neighbourhood of 30 per cent.

E. CONCLUSIONS

The most promising trusses appeared to be Types B and C from the standpoint of deflection characteristics and ultimate strengths. Type A truss was the least rigid and weakest of all the trusses tested as well as being the most expensive to build. Type D truss, although somewhat stronger and more rigid than Type A, was not as rigid or strong as Types B and C. Type D was far cheaper than any other by a considerable margin (approximately 15 per cent) and because of this it would appear to justify further development to attempt to improve the strength and deflection characteristics.

It is difficult to say just how strong a truss should be in order to be adequate. One of the principal reasons why tests on conventional constructions were conducted was to attempt to establish a criterion by which to evaluate trusses. The wide variation in the strength of various conventional constructions made such a comparison for truss constructions difficult unless a minimum failure load is arbitrarily chosen from the range of failure loads shown for the various types of conventional constructions. For the time being this would probably be the most logical approach.

The second approach, which would be the use of the National Building Code snow loads with accepted design principles, would rule out the use of most if not all types of conventional roof framing as well as the use of all light-weight trusses.

The results of the tests carried out to date, therefore, suggest that more information is required on the behaviour of roof structures under simulated snow loads as well as information on snow loads that actually occur on house roofs.

SUMMARY OF TRUSSED-RAFTER TESTS

TEST NO.	TYPE OF TRUSS	TYPE OF END SUPPORTS	MOISTURE CONTENT AT AREA OF FAILURE (%)	DEFLECTIONS AT 40 psf SNOW LOAD								TYPE OF FAILURE	ULTIMATE LOAD	
				Trusses 16" o.c.				Trusses 24" o.c. *					Trusses 16"o.c. (psf)	Trusses 24"o.c. (psf) *
				L_1 (in.)	Mid Span (in.)	L_2 (in.)	U_2 (in.)	L_1 (in.)	Mid Span (in.)	L_2 (in.)	U_2 (in.)			
32	A	Rollers	10	0.43	0.47	0.42	0.43	0.71	0.78	0.71	0.69	Fig. 17	110	68
33	A	Rollers	11	0.39	0.48	0.42	0.40	0.66	0.82	0.75	0.70	Fig. 18	100	62
52	A	Rollers	7	0.47	0.51	0.49	0.29	0.88	0.99	0.92	0.62	Fig. 19	100	62
34	A	Fixed	7	0.26	0.36	0.28	0.30	0.45	0.63	0.54	0.53	Fig. 20	114	71
35	A	Fixed	13	0.34	0.36	0.34	0.29	0.61	0.66	0.61	0.52	Fig. 18	114	71
51	A	Fixed	9	0.41	0.50	0.45	0.36	0.81	1.01	0.90	0.71	Fig. 21	100	62
36	B	Rollers	10	0.28	0.34	0.26	0.20	0.44	0.51	0.42	0.34	Fig. 22	140	88
37	B	Rollers	12	0.24	0.29	0.27	0.22	0.40	0.46	0.43	0.32	Short Diagonal Joint Failure	180	115
38	B	Rollers	12	0.27	0.31	0.28	0.24	0.44	0.48	0.45	0.35	Fig. 23	150	95
39	B	Fixed	11	0.22	0.30	0.24	0.19	0.35	0.48	0.39	0.30	Fig. 24	170	108
40	B	Fixed	8.5	0.26	0.28	0.24	0.20	0.42	0.42	0.37	0.29	Fig. 25	170	108
53	B	Fixed	11	0.19	0.23	0.18	0.15	0.35	0.41	0.32	0.25	Figs. 26, 27	180	115
41	C	Rollers	7	0.11	0.13	0.13	0.11	0.17	0.20	0.21	0.18	Fig. 28	180	115
42	C	Rollers	8	0.09	0.14	0.12	0.08	0.16	0.21	0.19	0.13	Fig. 29	130	82
55	C	Rollers	7.5	0.10	0.14	0.10	0.06	0.16	0.19	0.16	0.10	Fig. 28	190	122
43	C	Fixed	12	0.10	0.14	0.11	0.10	0.16	0.21	0.19	0.14	Fig. 28	212	136
44	C	Fixed	12	0.14	0.18	0.14	0.11	0.20	0.24	0.20	0.17	Fig. 28	190	122
54	C	Fixed	8	0.09	0.13	0.11	0.08	0.16	0.20	0.19	0.14	Fig. 30	155	98
46	D	Rollers	9 - 12	0.30	0.37	0.29	0.27	0.52	0.64	0.51	0.46	Fig. 31	130	82
48	D	Rollers	7	0.27	0.28	0.24	0.20	0.48	0.57	0.44	0.35	Fig. 31	150	95
49	D	Fixed	14	0.29	0.40	0.28	0.22	0.55	0.78	0.56	0.45	Fig. 21	130	82
50	D	Fixed	8	0.39	0.44	0.35	0.29	0.66	0.78	0.63	0.51	Fig. 32	110	68
45	E	Rollers	12	0.37	0.46	0.33	0.31	0.60	0.73	0.54	0.53	Fig. 33	130	82
47	F	Rollers	14	0.32	0.48	0.36	0.29	0.63	0.83	0.62	0.53	Fig. 34	114	71

TABLE I

* These values were calculated from data obtained from tests on structures at 16 inches on centres

SUMMARY OF RAFTER-JOIST CONSTRUCTION TESTS

TEST NO.	TYPE OF CONST-RUCTION	LOAD CAUSING REVERSAL OF STRESS IN COLLAR TIE	RAFTER SIZE (in.)	TYPE OF END SUPPORTS	MOISTURE CONTENT AT AREA OF FAILURE	DEFLECTIONS AT 40 psf SNOW LOAD					TYPE OF FAILURE	ULTIMATE SNOW LOAD (psf)
						Deflections Perpendicular to Rafters				Ridge Deflection		
						6 ft. Left of Ridge	4 ft. Left of Ridge	4 ft. Right of Ridge	6 ft. Right of Ridge			
18	I	80-90	2x8	Rollers	8.5%	0.21"	0.18"	0.22"	0.23"	0.45"	Fig. 3	100
17	I	50-60	2x8	Rollers	9.5%	0.27"	0.23"	0.25"	0.34"	0.55"	Fig. 5	80
19	I	50-60	2x8	Rollers	8.5%	0.32"	0.26"	0.23"	0.27"	0.60"	Fig. 3	86
20	I	90-100	2x8	Fixed	10%	0.15"	0.21"	0.14"	0.15"	0.26"	Fig. 6	117
21	I	no change	2x8	Fixed	9%	0.14"	0.10"	0.14"	0.17"	0.19"	Fig. 7	120
22	I	no change	2x8	Fixed	8%	0.16"	0.14"	0.14"	0.16"	0.20"	Fig. 8	139
23	I	no change	2x4	Rollers	8%	0.69"	0.60"	0.48"	0.49"	0.86"	Fig. 9	57
24	I	no change	2x4	Rollers	8.5%	0.65"	0.53"	0.82"	0.90"	0.93"	Fig. 3	61
25	I	no change	2x4	Rollers	8-11%	0.73"	0.65"	0.62"	0.67"	1.01"	Fig. 3	49
26	I	no change	2x4	Fixed	17%	0.53"	0.53"	0.60"	0.67"	0.31"	Fig. 10	60
27	I	no change	2x4	Fixed	12%	0.55"	0.52"	0.68"	0.70"	0.23"	Fig. 11	77
28	I	no change	2x4	Fixed	12%	0.43"	0.41"	0.45"	0.47"	0.23"	Fig. 12	80
8	II	30-40	2x8	Rollers	9%	0.21"	0.18"	0.23"	0.25"	0.54"	Fig. 3	94
9	II	30-40	2x8	Rollers	11%	0.30"	0.27"	0.21"	0.22"	0.67"	Fig. 2	90
10	II	40-50	2x8	Rollers	9%	0.26"	0.22"	0.27"	0.30"	0.70"	Fig. 2	69
11	II	50-60	2x8	Fixed	9%	0.21"	0.18"	0.22"	0.24"	0.52"	Fig. 2	86
13	II	no change	2x4	Rollers	8%	0.95"	0.86"	0.88"	0.88"	2.01"	Fig. 2	44
14	II	no change	2x4	Rollers	9%	--	--	--	--	--	Fig. 4	40
15	II	no change	2x4	Rollers	14%	1.09"	0.93"	1.01"	1.10"	2.02"	Fig. 3	46
16	II	no change	2x4	Fixed	12.5%	--	--	--	--	--	Fig. 2	37
6	III	--	2x8	Rollers	9%	0.32"	0.30"	0.30"	0.35"	1.10"	Fig. 1	49
7	III	20-30	2x8	Rollers	8%	0.36"	0.30"	0.42"	0.46"	1.06"	Fig. 1	45
12	III	10-20	2x8	Rollers	9.5%	0.63"	0.50"	0.53"	0.66"	1.59"	Fig. 1	45
29	III	no change	2x4	Rollers	7%	--	--	--	--	--	Fig. 1	15
30	III	no change	2x4	Rollers	7%	--	--	--	--	--	Fig. 1	20
31	III	10-20	2x4	Rollers	8%	--	--	--	--	--	Fig. 1	20

TABLE II

COST DATA FOR VARIOUS TRUSSES

TYPE OF TRUSS	MATERIALS									LABOUR						TOTAL COST PER TRUSS
	QUANTITIES OF MATERIAL								MATERIAL COST *	LABOUR TIME			HOURLY WAGE		LABOUR COST	
	Bolts	Split Rings	1"x4" (fbm)	1"x6" (fbm)	2"x4" (fbm)	1/2 in. plywood (sq.ft.)	Glue (lb)	Nails (lb)		Cutting Plywood (hours)	Cutting Others (hours)	Assembly (hours)	Carpenter (per hr.)	Helper (per hr.)		
A	2 ($\frac{1}{2}$x4) 1 ($\frac{1}{2}$x8)	3 (2$\frac{1}{2}$")	-	-	57	-	-	2.0 (3$\frac{1}{2}$")	$8.28	-	0.416	0.416	$1.82	$1.00	$2.35	$10.63
B	-	-	-	11	39	7.33	-	1.47 (3") 0.15 (2$\frac{1}{2}$")	$7.74	0.069	0.140	0.516	$1.82	$1.00	$2.04	$ 9.78
C	-	-	2	-	47	8.60	0.40	0.461 (2$\frac{1}{2}$") 0.370 (1$\frac{1}{2}$")	$8.18	0.081	0.085	0.478	$1.82	$1.00	$1.82	$10.00
D	2 ($\frac{1}{2}$x8)	-	2	7	39	1.33	-	0.39 (3") 0.635 (2$\frac{1}{2}$")	$6.68	0.012	0.180	0.390	$1.82	$1.00	$1.64	$ 8.32

* The following unit prices were used in computing material costs in each truss:

Bolts - ($\frac{1}{2}$x4)......... $ .113
($\frac{1}{2}$x8)......... .216

Split Rings........... .20

Lumber - (1x4)........ .115 per fbm
(1x6)........ .120 per fbm
(2x4)........ .123 per fbm
($\frac{1}{2}$" plywood). .196 per sq.ft.

Glue............ $1.00 per lb.

Nails -
1$\frac{1}{2}$".......... ..118 per lb.
2$\frac{1}{2}$".......... .113 per lb.
3"........... .110 per lb.
3$\frac{1}{2}$.......... .109 per lb.

TABLE III

COST DATA FOR VARIOUS TYPES OF CONVENTIONAL CONSTRUCTION

TYPE OF CONSTRUCTION	RAFTER SIZE (in.)	QUANTITIES OF MATERIAL				UNIT PRICE OF MATERIAL				COST OF MATERIAL	
		2"x4" (fbm)	2"x6" (fbm)	2"x8" (fbm)	1"x6" (fbm)	2"x4"	2"x6"	2"x8"	1"x6"	Per 16" of Roof	Per 24" of Roof
I	2x8	8	26	37	1	$.123	$.125	$.132	$.12	$9.23	$13.84
I	2x4	27	26	--	1	$.123	$.125	--	$.12	$6.70	$10.05
II	2x8	10	28	37	1	$.123	$.125	$.132	$.12	$9.73	$14.60
II	2x4	29	28	--	1	$.123	$.125	--	$.12	$7.19	$10.78
III	2x8	10	29	37	1	$.123	$.125	$.132	$.12	$9.85	$14.77
III	2x4	29	29	--	1	$.123	$.125	--	$.12	$7.31	$10.96

* Costs do not include cost of fabrication or nails.

TABLE IV

CONDENSED SUMMARY OF TRUSSED-RAFTERS TESTS

TYPE OF TRUSS	TYPE OF END SUPPORTS	DEFLECTIONS AT 40 psf SNOW LOAD						ULTIMATE SNOW LOAD		COST PER TRUSS
		Trusses at 16" o.c.			Trusses at 24" o.c.			Trusses at 16" o.c. (psf)	Trusses at 24" o.c. (psf)	
		Mid Span Lower Chord	Deflection / Span	Ridge	Mid Span Lower Chord	Deflection / Span	Ridge			
A	Rollers	0.49"	1/590	0.37"	0.86"	1/335	0.67"	103	64	$10.63
A	Fixed	0.41"	1/700	0.32"	0.77"	1/370	0.59"	109	68	"
B	Rollers	0.31"	1/930	0.22"	0.48"	1/600	0.34"	157	99	$ 9.78
B	Fixed	0.27"	1/1070	0.18"	0.44"	1/650	0.28"	173	110	"
C	Rollers	0.14"	1/2060	0.08"	0.20"	1/1440	0.14"	167	106	$10.00
C	Fixed	0.15"	1/1920	0.10"	0.22"	1/1310	0.15"	186	119	"
D	Rollers	0.33"	1/870	0.23"	0.60"	1/480	0.40"	140	88	$ 8.32
D	Fixed	0.42"	1/690	0.25"	0.78"	1/370	0.48"	120	75	"

TABLE V

CONDENSED SUMMARY OF RAFTER-JOIST TESTS

TYPE OF CONSTRUCTION	RAFTER SIZE (in.)	TYPE OF END SUPPORTS	AVERAGE DEFLECTIONS FOR 40 psf SNOW LOAD			ULTIMATE SNOW LOAD (psf)	COST OF WOOD ONLY	
			Mid Span of Rafter	Deflection at Mid Span of Rafter ÷ Rafter Length	Ridge		Per 16" of Roof	Per 24" of Roof
I	2x8	Rollers	0.27"	1/580	0.53"	89	$9.23	$13.84
I	2x8	Fixed	0.16"	1/970	0.22"	125	"	"
I	2x4	Rollers	0.69"	1/230	0.93"	56	$6.70	$10.05
I	2x4	Fixed	0.56"	1/280	0.26"	72	"	"
II	2x8	Rollers	0.26"	1/600	0.64"	84	$9.73	$14.60
II	2x8	Fixed	0.23"	1/680	0.52"	82	"	"
II	2x4	Rollers	1.01"	1/150	2.01"	43	$7.19	$10.78
II	2x4	Fixed	--	--	--	40	"	"
III	2x8	Rollers	0.46"	1/340	1.25"	46	$9.85	$14.77
III	2x8	Fixed	--	--	--	46	"	"
III	2x4	Rollers	--	--	--	18	$7.31	$10.96
III	2x4	Fixed	--	--	--	18	"	"

TABLE VI

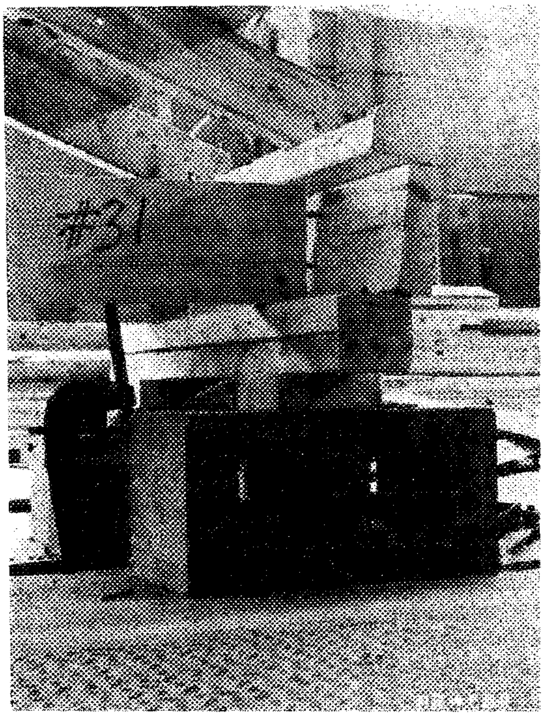


Fig. 1   Typical failure of Type III construction. Rafter plate and header pushed away from joists.

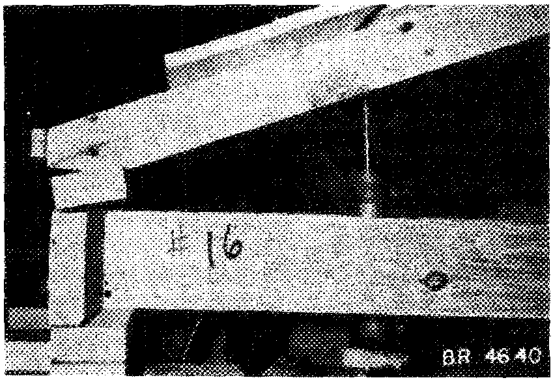


Fig. 2   Typical failure of Type II construction. Rafters pushed outward from rafter plate.

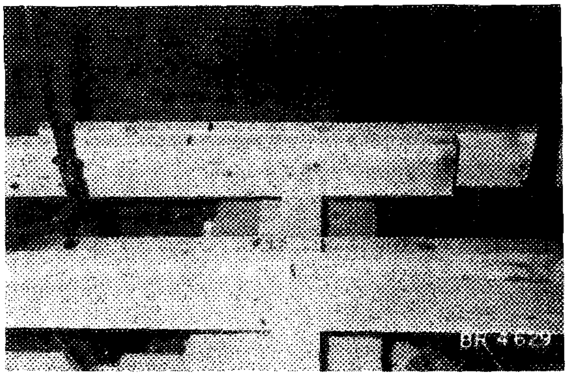
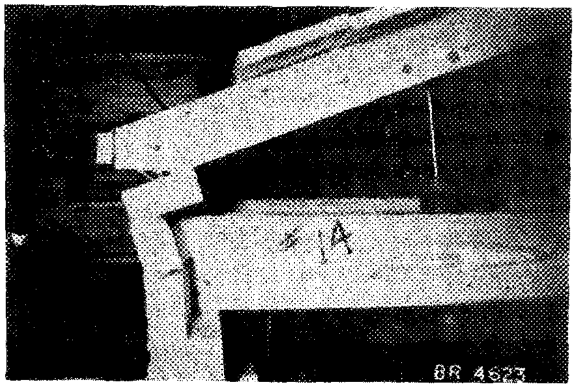


Fig. 3 Typical failure with Type I construction with roller supports. Joist lap at the centre of the span badly deformed by horizontal movement of joists.

Fig. 4 Failure of Type II construction. Headers split due to outward movement of rafter plate. Rafters also pushed away from rafter plate.



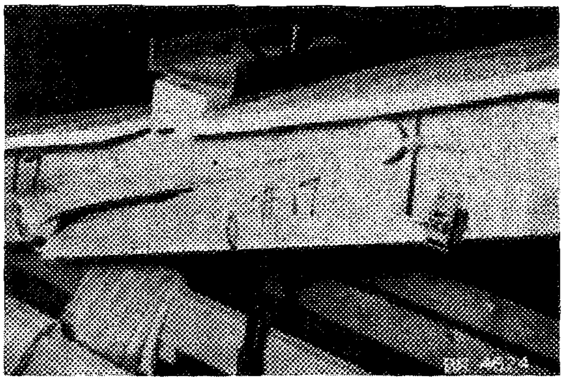


Fig. 5 Failure in Type I construction. Rafter broke near large knot.

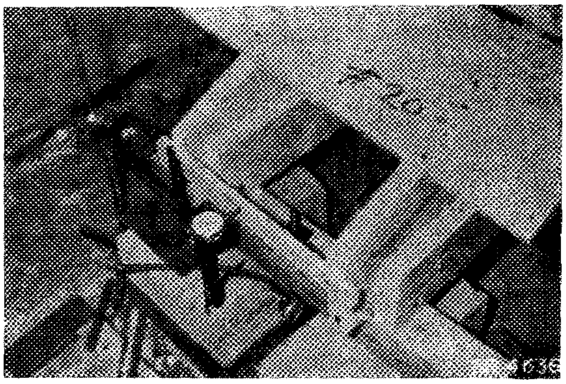
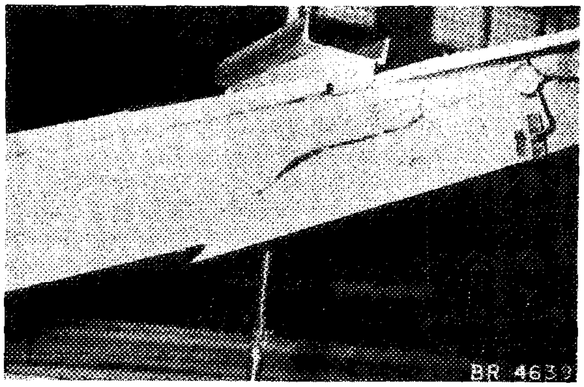


Fig. 6 Failure in Type I construction. Rafters pushed away from top plate and joist.

Fig. 7 Failure in Type I construction. Rafter broke near section of badly cross-grained wood.



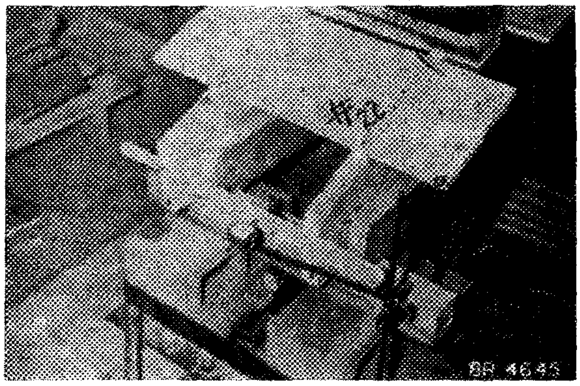


Fig. 8 Failure in Type I construction. Rafters pushed outward from top plate.

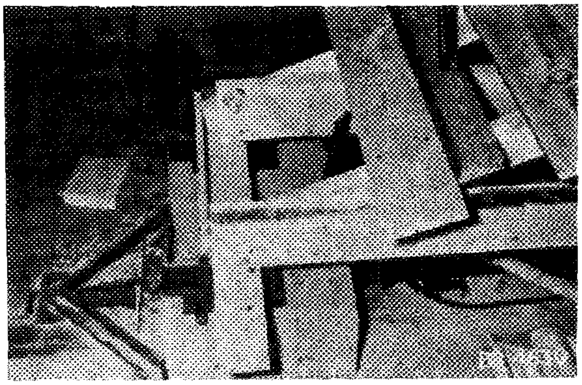


Fig. 9 Failure in Type I construction. Top plate pushed away from ceiling joist.

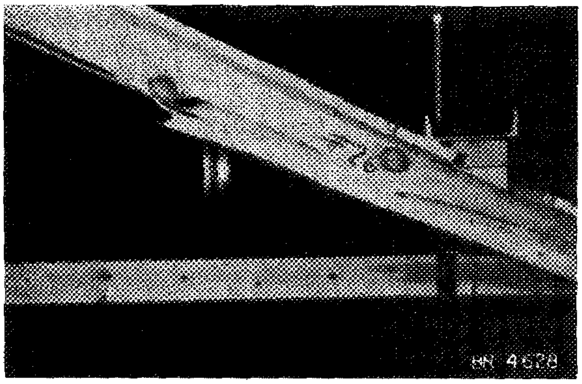


Fig. 10 Failure in Type I construction showing broken rafters.

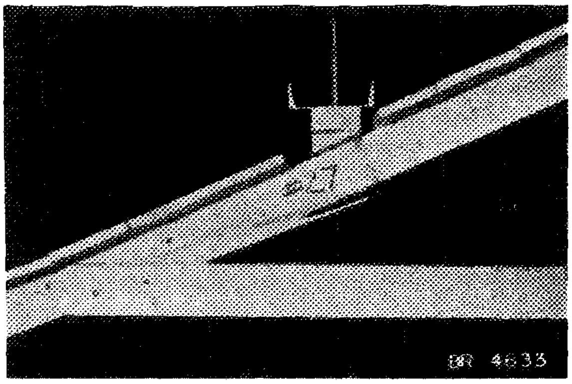


Fig. 11 Failure in Type I construction showing broken rafters.

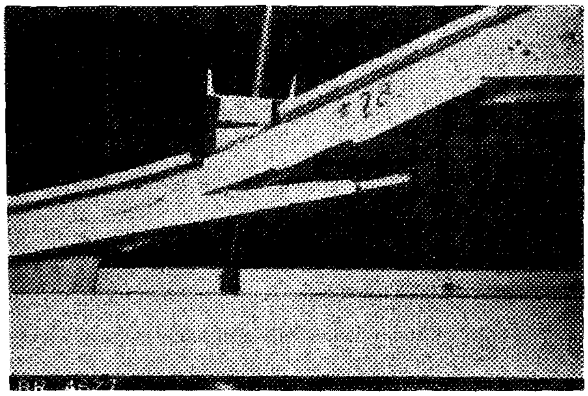


Fig. 12 Failure in Type I construction showing broken rafters.

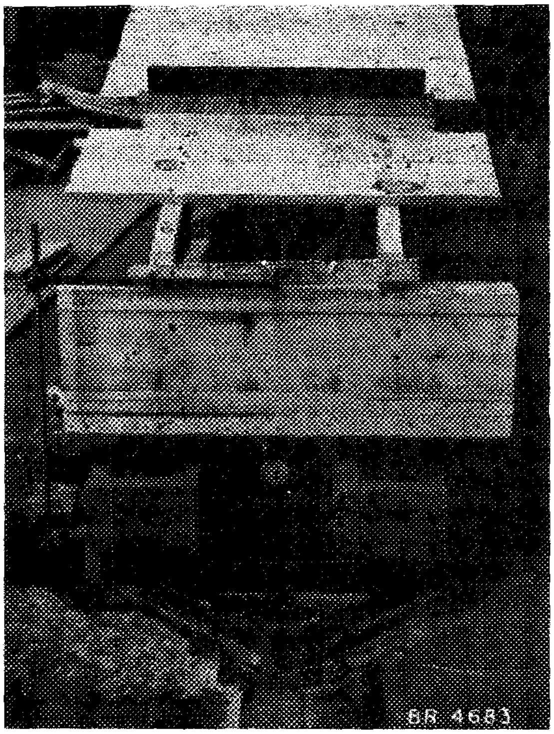
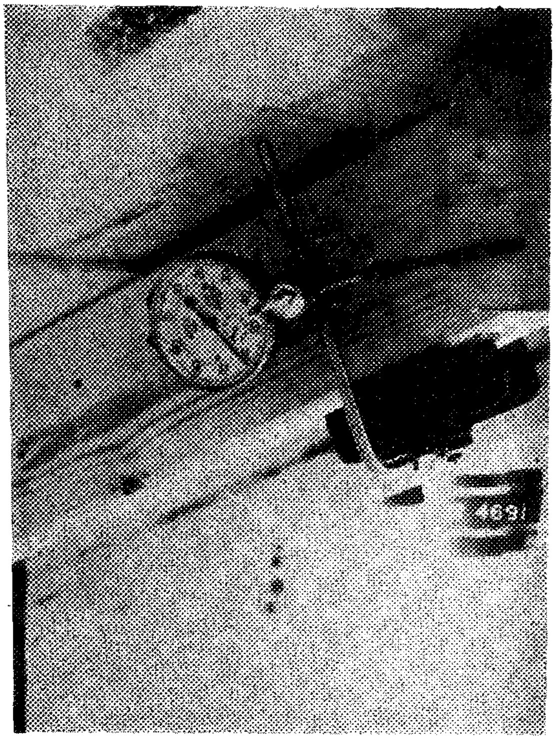


Fig. 13 Instrumentation at left end of typical test structure showing end brace, roller supports and dial gauges.

Fig. 14 Dial gauge attached to rafter near the collar tie to measure relative movement between collar tie and rafter.



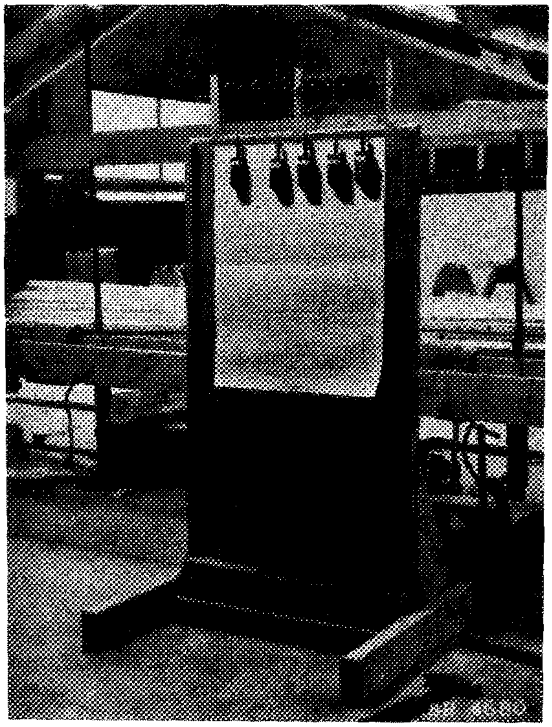


Fig. 15 Recording board showing deflection indicator weights and recording sheet.

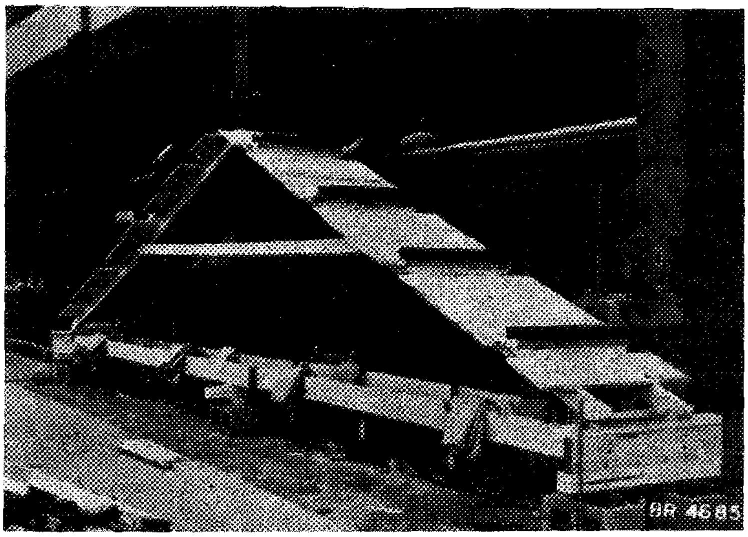


Fig. 16 Typical conventional test structure.

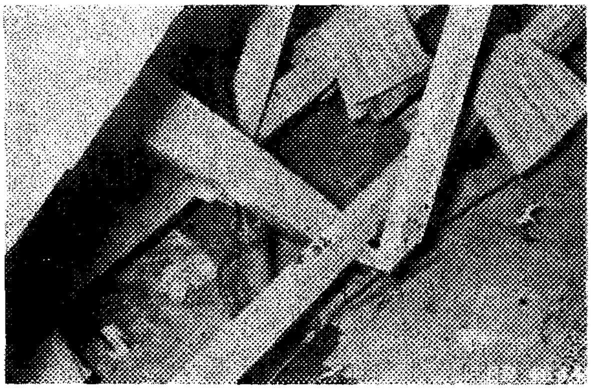


Fig. 17 Failure in Type A truss. Long diagonal pulled away from lower chord.

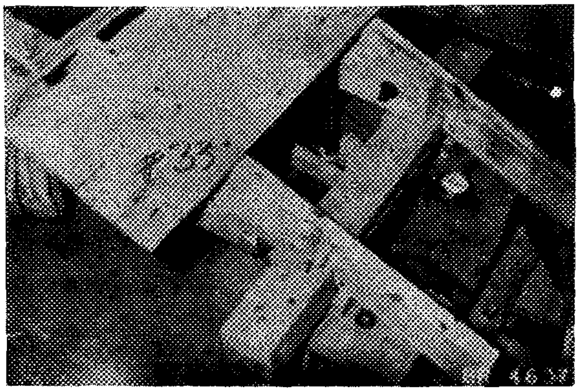


Fig. 18 Failure in Type A truss. Ends of bottom chord failed due to action of split rings.

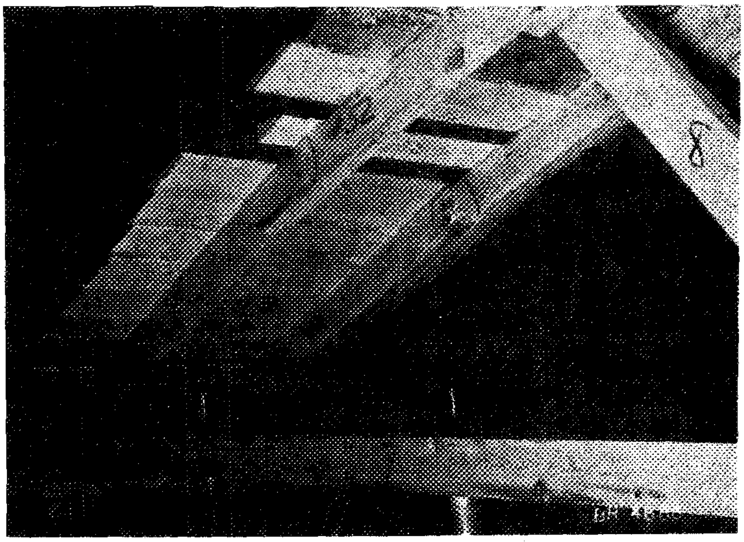


Fig. 19 Failure in Type A truss showing break in top chord at knot due to combination lateral and vertical bending of top chord.

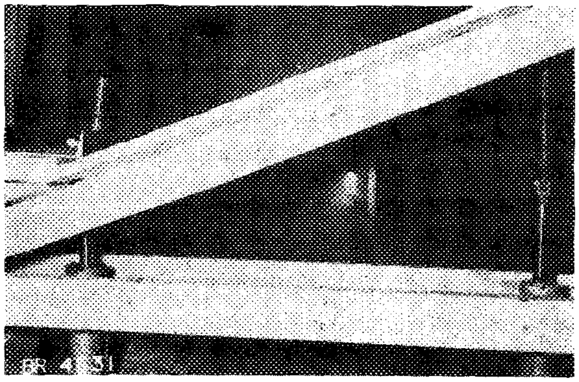


Fig. 20 Failure in Type A truss showing break in top chord.

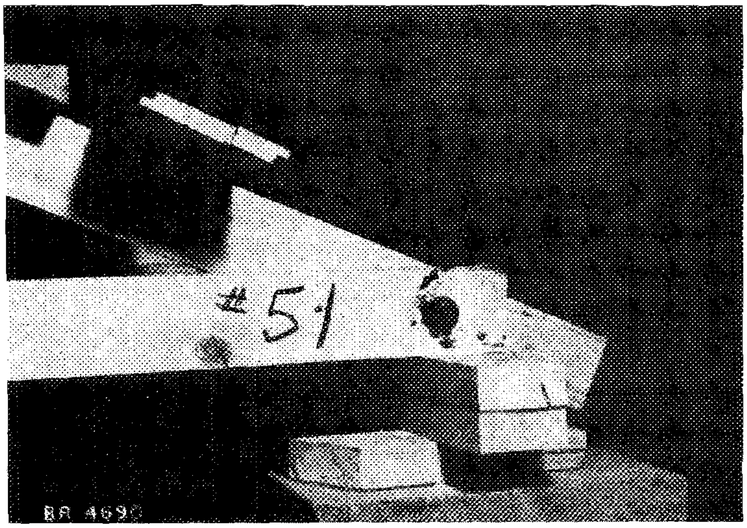


Fig. 21 Failure in Type A truss. Bottom chord broke in tension at split ring.

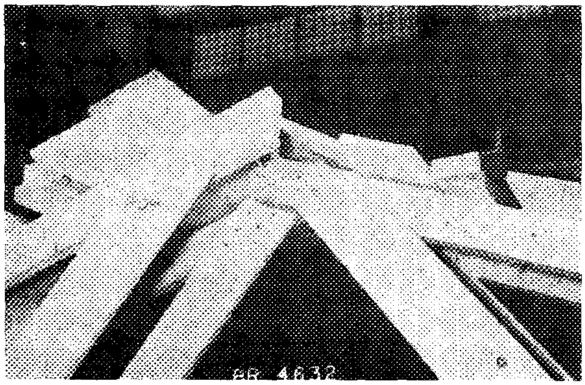


Fig. 22 Failure of Type B truss. Upper chords bowed laterally under load causing gusset plates at the peak of the top chord to fall off.

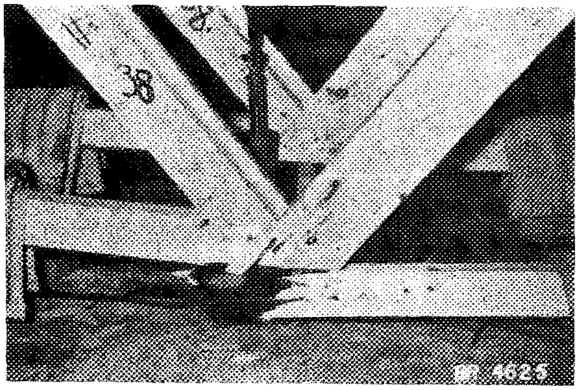


Fig. 23 Failure in Type B truss. Bottom chord failed in tension at junction of two diagonals.

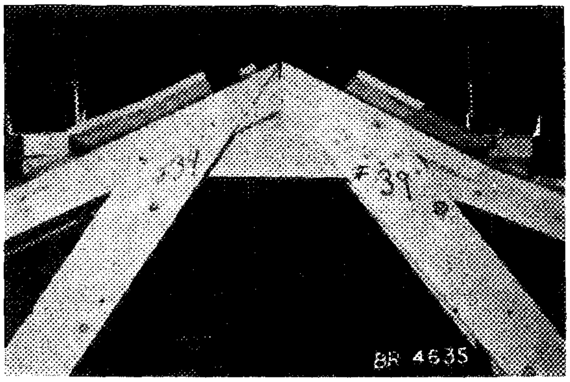


Fig. 24 Failure in Type B truss. Long diagonal pulled down with sufficient force to split the top of the upper chord.

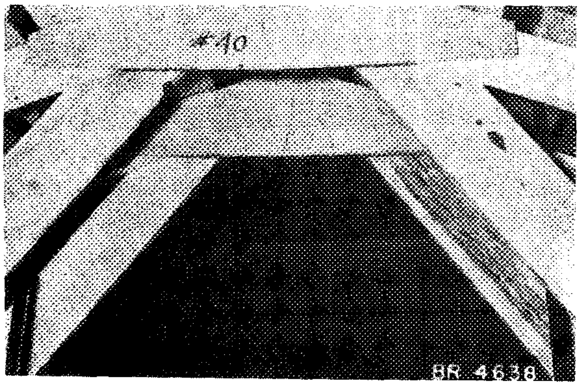
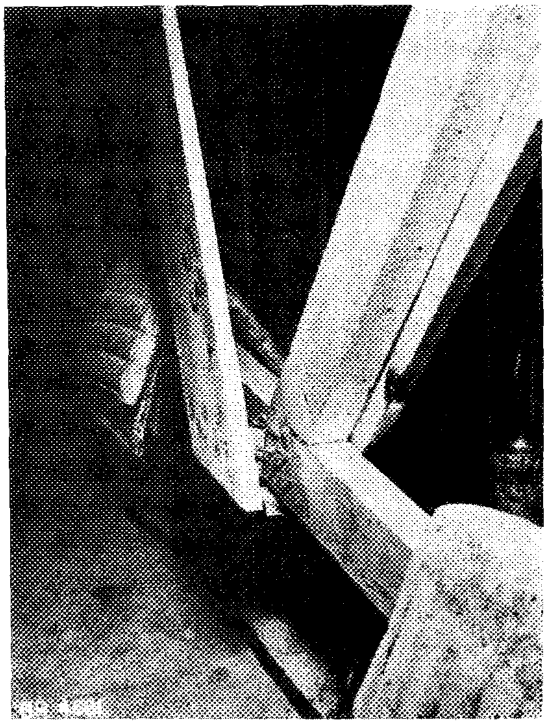
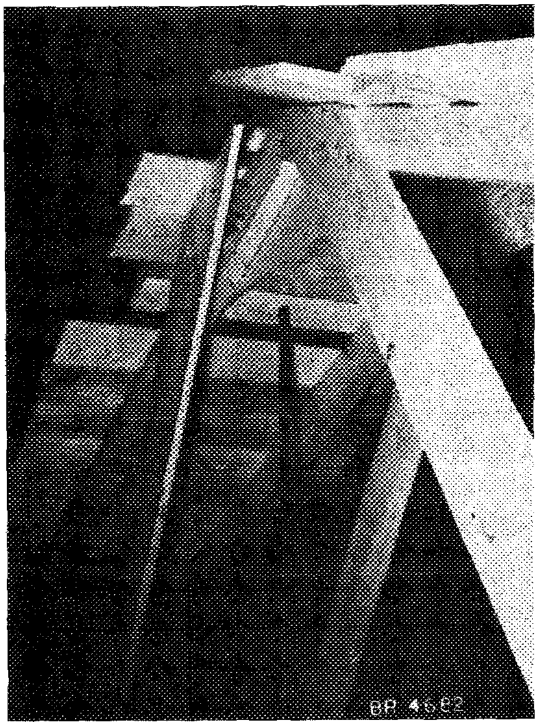


Fig. 25 Failure in Type B truss. Upper chord of truss bowed laterally under load causing the gusset plates at the peak to crack near the centre.





Figs. 26 and 27 Failure in Type B truss. Long diagonal pulled away from lower chord and upper chord simultaneously.

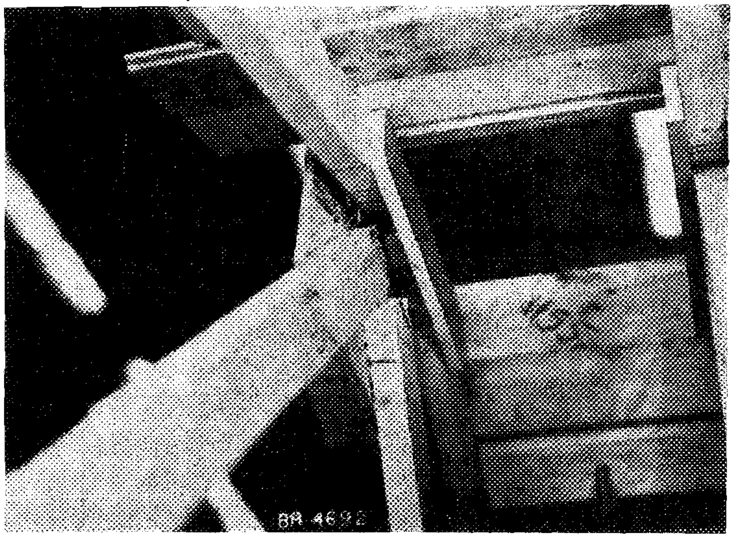


Fig. 28 Failure in Type C truss. Gusset plate at peak pulled away from both upper chords.

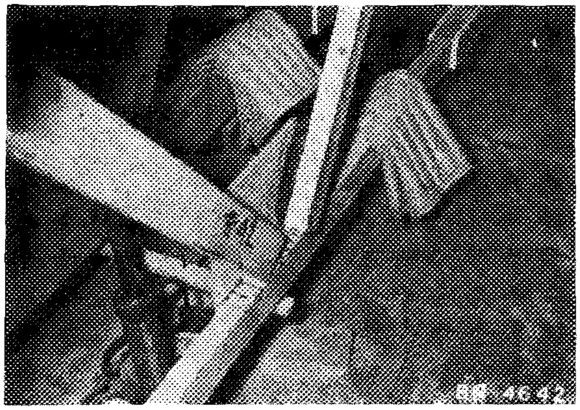


Fig. 29 Failure in Type C truss. Lower end of long diagonal pulled away from gusset plate.

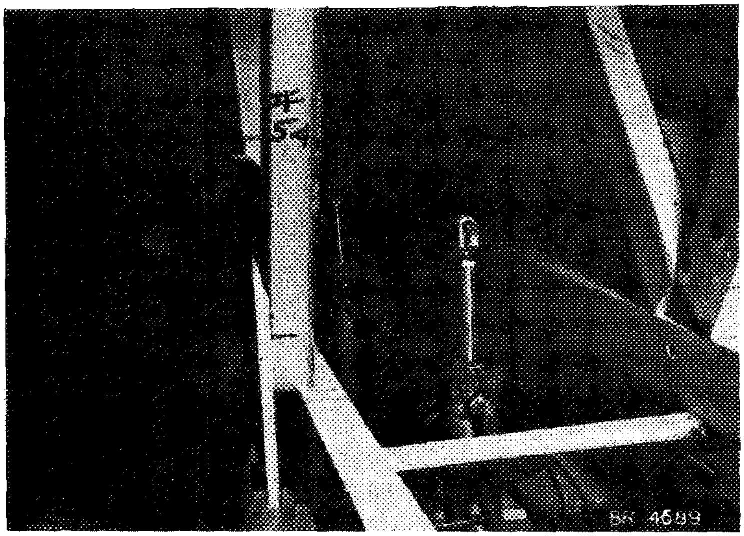


Fig. 30 Failure in Type C truss. Gusset plate connecting long and short diagonals to lower chord separated from lower chord.

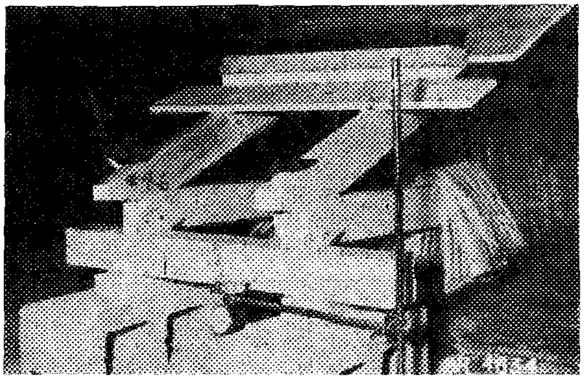


Fig. 31 Failure in Type D truss showing crushing of lower chord at the heel joint.

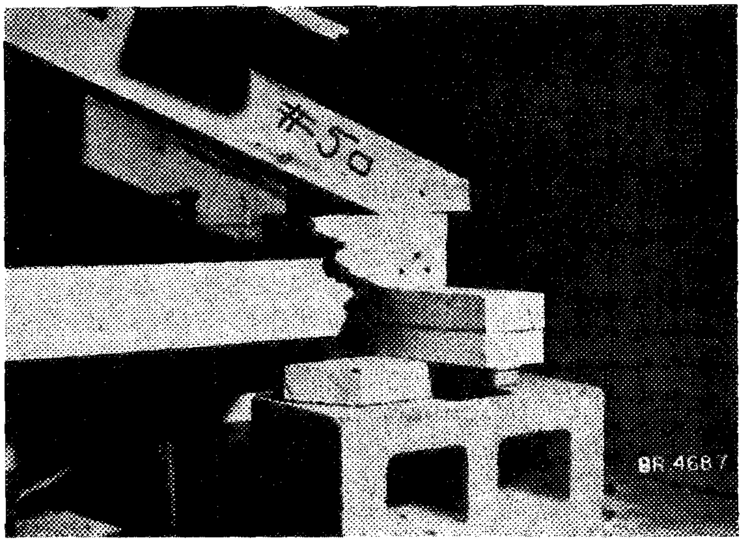


Fig. 32 Failure in Type D truss. Bottom chord failed in tension near bolt. Note the badly cross-grained wood.

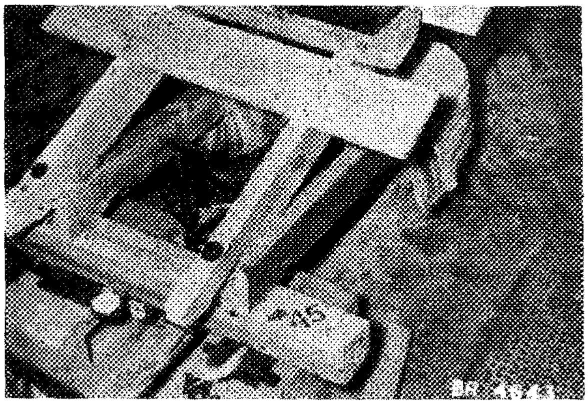


Fig. 33 Failure in Type E truss. Bottom chord crushed by upper chord at heel joint. Bottom chord also failed in tension.

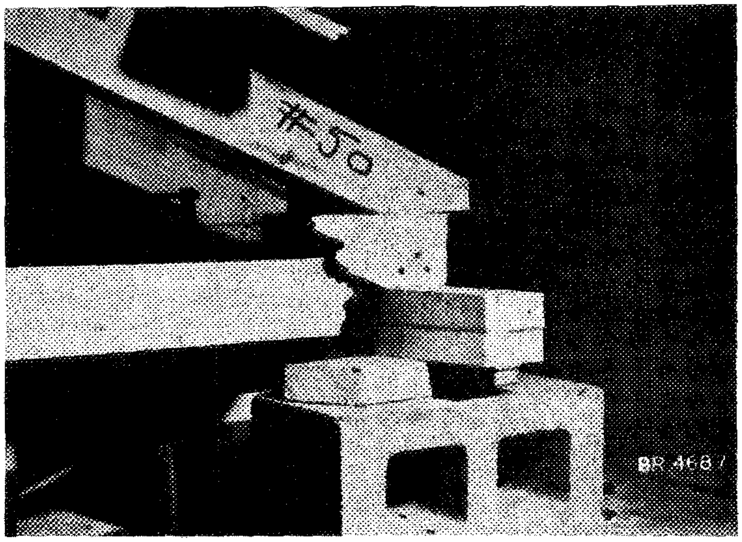


Fig. 32 Failure in Type D truss. Bottom chord failed in tension near bolt. Note the badly cross-grained wood.

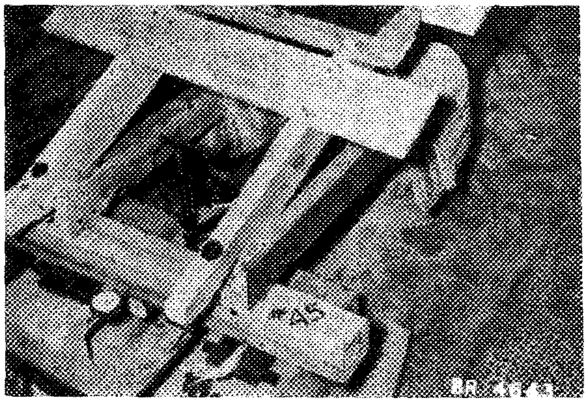


Fig. 33 Failure in Type E truss. Bottom chord crushed by upper chord at heel joint. Bottom chord also failed in tension.

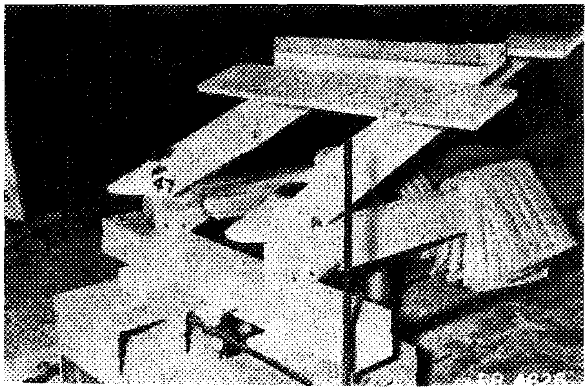


Fig. 34 Failure in Type F truss. Lower chord failed in tension near bolt.

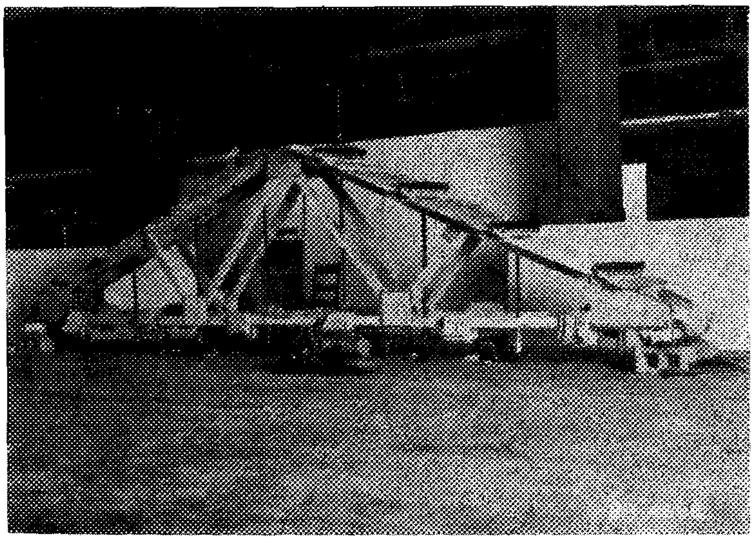
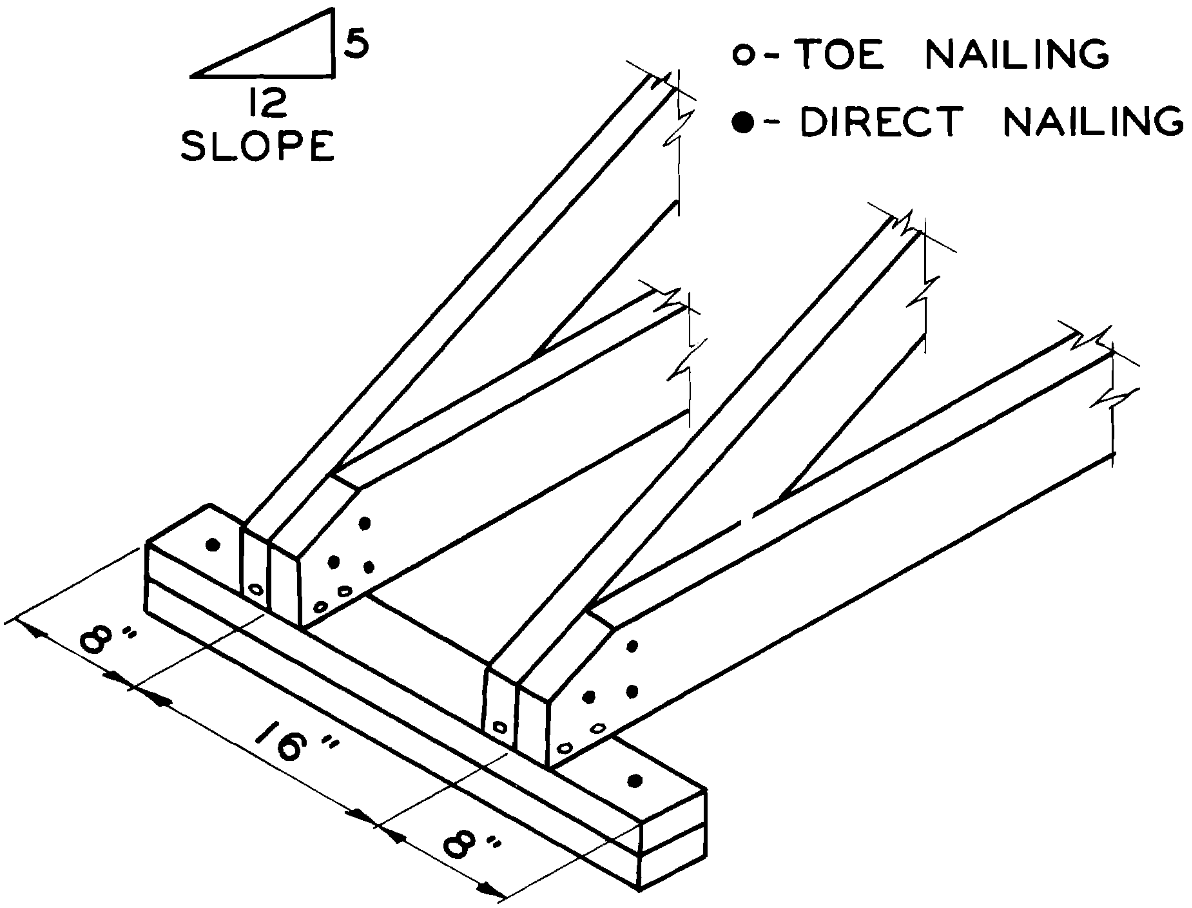


Fig 35 General arrangement of typical test structure for truss tests.



TYPE NO. 1

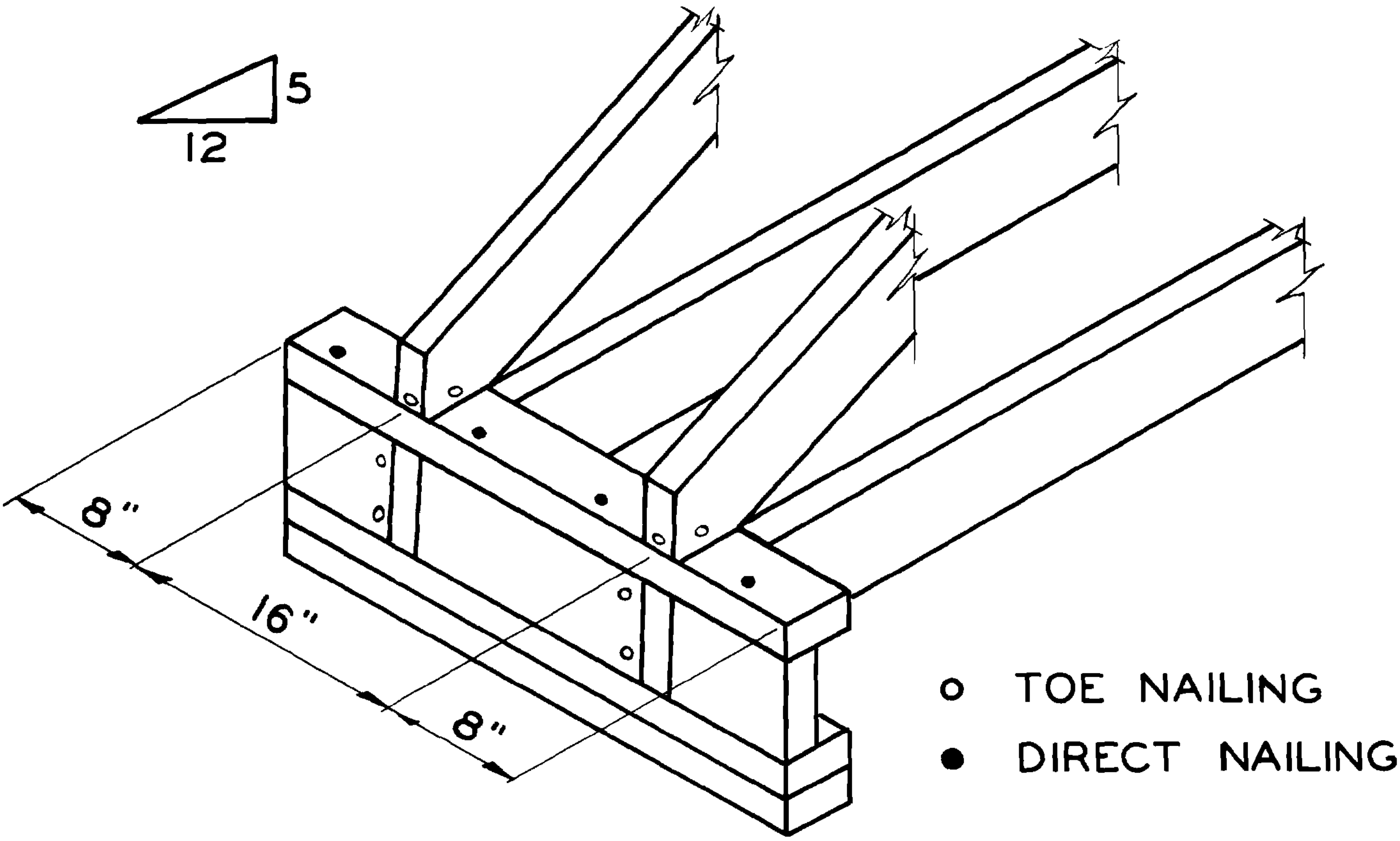
3-3½" NAILS, RAFTER TO JOIST

2-3½" NAILS, JOIST TO PLATE- TOE NAILED

3-3½" NAILS, RAFTER TO PLATE- TOE NAILED

2-3½" NAILS, PLATE TO PLATE

FIGURE 36



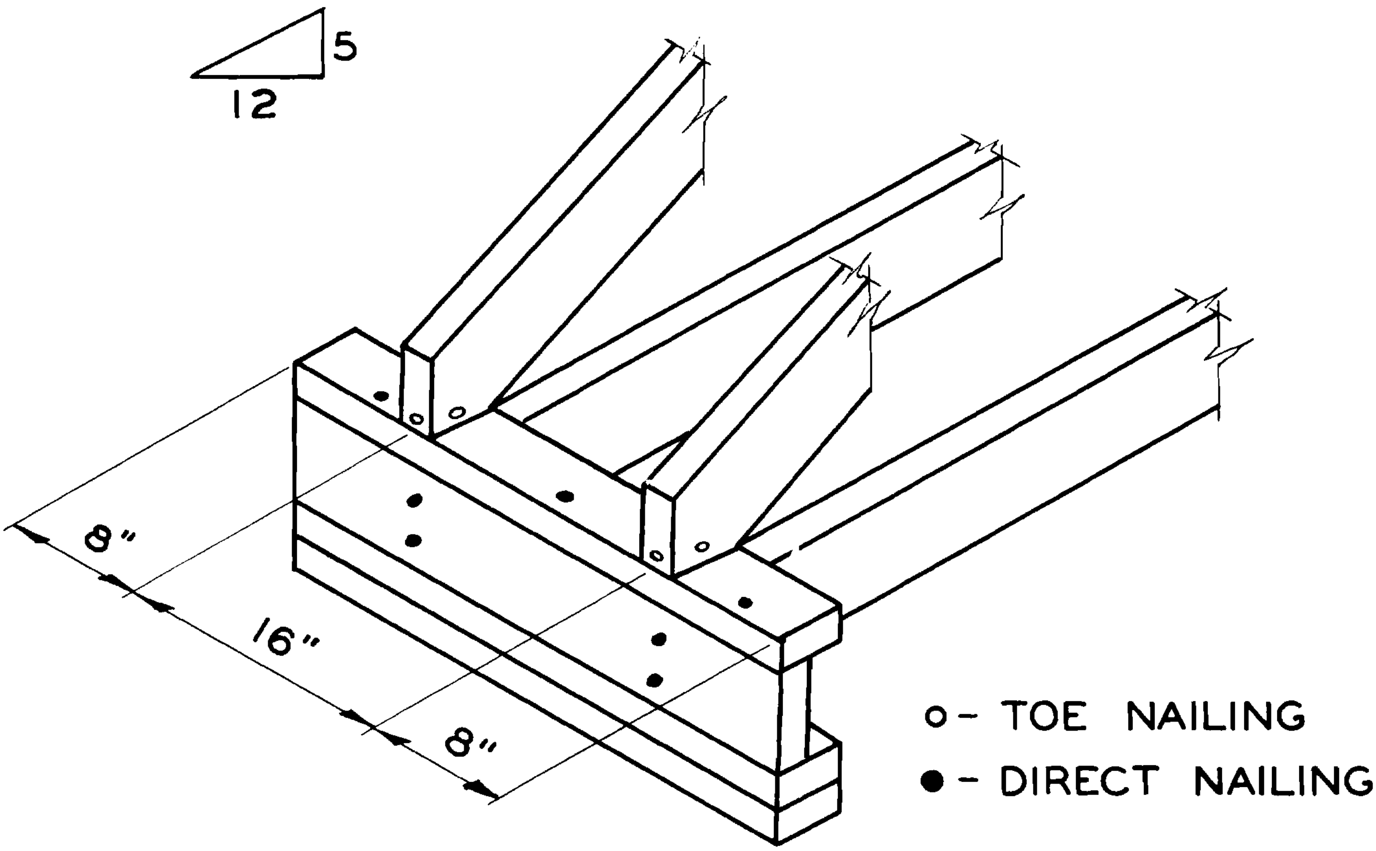
TYPE NO. 2

3-3½" NAILS, RAFTER TO RAFTER PLATE (TOE-NAILED)
1-4" NAIL, RAFTER PLATE TO RT. & LT. HEADER
2-4" NAILS, RAFTER PLATE TO CENTER HEADER
3-3½" TOE NAILS, SHORT HEADER (LT. END) TO JOIST
2-4" NAILS, JOIST (LT.) TO CENTER HEADER
3-3½" TOE NAILS, CENTER HEADER TO JOIST (RT.)
2-4" NAILS, JOIST (RT.) TO SHORT HEADER (RT.)
2-4" NAILS, RAFTER PLATE TO JOIST (EA. SIDE)
2-3½" NAILS, JOIST TO JOIST PLATE
2-3½" NAILS, PLATE TO PLATE

FIGURE 37

BR. 1034-2

DRB REPORT 81

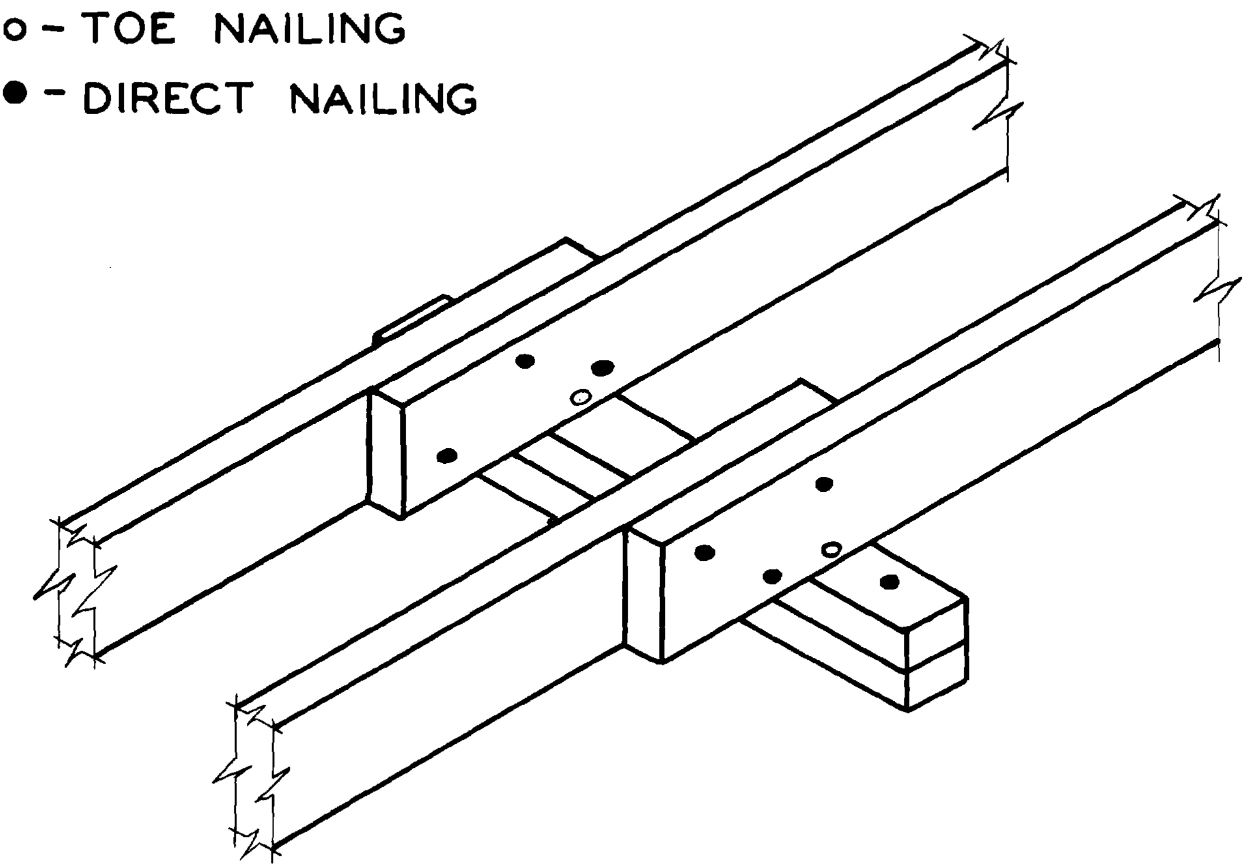


TYPE NO. 3

3-3½" NAILS, RAFTER TO RAFTER PLATE (TOE NAILED)
3-4" NAILS, RAFTER PLATE TO HEADER
1-4" NAIL, RAFTER PLATE TO JOIST
2-4" NAILS, HEADER TO JOIST END
2-3½" NAILS, JOIST TO JOIST PLATE
2-3½" NAILS, PLATE TO PLATE

FIGURE 38

BR. 1034-3



PARTITION SPLICE

3-3½" NAILS, JOIST TO JOIST

2-3½" NAILS, JOIST TO PLATE (TOE NAILS)

2-3½" NAILS, PLATE TO PLATE

FIGURE 39

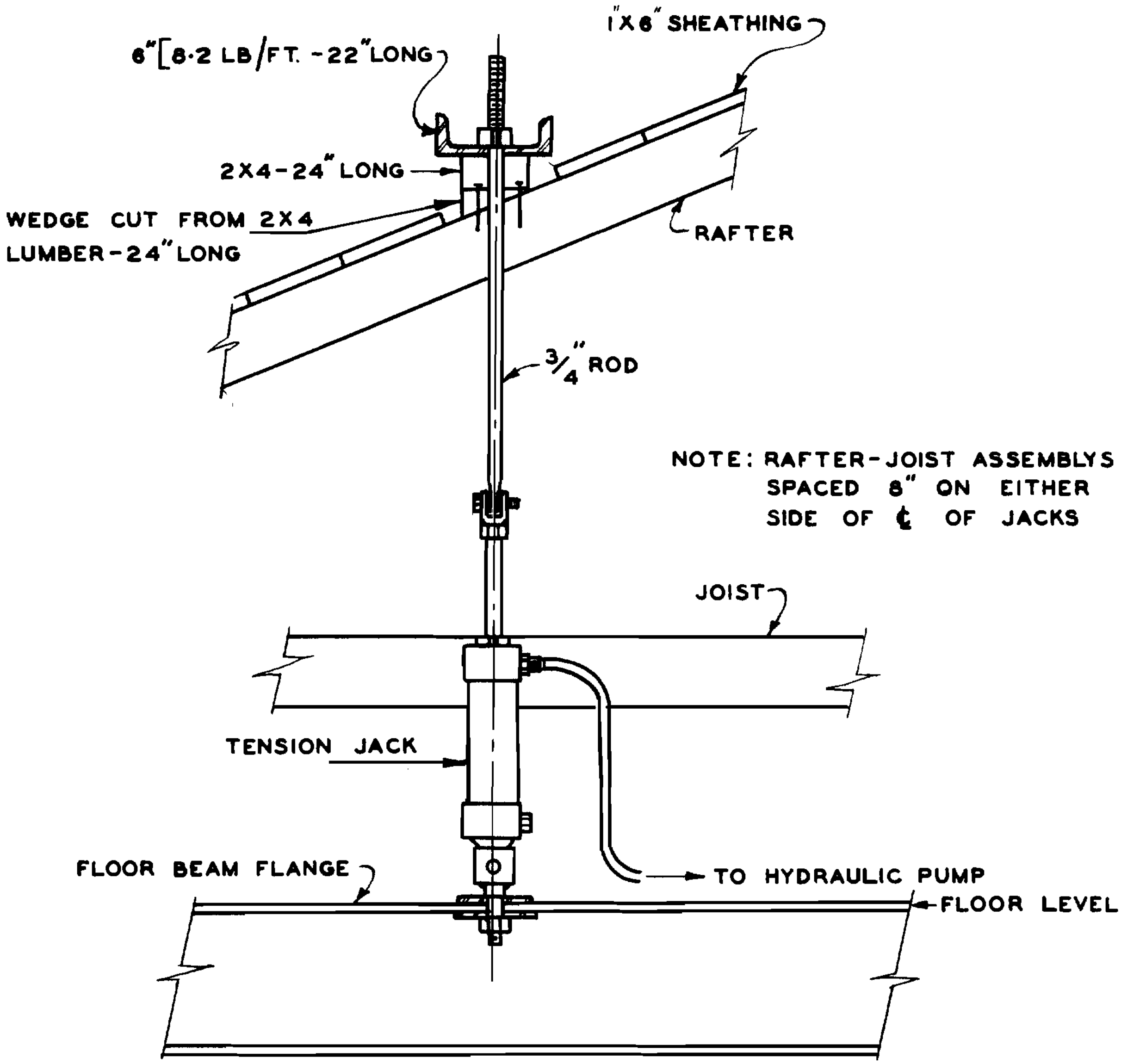
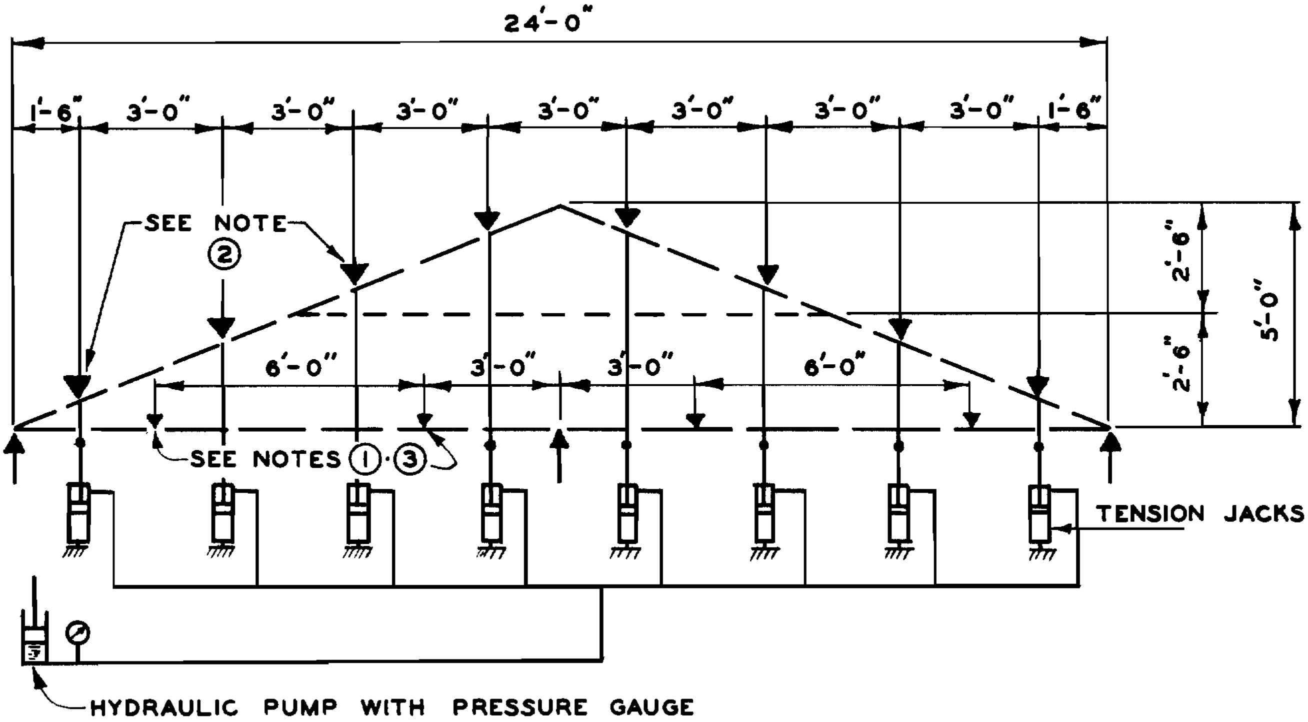


FIGURE 40

DETAIL OF JACK ASSEMBLY

BR. 1034-5



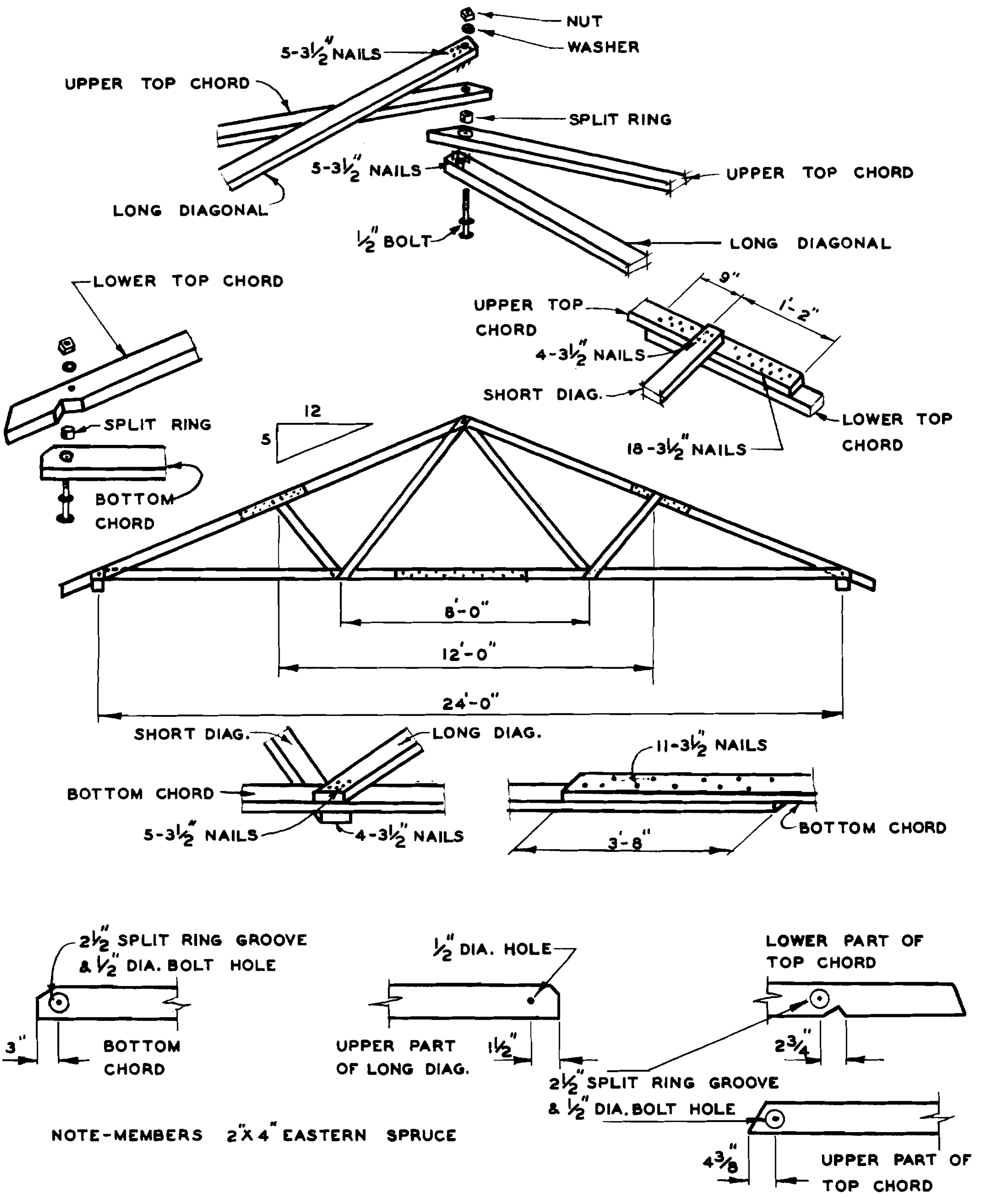
NOTE ① – CEILING LOAD APPLIED BY MEANS OF LEAD FILLED BAGS PLACED AS SHOWN WITH 80 LB. AT EACH POSITION INDICATED ON EACH OF 2 PAIRS OF JOISTS.

NOTE ② – LOADS EXERTED BY TENSION JACKS WERE 8W LB., WHERE W = APPLIED SNOW LOAD IN LB./SQ. FT.

NOTE ③ – THIS SKETCH OF LOADING APPARATUS APPLIES AS WELL TO TRUSS TESTING WITH ONE SLIGHT DIFFERENCE. THE CEILING LOADS IN THIS CASE WERE 50 LB. AT EACH POSITION. THE POSITIONS OF THESE LOADS FOR TRUSSES WERE 2', 6' AND 10' ON EACH SIDE OF THE ℄, RATHER THAN 3' AND 9' AS SHOWN.

FIGURE 41

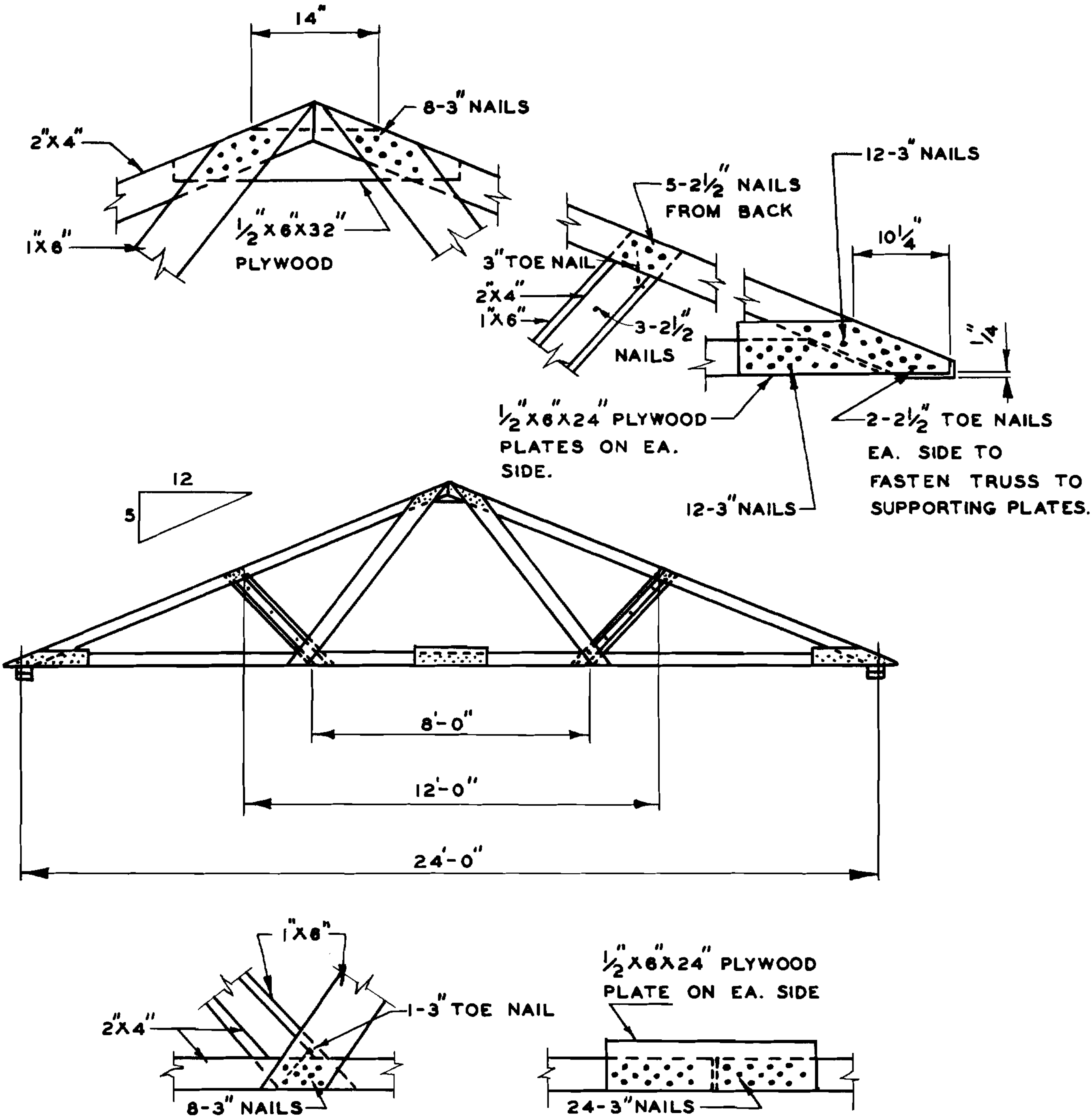
SCHEMATIC SKETCH OF LOADING ARRANGEMENT



NOTE-MEMBERS 2"X 4" EASTERN SPRUCE

FIGURE 42

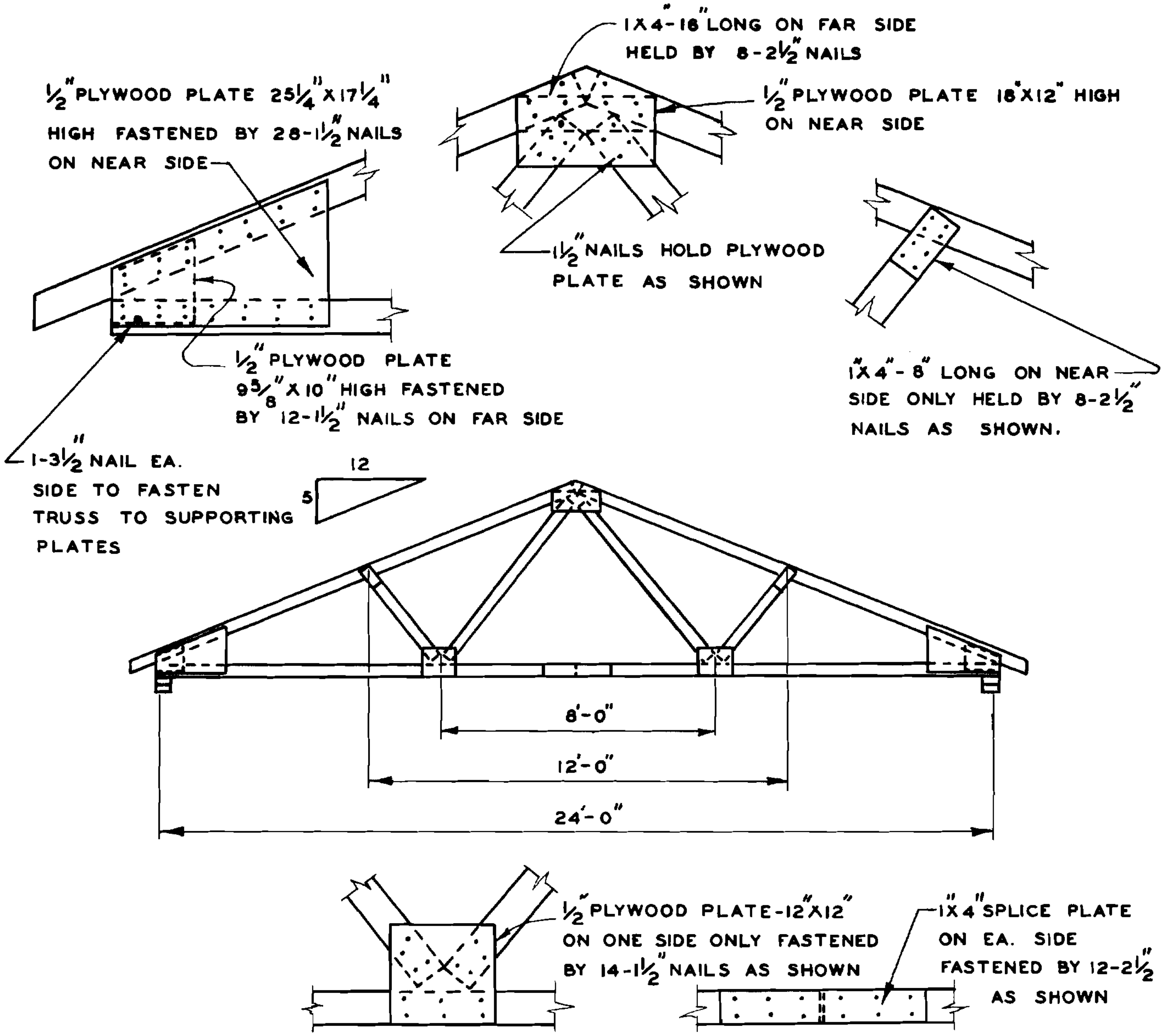
TYPE-A TRUSS



NOTE:- ALL MEMBERS MADE OF EASTERN SPRUCE.
PLATES- 5 PLY DOUGLAS FIR PLYWOOD.

FIGURE 43

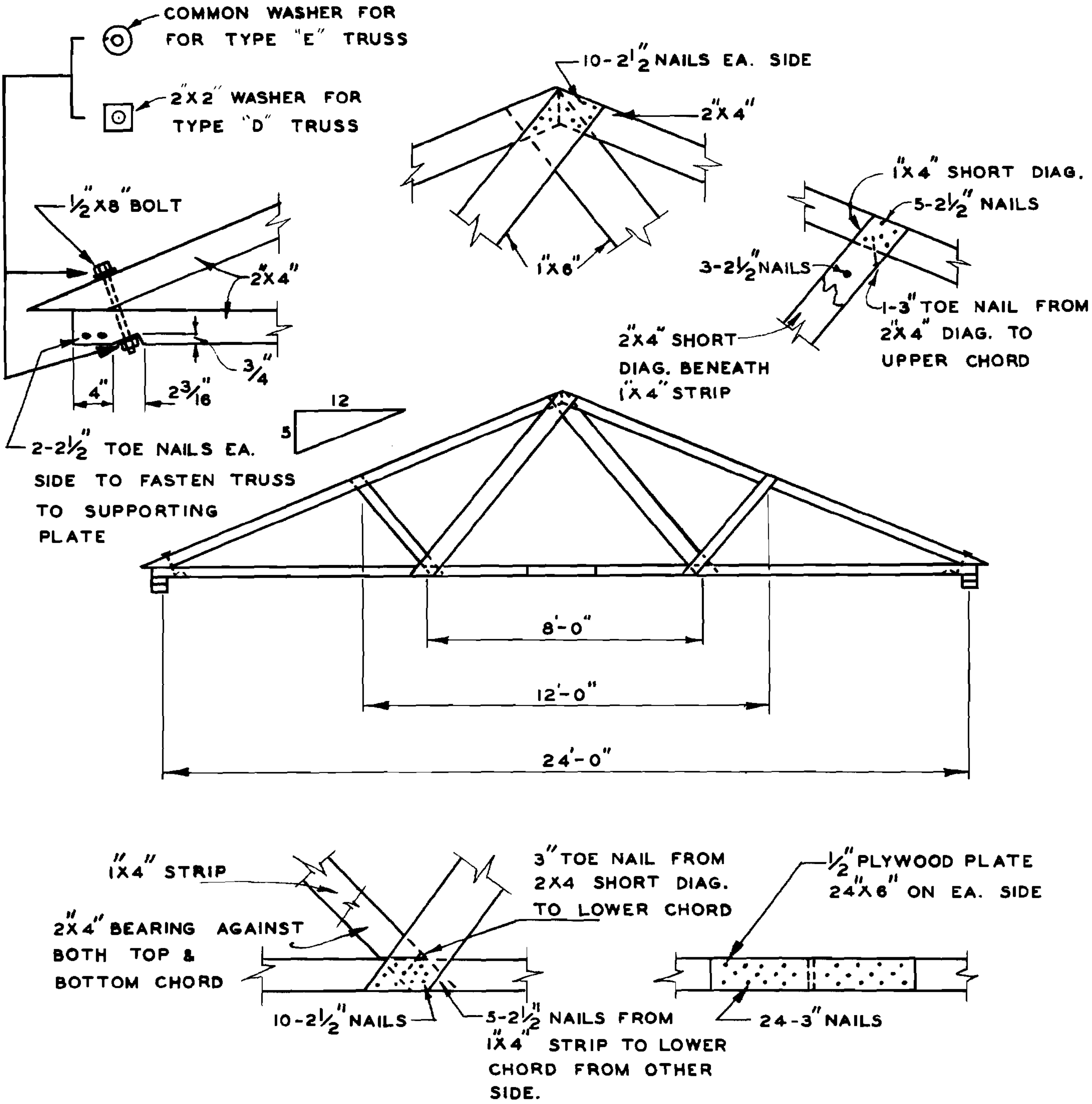
TYPE - B TRUSS



NOTES: ALL STRUCTURAL MEMBERS-2X4 EASTERN SPRUCE.
PLYWOOD PLATES-½"-5 PLY DOUGLAS FIR.
ALL JOINTS FASTENED WITH CASEIN GLUE. NAILS USED TO SUPPLY PRESSURE TO THE JOINT WHILE DRYING.

FIGURE 44

TYPE-C TRUSS



TYPE D & E TRUSS

(EXAMINE HEEL JOINT DETAIL FOR DIFFERENCE IN THE TWO TYPES.)

FIGURE 45

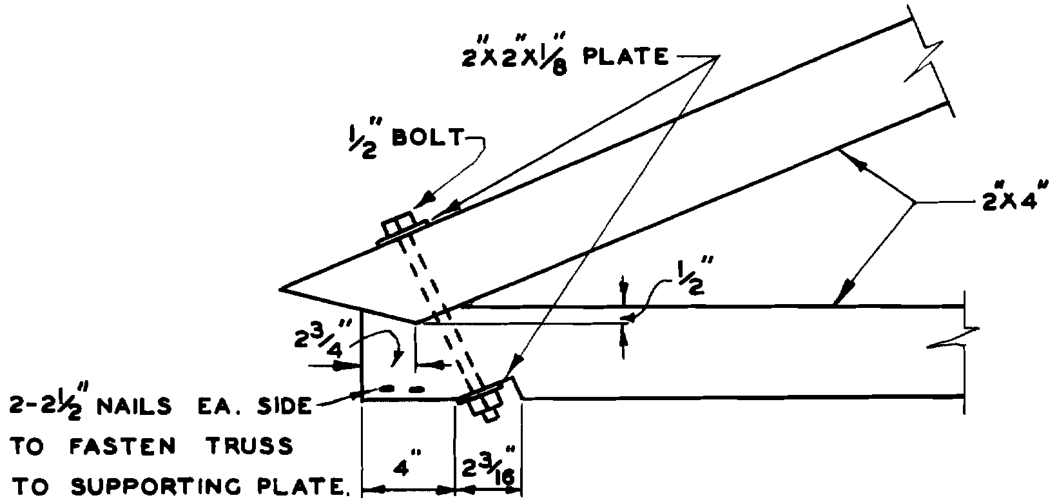


FIGURE 46

TYPE - F TRUSS

(NOTE OTHER DETAILS SIMILAR TO TYPE D & E.)

BR. 1034-11