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# LOADING TESTS ON CONVENTIONAL AND TRUSSED ROOF CONSTRUCTIONS

(Third Progress Report)

by

A.T. Hansen

ANALYZED

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## PREFACE

The structural performance of roof components is being studied by the Division as part of a reassessment of the structural design of houses. Previous reports have dealt with a structural test of a full scale house, the structural performance in the laboratory of a W truss design, and a comparison of trusses and conventional rafter constructions. This work has led to a reappraisal of the loading requirements for house roof systems and to a program of field measurements on the actual snow loads occurring on roofs.

In the meantime attention is being directed in the laboratory to further studies of the performance of roof constructions. The performance of a lightweight nailed truss design on three spans and two roof slopes under short-time loading is now reported. Results are given also for relatively long-term loading of the nailed truss and of one of the conventional constructions previously studied.

Ottawa,
June, 1957.

N.B. Hutcheon,
Assistant Director.

# LOADING TESTS ON CONVENTIONAL AND TRUSSED ROOF CONSTRUCTIONS

(Third Progress Report)

by

A.T. Hansen

## 1. Introduction

The preceding two reports on roof frame testing, DBR Internal Reports 81 and 113, have dealt chiefly with the short-term loading tests on various types of conventional roof framing systems and several types of lightweight roof trusses. These investigations have been limited mainly to structures having a 24-ft. span and a 5/12 roof slope. Most of the tests on trusses have been on trusses similar to those developed in the United States but made with materials common to Eastern Canada.

This report presents the results of investigations of a type of lightweight nailed W truss, the design of which has been influenced by the results of tests on conventional constructions, keeping in mind the need for producing a truss, both economical and simple to construct either in a factory or on the site. The economic considerations were influenced by the relatively high cost of plywood in Eastern Canada. For Western Canada some alterations in design could be made, to make greater use of plywood gusset plates. It is hoped that such designs may be included in future investigations on roof trusses.

A nailed construction was chosen rather than glued or split-ring constructions as this type, it is thought, lends itself to on-site construction more than the other types, and no special equipment is necessaty to fabricate it.

The report presents two phases of laboratory investigations. The first part describes the short-term loading of the nailed truss for various spans and slopes. The second part deals with the effects of relatively long-term loading on nailed trusses and conventional construction, as well as long-term recovery characteristics of the structures after the loads are removed.

## 2. Short-Term Tests

### (a) Description of Trusses and Criteria of Acceptable Performance

On the basis of the tests on conventional construction the writer considered that a reasonable minimum failure load for trusses should be 100 p.s.f. snow load plus 10 p.s.f. ceiling load and 5 p.s.f. roof dead load. This value was chosen because it is in the neighbourhood of the failure strength for the strongest conventional construction (see DBR Internal Report 113, page 4) and could therefore be considered a conservative figure.

It is not as easy, however, to establish the values for the maximum allowable deflections of trusses by direct comparison with conventional construction, since there are considerable differences in the deflection characteristics of the two types of framing. With conventional construction, for example, the roof loads are not transmitted to the ceiling joists in the same manner as with truss construction. The vertical deflections of the ceiling joists with conventional construction are caused solely by the ceiling loads (except when dwarf walls or knee walls are used to support the rafters), whereas with truss construction both the roof loads and ceiling loads produce deflections in the lower chords. The roof loads with the conventional construction cause the ceiling joists to spread horizontally and the ceiling joist splice to separate. This separation tends to crack the ceiling plaster. It is difficult, therefore, to judge what distortions in conventional construction would be equivalent to any given truss deflection in terms of similar degrees of damage to the ceiling finish.

The length of time that a test snow load should be applied to a structure to be representative of actual snow loads is not known. It is reasonable to assume, however, that the 5-minute loading intervals used in the short-term tests are not representative of actual snow load durations in practice. The 5-minute loadings, however, are convenient in carrying out rapid comparisons between structures. In order to use the 5-minute loadings to evaluate truss deflection characteristics, it was decided to set relatively high standards for the deflection under short-term loads. It was arbitrarily decided, therefore, that the maximum allowable deflection in these test structures should not be greater than 1/360 of the span with a snow load of 80 p.s.f., a dead load of 5 p.s.f. and a ceiling load of 10 p.s.f., after 5 minutes of loading.

On the basis of the tests reported in DBR Report 81, it was estimated that the nailing required to produce this stiffness should be the design nailing for a 35 p.s.f. snow load and roof dead load plus 10 p.s.f. ceiling load, using the National Building Code design requirements.

It was also decided that the top and bottom chords should be kept to a reasonable minimum size (2 by 4's) and this size would be increased only if the strength or deflection of the structures fell below the arbitrary limits.

No 1 Eastern spruce was used for the 2 by 4 members and No. 3 for the 1-in. thick members. No. 1 spruce is presumably the commonly used lumber for roof framing in Eastern Canada.

Figures 1 to 6 show the truss designs for 24-, 26- and 28-ft. spans and 4/12 and 5/12 slopes. The nailing for these trusses was calculated on the basis of 24-in. truss spacings.

(b) Description of Test Equipment

Simulated snow loads were applied to the top chords by means of 8 equally spaced hydraulic tension jacks, and the ceiling loads applied by means of lead-filled bags placed directly on the bottom chords in the same manner as described in DBR Report 81. The trusses, as in previous tests, were tested in pairs and sheathed with 1 by 6 lumber.

Previous tests on trusses indicated that the performance of nailed trusses was only slightly influenced by the type of end supports. The strength and stiffness of trusses tested on roller supports appeared to be roughly 10 per cent less than for trusses tested on supports bolted to the floor to restrict horizontal movement. The degree of restriction to horizontal movement of the roof structure that may be provided by the walls in a complete house is not known, but the value must lie between the extremes provided by roller supports and fixed supports. Since the effect of the type of end support on the performance characteristics of nailed trusses caused by the type of end support is relatively small, it was decided to test all trusses on roller supports only. The values so obtained should be conservative.

(c) Instrumentation

The truss deflections were measured by means of piano wire strung along the top and bottom chords of each truss and held taut by weights hung from the ends of the wire. Scales to measure the deflection of the trusses relative to the wire were placed at each panel point and at the centre of each panel. The peak deflections were measured by suspending indicator weights from the peaks and measuring the deflections of the weights on a recording board directly below.

The separation of the bottom chord at the splice and the horizontal spread of the trusses were measured by means of dial gauges.

(d) Testing Procedure

Since the weight of the test equipment suspended from the top chords was about equal to the shingle load, no extra allowance for this was made. All gauge readings were noted before the ceiling load was applied. A 10 p.s.f. ceiling load was then placed on the lower chord and allowed to remain for the duration of the test. Five minutes after the ceiling load was applied all readings were again noted.

The hydraulic loads were then applied in increments simulating 20 p.s.f. snow loads. Five minutes after each loading increment the readings were again noted. The loading was increased until a total of 40 p.s.f. snow load was applied, after which the hydraulic loads were reduced to zero. The loading was again increased in increments of 20 p.s.f. until failure occurred.

Three tests involving six trusses were carried out for each span and roof slope to provide average values.

(e) Recording of Results

All dial gauge readings were recorded to the nearest .001 inches and deflection measurements to the nearest .01 inches.

The results of the tests are given in Tables I and II.

The over-all deflections of the trusses at 0, 40, and 80 p.s.f. snow loads are shown graphically in Figs. 15 to 35.

The mid-span deflections of the lower chords are shown for the various applied loads for each test in Figs. 7 to 13. The curves do not show the residual deflections of the trusses after removal of the 40 p.s.f. snow loads during the first phase of the test, as the inclusion of these deflections complicates the curves. The residual deflections are recorded, however, in terms of per cent recovery, in Table I. The per cent recovery was calculated using the following formula:

Per cent recovery = [(deflection at 40 p.s.f. snow load and 10 p.s.f. ceiling load) -(residual deflection after snow load removal)] x 100 ÷ [(deflection at 40 p.s.f. snow load and 10 p.s.f. ceiling load) -(deflection with ceiling only)] .

It is to be noted that the ceiling load was allowed to remain for the duration of the test and the recovery is based on removal of snow load only.

Table II is a condensation of Table I. The results in Table I are the average values of the performance of the two trusses in each test.

(f) Results of Tests

(i) Failure Loads - As may be seen in Tables I and II, the failure loads compare quite favourably with the arbitrary 100 p.s.f. minimum. The failure loads varied from an average of 130 p.s.f. snow load for the 26-ft., 4/12 slope truss, to 165 p.s.f. for the 24-ft., 4/12 slope truss. The 24-ft. span trusses were stronger by a considerable margin with both the 4/12 and 5/12 slopes than the 26- and 28-ft. span trusses which appeared to be about the same strength for similar slopes.

When 2 by 5's were substituted for the 2 by 4's in the top chord of the 28-ft. span, 4/12 slope truss, the apparent average increase in the snow load at failure was about 8 per cent.

(ii) Deflections (see Tables I and II) - With one exception the deflection ratios of the lower chords of the trusses were quite consistent, regardless of the span. The exception was the 28-ft. span, 4/12 slope truss which was appreciably less rigid than the others.

The average deflection ratios for all other types of trusses varied from 1/910 to 1/870 with the 5/12 slope supporting a 40 p.s.f. snow load, and from 1/820 to 1/800 with a 4/12 slope with the same snow load. For an 80 p.s.f. snow load the average deflections varied from 1/440 to 1/460 for the 5/12 slope and from 1/390 to 1/400 for the 4/12 slope trusses.

The 28-ft. span, 4/12 slope trusses with 2- by 4-in. top chords, however, had an average deflection ratio of 1/600 for a 40 p.s.f. load and 1/320 for an 80 p.s.f. snow load. When 2- by 5-in. top chords were used in place of the 2 by 4's the average deflection ratios were 1/690 to 1/370 respectively. This amounted to an average increase in stiffness of about 15 per cent with the larger top chords.

As can be seen from Table II, the span appeared to have little influence on the stiffness of the trusses. There is a marked difference, however, between the deflection ratios for a change in roof slope, with the 5/12 slope trusses generally being about 10 per cent stiffer than the 4/12 slope trusses.

It is necessary to add a word of caution here about accepting these observations as statistically correct since the number of tests were relatively small. It is believed, however, that the variations within the groups of three tests were small enough compared to the spread of general results obtained to justify confidence in the conclusions. Little attempt was made to correlate the effect of moisture content or wood density to the deflection characteristics. This could be quite an involved study and, therefore, it was not attempted at this time.

(iii) Lower Chord Splice Separation - The measurements taken during the tests indicated that there was very little relative movement at the lower chord splice, the value being in the order of a few thousandths of an inch in most cases for loads up to 80 p.s.f.

(iv) Recovery (see Tables I and II) - The per cent recovery observed in these tests and averaged for all trusses of one kind varied from 69 per cent to 78 per cent. As might have been expected, there appeared to be little correlation between the stiffness of the trusses and the per cent recovery after load removal. In fact, the trend seemed to be that the less stiff the truss the higher the per cent recovery.

(v) Horizontal End Movement (see Tables I and II) - The horizontal end movement, which may be considered in part as a measure of the stress in the bottom chord, behaved as could be expected with the average movement at 40 p.s.f. snow load in the 24-ft. span, 5/12 slope truss of .062 in. to .106 in. for the 28-ft. span, 4/12 slope truss (2- by 4-in. top chords). The results showed that the average end movement became greater as the span increased and decreased as the slope became steeper.

(vi) Types of Failure - Almost all failures were due to failure in the main members with no failures occurring due to nailing. Of the 21 tests reported, 15 failures were caused by the upper chord breaking, 3 were due to tension failures in the lower chord, 2 were due to lateral buckling of the top chords and 1 caused by a shear failure in the heel gusset plate. This would indicate that the nailing was quite adequate to develop the full strength of the truss members.

(vii) Moisture Content - With the small range of moisture contents of the wood in these tests (approximately 7 to 12 per cent), no conclusions can be drawn as to the effect of moisture content on the strength or stiffness of the trusses.

## 3. Long-Term Tests

### (a) Introduction

It was thought that it would be useful to obtain data on the deflection characteristics of trusses and conventional construction under relatively long-term loads. Since long-term loading does not lend itself to the hydraulic loading method, it was decided to use concrete blocks to simulate the snow and dead roof loads.

The purpose of these tests was to determine the long-term deflection characteristics inherent in this particular truss design and to relate the deflection characteristics observed in short-term tests to those that may be expected under long-term loading. At the same time, it was convenient to examine the characteristics of conventional and truss construction on a comparative basis and to attempt to relate the relative sufficiency of each on a long-term basis.

(b) Description of Test Structures

(i) Trusses - Due to limited working space it was possible to test only a limited number of structures in this series. The 26-ft. span, 4/12 slope truss, spaced 24 in. o.c., was chosen for the test trusses (Fig. 4). The 26-ft. span was selected as it represented an average of those trusses tested in the short-term test. The 4/12 slope was selected principally because it produced greater deflections than the 5/12 slope.

(ii) Conventional Constructions - The conventional test structures were of Type I construction (see Figs. 36 and 39 in DBR Internal Report 81), i.e., of 24-ft. span 5/12 slope with 2- by 6-in. rafters and joists placed 16 in. o.c. and 2- by 4-in. collar ties at mid-rafter height. Field observations have shown this to be the most commonly built conventional construction.

The long-term tests on trusses revealed some lateral instability of structures tested in pairs, as the top chords tended to buckle under sustained loading expecially at higher loads. It was decided, therefore, to increase the lateral stiffness by sheathing the conventional structures with 1/2-in. plywood sheets instead of the 1- by 6-in. board sheathing.

In all cases roller supports were used under one end of the test structures for both conventional and truss constructions.

(c) Test Equipment

(i) Trusses - Loading for the first hour was applied in the same manner as in the short-term tests (i.e., snow loads were applied by tension jacks and ceiling loads by lead-filled bags). After one hour, the hydraulic loading equipment was removed and loads were re-applied using concrete block for the snow loads and lead-filled bags for the ceiling loads.

(ii) Conventional Constructions - The loading equipment for the conventional constructions consisted of concrete block for the roof loads and the usual lead-filled bags for the ceiling loads. The hydraulic jacks were not used for reasons explained later.

(d) Instrumentation

(i) Trusses - The deflections were measured against a piano wire strung along the top and bottom chords in the same way as in the short-term tests. The peak deflections were measured by a rigid strap fastened to the peak and hung so that the lower end projected below the horizontal wire against which the lower chord deflections were measured. A graduated paper was taped to this strap so that its movement relative to the wire could be read. This means of measuring the peak deflections was decided on because, as the trusses were to be moved during the process of testing, it was necessary to take all measurements independent of any floor supports. No measurements were taken of the long-term bottom chord splice separation or the horizontal movement of the trusses, as it was thought that gauges placed for these measurements would be disturbed when the trusses were moved from the hydraulic testing area to the location where they were loaded with concrete blocks.

(ii) Conventional Construction - The rafter deflections were taken at the mid-rafter span by piano wire strung along the lengths of the rafters. Dial gauges were also placed to measure the separation of the ceiling joist splice and the horizontal end movement of the structures. The peak deflections were measured by suspending indicator weights from the peaks and measuring their positions on graduated paper gauges immediately below.

(e) Testing Procedure

(i) Trusses - Four pairs of trusses were test loaded. The trusses were tested at 24 in. o.c. and sheathed with nominal 1-in. sheathing. One pair was loaded with 20 p.s.f. snow load, one with 40 p.s.f. snow load, one with 60 p.s.f. snow load and one with 80 p.s.f. snow load. These loads are in addition to the shingle load and 10 p.s.f. ceiling load that were applied in each case.

The trusses were first loaded by tension jacks and lead-filled bags as in the short-term tests. Deflection readings were taken immediately after the loads were applied and after 5 minutes, 15 minutes, 30 minutes, and 60 minutes of loading. After one hour, the loads and test equipment were removed and the structure was then loaded with concrete block (roof loads) and lead-filled bags (ceiling load). These loads remained for 30 days during which deflection readings were taken at increasing intervals of time.

The structures were first loaded with the tension jacks before being loaded with concrete block to provide an accurate deflection curve for the first hour of loading, since the placing of concrete block on these structures took 15 to 30 minutes. This procedure incidentally, provided an opportunity for comparison of the loads provided by the two methods.

After the structures had been loaded for 30 days, the loads were removed and residual deflections in those structures which had survived the test loading were noted at increasing time intervals for 30 days.

The loads were then re-applied to these test structures and deflection measurements noted periodically to determine the time necessary to reach the maximum deflections recorded in the original 30-day loading test.

(ii) Conventional Construction - Two conventional structures were loaded, one with a 20 p.s.f. snow load and another with a 40 p.s.f. snow load. These loads were in addition to the dead roof loads and ceiling loads.

Since the joists and rafters were spaced only 16 in. o.c. and the span only 24 ft., the total number of blocks necessary in each loading was reduced. It was thought, therefore, that the loads could be applied rapidly enough with the concrete blocks so that hydraulic loading would not be necessary during the first hour as in the long-term truss tests.

The roof loads were applied, therefore, by means of concrete block and the ceiling loads by lead-filled bags.

Readings of the gauges were taken immediately after completion of the loading and after 5 minutes, 15 minutes, 30 minutes, 1 hour and at increasing time intervals for 30 days. Then the loads were removed and the residual distortions in the structures recorded at increasing time intervals for 26 days.

(f) Recording of Results

(i) Trusses - The results of the long-term truss tests are recorded in Table III. The deflections of the mid-span of the lower chord are plotted against time in Fig. 36.

(ii) Conventional Construction - The results of the long-term tests on conventional constructions are recorded in Table IV.

The conventional constructions were supported at mid-span to simulate the support of a bearing partition. Measurements of the deflection of the lower chord of these structures, therefore, did not provide a suitable basis for comparison with trusses. As a measurement of the long-term performance, it was decided to plot the peak deflections and the separation at the joist splice against time (Fig. 37).

(g) Results of Long-Term Tests

(i) Trusses Under Load - Observations of deflections after 5 minutes' loading in the hydraulic test phase showed that the trusses in these tests were considerably less rigid than for identical trusses loaded for the same time interval in the short-term tests. For example, in the short-term test results reported in Table II, the deflection ratios averaged 1/390 and 1/800 for 80 p.s.f. and 40 p.s.f. snow loads respectively. The deflection ratios of the trusses in the long-term test series after 5 minutes of loading were 1/280 and 1/570 for the same loads (calculated from Table III). This was quite unexpected as the moisture contents of all the trusses showed little difference. Upon closer examination of the structural elements of the trusses it was noted that the wood in the long-term test trusses appeared to be a fast-growth, light-density spruce, while for the short-term test trusses the wood was usually more dense. As this was the only apparent difference between the short-term and long-term test trusses it is suspected that the difference in wood densities might have caused the difference in rigidity in structures.

After a period of 12 days the trusses loaded with the 80 p.s.f. snow load collapsed due to lateral buckling of the top chords. The failure was not a true structural failure since at the time of the collapse there was very little visual damage to the joints or members. The collapse occurred because the 1- by 6-in. sheathing did not provide sufficient lateral stability for the trusses (Fig. 44). At the end of 22 days' loading, the trusses loaded with 60 p.s.f. snow load collapsed due to the same lateral instability (Fig. 45). It is thought that neither collapse would have occurred if a sufficient number of trusses tied together with sheathing had been loaded instead of a single pair. Sheathing the trusses with sheets of plywood rather than board sheathing would have the same effect.

The results of the long-term truss tests are shown in summary in Table III and Fig. 36.

It can be seen from Table III that, except for the first hour's loading, the per cent increase in truss deflections for any given time interval is remarkably the same for all applied snow loads. The per cent increase in deflection is approximately 6 per cent after 1 hour's loading, 26 per cent after 1 day, 55 per cent after 1 week, and 96 per cent after 1 month when compared to the deflections after 5 minutes.

Whether or not these values also would apply to trusses of different spans and slopes is questionable, but it is thought that they give at least an approximation of what to expect with long-term loads. In a rough way, therefore, one can apply these per cent deflection increases to the short-term test results to obtain some indication of the probable deflection of the trusses loaded for any given time interval up to 1 month.

(ii) Trusses After Load Removal - Table III summarizes the observations made after the loads were removed. These results are for the trusses loaded with 20 p.s.f. and 40 p.s.f. snow loads since the other structures did not survive the 30-day loading period.

It was interesting to note that the recovery of the trusses was not instantaneous but continued for a long time after the loads were removed. For example, with the 20 p.s.f. snow load and 10 p.s.f. ceiling load removed, the instantaneous recovery was 43 per cent, after 1 hour it was 47 per cent, after 1 day, 51 per cent, after 1 week, 56 per cent, and after 1 month 58 per cent. The recovery after the 40 p.s.f. snow load and 10 p.s.f. ceiling load were removed was 51 per cent immediately after load removal, 54 per cent after 1 hour, 58 per cent after 1 day, 63 per cent after 1 week, and 67 per cent after 1 month. At the end of 1 month the rate of recovery was very small and, in the case of the trusses that had supported 20 p.s.f. snow load, it appeared to have almost stopped.

(iii) Trusses Reloaded - The results of these tests are shown in Table III. When the trusses were again loaded with the same loads they had supported in the original 30-day loading phase, the immediate deflection of the trusses supporting the 20 p.s.f. snow load was about 87 per cent of the maximum reached in the first 30-day loading period. After 1 week, the deflection reached 97 per cent, and after 12 days it was equal to the maximum deflection during the 30-day loading.

The trusses reloaded with the 40 p.s.f. snow load immediately reached 84 per cent of the maximum deflection recorded in the 30-day loading test, After 1 week this value reached 94 per cent and after 12 days the deflection reached about 97 per cent of the maximum deflection of the 30-day test.

It was not possible to continue observations on these structures as planned, as they had to be dismantled to permit the construction of a test floor in this laboratory.

(iv) Conventional Construction Under Load - The results of the loading tests are shown in Table IV and Fig. 37. It can be seen that with the structures loaded with 20 p.s.f. snow load the joist splice separation after 5 minutes was approximately 1/32 in. The increase in separation after 1 hour was 5 per cent; after 1 day it was 35 per cent; after 1 week, 81 per cent, and after 1 month 194 per cent.

The ceiling joist separation of the structures loaded with 40 p.s.f. snow load was approximately 7/64 in. after the load had been applied for 5 minutes. This separation increased 6 per cent after 1 hour, 32 per cent after 1 day, 114 per cent after 1 week, and 252 per cent after 1 month.

The peak deflections followed somewhat the same trend as seen in Table IV. This was expected since the peak deflections are to a large extent a measure of the splice separation.

(v) Conventional Construction After Load Removal - The results of these observations may be seen in Table IV. The initial recovery in the joist splice separation after the 20 p.s.f. snow load and 10 p.s.f. ceiling load was removed, was about 45 per cent, after 1 day 48 per cent, after 1 week 56 per cent, and after 26 days about 57 per cent.

The corresponding per cent recovery of the separation in the joist splice after the 40 p.s.f. snow load was removed was 17 per cent immediately, 19 per cent after 1 week and 21 per cent after 26 days.

It was necessary to discontinue observations after 26 days as the structures had to be dismantled to permit the construction of a test floor in this area.

## 4. Economic Study

The results of this study are reported in Table V. The trusses were built by one man, a carpenter, and the time does not include any allowances for the cutting of pattern pieces or the original laying-out of the truss pattern. A power hand saw was used for cutting all structural members and a bench saw for cutting the gusset plates.

No jigs were used in fabricating the trusses since the number of any one type of truss was quite small (6 of each). The truss outline in each case was marked out on the floor.

It is believed that the use of jigs would enable speedier assembly and the use of two or more carpenters instead of one might increase the over-all efficiency. It was thought, however, that the inclusion of the cost study might be useful in this report to give at least a rough idea of the cost of such structures.

For the sake of uniformity, the wage rates and costs of materials are the same as quoted in DBR Internal Report 81, Table 3.

## 5. Discussion of Results

The question of how long a structure should be loaded or how great the load should be to simulate actual snow load conditions is still a matter of opinion so, at present, the best one can do in assessing the adequacy of truss performance is to compare its performance as far as possible with conventional construction.

From the standpoint of strength, there is little doubt that the performance of these trusses is better than the strongest type of conventional construction.

There is, naturally, no direct comparison to be made in the deflection characteristics since conventional and truss constructions differ. The conventional construction is designed to have a bearing partition somewhere near the centre of the span while the truss is not. Probably one of the first places in which damage would become apparent in the interior finish, due to structural distortion in conventional construction, would be near the joist splice at the bearing partition. The relative movement at the partition splice necessary to produce visible damage to the ceiling finish is believed small; its actual value can only be estimated. With the conventional structures loaded with 20 p.s.f. snow load the splice displacement after 1 hour is approximately .039 in., while after 1 month it is about .104 in. or approximately 3/32 in. For a 40 p.s.f. snow load the displacement is .116 in. after 1 hour's loading and .371 in. or nearly 3/8 in. after 1 month. Any of these values should produce visible cracking in the ceiling.

One must remember, however, that these values are for structures loaded on roller end supports, a condition which assumes that in practice the side walls of a house offer no lateral support against the horizontal spread of the rafters. While this assumption may be close to the truth it is not entirely correct in that the walls must offer some resistance. However, it also must be remembered that in the short-term tests where the ends of the structure were bolted securely to the floor there was still some movement even with the lower snow loads (see DBR Internal Report 113), which for similar constructions amounts to about 1/32 in. at a 40 p.s.f. snow load (see Table 2, Report 113) after 5 minutes of loading. It also should be kept in mind that these figures are for 24-ft. spans with nailing according to the nailing schedule in the National Building Code. In practice the spans may be up to 28 ft. or even 32 ft. in width and the nailing quite inferior to the recommended nailing.

The bottom chord splice separation with trusses is small in relation to conventional construction (in the neighbourhood of a few thousandths of an inch for short-term loads), and for all practical purposes may be ignored. The deflections of the bottom chord then become the critical ones for estimating acceptable performance.

The deflection, which may be tolerated before plaster cracking results, is assumed to be 1/360 of the span. If the comparison between conventional and truss construction is made solely on the characteristics of the 26-ft. span, 4/12 slope truss, which was used in the long-term tests, it may be noted in Fig. 36 that the trusses loaded with the 20 p.s.f. snow load never reached this limiting deflection after 1 month's loading, while the trusses with the 40 p.s.f. snow load reached the limiting deflections after 12 days' loading, and the trusses with the 60 p.s.f. snow load after 5 hours' loading.

The deflections of the long-term structures explained earlier, appeared to be greater than would normally be expected since previous short-term tests on identical trusses showed deflections considerably less than for the trusses in the long-term tests. If the increase in deflection is due mainly to an unusually light-density spruce, then ordinarily one could expect even better performance. For example, if the per cent increase in deflections determined in the long-term tests on trusses were applied to the short-term deflections, listed in Table II, it may be seen that all the trusses (with the exception of the 28-ft. span, 4/12 slope truss) would not exceed the limiting deflection after 1 month's loading with a 40 p.s.f. snow load.

During the course of experiments the question arose as to what would be a reasonable period of time to subject the structures to snow loads, assuming that the proper magnitude of snow load could be arrived at. The maximum design load would probably not occur frequently although lesser snow loads would occur periodically. The loadings occurring in the winter months are followed by relatively long recovery periods in the summer months.

The period of 30 days' loading, used in the long-term tests, was selected as a convenient test time. Whether this is longer or shorter than the time for which trusses should be expected to carry the full load must remain a matter of opinion until further information on actual measured snow loads becomes available.

The results show that after the 30-day loads were removed from the trusses the recovery in deflection continued for a considerable time, and there was recovery even after one month. Upon re-application of load the time required for the trusses to reach the maximum deflection again was 12 days in the case of the trusses with the 20 p.s.f. snow load and would have been even longer in the case of the 40 p.s.f. snow load.

This pattern of recovery may be expected to change with further repetition of the loading cycles. There is no reason to believe as long as the recovery is time dependent, that any simple relationship will be found between the deflections occurring with time under a number of loading cycles and those occurring under a single prolonged loading time.

It would appear that the duration of time of periodically applied loads and the period of recovery are both important in determining the total deflection which may occur. It would be interesting to conduct additional tests to determine the effects produced by such cycling of loads.

Unfortunately, time did not allow the investigation of the effect on truss deflections of partitions located at different positions under the lower chord, but this should be done when time permits. The effect that sheathing and gable end walls would have on the strength and deflection characteristics in an entire roof is also an unknown factor that needs clarification.

## 6. Conclusions

On the basis of the comparisons of deflection and strength characteristics of conventional and trussed constructions that have been made, it would appear that the trusses tested are stronger than the strongest type of conventional construction. The deflections of the trusses probably would not cause as much plaster damage under a given load as would occur in conventional construction.

Due to the greater deflections in the 28-ft. span, 4/12 slope trusses, the upper chords for trusses should be a minimum size of 2 in. by 5 in. to provide comparable deflections to the other trusses tested.

TABLE I

SUMMARY OF RESULTS OF SHORT TERM TRUSS TESTS

| Test No. | Span (ft.) | Slope | Lower Chord Deflection Ratio for 40 psf Snow Load | % Recovery After 40 psf Snow Load Removed | Lower Chord Deflection Ratio for 80 psf Snow Load | Horizontal End Movement (ins.) | Type of Failure | Failure Snow Load (psf) | Moisture Content at Failure Area |
|---|---|---|---|---|---|---|---|---|---|
| 76 | 24 | 5/12 | 1/1070 | 72% | 1/520 | .049 | See Fig. 38 | 157 | 11-12% |
| 81 | 24 | 5/12 | 1/750 | 65% | 1/390 | .069 | See Fig. 38 | 152 | 10-12% |
| 80 | 24 | 5/12 | 1/910 | 70% | 1/460 | .069 | Upper chords buckled laterally | 180 | 11-12% |
| 78 | 24 | 4/12 | 1/910 | 78% | 1/440 | .040 | See Fig. 38 | 159 | 10-13% |
| 82 | 24 | 4/12 | 1/750 | 79% | 1/380 | .090 | Upper chords buckled laterally | 177 | 10-12% |
| 83 | 24 | 4/12 | 1/790 | 78% | 1/390 | .087 | See Fig. 38 | 160 | 10-12% |
| 84 | 26 | 5/12 | 1/990 | 73% | 1/470 | .082 | See Fig. 38 | 145 | 10-12% |
| 85 | 26 | 5/12 | 1/830 | 69% | 1/430 | .080 | See Fig. 38 | 140 | 13% |
| 86 | 26 | 5/12 | 1/800 | 66% | 1/410 | .086 | See Fig. 38 | 143 | 10% |
| 87 | 26 | 4/12 | 1/760 | 77% | 1/365 | .053 | See Fig. 38 | 132 | 15% |
| 88 | 26 | 4/12 | 1/770 | 73% | 1/380 | .116 | See Fig. 39 | 111 | 9% |
| 89 | 26 | 4/12 | 1/880 | 74% | 1/430 | .063 | See Fig. 38 | 147 | 12% |
| 90 | 28 | 5/12 | 1/870 | 70% | 1/440 | .092 | See Fig. 38 | 145 | 12% |
| 91 | 28 | 5/12 | 1/920 | 72% | 1/470 | .076 | See Fig. 38 | 139 | 12% |
| 92 | 28 | 5/12 | 1/880 | 73% | 1/450 | .072 | See Fig. 40 | 152 | 12% |
| 93 | 28 | 4/12 | 1/550 | 73% | 1/300 | .115 | See Fig. 38 | 137 | 10% |
| 94 | 28 | 4/12 | 1/610 | 63% | 1/340 | .090 | See Fig. 41 | 140 | 10-12% |
| 95 | 28 | 4/12 | 1/650 | 76% | 1/330 | .114 | See Fig. 40 | 115 | 10-12% |
| 101 * | 28 | 4/12 | 1/710 | 73% | 1/380 | .113 | See Fig. 42 | 140 | 10% |
| 102 * | 28 | 4/12 | 1/690 | 76% | 1/370 | .097 | See Fig. 43 | 132 | 10% |
| 105 * | 28 | 4/12 | 1/680 | 82% | 1/360 | .093 | See Fig. 38 | 155 | 10% |

* With 2" x 5" top chord.

TABLE II

CONDENSED SUMMARY OF RESULTS OF SHORT-TERM TESTS

| Span (ft.) | Slope | Lower Chord Deflection Ratio for 40 psf Snow Load | % Recovery After 40 psf Snow Load Removed | Lower Chord Deflection Ratio for 80 psf Snow Load | Horizontal End Movement at 40 psf Snow Load (ins.) | Failure Snow Load (psf) | Cost ** Per Truss |
|---|---|---|---|---|---|---|---|
| 24 | 5/12 | 1/910 | 69% | 1/460 | .062 | 163 | $ 9.10 |
| 24 | 4/12 | 1/820 | 78% | 1/400 | .072 | 165 | 9.25 |
| 26 | 5/12 | 1/870 | 69% | 1/440 | .083 | 143 | 10.05 |
| 26 | 4/12 | 1/800 | 75% | 1/390 | .077 | 130 | 9.73 |
| 28 | 5/12 | 1/890 | 72% | 1/450 | .080 | 145 | 10.59 |
| 28 | 4/12 | 1/600 | 71% | 1/320 | .106 | 131 | 10.30 |
| 28 * | 4/12 | 1/690 | 77% | 1/370 | .101 | 142 | 11.14 |

* With 2" x 5" top chord.

** See Table V.

TABLE III

SUMMARY OF LONG-TERM TRUSS TESTS (26' SPAN, 4/12 SLOPE)

| Loading Phase | Applied Snow Loads (psf) | MID SPAN DEFLECTIONS OF LOWER CHORDS | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0 Minutes | | 5 Minutes | | 1 Hour | | 1 Day | | 1 Week | | 1 Month | |
| | | Ins. | % Increase Over 5 Min. Deflections | Ins. | % Increase Over 5 Min. Deflections | Ins. | % Increase Over 5 Min. Deflections | Ins. | % Increase Over 5 Min. Deflections | Ins. | % Increase Over 5 Min. Deflections | Ins. | % Increase Over 5 Min. Deflections |
| First Application of Long Term Snow Loads and Ceiling Loads | 20 | .295 | --- | .30 | 0% | .315 | 5% | .385 | 28% | .48 | 60% | .60 | 100% |
| | 40 | .535 | --- | .55 | 0% | .58 | 5% | .68 | 24% | .83 | 51% | 1.06 | 93% |
| | 60 | .755 | --- | .795 | 0% | .84 | 6% | .985 | 24% | 1.19 | 50% | *** | |
| | 80 | 1.045 | --- | 1.12 | 0% | 1.21 | 8% | 1.42 | 27% | 1.755 | 57% | **** | |
| Snow Loads and Ceiling Loads Removed | | Ins. | % Recovery | Ins. | % Recovery | Ins. | % Recovery | Ins. | % Recovery | Ins. | % Recovery | Ins. | % Recovery |
| | * | .345 | 43% | .335 | 44% | .32 | 47% | .295 | 51% | .265 | 56% | .25 | 58% |
| | ** | .515 | 51% | .505 | 52% | .49 | 54% | .445 | 58% | .39 | 63% | .35 | 67% |
| Second Application of Long Term Snow Loads and Ceiling Loads | | Ins. | % of Original 1 mo. Deflections | Ins. | % of Original 1 mo. Deflections | Ins. | % of Original 1 mo. Deflections | Ins. | % of Original 1 mo. Deflections | Ins. | % of Original 1 mo. Deflections | Ins. | % of Original 1 mo. Deflections |
| | 20 | .52 | 87% | .525 | 88% | .545 | 91% | .555 | 93% | .58 | 97% | --- | --- |
| | 40 | .895 | 84% | .905 | 85% | .915 | 86% | .94 | 89% | 1.00 | 94% | --- | --- |

* Trusses originally loaded with 20 p.s.f. snow load.
** Trusses originally loaded with 40 p.s.f. snow load.
*** Structure collapsed due to lateral instability after 22 days.
**** Structure collapsed due to lateral instability after 12 days.

TABLE IV

SUMMARY OF LONG-TERM TESTS ON CONVENTIONAL CONSTRUCTION
(TYPE I CONSTRUCTION, 24' SPAN, 5/12 SLOPE)

| Loading Phase | Applied Snow Load (psf) | Joist Splice Separation (ins.) | | | | | Peak Deflections (ins.) | | | | | Mid Span Rafter Deflections, Perpendicular to Slope (ins.) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 5 Min. | 1 Hour | 1 Day | 1 Week | 1 Month | 5 Min. | 1 Hour | 1 Day | 1 Week | 1 Month | 5 Min. | 1 Hour | 1 Day | 1 Week | 1 Month |
| First Application of Long Term Snow and Ceiling Load | 20 | .037 | .039 | .050 | .067 | .109 | .12 | .13 | .16 | .23 | .35 | .08 | .08 | .10 | .14 | .19 |
| | 40 | .109 | .116 | .144 | .233 | .384 | .39 | .43 | .55 | .77 | 1.09 | .22 | .24 | .30 | .43 | .63 |
| Snow Loads and Ceiling Loads Removed | 0 * | .060 | .059 | .057 | .048 | ☆ .047 | .20 | .20 | .20 | .19 | ☆ .19 | .10 | .10 | .09 | .09 | ☆ .08 |
| | 0 ** | .319 | .316 | .312 | .310 | ☆ .303 | .82 | .81 | .80 | .78 | ☆ .78 | .42 | .40 | .39 | .38 | ☆ .35 |

* Structure Originally Loaded with 20 p.s.f. Snow Load.

** Structure Originally Loaded with 40 p.s.f. Snow Load.

☆ Structure loaded for 26 days only.

TABLE V

COST DATA FOR NAILED TRUSS

| SPAN | SLOPE | MATERIALS 2"x5" (fbm) | 2"x4" (fbm) | 1"x8" (fbm) | 1"x5" (fbm) | ½" Plywood (sq.ft.) | Nails (lb.) 3" | Nails (lb.) 2½" | Materials Cost | LABOUR Cutting (hrs.) Structural Pieces | Cutting (hrs.) Plywood Plates | Assembly (hrs.) | Total Time (hrs.) | Total Labour Cost | TOTAL COST OF TRUSS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 24' | 5/12 | - | 41 | 10 | 3½ | 4.8 | 1.38 | .78 | $7.82 | .14 | .06 | .50 | .70 | 1.28 | $ 9.10 |
| 24' | 4/12 | - | 40 | 9½ | 3½ | 6.0 | 1.73 | .82 | 7.92 | .14 | .06 | .53 | .73 | 1.33 | 9.25 |
| 26' | 5/12 | - | 44 | 11 | 4 | 6.4 | 1.52 | .82 | 8.72 | .14 | .06 | .53 | .73 | 1.33 | 10.05 |
| 26' | 4/12 | - | 43 | 10 | 3½ | 6.0 | 1.84 | .86 | 8.36 | .14 | .06 | .55 | .75 | 1.37 | 9.73 |
| 28' | 5/12 | - | 47 | 11½ | 4½ | 6.4 | 1.65 | .86 | 9.22 | .14 | .06 | .55 | .75 | 1.37 | 10.59 |
| 28' | 4/12 | - | 46 | 10½ | 4 | 6.0 | 2.00 | .90 | 8.88 | .14 | .06 | .58 | .78 | 1.42 | 10.30 |
| 28' | 4/12 | 28½ | 23½ | 10½ | 4 | 6.5 | 2.00 | .90 | 9.72 | .14 | .06 | .58 | .78 | 1.42 | 11.14 |

COST OF MATERIALS

2½" nails - 11.3¢ per lb.

3" nails - 11.0¢ per lb.

1"x8" lumber - 12.0¢ per fbm

1"x5" lumber - 11.5¢ per fbm

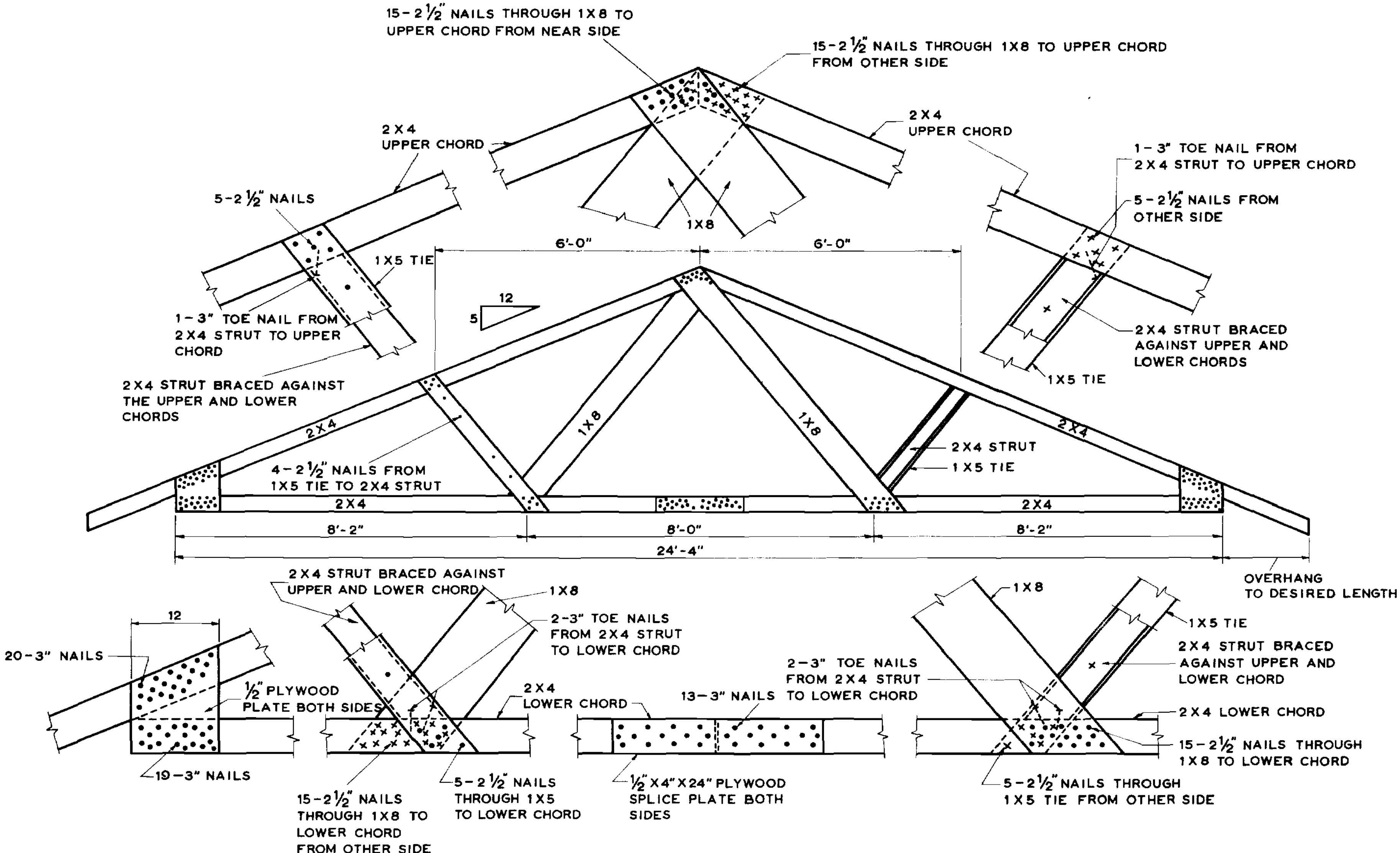
2"x4" lumber - 12.3¢ per fbm

2"x5" lumber - 12.3¢ per fbm

½" plywood - 19.6¢ per sq.ft.

COST OF LABOUR

Carpenter - $1.82 per hour



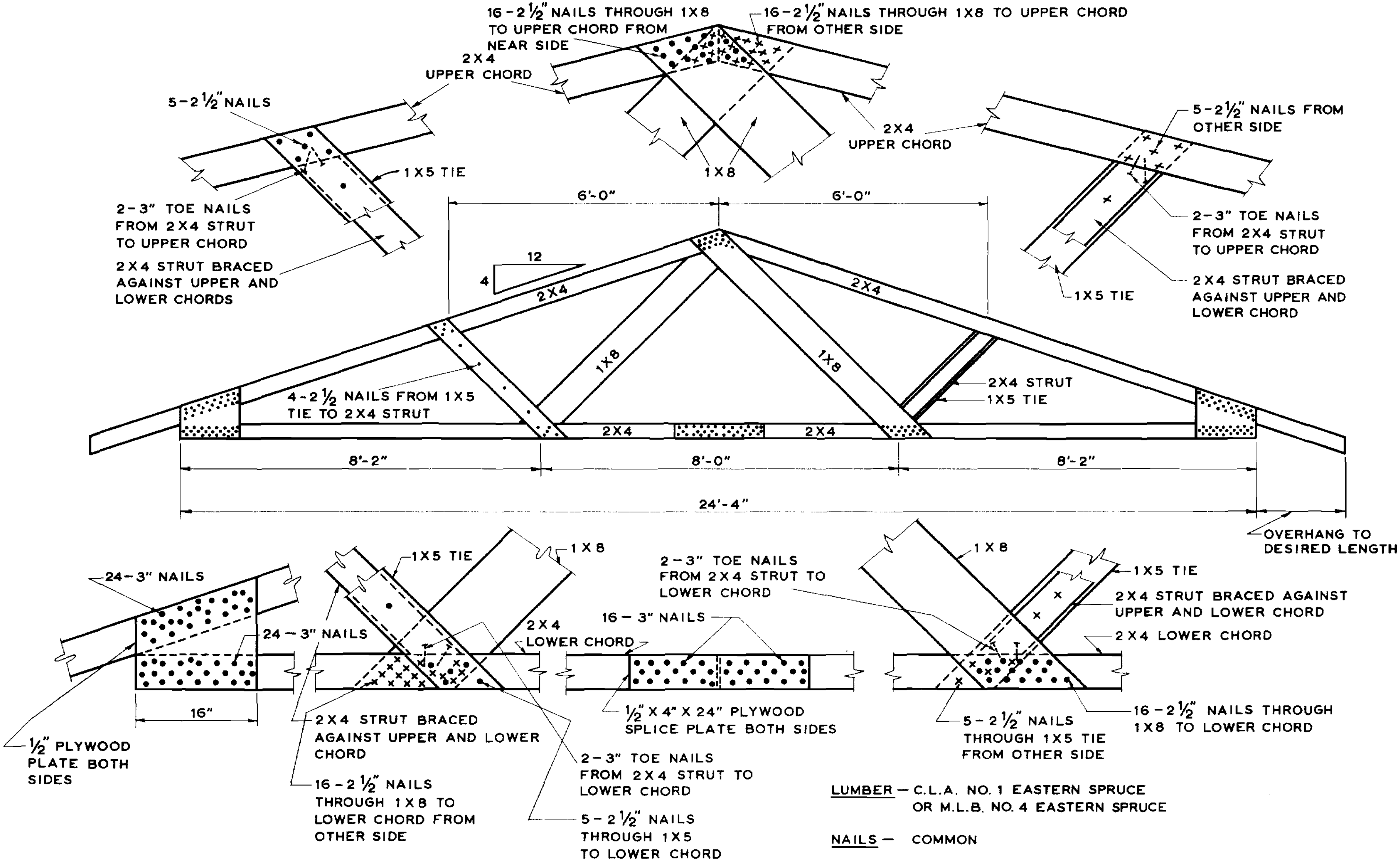

**LUMBER** — C.L.A. NO. 1 EASTERN SPRUCE OR M.L.B. NO. 4 EASTERN SPRUCE

**NAILS** — COMMON

— ALL ROWS OF NAILS ARE STAGGED IN THE DIRECTION OF THE GRAIN TO KEEP SPLITTING TO A MINIMUM

**NOTE** — TO ENSURE MAXIMUM STIFFNESS, THE UPPER CHORDS MUST BE IN GOOD BEARING CONTACT AT THE PEAK, AND THE 2X4 STRUTS IN GOOD BEARING CONTACT WITH THE TOP AND BOTTOM CHORD

**FIGURE I** NAILED "W" TRUSS 24' SPAN $5/12$ SLOPE 2'-0" O.C.



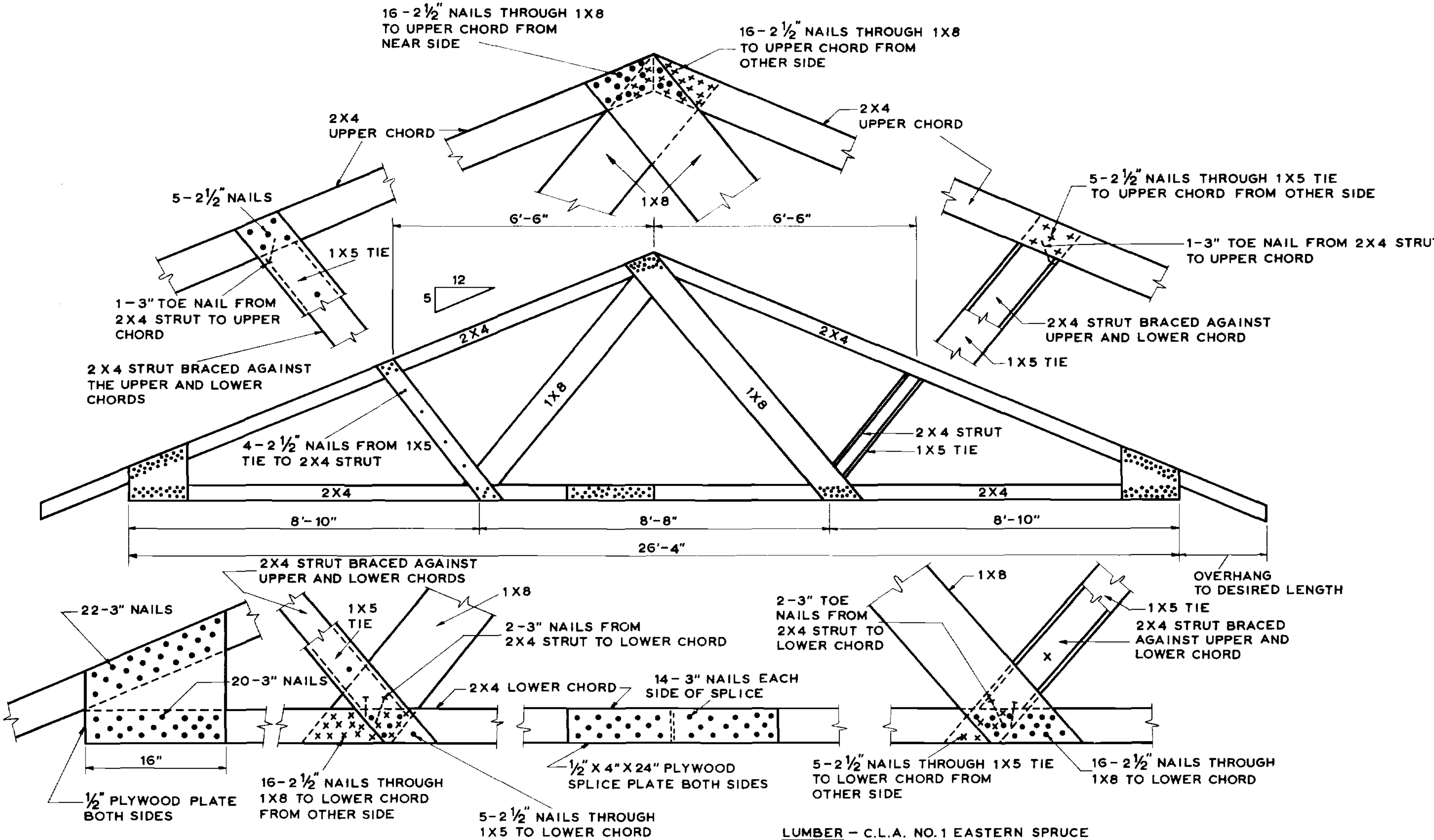

LUMBER — C.L.A. NO. 1 EASTERN SPRUCE OR M.L.B. NO. 4 EASTERN SPRUCE

NAILS — COMMON

— ALL ROWS OF NAILS ARE STAGGERED IN THE DIRECTION OF GRAIN TO KEEP SPLITTING TO A MINIMUM

NOTE — TO ENSURE MAXIMUM STIFFNESS, THE UPPER CHORD MUST BE IN GOOD BEARING CONTACT AT THE PEAK, AND THE 2X4 STRUTS IN GOOD BEARING CONTACT WITH THE TOP AND BOTTOM CHORD

**FIGURE 2** NAILED "W" TRUSS 24' SPAN 4/12 SLOPE 2'-0" O.C.



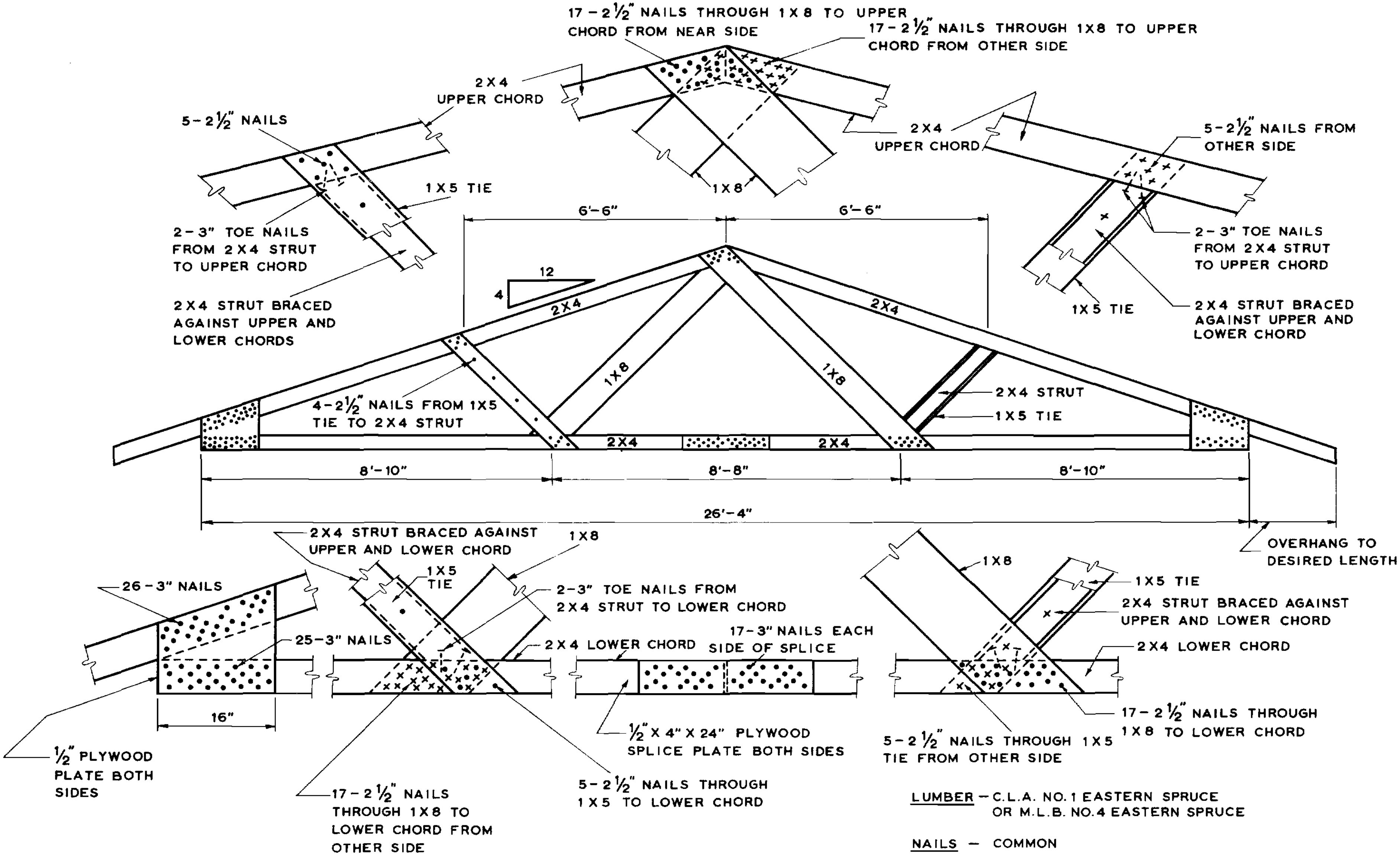

LUMBER – C.L.A. NO. 1 EASTERN SPRUCE OR M.L.B. NO. 4 EASTERN SPRUCE

NAILS – COMMON

– ALL ROWS OF NAILS ARE STAGGERED IN THE DIRECTION OF THE GRAIN TO KEEP SPLITTING TO A MINIMUM

NOTE – TO ENSURE MAXIMUM STIFFNESS, THE UPPER CHORDS MUST BE IN GOOD BEARING CONTACT AT THE PEAK, AND THE 2X4 STRUTS IN GOOD BEARING CONTACT WITH THE TOP AND BOTTOM CHORD

**FIGURE 3** NAILED "W" TRUSS 26' SPAN 5/12 SLOPE 2'-0" O.C.



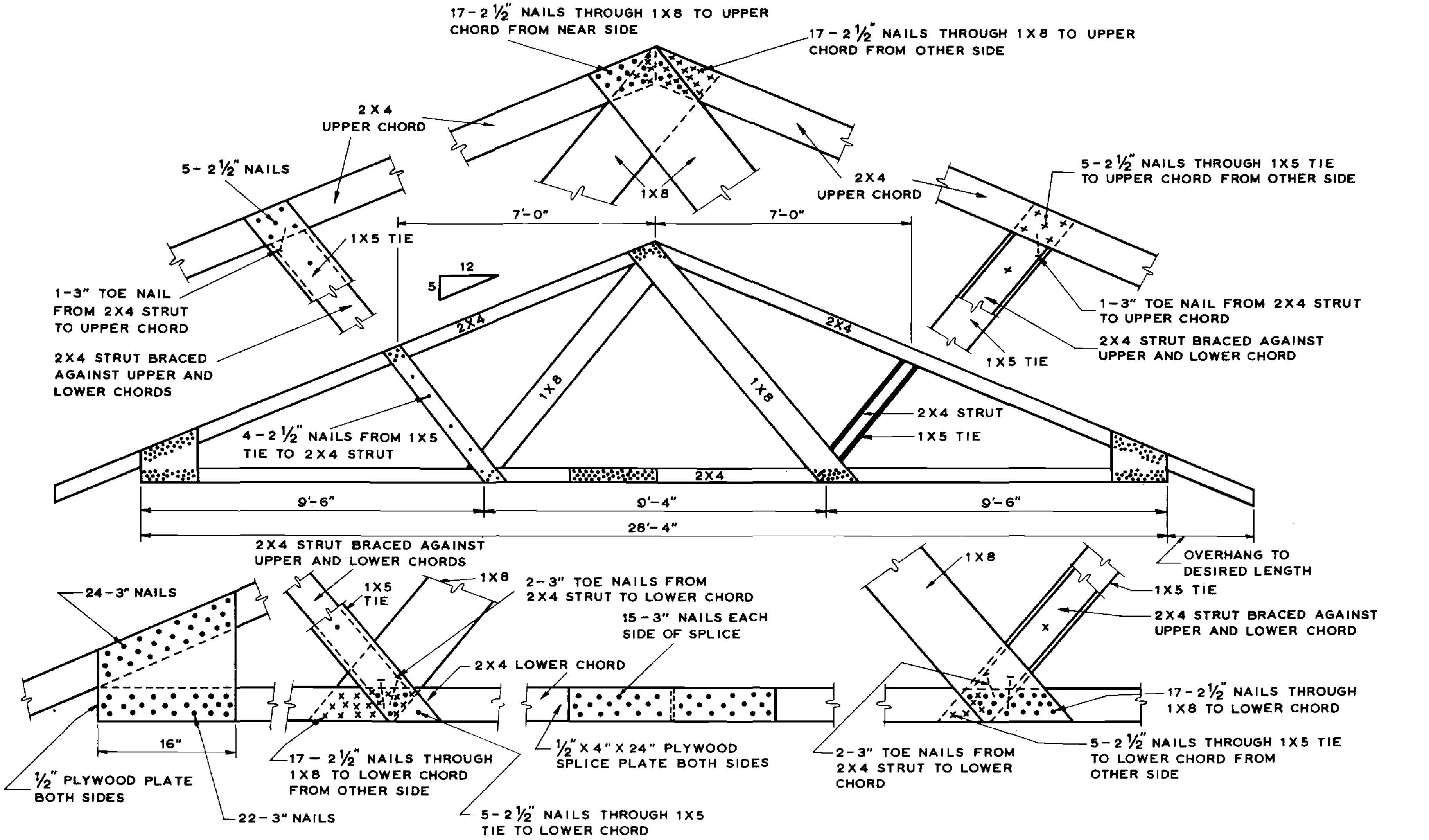

LUMBER – C.L.A. NO. 1 EASTERN SPRUCE OR M.L.B. NO. 4 EASTERN SPRUCE

NAILS – COMMON

– ALL ROWS OF NAILS ARE STAGGERED IN THE DIRECTION OF THE GRAIN TO KEEP SPLITTING TO A MINIMUM

NOTE – TO ENSURE MAXIMUM STIFFNESS, THE UPPER CHORDS MUST BE IN GOOD BEARING CONTACT AT THE PEAK, AND THE 2X4 STRUTS IN GOOD BEARING CONTACT WITH THE TOP AND BOTTOM CHORD

**FIGURE 4** NAILED "W" TRUSS 26' SPAN 4/12 SLOPE 2'-0" O.C.



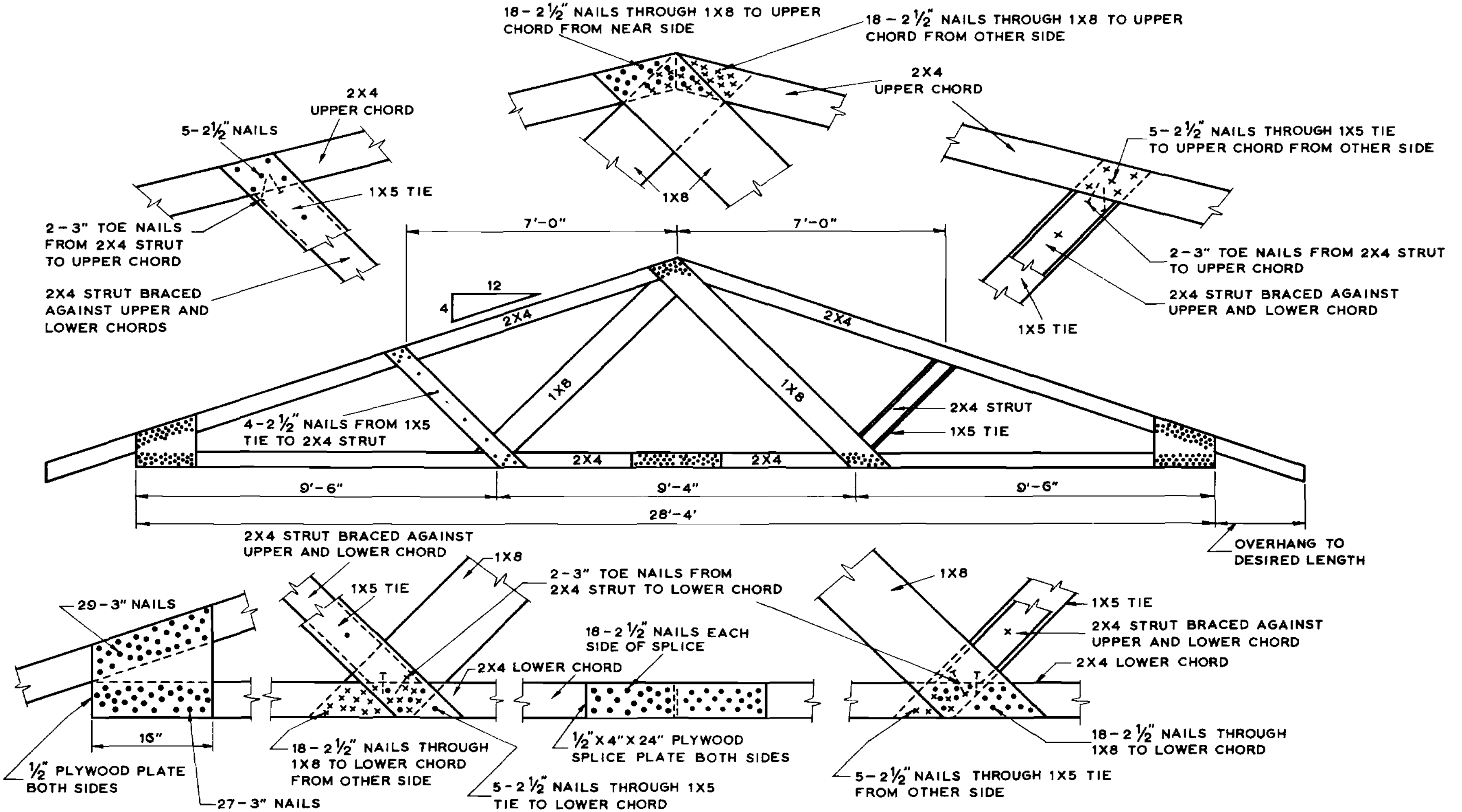

LUMBER – C.L.A. NO. 1 EASTERN SPRUCE OR M.L.B. NO. 4 EASTERN SPRUCE

NAILS – COMMON

– ALL ROWS OF NAILS ARE STAGGERED IN THE DIRECTION OF THE GRAIN TO KEEP SPLITTING TO A MINIMUM

NOTE – TO ENSURE MAXIMUM STIFFNESS, THE UPPER CHORDS MUST BE IN GOOD BEARING CONTACT AT THE PEAK, AND THE 2X4 STRUTS IN GOOD BEARING CONTACT WITH THE TOP AND BOTTOM CHORD

**FIGURE 5** NAILED "W" TRUSS 28' SPAN 5/12 SLOPE 2'-0" O.C.




LUMBER - C.L.A. NO. 1 EASTERN SPRUCE
OR M.L.B. NO. 4 EASTERN SPRUCE

NAILS - COMMON

- ALL ROWS OF NAILS ARE STAGGERED IN THE DIRECTION OF THE GRAIN TO KEEP SPLITTING TO A MINIMUM

NOTE - TO ENSURE MAXIMUM STIFFNESS, THE UPPER CHORDS MUST BE IN GOOD BEARING CONTACT AT THE PEAK, AND THE 2X4 STRUTS IN GOOD BEARING CONTACT WITH THE TOP AND BOTTOM CHORD

**FIGURE 6** NAILED "W" TRUSS 28' SPAN 4/12 SLOPE 2'-0" O.C.

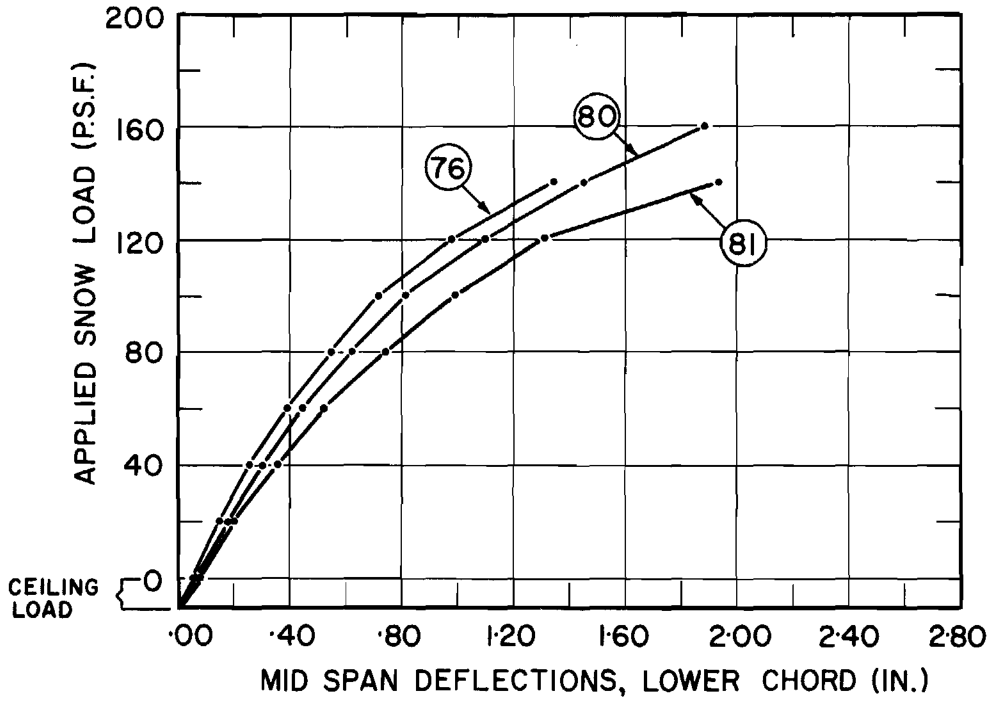



FIGURE 7
LOAD VS. DEFLECTION CURVES 24' SPAN NAILED
"W" TRUSS 5/12 SLOPE, SPACED 2'-0" O.C.
(TESTS No. 76, 80, 81)

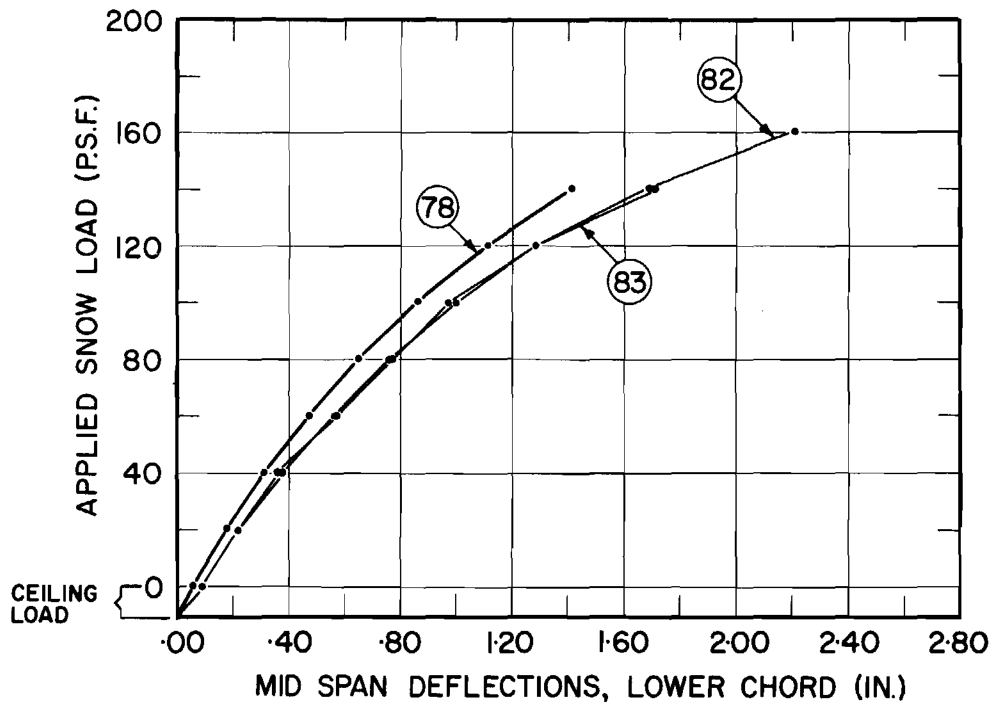



FIGURE 8
LOAD VS. DEFLECTION CURVES 24' SPAN NAILED
"W" TRUSS 4/12 SLOPE, SPACED 2'-0" O.C.
(TESTS No. 78, 82, 83)

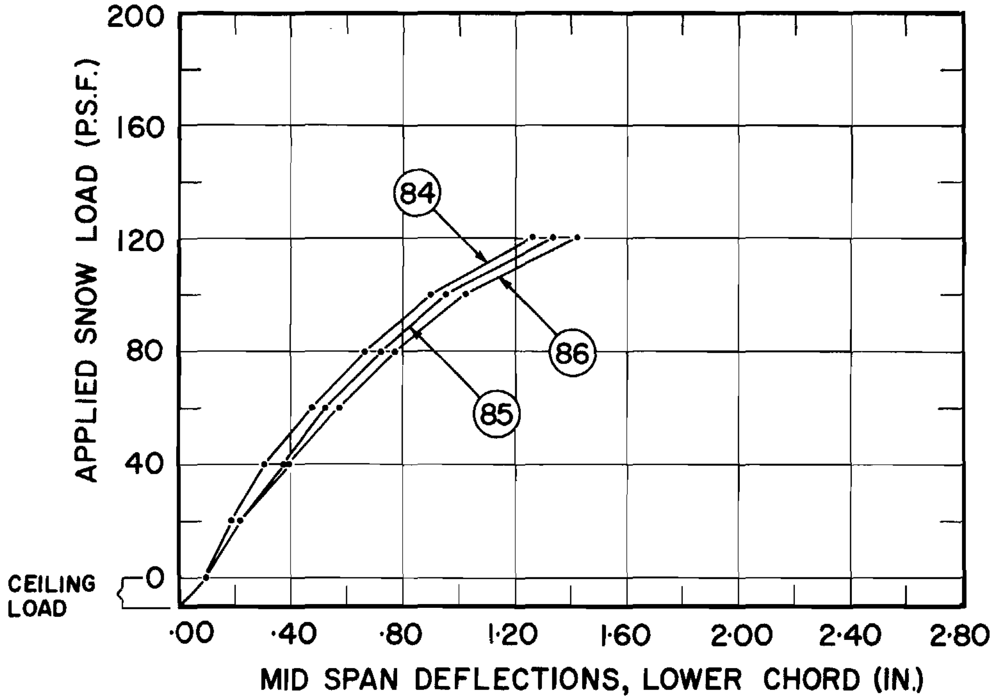



FIGURE 9

LOAD VS. DEFLECTION CURVES 26' SPAN NAILED "W" TRUSS 5/12 SLOPE, SPACED 2'-0" O.C. (TESTS No. 84, 85, 86)

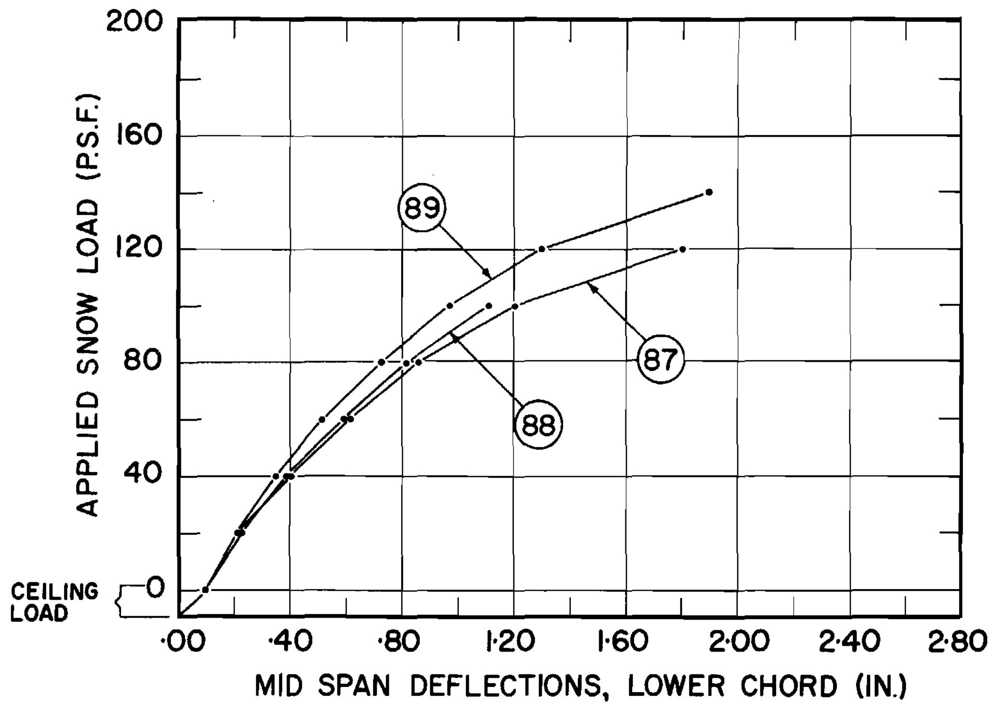



FIGURE 10

LOAD VS. DEFLECTION CURVES 26' SPAN NAILED "W" TRUSS 4/12 SLOPE, SPACED 2'-0" O.C. (TESTS No. 87, 88, 89)

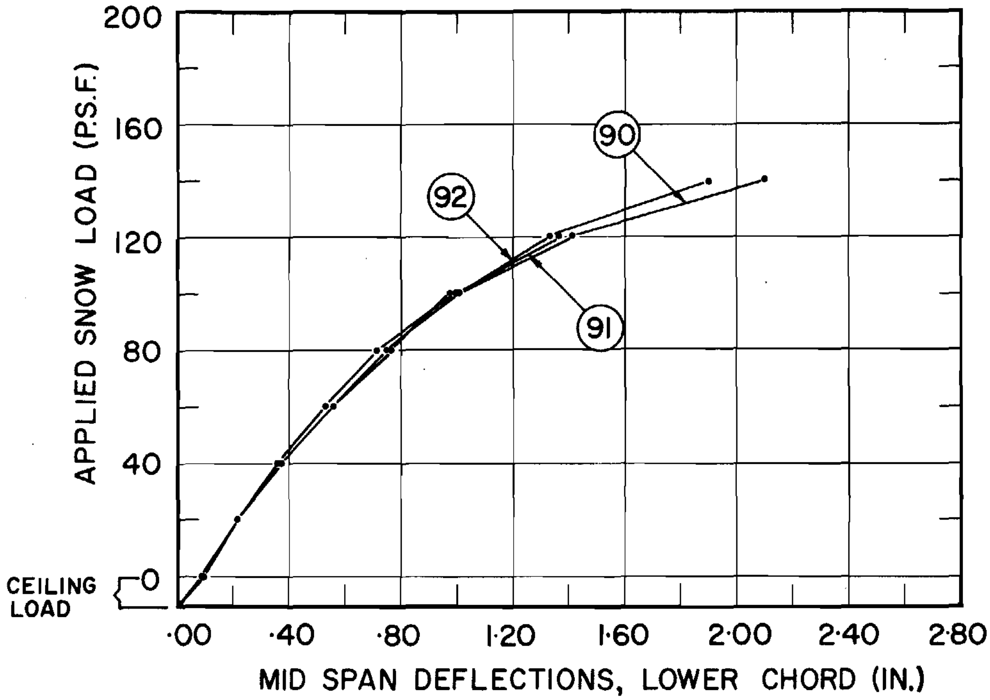



FIGURE 11

LOAD VS. DEFLECTION CURVES 28' SPAN NAILED "W" TRUSS 5/12 SLOPE, SPACED 2'-0" O.C. (TESTS No. 90, 91, 92)

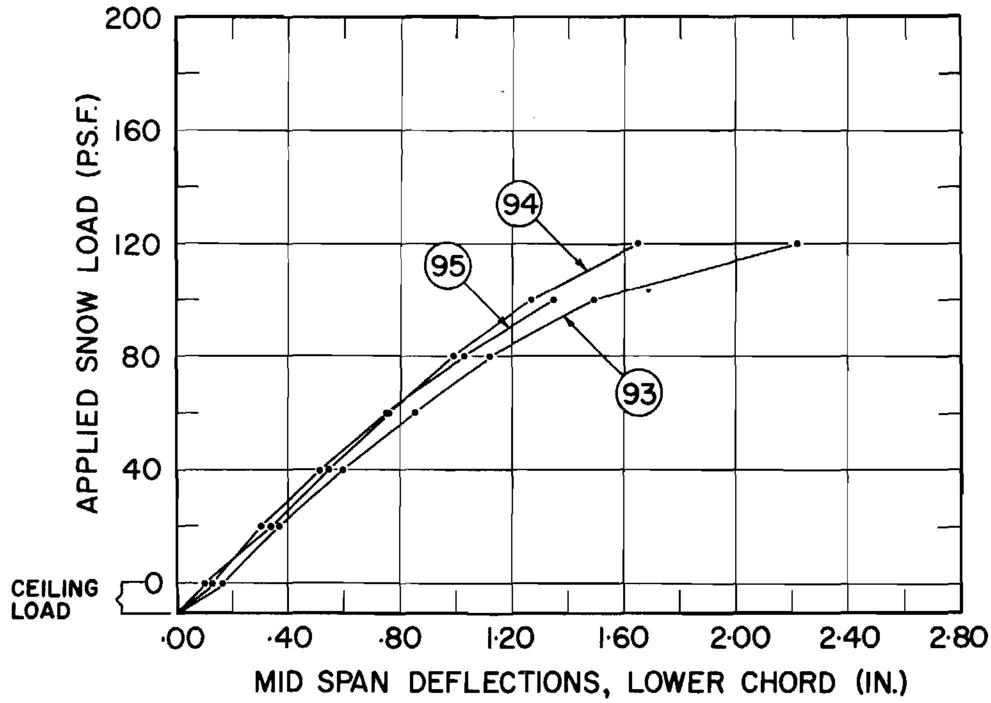



FIGURE 12

LOAD VS. DEFLECTION CURVES 28' SPAN NAILED "W" TRUSS 4/12 SLOPE, SPACED 2'-0" O.C. (TESTS No. 93, 94, 95)

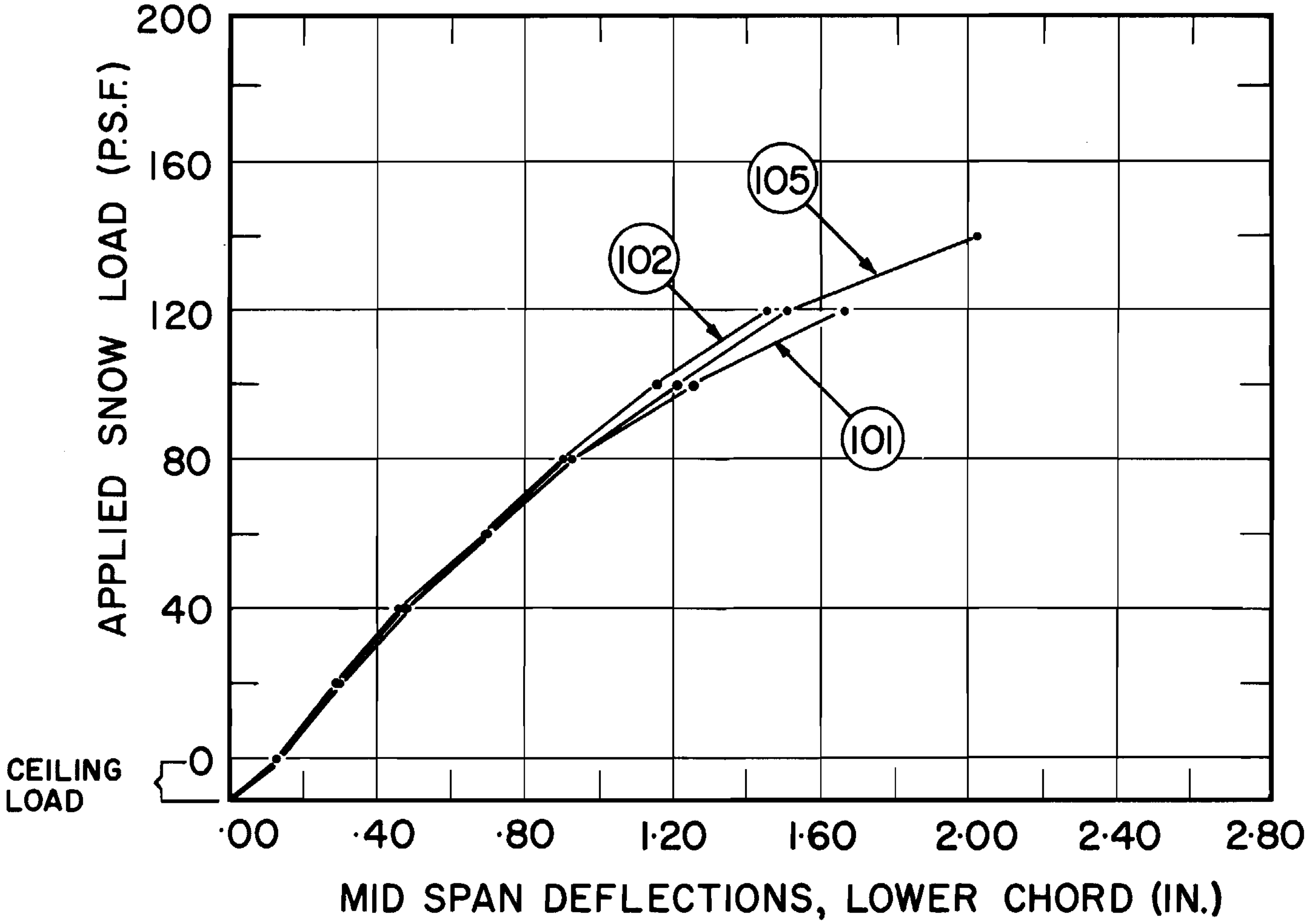
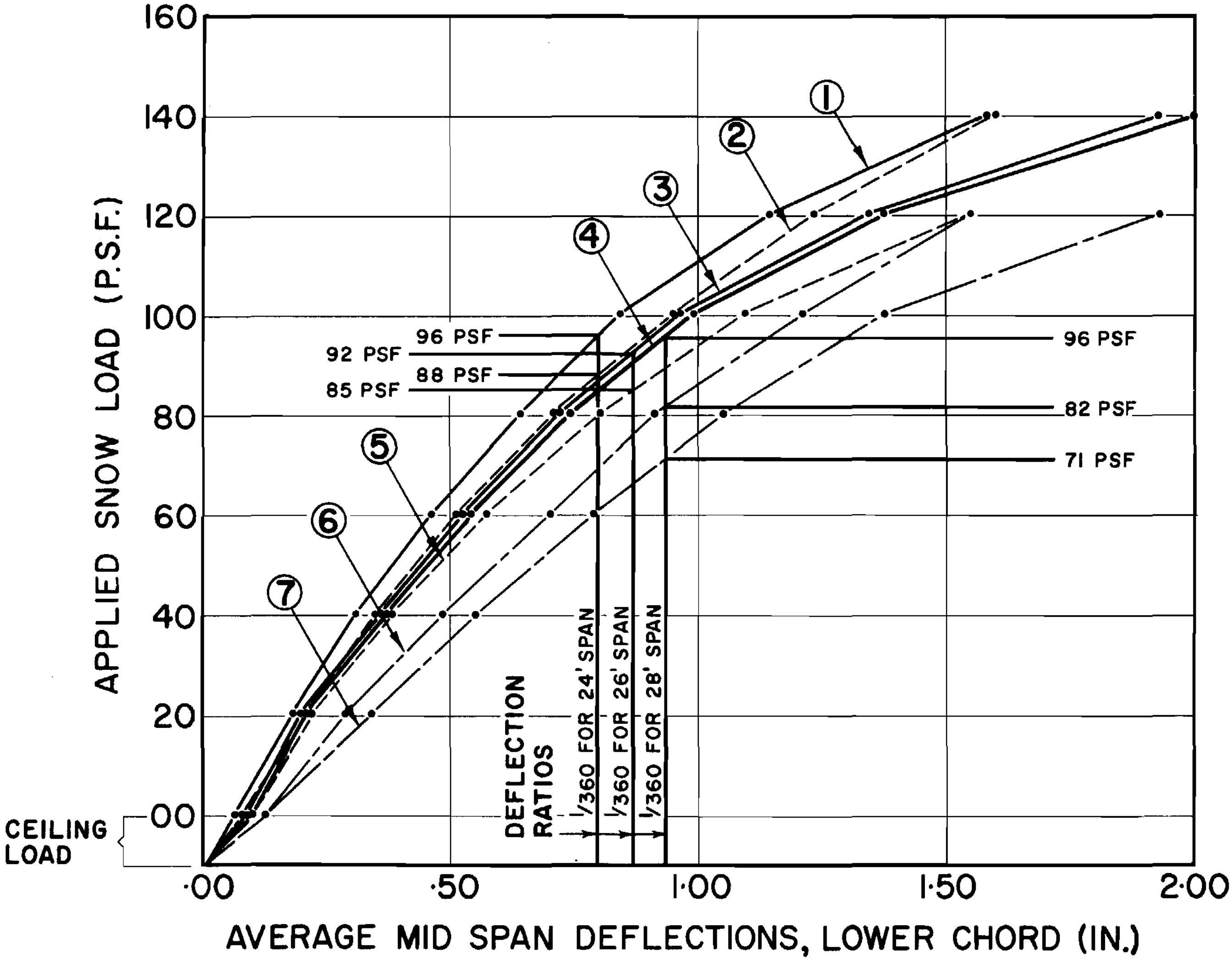



FIGURE 13

LOAD VS. DEFLECTION CURVES 28' SPAN NAILED "W" TRUSS 4/12 SLOPE, SPACED 2'-0" O.C. 2 x 5 TOP CHORDS.

(TESTS No. 101, 102, 105)



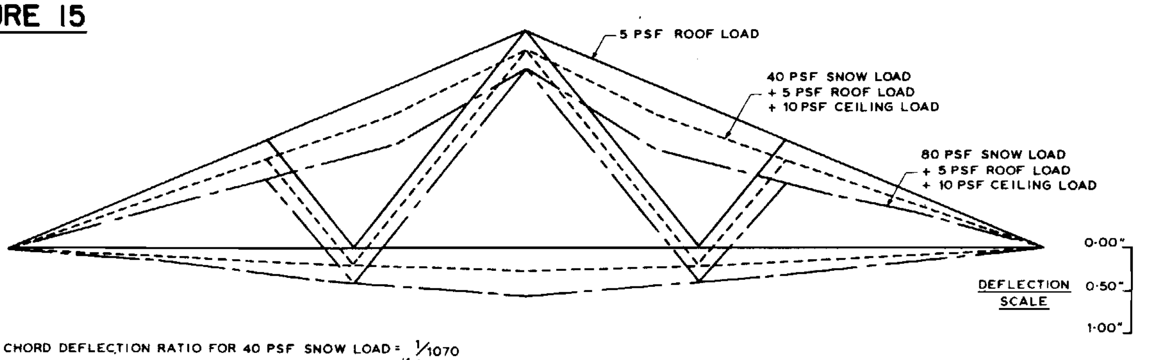

① 24' SPAN 5/12 SLOPE
② 24' SPAN 4/12 SLOPE
③ 26' SPAN 5/12 SLOPE
④ 28' SPAN 5/12 SLOPE
⑤ 26' SPAN 4/12 SLOPE
⑥ 28' SPAN 4/12 SLOPE (2"x5" TOP CHORD)
⑦ 28' SPAN 4/12 SLOPE (2"x4" TOP CHORD)

FIGURE 14

LOAD VS. DEFLECTION CURVES OF AVERAGE DEFLECTIONS FOR VARIOUS SPANS AND SLOPES FOR 5 MIN. LOAD APPLICATIONS.

TEST No. 76

## FIGURE 15

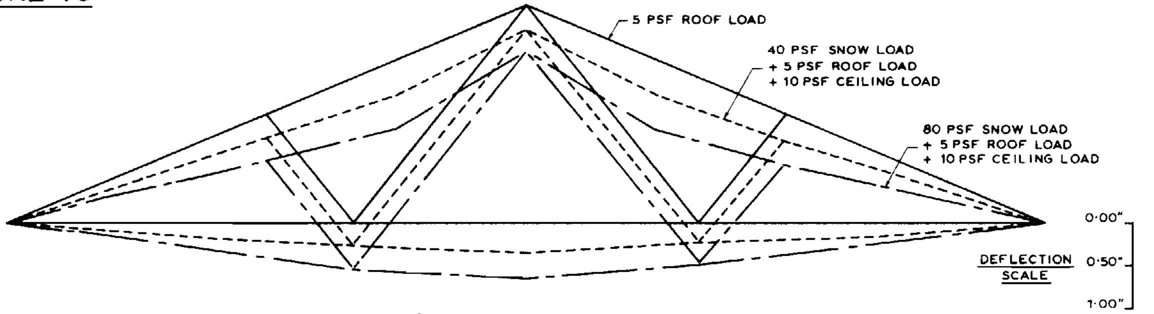



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/1070
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/520

FAILURE LOAD = 157 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 80

## FIGURE 16



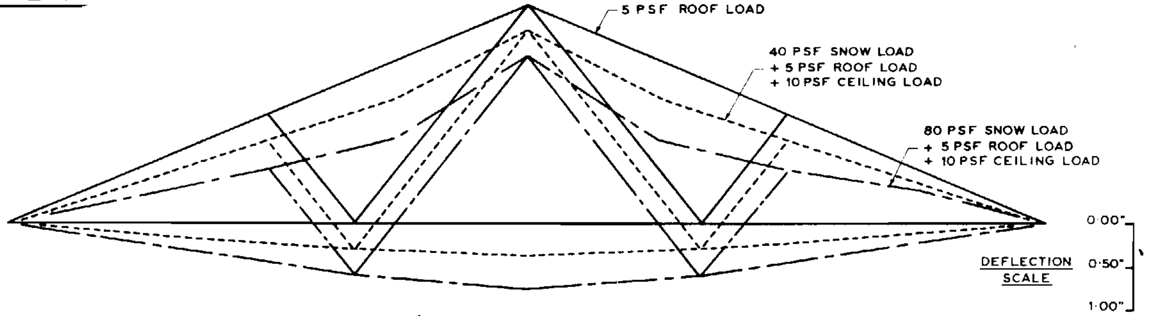

BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/910
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/460

FAILURE LOAD = 180 PSF SNOW LOAD *
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

* AT THIS LOAD THE STRUCTURE BUCKLED LATERALLY

TEST No. 81

## FIGURE 17



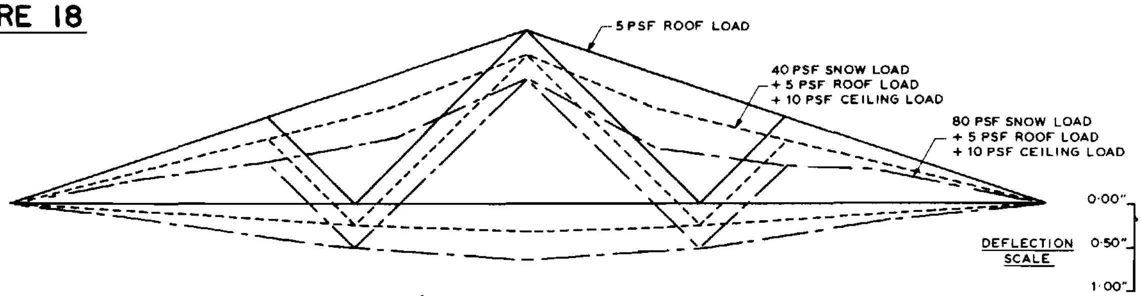

BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/750
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/390

FAILURE LOAD = 152 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

# DEFLECTION CHARACTERISTICS
# NAILED "W" TRUSS 24' SPAN 5/12 SLOPE 2'-0" O.C.

TEST No. 76

## FIGURE 18

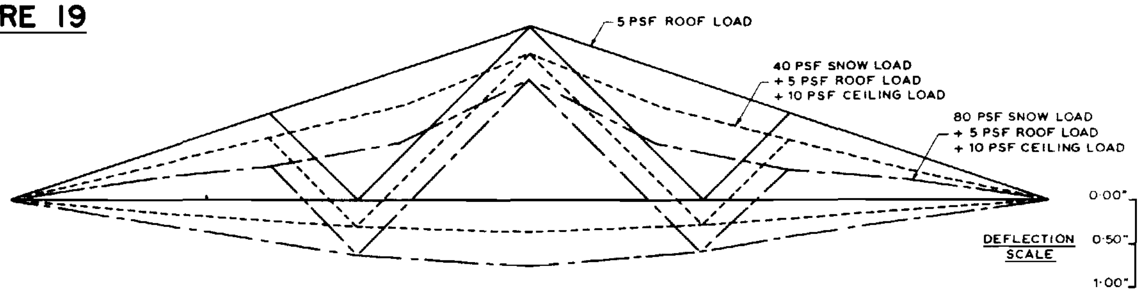



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/910
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/440

FAILURE LOAD = 159 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 82

## FIGURE 19

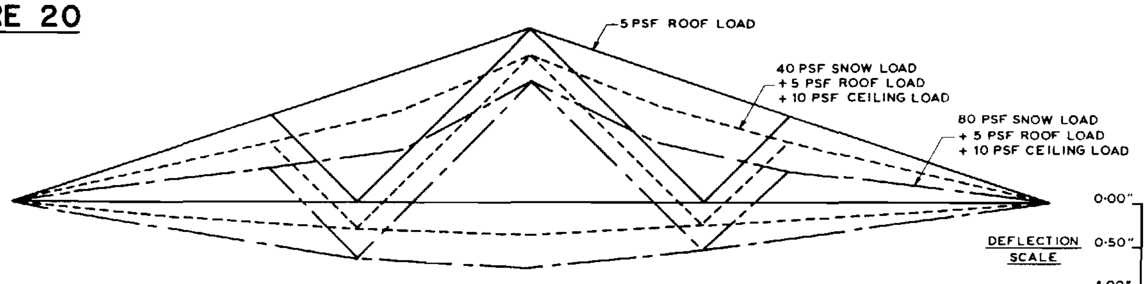



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/750
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/380

FAILURE LOAD = 177 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 83

## FIGURE 20



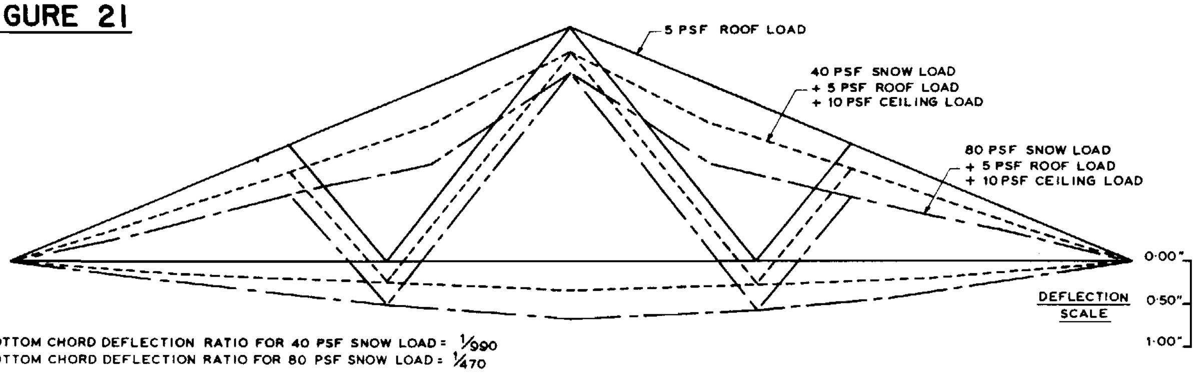

BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/790
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/390

FAILURE LOAD = 160 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

# DEFLECTION CHARACTERISTICS
# NAILED "W" TRUSS 24' SPAN 4/12 SLOPE 2'-0" O.C.

TEST No. 84

## FIGURE 21

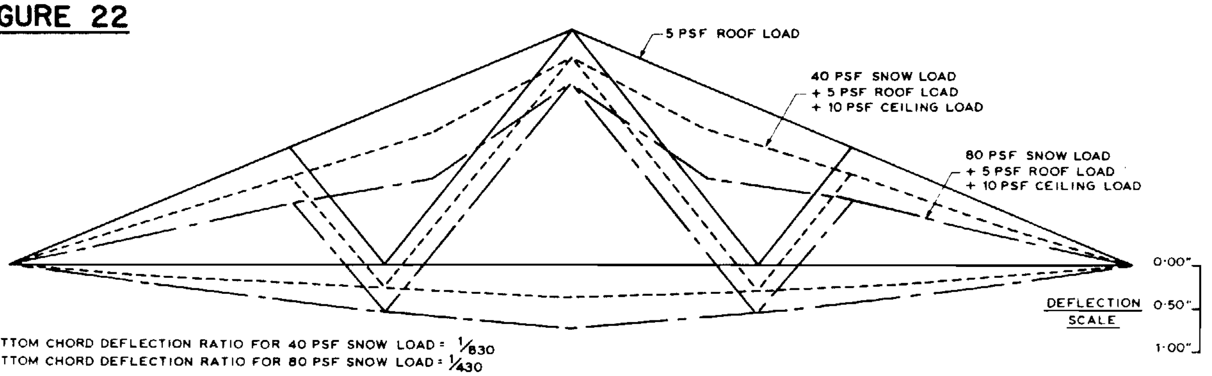



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/990
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/470

FAILURE LOAD = 145 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 85

## FIGURE 22

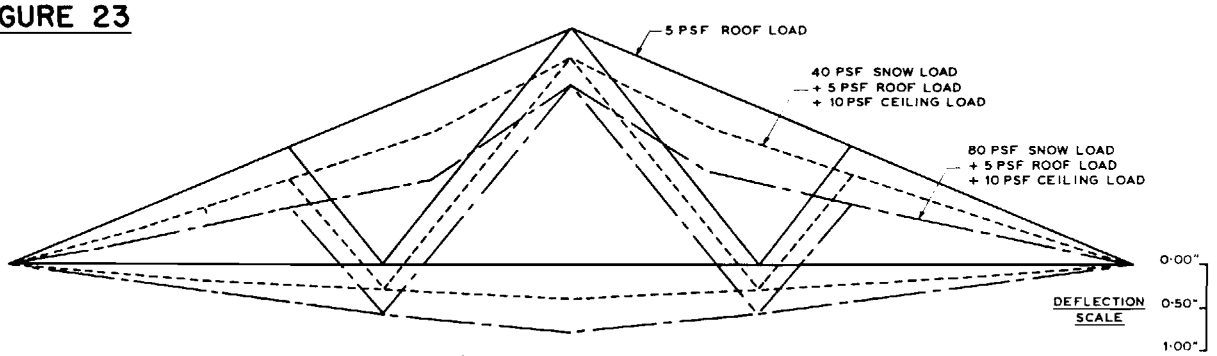



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/830
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/430

FAILURE LOAD = 140 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 86

## FIGURE 23




BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/800
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/410

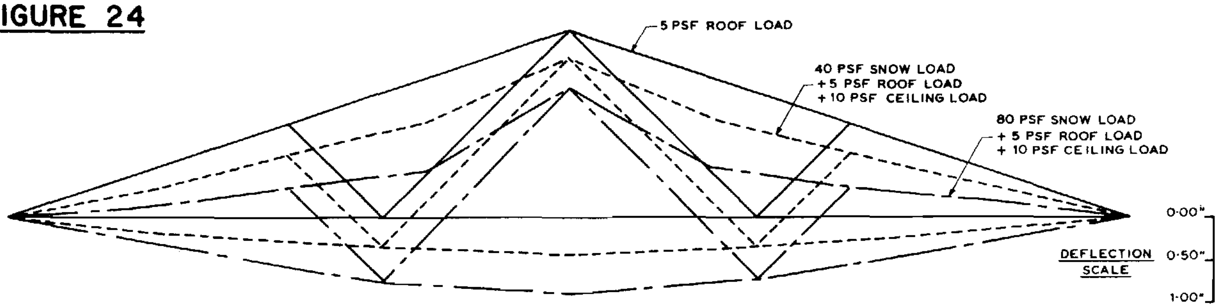
FAILURE LOAD = 143 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

# DEFLECTION CHARACTERISTICS

## NAILED "W" TRUSS 26' SPAN 5/12 SLOPE 2'-0" O.C.

TEST No. 87

## FIGURE 24

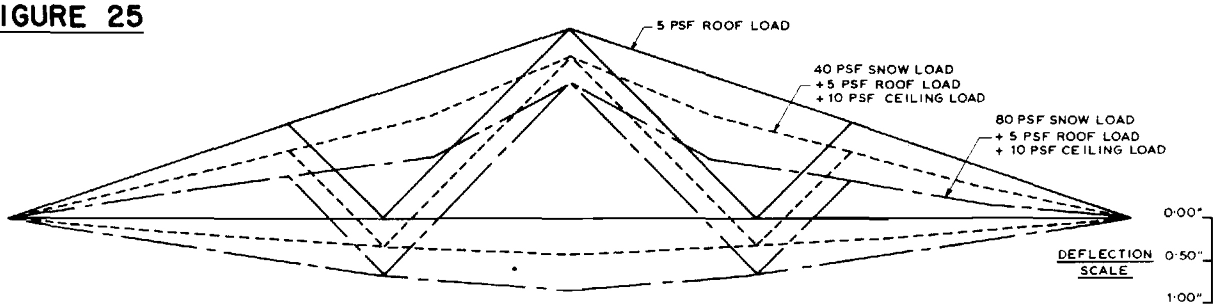



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/760
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/365

FAILURE LOAD = 132 PSF SNOW LOAD
+5 PSF ROOF LOAD
+10 PSF CEILING LOAD

TEST No. 88

## FIGURE 25

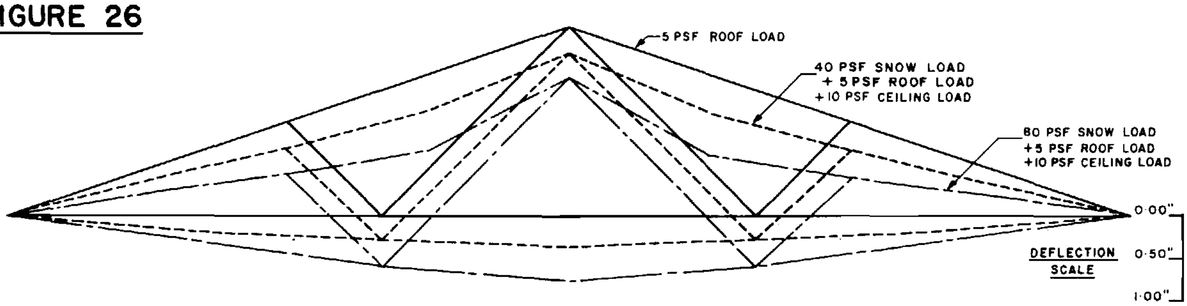



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/770
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/380

FAILURE LOAD = 111 PSF SNOW LOAD
+5 PSF ROOF LOAD
+10 PSF CEILING LOAD

TEST No. 89

## FIGURE 26



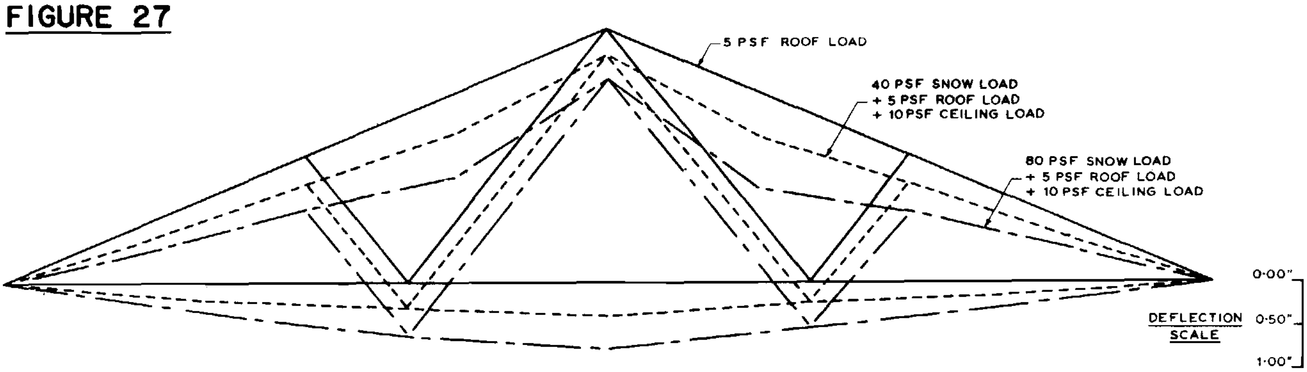

BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/880
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/430

FAILURE LOAD = 147 PSF SNOW LOAD
+5 PSF ROOF LOAD
+10 PSF CEILING LOAD

## DEFLECTION CHARACTERISTICS
## NAILED "W" TRUSS 26' SPAN 4/12 SLOPE 2'-0" O.C.

TEST No. 90

## FIGURE 27

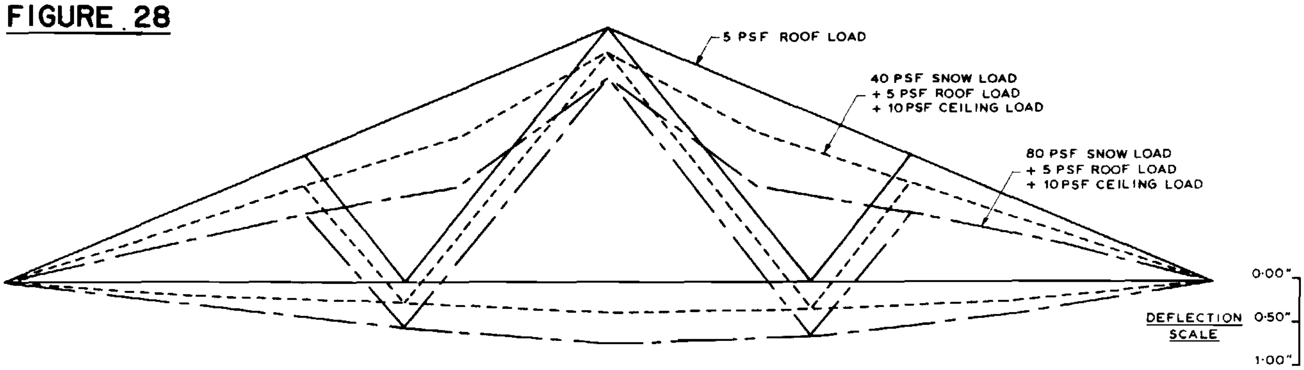



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/870
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/440

FAILURE LOAD = 145 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 91

## FIGURE 28

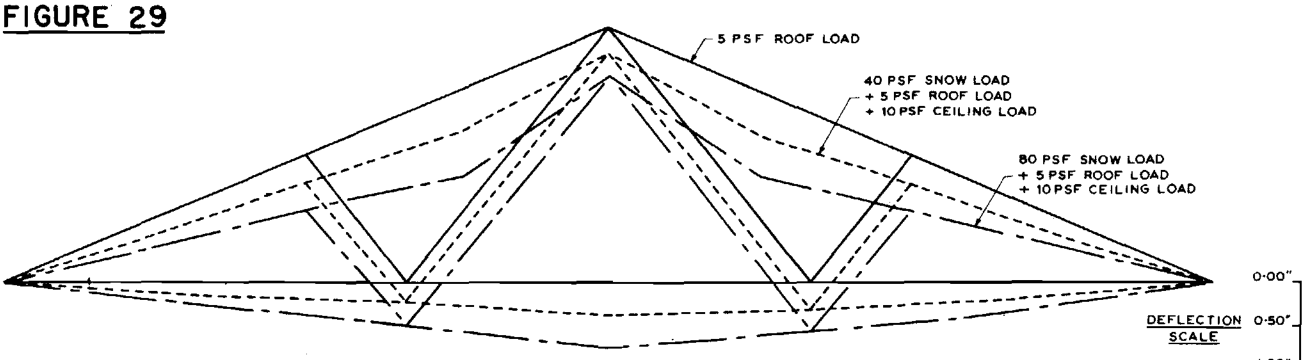



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/920
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/470

FAILURE LOAD = 139 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 92

## FIGURE 29



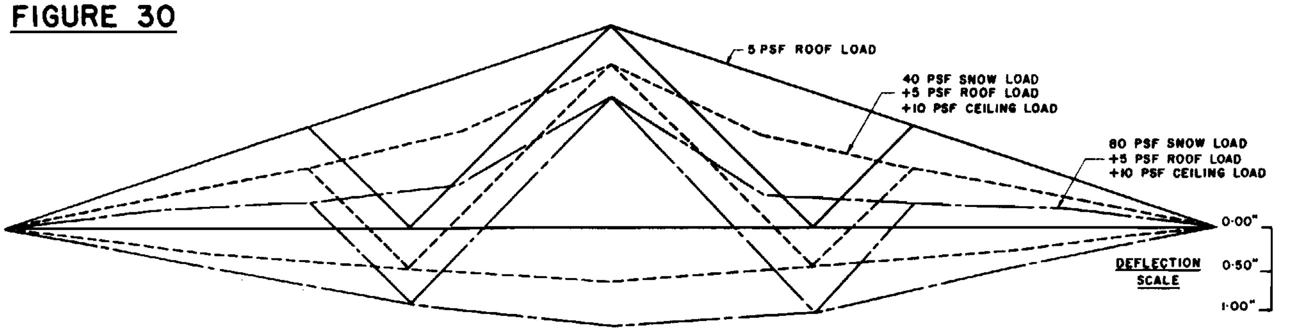

BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/880
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/450

FAILURE LOAD = 152 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

# DEFLECTION CHARACTERISTICS
# NAILED "W" TRUSS 28' SPAN 5/12 SLOPE 2'-0" O.C.

TEST No. 93

## FIGURE 30

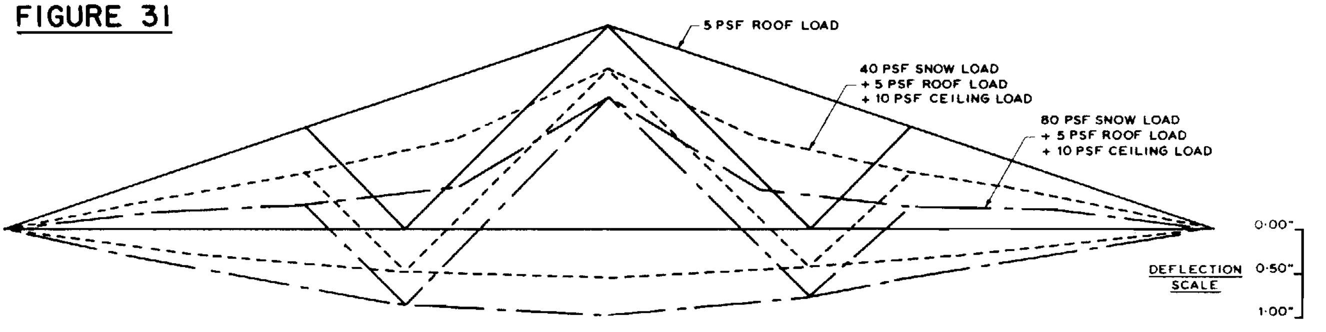



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/550
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/300

FAILURE LOAD = 137 PSF SNOW LOAD
+5 PSF ROOF LOAD
+10 PSF CEILING LOAD

TEST No. 94

## FIGURE 31

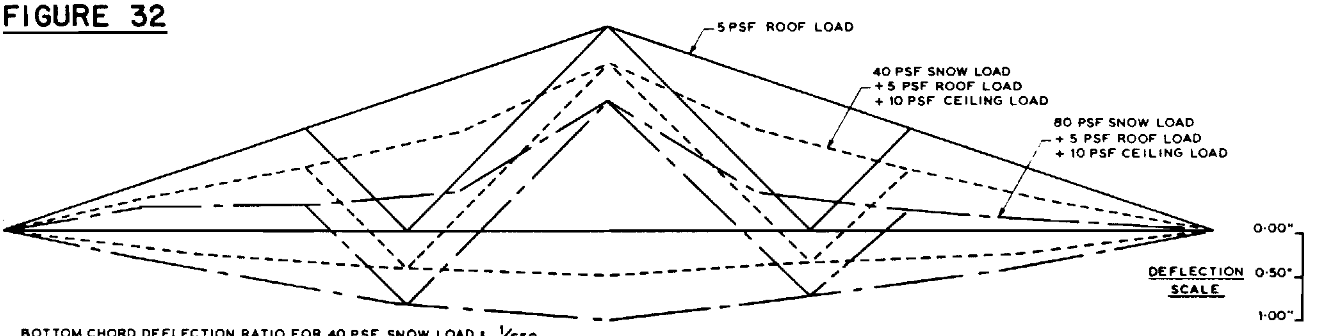



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/610
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/340

FAILURE LOAD = 140 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST No. 95

## FIGURE 32



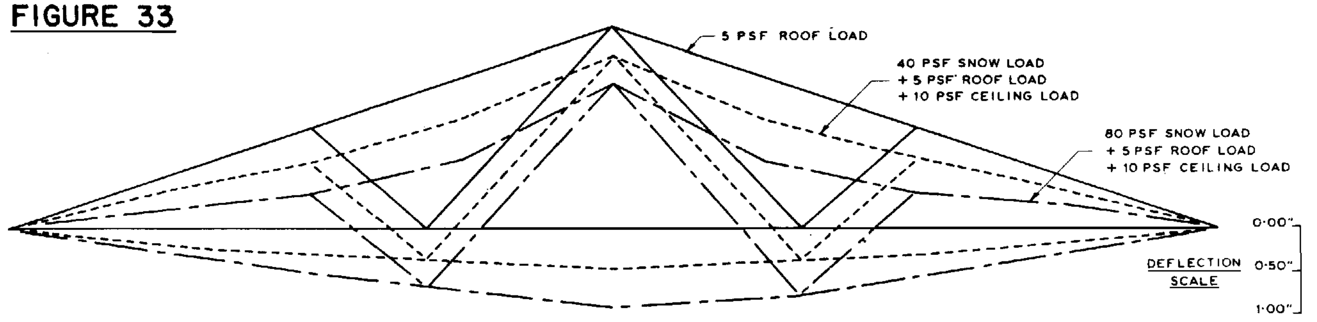

BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/650
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/330

FAILURE LOAD = 115 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

# DEFLECTION CHARACTERISTICS
# NAILED "W" TRUSS 28' SPAN 4/12 SLOPE 2'-0" O.C.

TEST NO. 101

## FIGURE 33

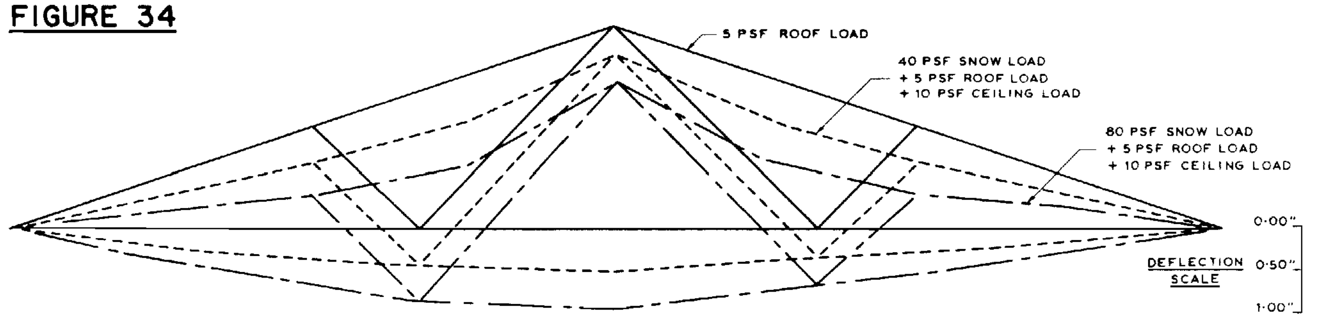



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/710
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/380

FAILURE LOAD = 140 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST NO. 102

## FIGURE 34

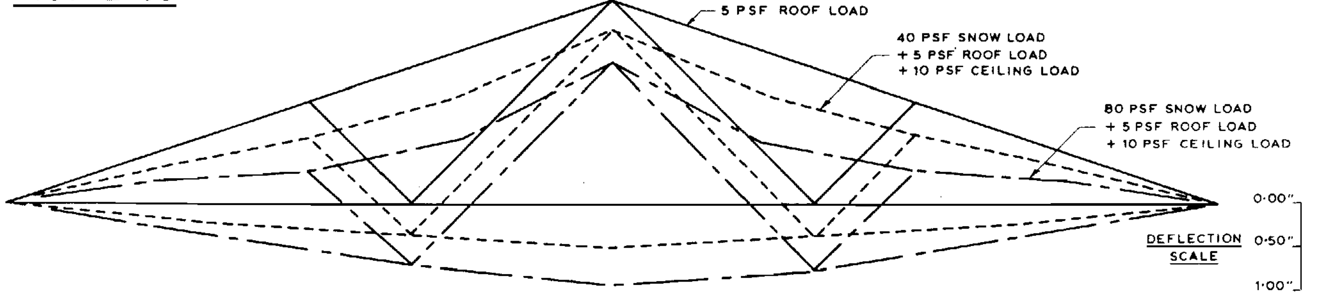



BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/690
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/370

FAILURE LOAD = 132 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

TEST NO. 105

## FIGURE 35




BOTTOM CHORD DEFLECTION RATIO FOR 40 PSF SNOW LOAD = 1/680
BOTTOM CHORD DEFLECTION RATIO FOR 80 PSF SNOW LOAD = 1/360

FAILURE LOAD = 155 PSF SNOW LOAD
+ 5 PSF ROOF LOAD
+ 10 PSF CEILING LOAD

## DEFLECTION CHARACTERISTICS
## NAILED "W" TRUSS 28' SPAN 4/12 SLOPE 2'-0" O.C.
## 2 X 5 TOP CHORDS

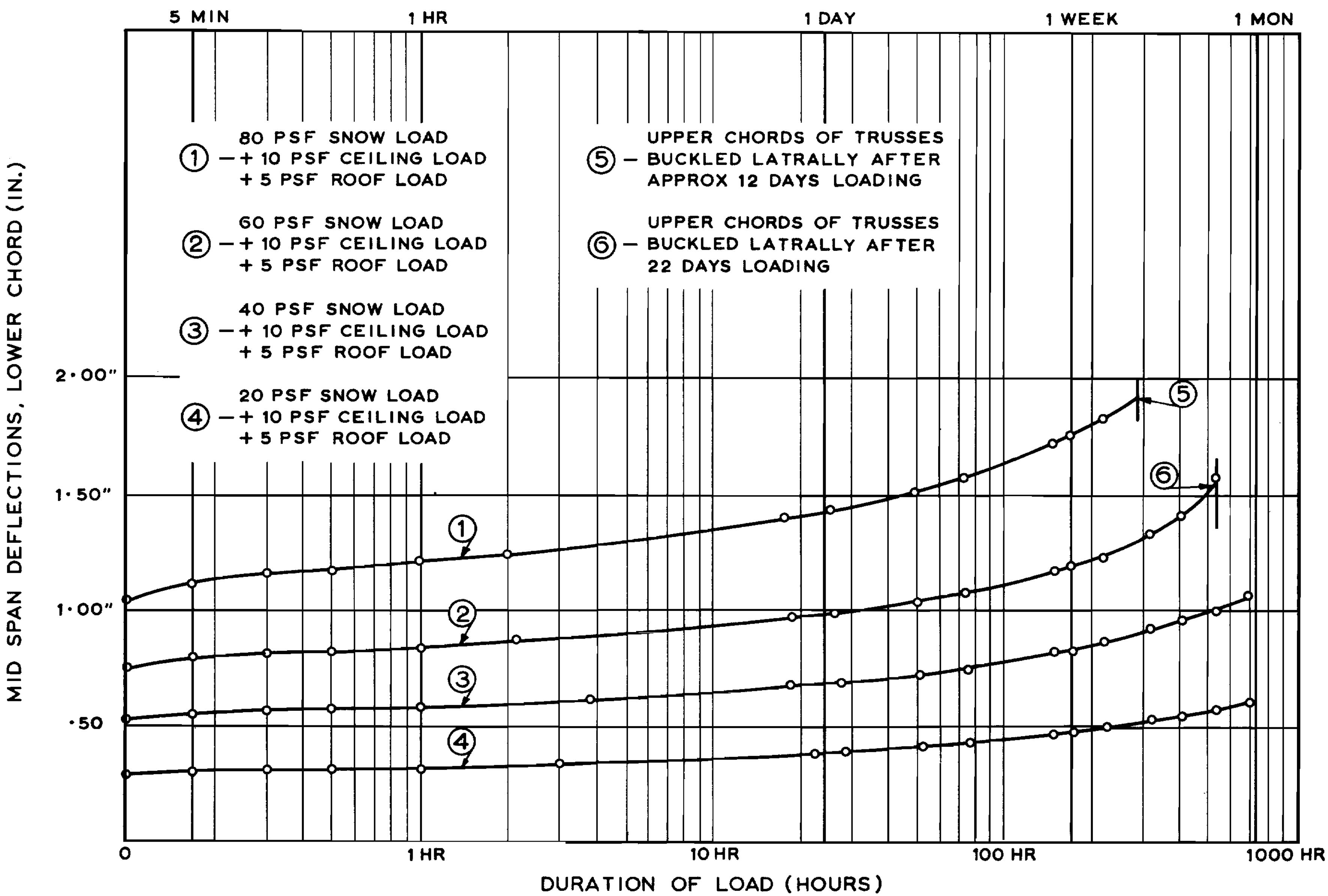



FIGURE 36 DEFLECTION VS TIME CURVES FOR NAILED W TRUSSES OF 26' SPAN, 4/12 SLOPE WITH VARIOUS LOADINGS

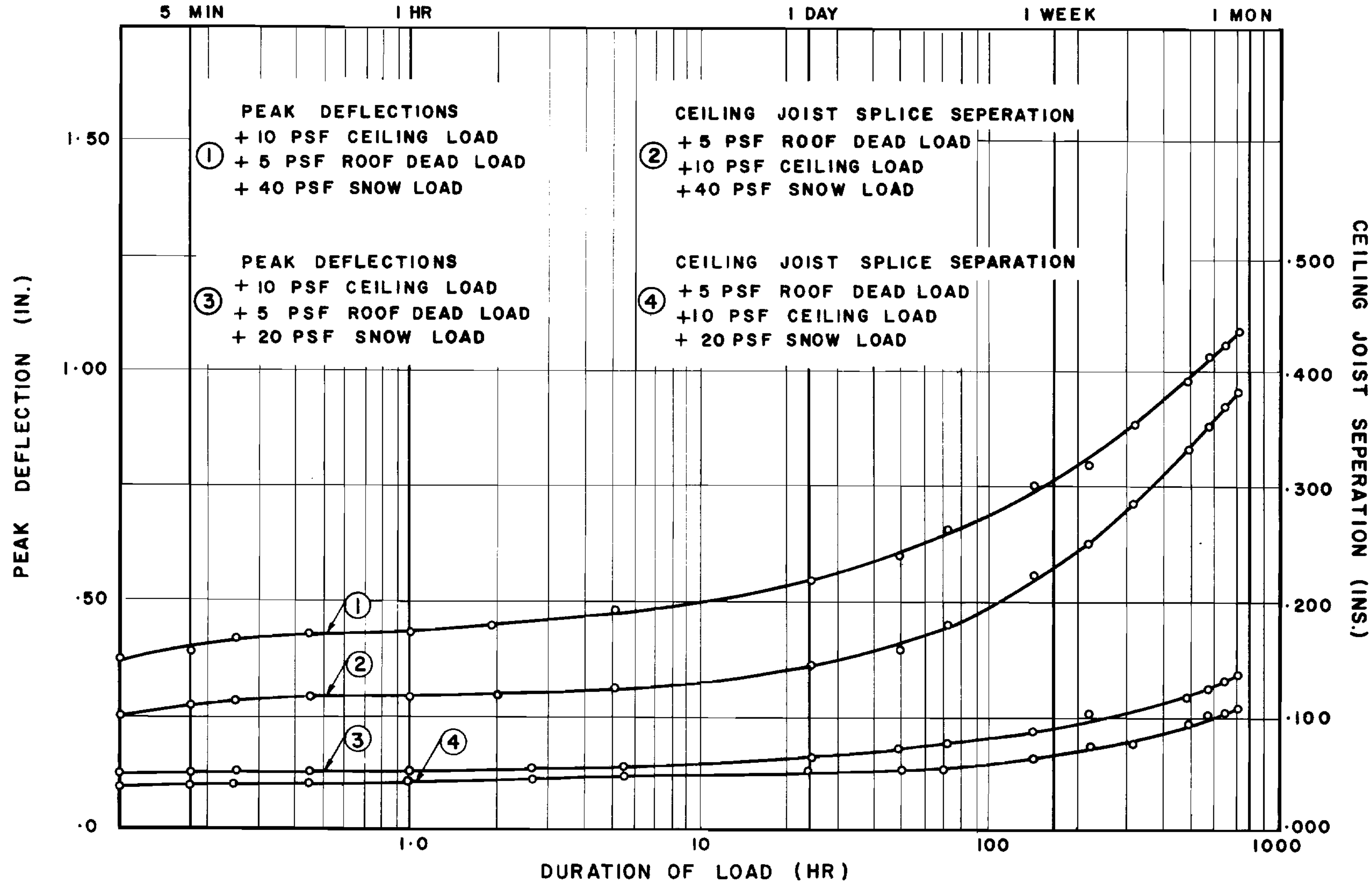



**FIGURE 37 LONG TERM DEFLECTION AND CEILING JOIST SPLICE SEPARATION**

**TYPE I CONVENTIONAL CONST. 24' SPAN, 5/12 SLOPE, ROLLER SUPPORTS**

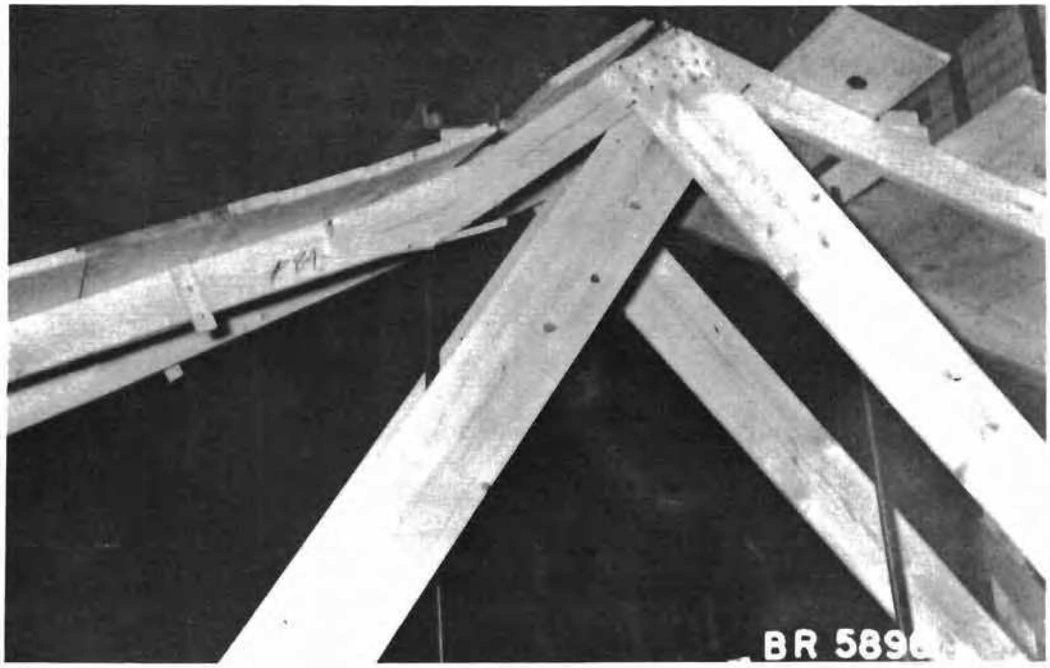
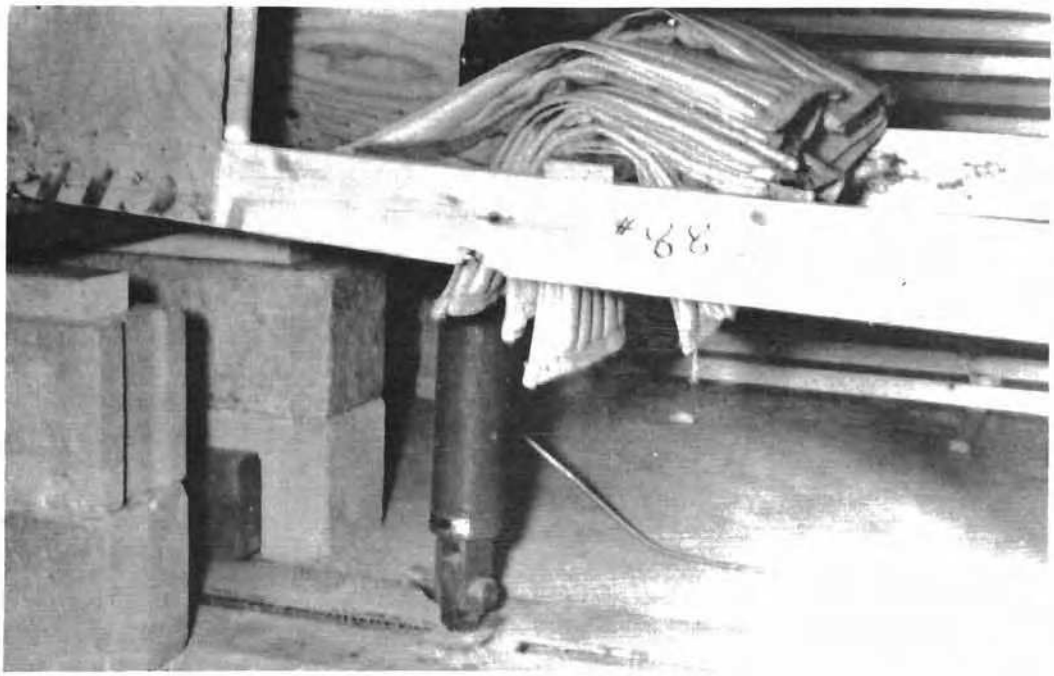



Figure 38. Failure caused by upper chord breaking near peak.




BR 5893

Figure 39. Failure caused by the lower chord breaking in tension near the heel joint.

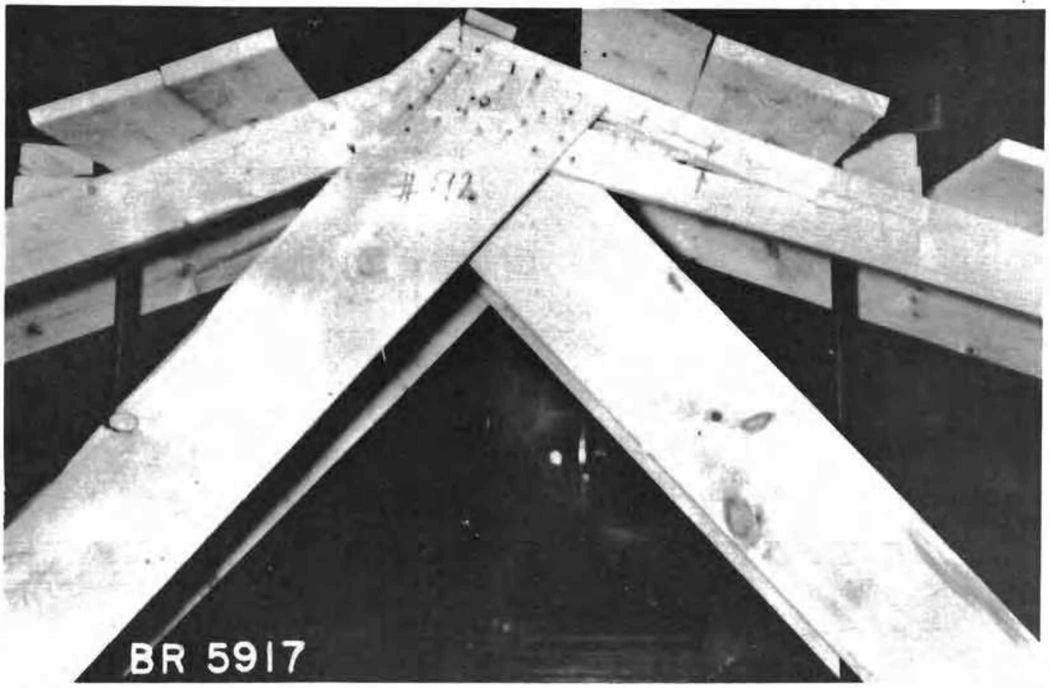



Figure 40. Failure in the upper chord near the peak, caused by the tension action of the long diagonal splitting the top chord.


Figure 41. Shear failure in the heel gusset plates in Test No. 94.

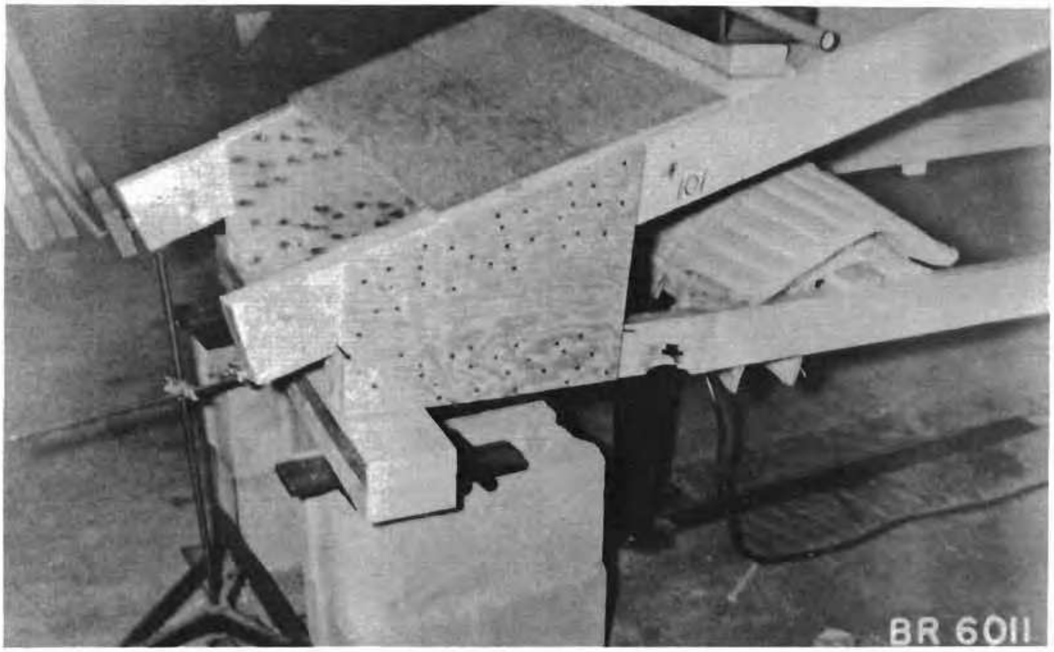



Figure 42. Failure caused by the lower chord breaking in tension near the heel joint.

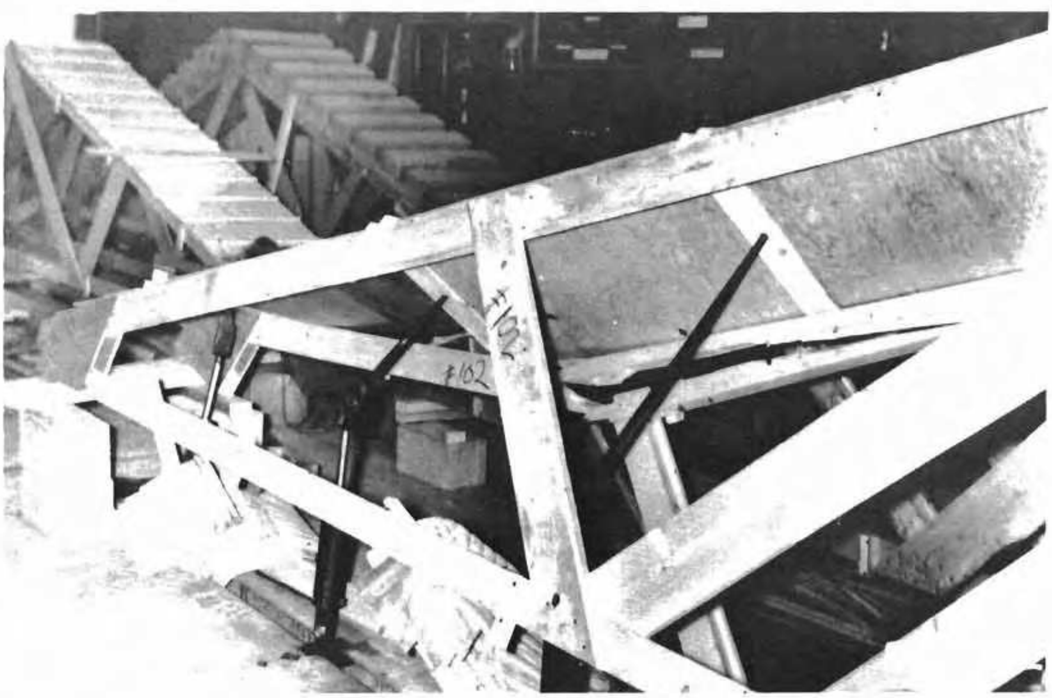


Figure 43. Failure in Test No. 102 caused by the lower chord breaking in tension near the heel joint. The upper chord broke immediately afterward.

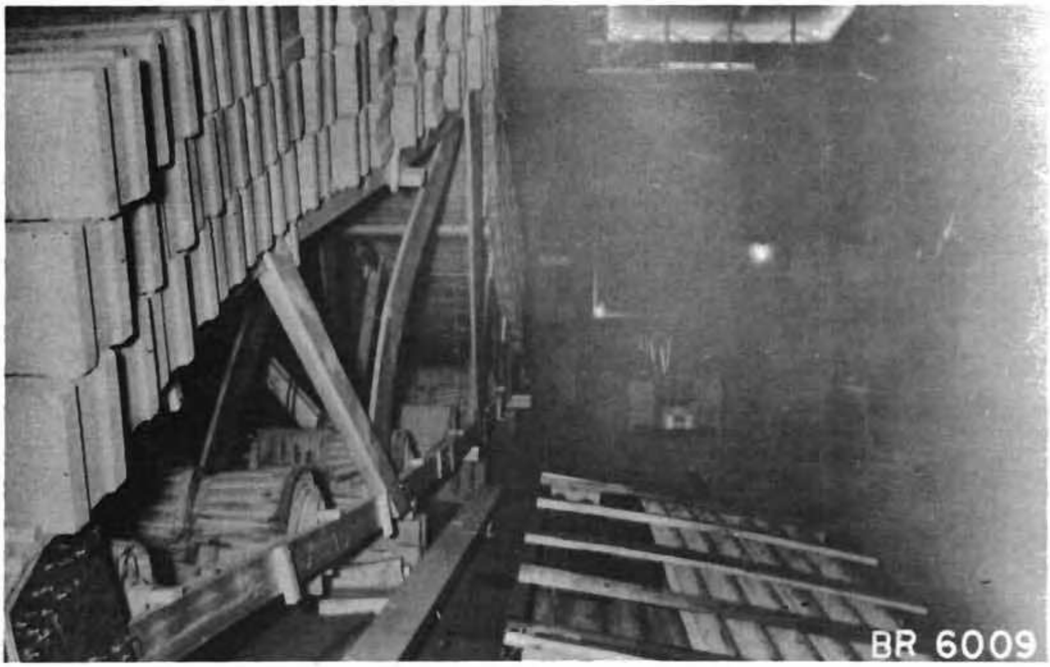



Figure 44. Collapse of trusses loaded with 80 p.s.f. snow load after 12 days loading. Failure due to lateral instability.

BR 6010

Figure 45. Collapse of trusses loaded with 60 p.s.f. snow load after 22 days loading. Failure due to lateral instability.

BR 5913

Figure 46. Photo showing trusses loaded with 60 p.s.f. snow load (front pair) and 40 p.s.f. snow load (rear pair).

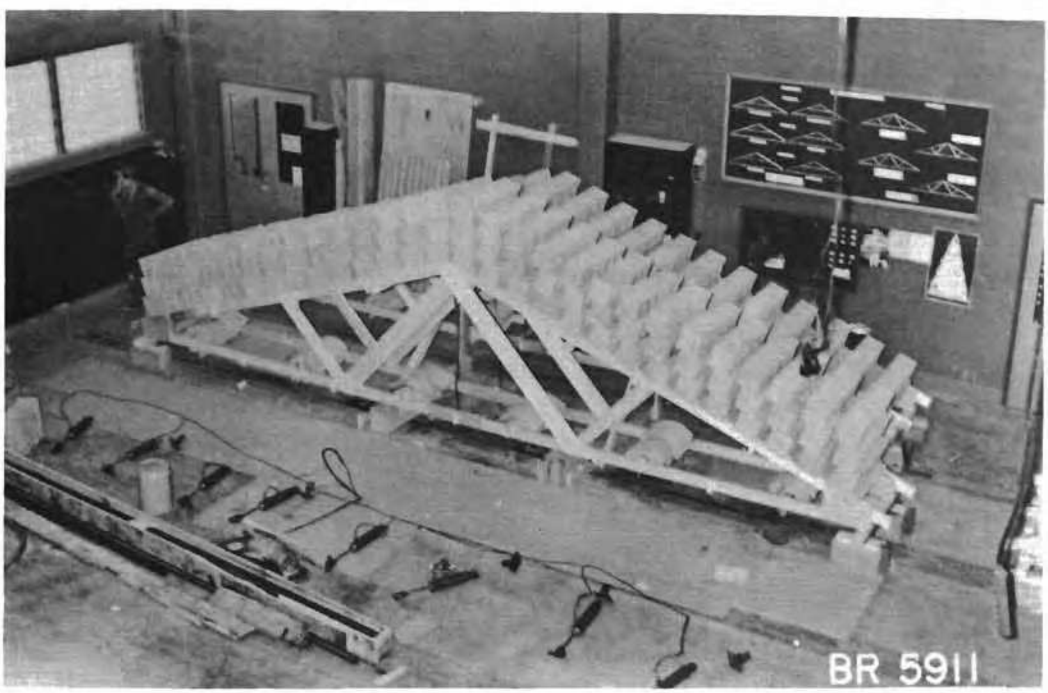



Figure 47. Photo showing trusses loaded with 80 p.s.f. snow load (front pair) and 20 p.s.f. snow load (rear pair).


Figure 48. Photo showing conventional construction loaded with 20 p.s.f. snow load (centre of photo).

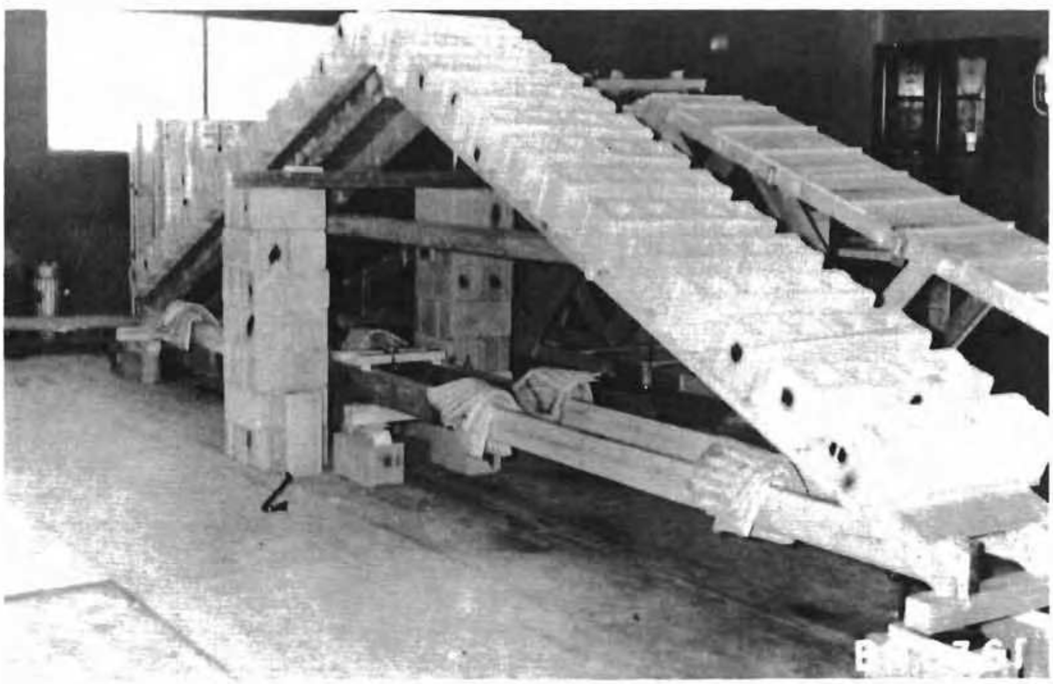


Figure 49. Photo showing conventional construction loaded with 40 p.s.f. snow load (foreground).