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DIVISION OF BUILDING RESEARCH

STRUCTURAL TESTING OF TWO W-TRUSSES

by

D.B. Dorey

(A co-operative project with the Forest Products Laboratories, Department of Northern Affairs and National Resources, Ottawa)

Report No. 77
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Ottawa
December 1955

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.

This report describes the first experimental work carried out by the Division in this field, utilizing a design developed in the United States, but subject to the more severe loadings which have to be anticipated in Canada.

The work described is only a beginning, but it did give useful information and provided good experience in experimental techniques which can be applied in further work in this field which has already been started. The research work herein described has opened up several useful avenues of inquiry, the first results of which the Division hopes to be able to publish in 1956.

In all its work in which wood is involved as a material, the Division has the pleasure and privilege of working closely with the Forest Products Laboratory of the Department of Northern Affairs and National Resources of which Colonel J.H. Jenkins is Chief. Although the Division of Building Research accepts full responsibility for the work herein described, it was carried out in close consultation with the Forest Products Laboratories, and it is for this reason that this liaison is indicated on its title page.

The author, D.B. Dorey, is an Assistant Research Officer and was a member of the Building Design Section (W.R. Schriever, Head) when the work described herein was done. Mr. Dorey is now Officer in Charge of the Atlantic Regional Station of the Division at Halifax.

R.F. Legget
Director.

Ottawa
December 1955

STRUCTURAL TESTING OF TWO W-TRUSSES

by D.B. Dorey

Home builders in Canada are always on the alert for new developments which will facilitate their activities and improve the quality of their finished products. Among more recent developments, the trussed roof is receiving considerable attention as an alternative to the conventional rafter systems. This attention is largely due to the advantages which trusses offer to a builder when the economics of his operations justify the following:

(i) Precutting and pre-assembling of house components at a convenient location (a) when shop facilities are available and (b) when there is a shortage of skilled workmen to do on-site cutting and framing;

(ii) Speed of erection, such as (a) when seasonal weather hampers continuous work or (b) when specialized crews move from house to house doing one operation;

(iii) Clear spans for flexible interior layout.

Trussed roof construction for dwellings in Canada has been influenced greatly by the work of research organizations in the United States. Notable among these organizations has been the Small Homes Council of the University of Illinois. The Small Homes Council has published several working sheets of data on house trusses which can be readily used by home builders to mention only one example of the useful information that has been made available. A review has been made of this information to assist in similar work at the Building Research Centre. In this work the more severe loading conditions imposed by the Canadian climate are taken into account.

2. DESIGN OF TWO W-TRUSSES

To acquaint home builders attending the Conference on Building Research (1953), and also those attending Open House activities (1954), with the features of house truss design, a full-scale model of an end section of a house was erected in the large project area of the Building Research Centre.

This model consisted of two 8-foot sections for side walls, end wall, and three W-trusses having a 6 in 12 slope. The trusses were designed according to the recommendations of the 1953 edition of the National Building Code of Canada. Two trusses were of identical construction; the third truss differed from the first two in the construction of the lower chord members. The upper chords were fabricated from 2- by 8-inch stock and the remaining members were made from 2- by 4-inch, except for the third truss which had a lower chord made from 2- by 6-inch stock. All the stock used was air-dried No. 1 Eastern Spruce. Joints in all three trusses were made by $2\frac{1}{2}$-inch split ring connectors and $\frac{1}{2}$ inch diameter bolts. The details of the two identical trusses which will be dealt with later are shown in Fig. 1.

It was found that, although it would be desirable to use 2- by 6-inch material with a lap joint at the uppermost panel point, 2- by 6-inch stock would be overstressed in the upper chords. The size of the upper chords, therefore, had to be increased, and since lumber is usually manufactured in even widths, a 2- by 8-inch nominal section was selected. This obviated a departure from the design used by the Small Homes Council because of the limited strength of a lapped joint at the top panel point. To overcome this difficulty, and to improve the strength of the top joint sufficiently, the 2- by 8-inch upper chord members were given a common rafter cut and were butted together. It was also found necessary to change the design of the lower chords because of the increase in the design loadings over those used by the Small Homes Council. Imposed forces in the lower chords required the use of two nominal 2- by 4-inch members to connect the heel joints to third points of the lower chords.

When the useful purpose of the display house model was at an end, it was disassembled and the two identical trusses were retained for structural testing.

3. TESTING OF W-TRUSSES

(a) Purpose of Test

Since this test was the first of this type to be carried out by the Division, it was important to learn as much as possible about the necessary testing technique. The objectives of the test were therefore as follows:

(i) To study the performance of two W-trusses designed according to the National Building Code (1953), when subjected to loads up to failure; and

(ii) To study a proposed method of testing as a means of testing similar house components in the large project area.

(b) Preparing Test Assembly

As opposed to testing each truss separately, the two trusses were mounted side by side on supports having the approximate lateral stability of one-storey house walls. The spacing on the supports was set at the design spacing of 24 inches on centres. The upper chords were joined together with 4-foot lengths of sheathing boards to provide the required tributary roof area for loading and also to simulate the working conditions of trusses in a house roof as nearly as possible. Furring strips were nailed to the lower chords in a manner duplicating the furring arrangement in the ceiling of a house. To prevent sidesway and possible overturning of the test assembly when loaded, a stabilizing framework was anchored to the floor beams of the project laboratory.

Each panel point of both trusses was assigned a reference letter and a system of wires and pulleys was installed to record vertical and horizontal movements at the panel points on W-truss No. II. A deflection board was securely anchored to the floor of the project laboratory and each deflection wire was appropriately referenced and tensioned by a standard reference weight. Figure 2 shows the test assembly with the ceiling load and the equivalent dead load of the shingles in place on the trusses.

(c) Method of Loading and of Recording Deflections

Standard 1-, 10-, and 20-lb. lead-filled bags were uniformly distributed on the tributary roof area and on the furring strips of the lower chords to simulate one times the design load plus dead load. Since the number of lead-filled bags was limited, loads in excess of one times the design live load for symmetrical loading were superimposed by applying pea-sized crushed stone. The crushed stone was confined to the tributary roof area by specially constructed bulkheads. Figure 2a shows the test assembly with the bulkheads in position.

Deflections, both horizontal and vertical, were measured at each panel point on W-truss No. II by a system of wires running over small pulleys. Dial gauges were also used to read deflections on the lower chords of each truss but were removed when twice the design live load was reached. The system of wires and pulleys provided a continuous record of the deformed shape of W-truss No. II; the dial gauges provided a record of the initial deflections in the lower chords and served as a check on the wire and pulley system. In addition to the mechanical methods used for measuring deflections, the Photogrammetry Section of the Division of Physics measured deformations by a photogrammetric method. The latter method is the subject of a separate report.

(d) Testing Procedure

The sequence followed in applying the loads was as outlined in Table I. The ceiling load of 10 lb. per square foot and the additional dead load of 210 lb. per square (10 by 10 feet) for asphalt shingles were applied to the trusses. This loading remained in place for the duration of all loading phases. Deflection readings were taken each day for twenty days before any live loads were added. This period was allowed in order to give sufficient time for the trusses to "bed down" at the joints and supports. Since the records show a pattern of uniform deflections on the lower chords for the 20-day period, only those deflections taken at five-day intervals are given in Table II.

At the end of the 20-day period of the first loading phase, the N.B.C. design snow loading of 50.8 lb. per square foot of the horizontally projected roof area was applied in increments of one-half the design live load and was allowed to remain on the trusses for a period of seven days. The lead-filled bags were applied manually and simultaneously to both roof slopes from a platform erected beside the test assembly. Deflection records were made from both the dial gauge readings and the deflection board. The dial gauge readings have been reduced in Table III to show the net deflections at the lower chord panel points with respect to the end panel points. Figure 3 shows the trusses loaded with the design live load plus dead loading and ceiling load.

When the seven days for the second loading phase had elapsed, the design live load was removed and twenty-four hours were allowed for the trusses to recover before further loading proceeded. The net deflections and the residual settlements at the supports are also given in Table III for immediately after the removal of the design live load and 24 hours later. Moisture content readings for the truss members taken daily showed that the moisture in the wood remained at approximately 7 per cent.

For the third loading phase, the design live load was re-applied and allowed to remain for a period of 24 hours. This loading phase was designed to serve as a check on the ability of the trusses to give repeated deflection readings for the same magnitude of loading. After 24 hours, the load was removed and an additional 24 hours were allowed for recovery of deflections. The deflections for the period under load and after the recovery period are given in Table IV.

The fourth and fifth loading phases were designed to produce the effect of a build-up of snow on one slope while the opposite slope remained bare. To approximate this arbitrary loading condition, 1.33 times the design live load (snow load) was applied, first to one slope with no live load on the opposite slope and then to the other for the reverse condition. For each of these loading phases, the loads were left in place for 24 hours and a minimum of 24 hours was allowed for recovery of deflections. The deflections resulting from these unsymmetrical loading phases are given in Table V for the sustained loading period and for the recovery period. The moisture content of the members in the trusses remained at approximately 7 per cent.

Since the available supply of lead-filled bags was limited, and not sufficient to carry the test to failure, the test assembly had to be altered to accommodate crushed stone before the sixth loading phase could be carried out. Sideboards and bulkheads were made up and mounted parallel with the roof slopes on the stabilizing framework in such a way as to form a bin around the tributary roof area. The approximate capacity of this bin in terms of crushed stone was equivalent to four times the design live load plus dead load. The total depth of the sideboards and bulkheads to give this capacity was 28 inches, 4 inches of which extended below the roof sheathing boards to allow for deflections which would take place under loading. Clearance to the extent of $\frac{1}{4}$ inch was allowed around the edges of the roof sheathing boards so that the trusses could deflect when loaded without being restrained by the surrounding framework. Adjustable tie rods were put through the sideboards to prevent them from spreading under the outward thrust of the crushed stone. All joints in the framework were secured and crossbeams were put through under the upper chords of the trusses and bolted to the stabilizing framework in order to prevent abrupt and complete collapse of the assembly. A space of approximately 4 inches was allowed between the upper chords of the trusses and the crossbeams for the trusses to deflect before failure.

To transfer the crushed stone from the stockpile on the floor of the project area to the tributary area of the trusses, a special bucket was designed which could be dumped from the floor with a small rope while suspended from the overhead crane. The capacity of this bucket was approximately two cubic feet of pea-sized crushed stone. To maintain an accurate record of the material being placed on the trusses, a set of platform scales was provided for weighing of each bucketful of crushed stone.

During the sixth and final loading phase a minimum of five men were required. Two men were required to fill and weigh the bucket, one to operate the overhead crane, one to dump the bucket, and one to spread the crushed stone uniformly on the tributary roof area.

The crushed stone was applied to the test assembly in increments of one-half the design live load until failure occurred. Deflections were recorded at the end of each loading increment and regular inspections were made of the two trusses. When four times design live load was reached, the supply of crushed stone was exhausted. Loading was continued therefore by using the lead-filled bags which were placed on top of the crushed stone.

When the superimposed load reached a value equivalent to five times the design live load or a total of 254 lb. per square foot of horizontally projected roof area, failure occurred very suddenly in the lower chords of both trusses simultaneously near the panel point "G" (Figs. 1 and 14-17). Complete collapse was prevented by the upper chords of the trusses coming to rest on the crossbeams. Additional supports were installed as a precautionary measure against local failures in weakened joints and end supports. By preventing complete collapse of the test assembly at failure of 24,443 lb., it was possible to obtain detailed photographic records of the points of most interest, namely, the joints near panel point "G" and the supports where spreading in the direction of the span occurred. The deformed shapes of W-truss No. II for each increment of live load as plotted from the deflection board records are shown in Figs. 4 to 13.

4. DISCUSSION OF RESULTS

Although a period of twenty days was allowed for the joints in the trusses to bed down in the first loading phase, the changes in deflection during this period were small, as shown in Table II.

Under dead load plus ceiling load plus design live load, the largest net deflection at the end of the seven days of sustained loading occurred at the point "F" (see Table III) on the lower chord of W-truss No. I. At this point the maximum value reached was 0.246 inch. This value is approximately one-third of the allowable deflection for a plastered ceiling. The recovery of deflection at this point, 24 hours after the removal of the live load was, however, only 48.7 per cent. Similar values for the net deflections and recoveries were obtained for the remaining three points on the trusses.

By comparing the deflections for the sixth and seventh days in Table III with the deflections in Table V for the initial readings and the readings after 24 hours, it can be seen that there was no significant change in the trend indicated in Table III when the live load was re-applied for the third loading phase.

Under a symmetrical loading, the largest net deflection occurred again at point "F" on the lower chord of W-truss No. I, when 1.33 times the live load was applied to the upper chords from "C" to "E". This maximum value was 0.267 inch (see Table V), or approximately 1/1080 of the span. The recovery of deflection at this point, 24 hours after the superimposed load was removed, was 41.9 per cent. The corresponding value for the largest net deflection on the lower chords when the same loading was applied to the opposite roof slope was 0.244 inch at point "G", and the recovery 24 hours after the removal of the superimposed load was 42.7 per cent.

During the test to failure, i.e. sixth loading phase, the wire and pulley deflection apparatus was used to obtain the deformed shape of W-truss No. II as the increments of one-half design live load were applied. The maximum vertical deflection reached before failure occurred took place at point "G" (Fig. 13) and reached a value of 0.56 inch. The corresponding value for point "F" was 0.50 inch. Both values were below the allowable deflection for a plaster ceiling of 1/360 of the span, or 0.80 inch. The fact that greater deflections did not occur can be explained by the resistance offered to vertical deflections by the tension developed in the lower chords under superimposed loads.

It will be noted when reviewing Figs. 4 to 13 that there was a trend toward progressive spreading at the supports as successive increments of loading were applied. The total outward movement in the direction of the span at point "A" prior to failure was 1/10 of an inch and the corresponding movement at point "E" was approximately 2/10 of an inch, thus giving a total elongation of the lower chord of 3/10 of an inch. The total deflections of the upper chords measured at right angles to the slope at point "B" and point "D" were 0.50 inch and 0.75 inch respectively.

When the lower chords failed under dead load plus ceiling load plus five times design live load, the test assembly and loading were supported so that photographs could be taken of the points of interest. Both lower chords of the two trusses failed at almost the same instant and at the same location. Figures 14 and 15 show the failures in W-truss No. I and No. II respectively. Additional features of the joint failures are shown in Figs. 16 to 19, which show the individual members that failed, after the trusses had been disassembled. Failure in each member was by shear and tension; the exact sequence of failure by these forces is difficult to determine.

5. CONCLUSIONS

(a) Trusses

Observations made on the performance of the two trusses tested would not normally provide sufficient evidence upon which to base definite conclusions, but, making due allowance for the variability in wood, the general remarks which follow would apply in the over-all picture of similar components:

> Firstly, because of the tensile forces induced in the lower chord members when the loads were applied, the vertical deflections of the members which normally support the ceiling in a house did not exceed the maximum allowable deflection of 1/360 of the span for exterior plaster finish, even at incipient failure;
>
> Secondly, the method used for fastening the joints of the trusses appears to be satisfactory, i.e. there was very little relative movement in the form of slippage between members until twice design load was reached;
>
> Thirdly, since five times the design live load recommended by the National Building Code, 1953, was reached before failure occurred, the strength of the trusses was more than adequate.

(b) Testing Method

In general, the method of testing the trusses was satisfactory. It was found that the standard lead weights were very convenient for this type of loading test. The loads could be applied and removed without difficulty, and when the weights were not being used they could be neatly piled to one side of the test assembly.

For the test to failure it was found that the pea-size crushed stone could be taken from the floor of the project area and transported by the overhead crane to the trusses without undue difficulty. Transporting of the crushed stone by the crane, however, was slow. Moreover, considerable preparations were necessary to confine the crushed stone to the tributary roof area, and, unless the crushed stone was dampened by water before it was shovelled into the bucket and dumped on the trusses, the dust from it was objectionable.

The accuracy of the load on the trusses at any particular time was questionable because of the frictional losses between the crushed stone and the bulkheads. The exact amount of friction developed during this test is not known; however, it is thought that the effect on the results is not appreciable.

6. GENERAL DISCUSSION

It may be useful to add some general thoughts on possible future work in this field.

The tests on these two trusses, which were designed according to accepted design methods, that is, using allowable stresses, showed that an excessively high load safety factor resulted (the load safety factor being the ratio of load at failure to design load). In many structures the load safety factor exceeds the stress safety factor (the stress safety factor being the ratio of stress at failure to the allowable stress in the material). Thus the problem of the choice of an adequate load safety factor arises in the use of the test results. In other words, what multiple of the design load should the trusses be able to carry ?

In considering this question it seems that a comparison of roof trusses with conventional roof constructions (joist and rafter construction) should be included in this study, as at the present time only trusses have to be designed according to accepted engineering principles. The test of the two trusses described in this report can, therefore, be regarded as the first step only in a general investigation of the strength provided by conventional as well as truss construction. It is the intention of this Division to continue the work reported here with further studies aimed at the development of an equitable basis for a balanced design in conventional and trussed roof constructions.

7. BIBLIOGRAPHY

Dalgleish, D. Timber Roof Trusses for Domestic Buildings. Commonwealth Experimental Station, Sydney, Australia. September, 1949.

Expansion-Attic Truss. House and Home. September, 1952. p. 110.

Hansen, Howard J. Design Loads for Wooden Roof Trusses. Agricultural and Mechanical College of Texas. Bulletin No. 74. Fourth Series, Vol. 13, No. 17. December 1, 1942.

Housing and Home Finance Agency. Wood Roof Trusses for Small Dwellings. Reprinted from HHFA Technical Bulletin No. 2 (January, 1948). United States Government Printing Office. Technical Reprint Series No. 2. Washington, U.S.A. February, 1950.

Stern, E. George. Nailed Trussed Rafter of 24-ft. Span and 3 in 12 Pitch. Virginia Polytechnic Institute, Wood Research Laboratory, Blacksburg, Virginia. April, 1954.

Timber Engineering Company. Design Manual for Teco Timber Connector Construction. Teco Technical Series No. 1. Washington 6, D.C. 1943 Ed., Revised 1950.

Ulrey, Harry F. Nails Make Good Joints for Trussed Rafters. Practical Builder. October, 1953. p. 111.

University of Illinois. Trussed Roof-Frame for 1½ Storey Houses. Small Homes Council. Urbana, Illinois.

University of Illinois. "W" Truss for 20'-8" to 32'-8" spans. Small Homes Council. Urbana, Illinois.

University of Illinois. 3/12-Slope for 20'-8" to 30'-8" spans. Small Homes Council. Urbana, Illinois.

TABLE I

LOADING SCHEDULE

Loading Phase	Magnitude of Test Load	Duration of Loading	Recovery Period
1	ceiling load plus dead load (shingles)	20 days	None (load to remain to failure)
2	ceiling load plus dead load (shingles) plus design snow load	7 days	24 hours
3	as in "2"	24 hours	24 hours
4	as in "1" plus 1.33 x design snow load C to E	24 hours	24 hours
5	as in "1" plus 1.33 x design snow load A to C	24 hours	24 hours
6	test to failure in increments of ½ x design live load (snow load)		

TABLE II

DEFLECTIONS UNDER DEAD LOAD PLUS CEILING LOAD

(Moisture Content = 7% and under)

Time	W-truss No. I				W-truss No. II			
	Settlement at Supports		Net Deflections		Settlement at Supports		Net Deflections	
	A	E	G	F	A	E	G	F
Initial	0.029	0.009	0.041	0.067	0.022	0.033	0.026	0.026
After 5 days	0.035	0.015	0.047	0.072	0.024	0.038	0.029	0.030
" 10 "	0.036	0.016	0.050	0.073	0.030	0.040	0.032	0.033
" 15 "	0.036	0.016	0.051	0.074	0.030	0.040	0.035	0.036
" 20 "	0.037	0.016	0.052	0.075	0.031	0.041	0.036	0.036

Note: All deflections are given in inches.

TABLE III

DEFLECTIONS UNDER DEAD LOAD PLUS CEILING LOAD PLUS ONE TIMES LIVE LOAD

(Moisture Content = 7% and under)

Time	W-truss No. I				W-truss No. II			
	Settlement at Supports		Net Deflections		Settlement at Supports		Net Deflections	
	A	E	G	F	A	E	G	F
Initial	0.145	0.099	0.215	0.232	0.126	0.136	0.186	0.207
After 1 day	0.151	0.103	0.222	0.237	0.133	0.140	0.195	0.215
" 2 days	0.153	0.105	0.225	0.239	0.135	0.141	0.197	0.217
" 3 "	0.153	0.105	0.227	0.240	0.136	0.142	0.200	0.220
" 4 "	0.154	0.106	0.227	0.240	0.137	0.143	0.201	0.221
" 5 "	0.155	0.107	0.229	0.244	0.139	0.144	0.201	0.221
" 6 "	0.156	0.107	0.231	0.245	0.139	0.144	0.202	0.222
" 7 "	0.156	0.108	0.235	0.246	0.140	0.144	0.205	0.224
ONE TIMES LIVE LOAD REMOVED								
Immediate	0.119	0.071	0.117	0.135	0.105	0.100	0.092	0.103
After 24 hours	0.114	0.067	0.112	0.126	0.101	0.100	0.086	0.096
Recovery after 24 hours:			52.3%	48.7%			58.0%	57.6%

Note: All deflections are given in inches.

TABLE IV

DEFLECTIONS UNDER DEAD LOAD PLUS CEILING LOAD PLUS ONE TIMES LIVE LOAD

(Moisture Content = 7% and under)

Time	W-truss No. I: Settlement at Supports A	W-truss No. I: Settlement at Supports E	W-truss No. I: Net Deflections G	W-truss No. I: Net Deflections F	W-truss No. II: Settlement at Supports A	W-truss No. II: Settlement at Supports E	W-truss No. II: Net Deflections G	W-truss No. II: Net Deflections F
Initial	0.152	0.105	0.235	0.249	0.135	0.141	0.205	0.222
After 24 hours	0.155	0.108	0.238	0.258	0.141	0.144	0.216	0.234
ONE TIMES LIVE LOAD REMOVED								
Immediate	0.120	0.072	0.127	0.140	0.105	0.099	0.099	0.111
After 24 hours	0.116	0.068	0.122	0.135	0.102	0.096	0.092	0.103
Recovery after 24 hours:			48.7%	47.7%			57.4%	55.9%

Note: All deflections are given in inches.

TABLE V

UNSYMMETRICAL LOADING

(Moisture Content = 7% and under)

Time	W-truss No. I				W-truss No. II			
	Settlement at Supports		Net Deflections		Settlement at Supports		Net Deflections	
	A	E	G	F	A	E	G	F
DEAD LOAD PLUS 1.33 TIMES LIVE LOAD C TO E, DEAD LOAD A TO C								
Initial	0.131	0.104	0.149	0.262	0.114	0.141	0.131	0.235
After 24 hours	0.132	0.108	0.152	0.267	0.117	0.145	0.133	0.243
1.33 TIMES LIVE LOAD REMOVED C TO E								
Immediate	0.118	0.072	0.114	0.157	0.104	0.101	0.092	0.121
After 24 hours	0.116	0.070	0.113	0.155	0.103	0.099	0.091	0.117
Recovery after 24 hours:			25.6%	41.9%			31.6%	51.7%
DEAD LOAD PLUS 1.33 TIMES LIVE LOAD A TO C, DEAD LOAD C TO E								
Initial	0.153	0.088	0.240	0.173	0.136	0.118	0.204	0.153
After 24 hours	0.155	0.089	0.244	0.173	0.139	0.119	0.212	0.156
1.33 TIMES LIVE LOAD REMOVED A TO C								
Immediate	0.120	0.072	0.145	0.134	0.106	0.100	0.101	0.113
After 24 hours	0.117	0.069	0.140	0.133	0.103	0.096	0.097	0.113
Recovery after 24 hours:			42.7%	23.1%			54.2%	27.6%

Note: All deflections are given in inches.

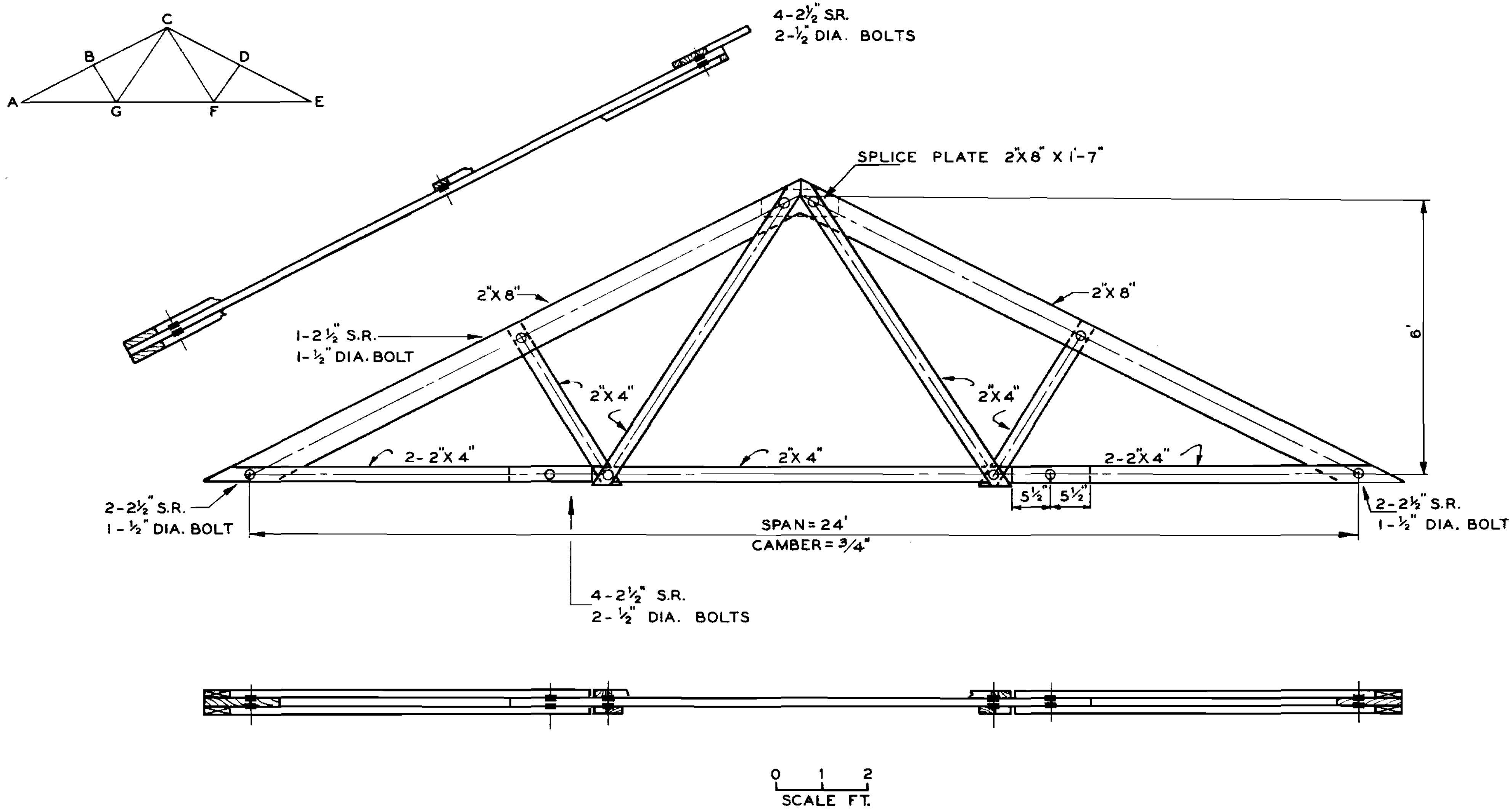


FIGURE 1

DETAILS OF W-TRUSS NOS. 1 & 2

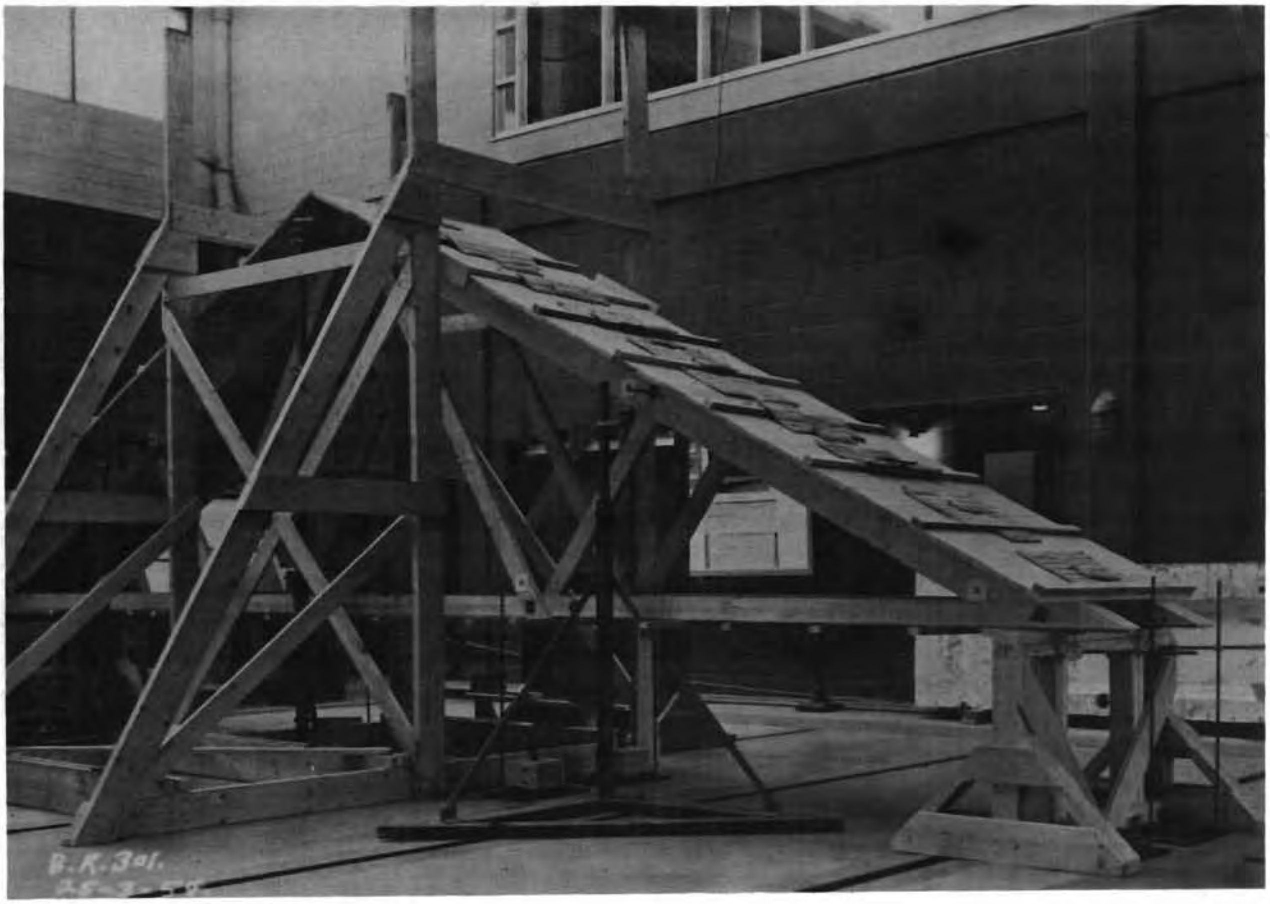


Fig. 2. Test assembly with ceiling load and dead load applied. Trusses free to deflect independently of stabilizing framework.

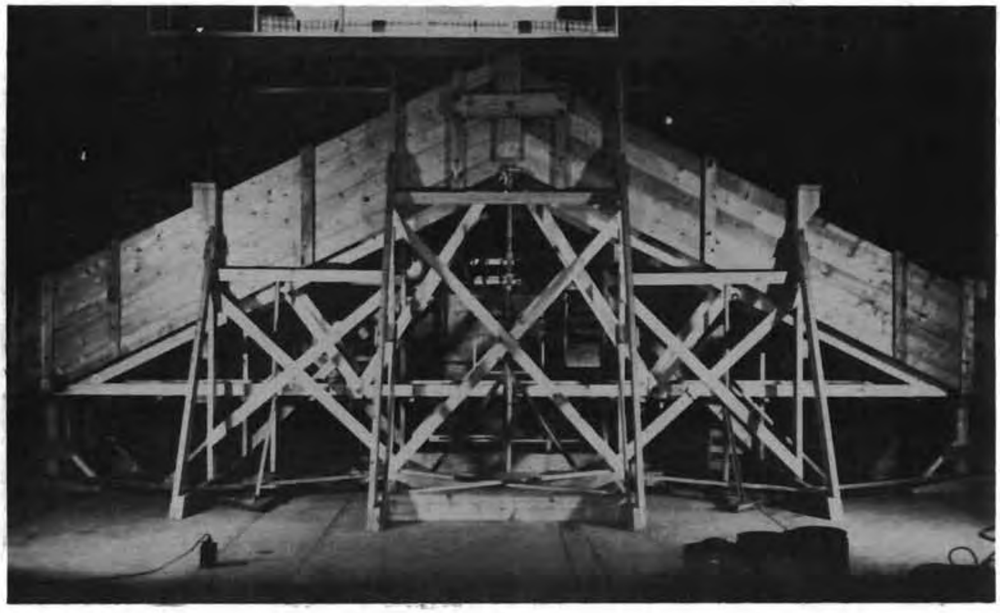


Fig. 2a. Test assembly with bulkheads in position.

(Photograph: Photogrammetry Section, Division of Applied Physics, N.R.C.)

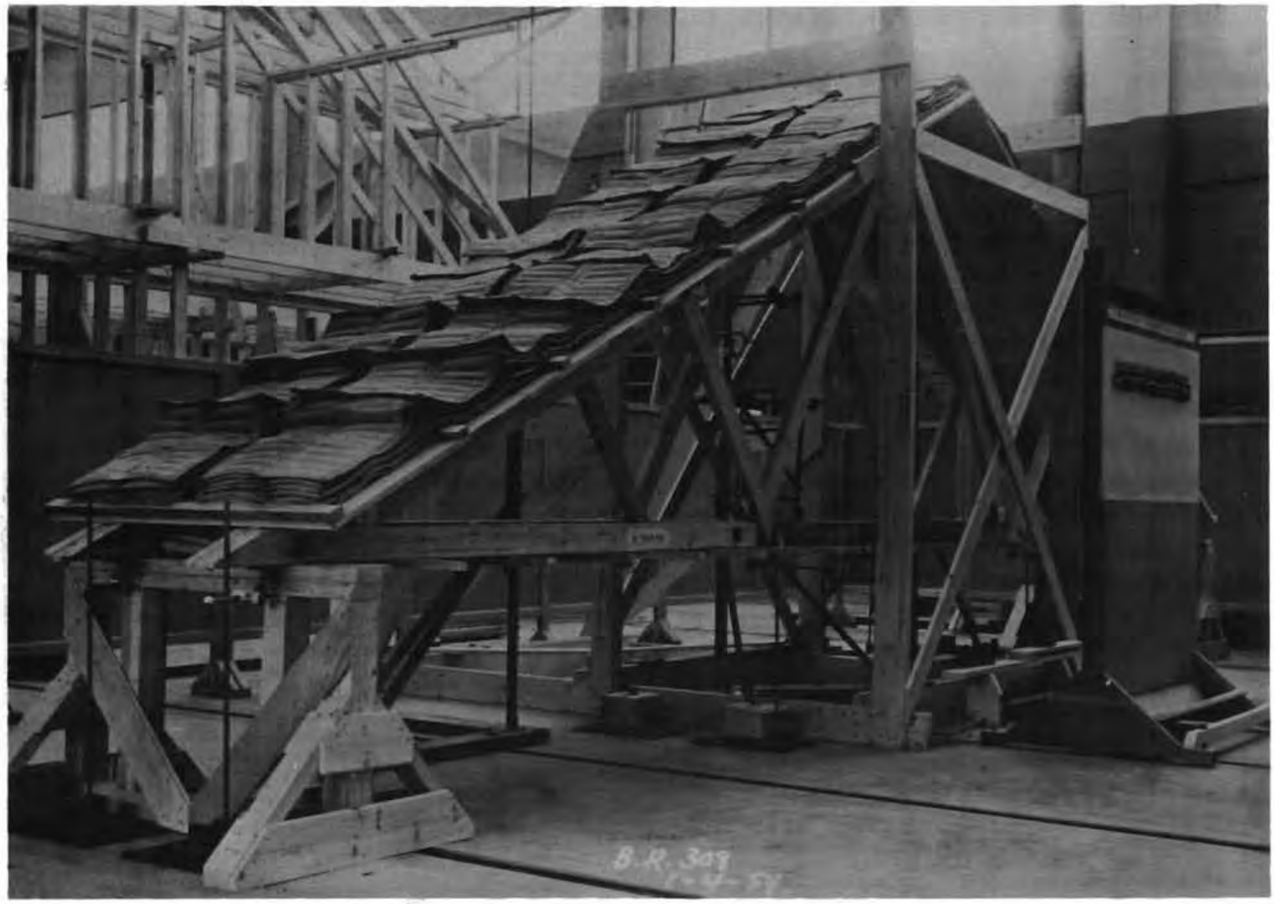


Fig. 3. Trusses loaded with ceiling load (10# per square foot) plus dead load (shingles = 2.1# per square foot) and design live load (snow load = 50.8# per square foot of horizontally projected roof area). Note deflection board and wire and pulley arrangement for recording deflections.

DEFORMED SHAPE OF W-TRUSS NO. 11

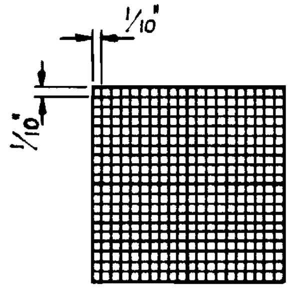
TEST TO FAILURE

LEGEND:

—— BEFORE LOADING

- - - AFTER LOADING

SCALE OF LENGTHS: 1" = 5'-0"



SCALE OF DEFLECTIONS

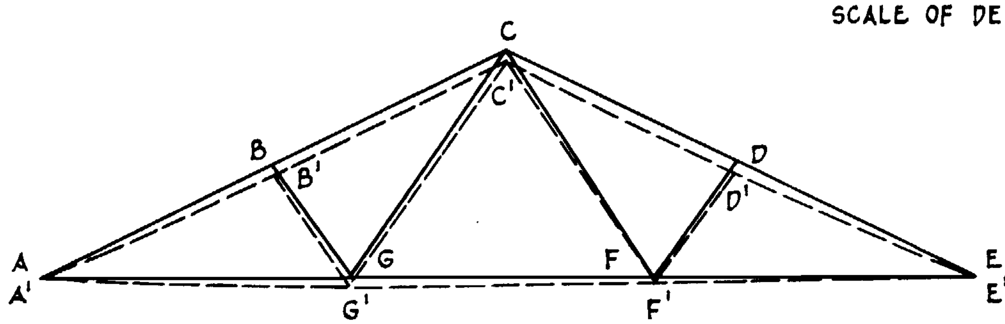


FIGURE 4 DEAD LOAD + CEILING LOAD + 1/2 LIVE LOAD

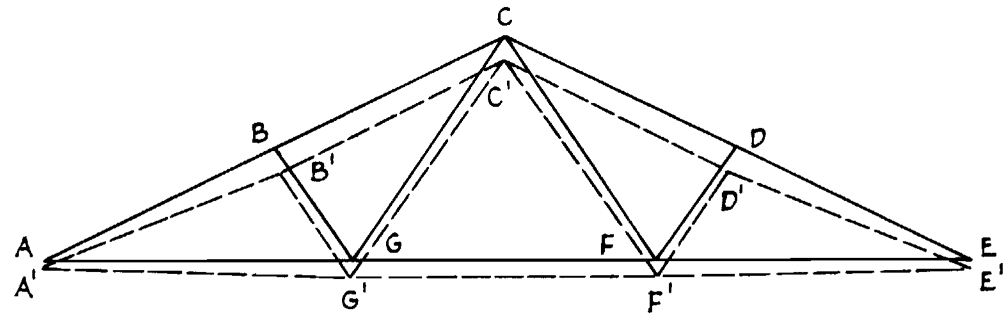


FIGURE 5 D.L. + C.L. + 1 SNOW LOAD

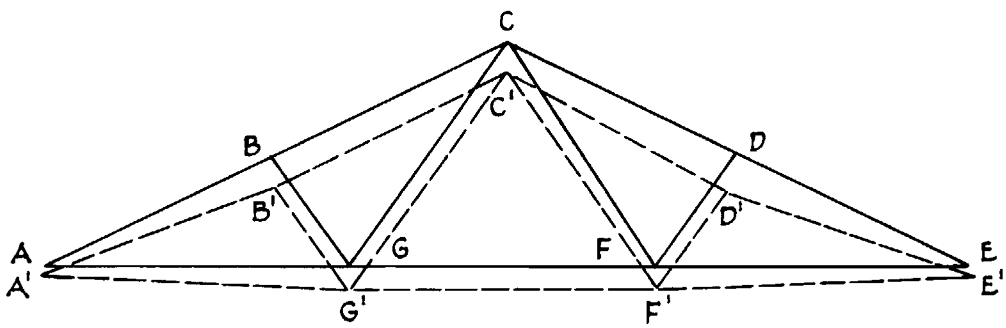


FIGURE 6 D.L. + C.L. + 1½ SNOW LOAD

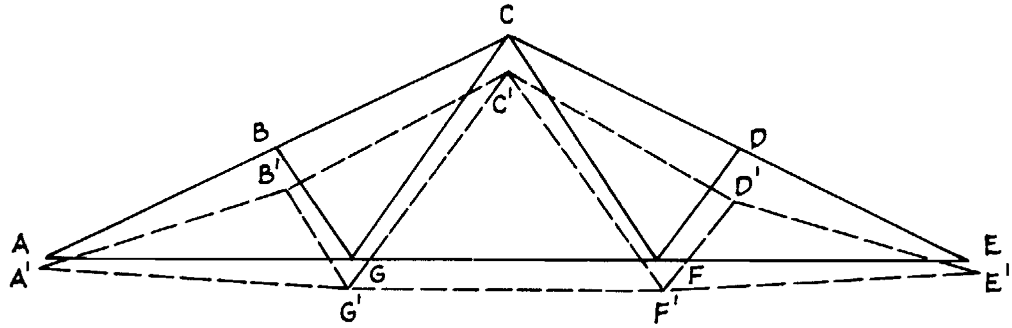


FIGURE 7 D.L. + C.L. + 2 SNOW LOAD

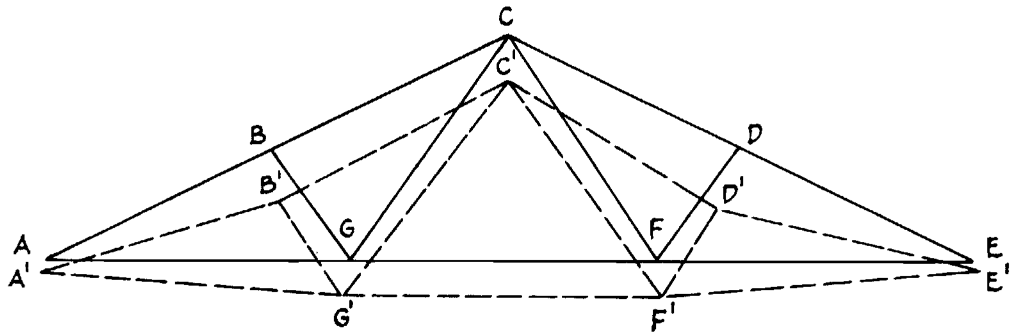


FIGURE 8 D.L. + C.L. + 2½ SNOW LOAD

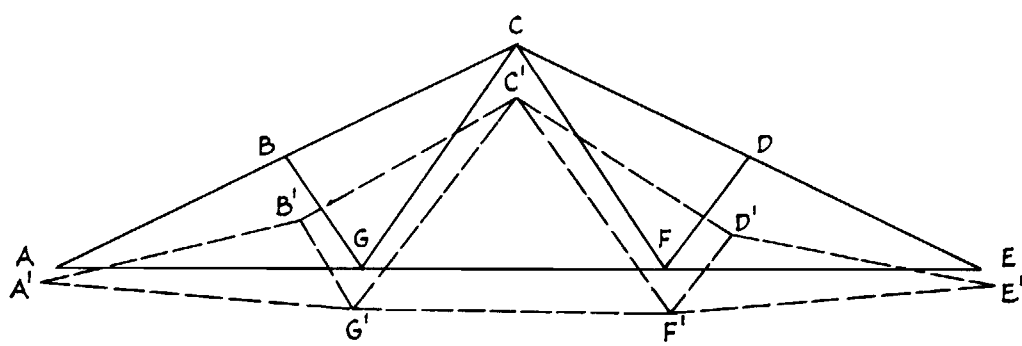


FIGURE 9 D.L. + C.L. + 3 SNOW LOAD

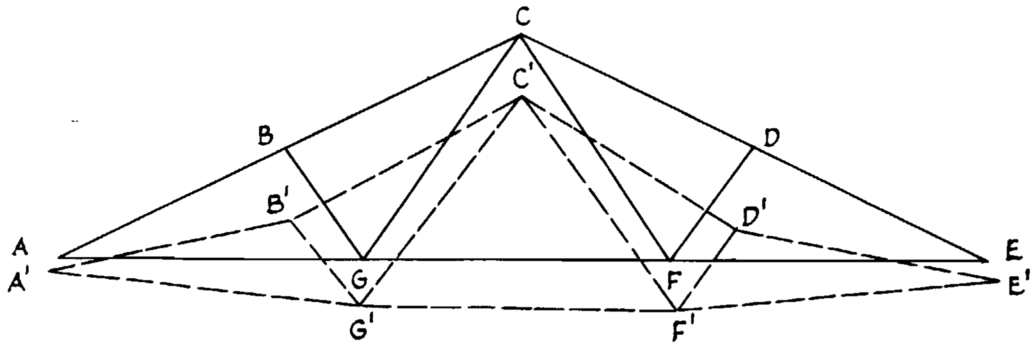


FIGURE 10 D.L. + C.L + 3½ SNOW LOAD

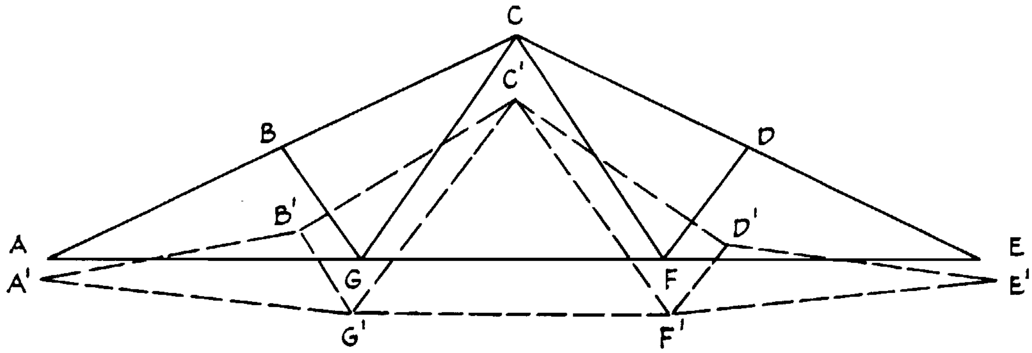


FIGURE 11 D.L. + C.L. + 4 SNOW LOAD

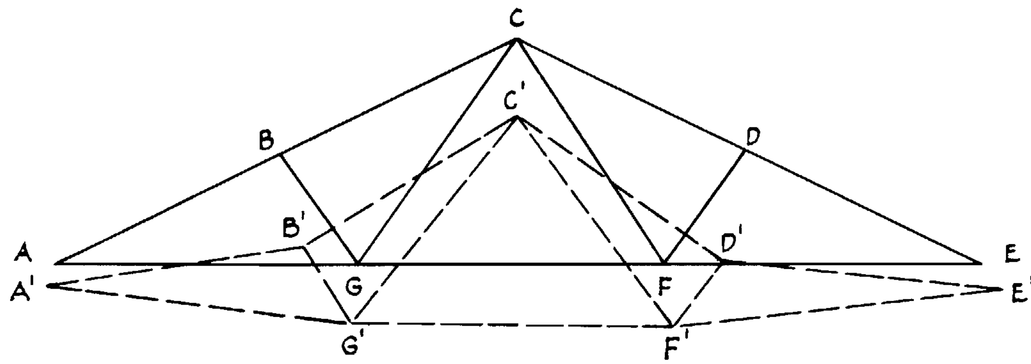


FIGURE 12 D.L. + C.L. + 4½ SNOW LOAD

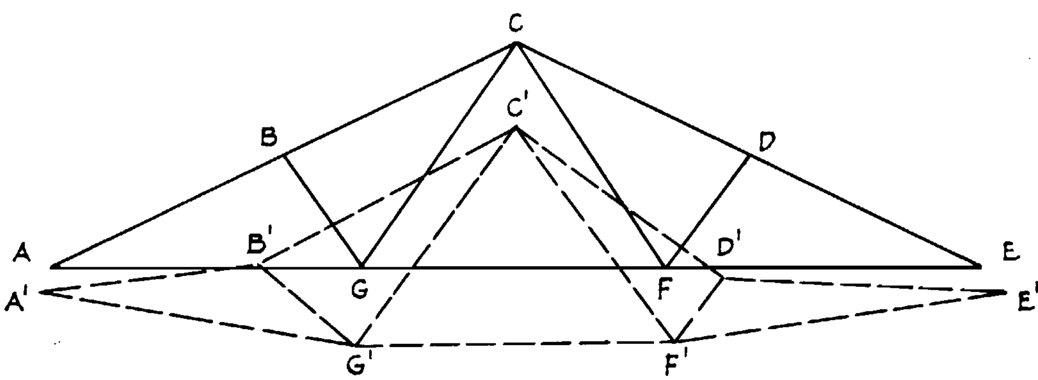


FIGURE 13 D.L. + C.L. + 5 SNOW LOAD
INCIPIENT FAILURE

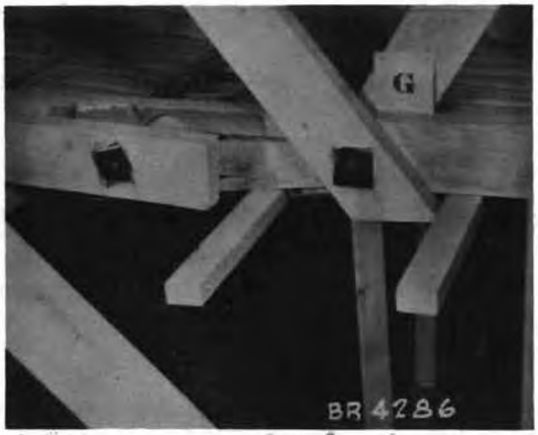


Fig. 14. Failure at joint in lower chord of W-truss No. I under dead load plus ceiling load plus five times live load (snow load)

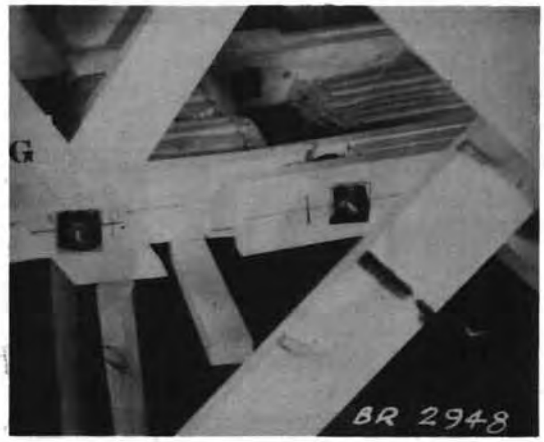


Fig. 15. Failure at joint in lower chord of W-truss No. II under dead load plus ceiling load plus five times live load (snow load)

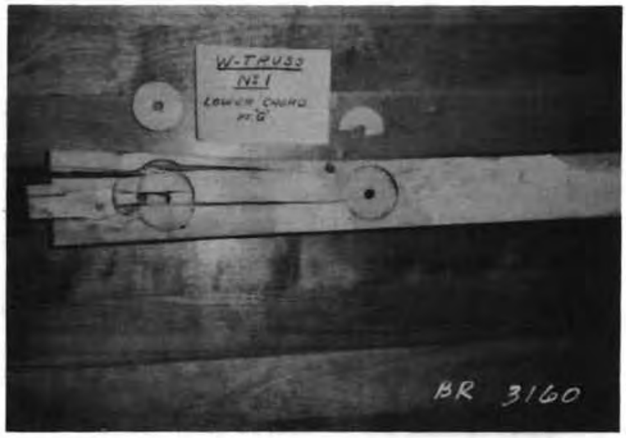


Fig. 16. Failure in lower chord member of W-truss No. I, top view of member

Fig. 17. Failure in lower chord member of W-truss No. I, bottom view of member

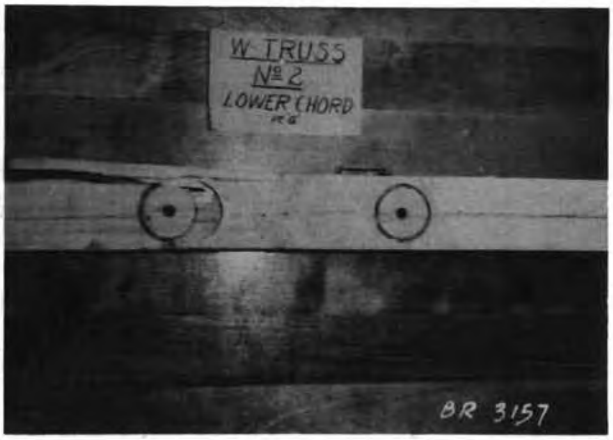


Fig. 18. Failure in lower chord member of W-truss No.II, top view of member

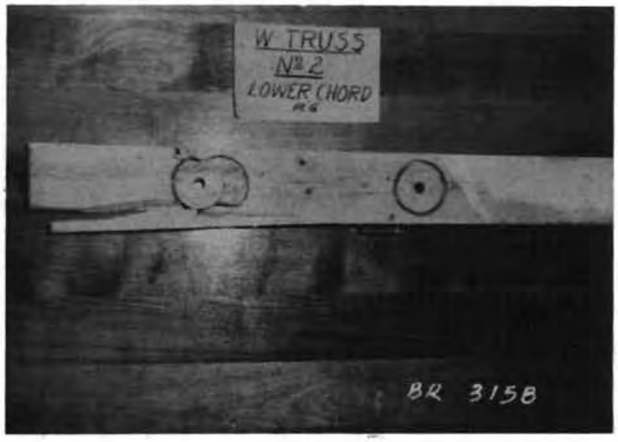


Fig. 19. Failure in lower chord member of W-truss No.II, bottom view of member