1939

Elastic properties of glued laminated rafter sections

James W. Martin

Iowa State College

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ELASTIC PROPERTIES OF GLUED LAMINATED RAFTER SECTIONS

by

James W. Martin

A Thesis Submitted to the Graduate Faculty
for the Degree of

MASTER OF SCIENCE

Major Subject Agricultural Engineering
(Farm Structures)

Signatures have been redacted for privacy

Iowa State College
1939
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INTRODUCTION

The Project

History

This study is a part of the general project of the Iowa Agricultural Experiment Station, "An Investigation of Farm Building Losses Due to Wind and Fire."

The investigation was undertaken as a major project in the year 1930. Since then the work has been pursued in several fields of activity. Briefly, the work in connection with this division of the project may be summarized as follows:

1. Field observations of wind damage
2. Statistical study of wind losses to Iowa farm buildings
3. Aerodynamic studies of wind pressure distribution on farm buildings
4. Structural analysis involving wind loads, dead loads, and combination of dead and wind loads
5. Laboratory tests of model structures subjected to loads approximating actual loading conditions
6. Building designs and recommendations applicable to Iowa conditions.

The statistical study and field observations of wind damage have shown very definitely the need for more
wind-resistant construction.

The aerodynamic phase of the problem has been confined largely to adapting the results of other investigators with regard to the nature and distribution of wind pressure.

Proper shape to provide stability under dead loads and the determination of reactions and bending moments have been the objectives for the structural analysis. Several fundamental assumptions are usually necessary before a structural analysis can be completed. These assumptions are necessary because of the indeterminate nature of most farm buildings. The closer these assumptions can approach actual field conditions the more valuable the solution.

The laboratory work consists of detailed study and tests of conventional types of barn rafters. Also, conventional methods of joining members have been studied in detail. After field observations disclose certain weaknesses, equipment is designed and constructed to place loads on model structures similar to that imposed by the wind. Design calculations are checked and verified, and various suggested improvements are tested, in an attempt to secure economical and practical design and construction standards. Building plans and recommendations are then issued according to the results of these tests.
Purpose

The purpose of this division of the project is to formulate practical and economical building design and construction standards that will reduce the losses resulting from wind damage.

Justification for the Present Study

The statistical studies seem to indicate that some basic studies of barn roof design are necessary before such structures can be made satisfactory as well as economical.

The Iowa Mutual Tornado Insurance Association alone paid claims amounting to $731,152.15 in 1936 and has paid average annual claims for the past nine years of $323,308.67. This high annual wind damage to Iowa farm buildings indicates a very real need for improved design and careful construction of farm buildings in order that these losses may be reduced to a minimum.

For the four-year period 1930 to 1933, wind damage to barns accounts for over 55 per cent of the total losses to farm buildings. Since the barn is by far the largest single item of loss and since our efforts can be directed more effectively to a single item rather than to a number of items, the barn has received the major portion of the attention. However, no less effort should be directed towards reducing the damage to minor buildings.
Barn roof trusses are generally classified under three general headings, namely: (1) gable, (2) gambrel, and (3) Gothic. During the past few years the Gothic roof has enjoyed a wide popularity. This is due to the pleasing appearance and clear mow space it provides, as well as to economy of materials and labor for its construction. There are approximately 42 board feet of lumber in a 36-foot curved rafter of the type investigated by Test (17), 35 board feet of which are above the plate line. The braced rafters as tested by Pickard (14) contain an average of 46 board feet. The use of the glued, laminated rafter allows a 24 per cent saving in material.

During a recent field trip several Iowa farmers were interviewed personally. Every farmer was interested in the Gothic arch barn. They expressed the opinion that if the curved roof barn were made as strong as the braced rafter roof their preference would be for the curved roof.

Field observations have shown that a great many barns made by using laminated, bent rafters have sagged at the ridge. Experience has indicated that this condition is wholly unnecessary provided care is exercised in their design and construction. Investigation of such failures has shown the cause to be, for the most part, improper shape, slippage between laminations, and slighting of materials in construction.

The ideal shape for a curved roof barn would be such that the line of thrust of the loads falls within the
boundaries of the material. This result cannot be accomplished when the rafters are circular.

Slippage between laminations is possible when the rafters are constructed by using only nails to fasten the laminations together. Bolts add to the strength and stiffness of laminated rafters, but a 1/4-inch bolt hole removes 14.3 percent of the cross-sectional area of the member. The use of glue in laminated rafter construction increases the stiffness of the rafter more than any other type of fastening. The effect of the glue is to give the laminated rafters an elasticity similar to that of solid timbers. This investigation is concerned with the proper amount and location of the material in the rafter.

The more recent design of the glued rafter calls for sections extending from the sill to the ridge. This construction requires a rafter approximately 36'-0" in length. In using lumber of standard dimensions, it becomes necessary to have joints in the rafter section. These joints materially weaken the rafter. How much these joints affect the strength and elastic properties of the rafter is a question to be answered by this investigation.

General objectives

From the foregoing discussion it is evident that a great deal of research work has already been conducted in connection with this project. The six fields of activity indicate the
magnitude and scope of this important problem. Information has been accumulating in the form of several theses during the years since 1930. Naturally such information has not been correlated with more recent experience in the field. The increased popularity of the Gothic arch barn roof, with its pleasing appearance, clear mow space, and saving in materials, indicates the need for continued research and investigation.

The general objectives set forth for this study are enumerated as follows:

1. To summarize the work of previous investigators
2. To study the elastic properties of glued, laminated rafter sections
HISTORICAL

History of Barn Framing

Braced rafter roof

The wood-frame construction of the modern barn has been a gradual evolution from the early barns of New England and the Central European countries. These early barns were massively framed affairs using lumber sawed or hewn from native trees. The heavy hewn timbers used by our forefathers have been hard to supplant.

The increasing scarcity of suitable timber and the demand for a simple construction that could be made quickly and easily have led to the gradual abandonment of the heavy timber framing and the introduction of the newer types.

Gothic arch

The Gothic type of barn roof is defined as one having rafters of a circular curvature meeting at a peak. It is believed (8) that this type of construction originated in Michigan. Barns with roofs of this shape were built as early as 1885 in Isabella County, Michigan, and in that county today there are several townships in which there is almost no other type of barn roof construction.
Sawed rafters. The first rafters for Gothic arch barns were made of 2" x 12" planks with the top edges sawed to the desired curvature. Later they were made with a greater curvature by using one-inch boards 8 or 10 inches wide and three or four feet in length. These boards were sawed to the proper curvature and then nailed together. The number of laminations depended upon the judgment of the carpenter building the structure. Both of these methods were wasteful of material and labor.

Sprung rafters. To avoid this wastefulness and yet preserve the appearance and other features of the Gothic roof, experiments were made with sprung rafters. The first barn embodying this construction, so far as is known, was erected in 1892 (8), but the method did not come into extensive use until after 1900. By this time a sufficient number of barns had been erected to demonstrate the success of this type of construction.

The sprung or bent rafters were first made by bending plies of one-inch material into the arc of a circle. The first sprung rafters were made of 1" x 4" or 1" x 6" members, each lamination being securely nailed to an adjacent lamination. Unless extreme care was used in fastening the laminations together, the ridge had a tendency to sag out of shape. This tendency to sag resulted from one lamination's slipping past another. To overcome this difficulty, bolts were substituted for part of the nails. The bolts prevented any
appreciable movement between adjacent laminations.

In the central states, architects' plans for bent-rafter roofed barns were first published in 1916 (11). During the past 20 years glue has been used extensively in the construction of wood parts of unusual properties, dimensions, and shapes. This increased use of glue suggested the possibility of using it to increase the strength and rigidity of the laminated, bent rafter. Most of the more recent rafter designs specify the use of glue. In the case of factory prefabricated rafters, glue is applied to the rafter laminations, and clamps hold the members in proper shape until the glue dries. No nails or bolts are used. In case the rafter is to be made on the job, nails and bolts are usually used in addition to the glue. The nails and bolts hold the laminations in place while the glue dries. Once set, the glue takes all the horizontal shear developed between adjacent laminations. The nails and bolts then provide a factor of safety in case the glue should fail.

Value of Farm Buildings

Farm buildings in the United States represent an investment of nearly thirteen billion dollars (18). If depreciation and taxes be estimated conservatively at 6 per cent, an annual outlay of over 700 million dollars will be required just to maintain this investment. Iowa ranks first among the states
with a total investment in farm buildings slightly in excess of one billion dollars.

With the exception of the dwelling, the barn is usually the largest and most expensive structure on the farmstead. Even so, when compared with the new school house, the new post office, or the new courthouse, the cost is exceedingly small; consequently, the individual farmer has not been able to have the services of competent engineers and architects.

Magnitude of Iowa Wind Damage

The results of the statistical study reveal some important and interesting facts concerning the amount and distribution of wind damage. If one may judge from field observation of wind damage, the lifting effect of wind on roofs of structures is responsible for more failures of buildings to withstand storms than any other one cause. Of course, after a building has been completely demolished, it is difficult to determine just where failure first occurred.

Anyone at all familiar with farm construction knows that a most important problem is to build to withstand the strong wind pressure against sides and roof and still have the structure remain rigid and in alignment. Some of our farm buildings have been constructed without keeping these requirements in mind. Only within the past decade have designers of farm buildings taken into account the reduced pressure
on the leeward side as well as the impact pressure on the
windward side. This disregard for the reduced pressure has
caused a great many failures.

Table I shows the magnitude of the losses as paid by
the Iowa Mutual Tornado Insurance Company.

Table I. Magnitude of Wind Damage
for the Years 1930-1938

<table>
<thead>
<tr>
<th>Year</th>
<th>Amount of Losses</th>
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<tbody>
<tr>
<td>1930</td>
<td>$219,846.59</td>
</tr>
<tr>
<td>1931</td>
<td>272,065.82</td>
</tr>
<tr>
<td>1932</td>
<td>149,792.16</td>
</tr>
<tr>
<td>1933</td>
<td>403,180.67</td>
</tr>
<tr>
<td>1934</td>
<td>368,285.84</td>
</tr>
<tr>
<td>1935</td>
<td>104,224.08</td>
</tr>
<tr>
<td>1936</td>
<td>731,152.15</td>
</tr>
<tr>
<td>1937</td>
<td>443,324.71</td>
</tr>
<tr>
<td>1938</td>
<td>219,916.05</td>
</tr>
<tr>
<td>Total</td>
<td>$2,909,778.07</td>
</tr>
<tr>
<td>9-year Average</td>
<td>$323,308.67</td>
</tr>
</tbody>
</table>

The average annual loss ($323,308.67) for the period
1930-38 is about three-fourths of the average annual loss
($410,000.00) for the 5-year period up to 1930. This trend,
by years, of the total wind damage to Iowa farm buildings is
shown in Figure 1. Such loss is a great economic waste to
Figure 1. Total Wind Damage.
Trend by Years

Figure 2. Wind Damage by Month of Occurrence
the farmers of Iowa: also, such destruction may cause the loss of life.

Studies made by Schweers (15) and continued by Clark (5) covered the period from 1930 to 1933. Information concerning the year 1934 was added later. Along with the trend by years, the distribution of wind damage by month should be investigated. The greatest amount of loss occurs in the months of May, June, and July, as shown in Figure 2. During these months cyclonic winds of high velocity are common. These high winds usually accompany heavy thunder storms, which are frequently attended by hail. Loss by breakage of glass and damage to roofing materials often results. Tornadoes, also, generally occur during the summer months and add materially to the annual wind damage.

During the month of May and the early part of June barns are nearly certain to be empty, but during July mows are at least partially full of hay. In designing a barn, consideration must be given to these varying conditions.

Constructional damage to buildings represents an average annual loss of $199,079, or over 75 per cent of the average annual loss of $261,221 for the period 1930-34. In order to care for livestock and crops properly, every farm must have, in addition to the barn and dwelling, several minor buildings such as: hog house, poultry house, crib and granary, and machine shed. The number and magnitude of losses suffered on each type of building are shown in Figure 3.
Figure 3. Total Damage to Farm Buildings by Wind
Figure 4. Constructional Damage to Farm Buildings
Buildings demolished by wind account for the major portion of all constructional damage as shown in Figure 4. Buildings out of plumb rank second. These two items contribute over 70 per cent of all constructional damage to Iowa farm buildings due to wind. Buildings moved off their foundations and roofs blown off add another 13 per cent. These figures show clearly that most failures are directly attributable to lack of proper anchorage and wind bracing. Too often the roof of a building is held down by the rafters being toenailed to the studs. During a high wind the roof is carried away. Lack of wind bracing both above and below the mow floor is common. Also, too frequently the ends of the barn do not have sufficient bracing to remain vertical or plumb during a high wind.

Damage to doors, shingles, roofing, and so forth, while of minor importance, does contribute to the total wind damage. A small amount of annual repair and inspection would return large dividends in the way of increased life and service of farm buildings.

Of the $111,492 which is the average annual damage caused by demolition of buildings, destruction of barns covers 65 per cent, as shown in Figure 5. It is interesting to note that only three dwellings were demolished annually. Thus in comparison with barns, dwellings account for less than three per cent of the number of structures damaged and less than one-half of one per cent of the loss in dollars.
Figure 5. Farm Buildings Demolished by Wind

Figure 6. Constructional Damage to Barns
Although dwellings comprise over one-half of total investment in farm buildings, they suffer less than one percent of the damage resulting from demolition by wind. This fact demonstrates quite conclusively that it is possible to build to withstand wind storms. However, the added cost which would be necessary in order to make a farm barn completely wind-proof would not be justified. Constructional damage to barns only is shown in Figure 6.

Review of Literature

Barn framing requirements

Service requirements. There are a number of functional requirements which are common to all types of barn roofs. They are:

1. To provide shelter for animals
2. To provide adequate mow space for storage of feed
3. To provide clear height for convenient handling of feed
4. To provide sufficient width at the ridge to allow the use of standard hay carrier equipment

Structural requirements. Strength, stability, and rigidity must all have careful consideration when the structural requirements are determined. The barn must have sufficient strength to withstand the loads imposed upon it. The greatest loads imposed upon a barn are not those resulting
from the main purpose for which it was designed, but from wind loads. Because of the uncertainty of the direction and velocity of the wind, the probable wind stresses are very uncertain. The gusty nature and vibrations of the wind set up stresses of indeterminate nature. In designing roof members it is important to know whether impact pressure or reduced pressure occurs over the roof area supported by the members.

**Economic requirements.** Economy of materials and labor are two important considerations in any structure. In order to obtain economy of materials it is necessary: (1) to select barn size such that it meets the requirements of the farm and livestock, (2) to use standard dimension lumber, and (3) to use local building materials as much as possible.

Economy of labor can only be obtained through the use of a minimum number of skilled workmen. In general, farm labor is less expensive than workmen connected with the building trades. However, the first cost is not always the most important. Frequently, a few dollars saved in first cost require increased cost in repairs and depreciation.

**Appearance requirements.** Even in a barn, appearance is of utmost importance. Some very good barns are unsatisfactory to their owners because they do not present a pleasing appearance. A barn should be an individual unit, and harmony with other buildings need not be attempted. Certain farmers
have the impression that the size of the barn must match the size of their farm. In other words, a large farm requires a large barn. However, the size of the barn should be governed by stock and feed requirements. An impression of stability and permanence is gained by proper proportioning of rafter length to barn width.

Selection of a standard barn shell

Dimensions. There are three standard barn widths, namely: 32 feet, 34 feet, and 36 feet. Figure 7 shows the results of observations of barns and barn plans made by Barre (3): (1) 34- and 36-foot widths of dairy and general purpose barns are generally recommended; (2) the 36-foot barn is the most common; (3) the average of all barn widths is 33.9 feet. For Iowa conditions the 34-foot barn is considered standard. Also in the interests of economy and comfort of the animals housed, 34-foot barns should be used.

The width of the lower structure and the amount of storage space largely determine the principal dimensions of the barn. The height of roof is largely determined by the amount of feed storage necessary to take care of the livestock housed in the barn. New developments in feed storage practices may affect the dimensions to be used.

Economic use of materials. The determination of the proper amount of material depends upon the evaluation of
Figure 7. Observations of Barns and Barn Widths
dead loads and wind loads. The practice has been in the past to use just enough material, the amount depending upon the judgment of the carpenter building the structure. This practice causes overdesigning of some parts and stinting of materials in other parts.

It is impossible to specify exactly which sizes of members are best, because of the indeterminate nature of the structure, variation in methods of fastening members, variations in the character of the wood, and variations in the quality of workmanship.

The shape of roof to provide stability. Stability under dead loads is a fundamental requirement of all roof structures. The dead load on rafters is composed of the weight of the shingles and sheathing, plus the weight of the rafters themselves. Snow loads also contribute to the weight the rafters must support. Unless the rafters supporting the roof are sufficiently rigid, and unless the shape is such that the line of thrust of the loads falls within the boundaries of the materials, undesirable stresses and deflections will occur.

When rafters built in the form of an arc of a circle are used, it is almost impossible to make the line of thrust fall inside the materials. However, the shape resulting from using a 33'-0" radius produces very little bending in the rafters as a result of dead load thrust. No sagging at the ridge is expected under these conditions.
Wind pressure distribution

As previously stated, the greatest loads to which a barn frame is subjected are those resulting from the wind. These loads are classified as eccentric loads since they tend to produce bending in certain parts of the structure.

The successful design of a structure such as a barn frame depends upon the evaluation of the direction and magnitude of the wind load.

Theoretical calculations. Sir Isaac Newton was the first to give a theoretical treatment of the resistance of plates to the motion of fluids.

"Wind Stresses in Buildings" by Robins Fleming (7) gives a translation of Newton's Prop. XLVIII: "The velocities of pulses propagated in an elastic fluid are in a ratio compounded of the subduplicate ratio of the elastic force directly, and the subduplicate ratio of the density inversely; supposing the elastic force of the fluid to be proportional to its condensation."

According to Fleming (7), "This means that the velocity \( v \) varies as \( \sqrt{\frac{p}{\rho}} \) or \( p \) varies as \( dv^2 \). For wind pressure, the density of the air being constant, we have the law that the pressure varies directly as the square of the velocity, which has remained almost undisputed since Newton's day."

The pressure \( p \) in pounds per square foot on a plane
surface normal to the direction of flow of a fluid having a relative velocity \( v \) in feet per second is equal to the weight of a vertical column of the fluid one square foot in cross-sectional area and of height \( h \), in feet, equal to that distance through which a freely falling body must fall to acquire the velocity \( v \).

\[
p = \frac{whv^2}{2g}
\]

For air at a temperature of 15° C. and 760 mm. Hg, the weight per cubic foot is .07651 lbs.

\[
p = \frac{.07651v^2}{2 \times 32.2}
\]

\[
p = .001189v^2
\]

If \( V \) denotes the velocity of the wind in miles per hour,

\[
p = .001189(V \times 1.47)^2 = .00257V^2
\]

This equation may be expressed in the general form \( p = kV^2 \), in which \( k \) is an empirical coefficient.

W. J. M. Rankine is given credit for a second theoretical approach to this problem of wind pressure calculation. If the wind were directed as a finite stream against an infinitely large surface, so that the direction of the air is completely changed, an equation expressing the force against that surface may be obtained. From the laws of mechanics, momentum is defined as mass multiplied by velocity. \( \frac{dAv}{g} \) is the mass of the quantity of air striking the surface per second. \( \frac{dAv^2}{g} \) is the momentum of the quantity of fluid whose motion is deflected per second, where \( d \) is the density of
the fluid in pounds per cubic foot, \( v \) the velocity in feet per second and \( A \) the cross-sectional area of the air stream in square feet.

The total pressure against a surface may be found from the principle that force multiplied by time equals change in momentum. \( p = \frac{dA v^2}{g} \).

For air at a temperature of 15°C and 760 mm. Hg, the weight per cubic foot is .07651 lbs.

\[
p = \frac{.07651v^2}{32.2}
\]

If \( V \) denotes the velocity of the wind in miles per hour, \( p = .005V^2 \)

Newton and Rankine, by purely theoretical calculations and disregarding the suction created on the leeward side of any object, recommended values for \( k \) of .0025 and .005 respectively. This fact illustrates the reason for so much confusion because, starting with the same assumption, these two men found two widely different values. Recourse must therefore be made to experimentation.

For the calculation of wind pressure on surfaces inclined to the direction of the wind, Newton suggested the relation \( p_n = p \sin^2 \theta \), where \( p_n \) is the intensity of pressure on the inclined surface, \( p \) the pressure on a surface perpendicular to the wind, and \( \theta \) is the angle of inclination of the surface to the horizontal. This formula has been found to be erroneous and has been little used.
The empirical relation $p_n = p \sin \theta (1.84 \cos \theta - 1)$ suggested by Hutton has had wide use, also the more recently accepted formula of Duchemin, which gives

$$p_n = \frac{2 \sin \theta}{1 + \sin^2 \theta}$$

None of these older methods take into account the suction on the leeward side or the reduced pressure on the windward side.

Figure 8 shows the relationship between the formulas as recommended by various authorities on wind pressure. The results vary so widely that the average designer is at a loss to know just what values should be assigned to take care of the probable wind loads. Therefore, the use of these formulas is not suited to the design of such a structure as a barn. The more recent approach to the problem is to assign a wind pressure distribution diagram to the structure in question.

**Experimental determination.** The problem of obtaining wind pressure information can be approached from two ways: (1) by experiments on models in wind tunnels, and (2) by observation in natural winds.

Experiments on models have contributed much to our store of knowledge in the fields of hydraulics and aerodynamics. Studies of wind pressure on buildings have been made here and abroad, but full confidence has not been placed in the results, because of the uncertainty as to the behavior of a building in a natural wind.
Figure 8. Comparison of Formulas for Normal Wind Pressures
In this same connection, Fuller and Kerekes (9) may be quoted as follows: "Although no generally acceptable data are yet available, important structures such as armories, field houses, and the larger steel mill buildings should be investigated for suction effects as well as for positive wind pressure. . . ."

By the use of the wind tunnel the velocity and direction of the wind are easily controlled. The exact pressure distribution diagram can be determined. When the magnitude and distribution of the wind pressure are known, the total force on the model can be measured. The disadvantages of this method are: (1) that the fine detail of the actual building cannot be reproduced on the model and (2) that the pressure on the full-size building may be somewhat different from that at the corresponding location on the model because of the existence of what is known as "scale-effect."

When measurements are taken in a natural wind, the conditions are different, as the direction and velocity of the wind are no longer under control. It is no longer easy to obtain conditions favorable for measurement, because the speed and direction of the winds are changing continuously.

In U. S. Bureau of Standards Scientific Paper No. 523, Hugh L. Dryden and George C. Hill (6) have the following comment on the problem of wind stress analysis:

"One very important factor retarding the advance of our knowledge of wind pressure is the great complexity of the
subject. When we consider that the stresses due to wind pressure depend on the form of the structure, the size of the structure, the speed and direction of the wind, and the location of surrounding structures; when we consider the rapid fluctuations in speed and direction of the wind; and when we consider that there is no practical method by which the wind loads may be obtained from the stresses in particular members of complicated structures, it is not amazing that progress has been slow.

The problem of wind stress analysis, therefore, resolves itself into two distinct questions. First, what are the maximum loads produced by the wind and how often do they occur? Second, what are the stresses in the various members of a structure resulting from these loads?

As expressed by Hugh L. Dryden and G. C. Hill (6), no hope for advancement can be expected unless wind loads are investigated rather than wind stresses. The recent experiments carried out largely in artificial wind streams produced in wind tunnels have proved so useful in the development of aeronautics that a great many valuable investigations are now being conducted on structures.

According to the theoretical calculations as proposed by Newton, the maximum increase in pressure produced by the wind is equal to the kinetic energy of the wind striking a surface. Kinetic energy is expressed by the formula $1/2 \, mv^2$. This
increase in pressure is usually termed the velocity pressure or impact pressure. Since experiments have established the presence of reduced pressures over a certain portion of a surface, it is no longer possible to use this method when determining the pressure differences over an entire structure.

In aerodynamics it is convenient to express all observed pressure differences as ratios of the pressure difference to the velocity pressure. In order to establish this condition it is necessary to consider the component parts making up the total pressure upon an object in an air stream.

The absolute pressure on an object in an air stream is composed of static pressure \( p_s \), which is the barometric pressure, and the wind pressure \( p_w \) caused by the presence of the object in the air stream. The wind pressure \( p_w \) may be either positive, negative, or zero. In other words, \( p \), which by definition is equal to \( p_s - p_w \), may be either greater, equal to, or less than \( p_s \).

The expression \( \frac{p_w}{p_v} \) gives the ratio of the pressure difference to the velocity pressure \( p_v \),

\[
\frac{p_w}{p_v} = \frac{p_w}{(1/2mv^2)} = \frac{k(vld)}{u}
\]

where, \( d \) is the air density, \( v \) the wind speed, \( u \) the viscosity of the air, \( l \) a linear dimension fixing the scale, and \( k \) a constant.

The wind pressure \( p_w \) could be measured in any convenient unit, but there are advantages in using the velocity pressure \( p_v \) as the unit.
Expressing the wind pressure $p_w$ in the terms of the velocity pressure $p_v$ gives: 

$$p_w = \frac{k(vl_d)}{u} p_v.$$ 

The expression applies only to geometrically similar bodies. For bodies without curved surfaces and with sharp curves, $p_w$ is practically independent of the wind speed and size of the object; that is, $\frac{kvld}{u}$ is a constant for any part on the object. When scale-effect is considered to be small, $p_w = C p_v$, where $C = \frac{kvld}{u}$, and is a pure member independent of the units used so long as the pressures are all measured in the same units. Therefore, if the value of $C$ is found for any point on a model, that value will apply to a model of any size at any wind velocity.

Hugh L. Dryden of the U. S. Bureau of Standards is the outstanding authority on wind pressure distribution. The wind pressure distribution diagram as shown in Figure 9 has been submitted to Dr. Dryden, who expressed the opinion that this figure illustrates the exact pattern of the distribution of wind pressure.

**Wind-load approximations.** The actual wind loads to which a barn is subjected are non-uniformly distributed loads. This type of loading cannot be reproduced in the laboratory on scale models. However, a series of concentrated loads can be applied to the model which will approximate actual loading conditions. In the tests conducted by the previous investigators, a varying number of loads have been used. The first
Figure 9. Wind Pressure Distribution Diagram for Gothic Arch Barns
tests were conducted by using only a single concentrated load to approximate actual loading conditions. The usual procedure was to apply seven to nine loads to each half of the rafter.
THE INVESTIGATION

Justification for Continued Study of the
Glued, Laminated, Bent Rafter

The increasing use of the glued, laminated, bent rafter justifies the continued study.

During the past 10 years, glued sections and assemblies have become increasingly popular. Glue is now used in the construction of almost every type of farm building. A recently developed plywood brooder house constructed with glued, laminated, bent rafters is becoming increasingly popular. A recent development in larger structures is a hog house constructed by gluing the stud to the rafter. A double plywood gusset plate provides strength and rigidity to the plate joint and approaches continuous framing.

In using glued, laminated construction, several problems are encountered. W. D. Test (17) reported on the results of testing glued, laminated, bent rafters by saying, "When the load corresponding to a wind velocity of 110 miles per hour was applied, the two inner laminations of the leeward rafter failed in tension near the plate joint on the inside edge of the rafter. . . ."

Since a fracture usually starts in the extreme fibre it is important that the best material be used on the outer
lamination. Joints are a serious problem especially when they occur at the points of maximum bending moment.

Some designers have been concerned over the stresses introduced in the laminations as a result of bending to the desired curvature. Unless the curvature is very moderate, bending of laminations to the desired shape induces in them initial stresses of considerable magnitude. However, it is believed that these stresses equalize themselves after a period of use.

Specific Objectives

The specific objectives of this investigation are:

1. To determine the quality of glued joints after 8 years of service conditions
2. To study the effect of joints upon the elastic properties of a glued rafter section
3. To study the effect of initial bending stress upon the final strength of the rafter
4. To investigate the reactions and bending moments resulting from combined dead and wind loads
5. To investigate the stability of roofs under dead loads
6. To establish a basis for approximating the conditions of end support
Preliminary Considerations

Glued wood construction

Forest Products Laboratory in its "Wood Handbook" (19) makes the following statement with regard to glued laminated members:

"Laminated curved members are produced from dry stock by bending and gluing together in one operation several comparatively thin pieces without softening them by steam or hot water. This process has the following advantages over the bending of single-piece members: (1) the laminations can be made so thin that bending to the required radius involves only moderate stress and deformation of the wood fibers; consequently, the use of steam or hot water is unnecessary and less subsequent drying and conditioning is required; (2) because of the moderate stress induced in bending, stronger parts are produced; (3) the tendency of laminated members to change shape with changes in moisture content resulting from changes in relative humidity is less than that of single-piece bent members and can be made negligible by making the laminations thin in comparison with the radius; (4) ratios of thickness of members to radius of curvature that are impossible in bending single pieces can readily be obtained by laminating; also members having reversed curvature are more readily made by laminating thin
plies, and curved parts of any desired length can be produced by staggering the joints in the laminations.

Softwoods are ordinarily used for laminated bent structural members, and thin material of any of the softwoods can be satisfactorily bent for such purposes. The choice of species is dependent primarily upon the cost and required strength.

**Gluing properties of wood.** Because of the increasing use of glue in building construction due consideration must be given to the gluing properties of wood.

The strength and durability of glued joints depend upon: (1) the kind of wood and its preparation for use, (2) the kind and quality of glue and its preparation for use, (3) the details of gluing, (4) the type of joint, (5) the conditioning of the joints, and (6) the protection given in service.

Sometimes difficulty is experienced in gluing certain kinds of woods. The problems involved in gluing wood members are affected by the following factors: (1) the density of the wood, (2) the structure of the wood, and (3) the kind of glue.

Experience has shown that heavy woods are more difficult to glue than light woods and that hardwoods are more difficult to glue than softwoods.
Glues used in wood-making. Animal glue has long been used by cabinet makers, and starch glue has been used quite generally for veneering. Casein glue is a more recent development and is used extensively where water resistance is desired.

The increased use of cold casein glue has given new life to the design of all farm structures. The conventional nailed and bolted joints are now being replaced by glue. The glued, laminated, bent rafter is a typical example of the increased use of glue in the construction of farm buildings.

Construction of the glued, laminated, bent rafter. The individual contemplating the use of laminated rafters in barn construction is confronted with two options: (1) making the rafters on the job and (2) purchasing the prefabricated rafters.

The problem of transporting members 36 feet in length has been rather serious. It is believed that the greatest need at the present time is for reliable information relative to making the rafter on the job.

In making the rafter continuous from sill to ridge, the mow floor cannot be used as a form. A form can be made by using a base of planks laid on cross pieces of 2" x 6"'s spiked to posts driven solidly into the ground. This form provides a rigid, level surface on which to work.
After the arc of the rafter is laid out, a row of 2" x 4" blocks about 2 feet apart are spiked to the base. These blocks are points on the arc of a circle. The first lamination is nailed to the blocks with 6d finishing nails. Each succeeding lamination is held in place during construction by using 6d finishing nails as required. Cold casein glue is used between laminations, and the entire rafter is fastened together by using 1/4" x 6" bolts. Extreme care should be exercised in breaking the joints. The joints in the outer and inner laminations should not be in the center or near the ends.

The cold casein glue specified in the rafter construction dries very rapidly. The gluing operation should be hastened as much as possible, as the rafter should be completely assembled in 20 minutes or less.

The stresses resulting from bending the laminations

According to Forest Products Laboratory (19), "Tests have shown that even when the curvature is as sharp as can be accomplished without breakage of individual laminations, the glued member has about 75 per cent as great strength as a similar assembly glued together but not bent. For moderate curvatures, this ratio is higher; and with a radius of curvature some 150 or more times as great as the thickness of the laminae, the strength and stiffness ratios, as found from tests, have been 90 per cent or greater."
The shearing strength of glue

Object of the study. Since information is lacking on the important question of glue deterioration, the investigation was concerned with the strength of glued joints after a lapse of several years.

Test specimens. During the year 1931 one of the earlier investigators constructed some full-size sections of barn rafters of 6-1" x 3" and 5-1" x 4" laminations using cold casein glue in addition to nails. The glue was applied rapidly, as might be done in barn construction. A strip approximately one-half the board width was coated and no particular effort was made to secure a perfect glue joint. The glue was expected to carry only the horizontal shear developed between laminations.

The test specimens were originally 12'-0" in length. During 1931 the rafters were subjected to tests and loaded to fracture. In the following year a second series of tests was conducted to determine the quality of the glued joints after a lapse of a year. The rafters had been stored on a concrete floor of an unheated building and under a leaky roof.

After the second series of tests, the undamaged portions were sawed into short lengths of about 12" to 14" in length and stored on the same concrete floor and under the same leaky roof. Consequently, the rafters have been subjected to rather severe humid conditions.
Apparatus and method of procedure. The apparatus used for the test was a Buffalo U. S. Standard Scale. This scale is sensitive to two pounds, with a maximum capacity of 5100 pounds. The short beams were supported at each end as shown in Figure 10. The load was applied to the midpoint by means of a stirrup suspended from the scale beam. A micrometer screw at the scale beam made possible the application of the load. A circular scale, which turned with the screw, afforded a convenient means of measuring deflection. The scale was calibrated from 0 to .230 inches.

Round bearing contacts were used at each point of support with 2-inch bearing plates to prevent crushing of the wood. An initial load of 50 pounds was applied to take out any slack that might be in the apparatus; then loading was continued by using 50-pound increments until failure occurred or the capacity of the machine was reached.

This method of testing was selected because horizontal shear is the controlling factor in short wooden beams rather than vertical shear or extreme fibre stress in bending. By applying 5100 pounds to the center of the beam, shear values several times that allowable for wood could be obtained. Samples of the test specimens as used are shown in Figure 11.

In addition to testing the beams for horizontal shear, a pure shear test seemed advisable. Figure 12 shows a sketch of the specimens for the pure shear tests. The area over which the shearing load must be distributed is equal to twice
Figure 10. Method of Testing Short Beams

Figure 11. Short Beam Test Specimens
Figure 13. Horizontal Shear Equation
Applied to a Rectangular Beam.
the length multiplied by the width of the laminations. The ultimate load required to cause failure of the joint was found by testing the blocks in the laboratory of the Engineering Experiment Station. No deflections were recorded, as very little deflection occurs in a glued joint.

Figure 13 illustrates the method of applying the general equation for horizontal shear to a rectangular section. As a result the maximum horizontal shear is given by the equation

\[ \frac{3p}{4bd} \]

Results. From the data obtained during the tests, load deflection diagrams were drawn for each beam. These deflection diagrams are shown for the beams composed of 5- 1" x 4" laminations and for the beams composed of 6- 1" x 3" laminations in Figure 14.

Since the investigation was concerned primarily with the shearing strength of glue the beams were not loaded to destruction.

When the maximum load of 5100 pounds and the actual dimensions of each beam were used, the maximum horizontal shearing stresses developed were:

1. \[ S_s = \frac{3 \times 5100}{\frac{4}{3.625} \times 4} = 264 \text{ pounds per square inch at the neutral surface for the beam composed of 5- 1" x 4" laminations, and} \]
2. \( S_g = \frac{3}{4} \times \frac{5100}{2.75 \times 4.75} = 293 \) pounds per square inch at the neutral surface for the beam composed of 6-1" x 3" laminations.

The calculated horizontal shearing stresses developed in each beam are shown in Table II, along with remarks as to the observed condition of the test beam when removed from the testing machine.

Table II
Results of Horizontal Shear Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Specimen</th>
<th>Condition</th>
<th>Max. Load</th>
<th>Max. Shear</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6-1x3</td>
<td>Good</td>
<td>5100</td>
<td>293</td>
<td>Center laminations checked</td>
</tr>
<tr>
<td>2</td>
<td>6-1x3</td>
<td>Fair</td>
<td>5100</td>
<td>293</td>
<td>No sign of failure</td>
</tr>
<tr>
<td>3</td>
<td>6-1x3</td>
<td>Good</td>
<td>5100</td>
<td>293</td>
<td>No sign of failure</td>
</tr>
<tr>
<td>4</td>
<td>5-1x4</td>
<td>Good</td>
<td>5100</td>
<td>264</td>
<td>Center laminations checked</td>
</tr>
<tr>
<td>5</td>
<td>5-1x4</td>
<td>Good</td>
<td>5100</td>
<td>264</td>
<td>Laminations checked from shear</td>
</tr>
<tr>
<td>6</td>
<td>5-1x4</td>
<td>Good</td>
<td>5100</td>
<td>264</td>
<td>No sign of failure</td>
</tr>
<tr>
<td>7</td>
<td>5-1x4</td>
<td>Good</td>
<td>5100</td>
<td>264</td>
<td>Wood checked at knot</td>
</tr>
<tr>
<td>8</td>
<td>5-1x4</td>
<td>Good</td>
<td>5100</td>
<td>264</td>
<td>No sign of failure</td>
</tr>
<tr>
<td>9</td>
<td>6-1x3</td>
<td>Fair</td>
<td>5100</td>
<td>293</td>
<td>No sign of failure</td>
</tr>
<tr>
<td>10</td>
<td>5-1x4</td>
<td>Good</td>
<td>5100</td>
<td>264</td>
<td>No sign of failure</td>
</tr>
</tbody>
</table>
Of the ten specimens tested, only four showed signs of failure. These four were failures in wood and not in the glue. The remaining six specimens gave no signs of failure in either the wood or the glue. However, in the case of beam No. 3, an inspection of Figure 14 indicates that at a load of 4700 pounds an appreciable amount of horizontal movement took place without such increase in load. This movement would seem to indicate a slipping of one lamination over another or a slight glue failure.

Table III presents the results of the pure shear test. The specimen with 6- 1" x 3" laminations required a load of 23,040 pounds to cause failure. The specimen having 5- 1" x 4" laminations required 24,100 pounds to cause failure.

Table III

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Specimen</th>
<th>Condition</th>
<th>Max. Load (lbs.)</th>
<th>Max. Shear (lbs.)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6- 1 x 3</td>
<td>Good</td>
<td>23,040#</td>
<td>419#/n&quot;</td>
<td>Glue over approx.</td>
</tr>
<tr>
<td>2</td>
<td>5- 1 x 4</td>
<td>Good</td>
<td>24,100#</td>
<td>332#/n&quot;</td>
<td>1/2 surface of test specimens.</td>
</tr>
</tbody>
</table>

As has been stated previously, no attempt was made in the original rafters to secure a perfect glued joint. The glue was applied to a strip about one-half of the width of the rafter and was intended to carry only the horizontal shear developed between the laminations.
When the actual beam dimensions are used, the average shear values are as follows:

1. 6-1” x 3” laminations. Actual size, 2-3/4 x 10 = 27.5 square inches, area of one face
   2 x 27.5 sq. in. = 55 sq. in., area over which the shear is distributed
   Ultimate load = 23,040 pounds
   \[
   \frac{23,040}{55} = 419 \text{ lbs. per sq. in., average shear}
   \]

2. 5-1” x 4” laminations. Actual size, 3-5/8 x 10 = 36.3 sq. in., area of one face
   2 x 36.3 sq. in. = 72.6 sq. in., area over which the shear is distributed
   Ultimate load = 24,100 pounds
   \[
   \frac{24,100}{72.6} = 322 \text{ pounds per sq. in., average shear}
   \]

An inspection of the joint failures shows that the area over which the glue actually was effective is only about one-half of the actual beam size. A conservative estimate of the actual shearing stresses would be approximately 500 to 550 pounds per square inch.

Basic stresses in pounds per square inch for clear material as recommended by Forest Products Laboratory (19) are given in Table IV. These stresses are for material that is continuously dry or continuously wet and free of defects.
Table IV

Basic Stresses for Clear Material

<table>
<thead>
<tr>
<th>Species</th>
<th>Extreme Fiber in bending</th>
<th>Compression perpendicular to grain</th>
<th>Compression parallel to grain</th>
<th>Max. Horizontal Shear</th>
<th>Modulus of Elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ash black</td>
<td>1333</td>
<td>300</td>
<td>866</td>
<td>120</td>
<td>1,100,000</td>
</tr>
<tr>
<td>Beech</td>
<td>2000</td>
<td>500</td>
<td>1600</td>
<td>167</td>
<td>1,600,000</td>
</tr>
<tr>
<td>Cedar western red</td>
<td>1200</td>
<td>200</td>
<td>933</td>
<td>106</td>
<td>1,000,000</td>
</tr>
<tr>
<td>Cypress southern</td>
<td>1733</td>
<td>300</td>
<td>1466</td>
<td>133</td>
<td>1,200,000</td>
</tr>
<tr>
<td>Douglas Fir R.M.</td>
<td>1466</td>
<td>275</td>
<td>1066</td>
<td>133</td>
<td>1,200,000</td>
</tr>
<tr>
<td>Elm American</td>
<td>1466</td>
<td>250</td>
<td>1066</td>
<td>133</td>
<td>1,200,000</td>
</tr>
<tr>
<td>Fir white</td>
<td>1466</td>
<td>300</td>
<td>933</td>
<td>93</td>
<td>1,100,000</td>
</tr>
<tr>
<td>Oak white</td>
<td>1866</td>
<td>500</td>
<td>1333</td>
<td>167</td>
<td>1,500,000</td>
</tr>
<tr>
<td>Pine western</td>
<td>1200</td>
<td>250</td>
<td>1000</td>
<td>113</td>
<td>1,000,000</td>
</tr>
<tr>
<td>Redwood</td>
<td>1600</td>
<td>250</td>
<td>1333</td>
<td>93</td>
<td>1,200,000</td>
</tr>
<tr>
<td>Spruce red</td>
<td>1466</td>
<td>250</td>
<td>1066</td>
<td>113</td>
<td>1,200,000</td>
</tr>
</tbody>
</table>

As a basis for comparison, the average allowable horizontal shear for the eleven species of wood was calculated to be 130 pounds per square inch. This value was used for comparison with the calculated shearing stresses as developed.
in the test specimen. In each case the glue developed over twice the average allowable horizontal shear for wood.

The test results indicate that the effect of glue deterioration is not serious within a period of eight years. As shown by the pure shear test, the glue is capable of developing approximately five times the allowable shearing stress for wood.

Conclusions.

1. The quality of the glue has not been seriously affected by eight years of service conditions.

2. In the original tests conducted by Clark (5), shearing stresses as high as 334 lbs. per sq. in. were developed and after eight years of service conditions the glue still resists shearing stresses five times the allowable shearing stress of wood.

3. The shearing stresses in rafters in actual service conditions would not approach the value obtained during the tests. In the full-size rafter, fiber stress becomes the controlling factor.

4. A continued study over a longer time is desirable in order to obtain more conclusive data on the matter of glue deterioration.

5. It is not desirable to allow farmers to make their own laminated, bent rafters held only by glue; it is still desirable to specify the use of bolts in
addition to the glue, as farmers may become careless in mixing or applying the glue.

6. Prefabricated rafters made in a manufacturing plant under controlled conditions and by careful workmen should give years of satisfactory service to the user, even under the most severe conditions.

Experimental

The effect of joints upon the elastic properties of a glued, laminated rafter section

Object of the study. Since it is impossible to construct a 36'-0" rafter without joints in all of the laminations, the investigation was extended to include a determination of the effect of joints in the outer laminations upon the elastic properties of a glued, laminated, bent rafter.

Test specimens. Sometime during the past few years four full-size rafters were supplied to the Department of Agricultural Engineering. These rafters were made by a commercial company and are the same as those supplied to a dealer for retail trade. The rafters were sawed into eight test specimens (rafters No. 1 to No. 8, inc.) as shown in Figure 15. Test rafters No. 9 to No. 17, inclusive, were selected for the purpose of comparison. Three solid members were tested as a means of comparing the laminated sections with
Figure 15. Dimensions of Rafter Test Sections
the solid members of equal dimensions. Six straight sections were constructed for the purpose of checking the results obtained when using the sections from the commercial rafters. The straight sections were constructed with joints in approximately the same locations as those in the curved members. Glue was applied to only one side of each lamination for rafters No. 12, No. 13, and No. 14. Glue was applied to both sides of the laminations in rafters No. 15, No. 16, and No. 17.

Some designers have been concerned over the stresses introduced in the laminations as a result of bending to the desired curvature. Unless the curvature is small, bending of the laminations to the desired shape induces in them initial stresses of considerable magnitude. The results of the tests performed on rafters No. 12 to No. 17 are to be compared with the test results on rafters No. 1 to No. 8, inclusive. However, when making rafters on a commercial basis workmen arrange that the laminations are held to the desired curvature by clamps with an applied pressure of approximately 100 pounds per square inch (1) until the glue dries. In the laboratory only hand clamps were available. The lack of pressure will no doubt affect the strength of the rafter sections.

While the investigation is primarily concerned with the elastic properties of glued, laminated, bent rafters, the shearing strength of the glue is an important factor to be
considered. The results of the tests on short beams indicate that the glue has greater strength in shear than the wood fibers. Average allowable shear values for wood are between 90 and 150 pounds per square inch, while the shear value of glue is approximately 500 to 600 pounds per square inch. All test rafters will be inspected for possible glue failures.

**Apparatus and method of procedure.** The apparatus for the test is shown in Figure 19 and is the same as that described in connection with the shear test of the short beams. The rafters were supported at each end and load was applied to the midpoint by means of a stirrup suspended from the scale beam. Round contacts were used at each point of support with steel bearing plates to prevent crushing the wood. An initial load of 20 pounds was applied to take out any slack in the apparatus. The deflection was read after the addition of each 20-pound load increment and recorded on a convenient data sheet.

Too much emphasis cannot be placed on the importance of standard procedure in making strength tests on structural timbers (13). The effect of defects, such as knots and cross grain, on strength has been fairly well established and recognized in the basic grading rules and working stresses. In a beam tested under center loading, the maximum stress in bending occurs at the center. Defects have their maximum
effect at the center of the length on the bottom face and on the lower edges of the vertical faces of the beam. Careful attention must also be given to such factors as speed of test and kind of bearings at the supports and load points.

It is, of course, well known that the moisture content of wood has a tremendous effect on the strength of small clear pieces. In structural sizes, however, the development of defects tends to affect any increase in fiber strength that may take place as a result of a reduction in moisture content. Structural timbers, even after air seasoning for one to two years, are only partially dry.

The moisture content of the laminated rafter sections (No. 1 to No. 8, inclusive) was approximately eight per cent or less. A moisture meter was used for the determination of the moisture content. Since the instrument was calibrated to read from 8 per cent to 24 per cent, no readings could be taken below 8 per cent. The moisture content of the solid members was greater than 8 per cent and recorded at 10 per cent. The lumber used in the test rafter sections No. 12 to No. 17 had been purchased several months previously and stored in heated room. The moisture content of the lumber was reduced to approximately 8 per cent as recorded by the moisture meter.

The density of wood is an indication of its strength. Density is defined as the mass of a body per unit volume. When expressed in the metric system, it is numerically equal
to the specific gravity of the same substance. The density of the actual wood substance, that is, the material of which the cell walls are composed, it substantially the same for all species; its value in the metric system is about 1.54. Therefore, all woods would be of the same specific gravity were it not for the fact that, because of variation in the size of the cell cavities and in thickness of the cell wall, some have more wood substance than others.

By taking a sample piece of known dimensions and weighing the sample the approximate density was obtained. From this simple procedure the density of the laminated sections was calculated to be 32.5 lbs. per cubic foot and the solid member was 30 lbs. per cubic foot. The difference in density should also be remembered when comparisons are made.

In addition to the moisture content and density, the number of annular rings per inch has a marked influence upon the strength of a structural timber. When results of the experimental tests are used for comparison, the test specimens are selected for rate of growth which requires the number of annual rings per inch to be within a specified range. However, according to Markwardt and Wilson (12), "Rate of growth does not have a definite relation to the strength in the sense of strength being proportional, either directly or inversely, to the rate of growth."

An inspection of the photographs reveals considerable
cross-grain in several of the test sections. The fibers in a cross-grained piece of wood are at an angle to the axis of the piece. The principal types of cross-grain are spiral grain and diagonal grain. Other less important types are wavy, dipped, interlocked, and curly grain. Cross-graining materially reduces the strength of the member and accounts for some of the unusual failures.

Results. Table V gives the results of the tests on the rafter sections in tabular form with a remark regarding the action of each test section. A load deflection diagram was plotted for each rafter. The diagrams are shown in Figure 16 and give the comparative strength and rigidity of each test rafter.

The results of the tests on each rafter section will be discussed separately and then an analysis of all the tests will be made. A brief description of each rafter will accompany this discussion.

Rafter No. 1
As shown in Figure 15, rafter No. 1 was composed of 7 1-3/4 x 25/32" laminations with a joint in the top lamination at the center of the span. Load was applied in 20-pound increments. At 600 pounds the top lamination failed in tension and following this failure the deflection was not at all in proportion to the load. The load deflection diagram as shown in Figure 16 assumes a stairstep appearance illustrating
Table V
Comparative Strength of Rafter Sections

<table>
<thead>
<tr>
<th>Rafter No.</th>
<th>Max. Load (Lbs.)</th>
<th>Max. Deflection (Inches)</th>
<th>Max. Stress (Lbs./sq. in.)</th>
<th>Max. Horizontal Shear (#/sq.&quot;&quot;)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1300</td>
<td>2.006</td>
<td>4450</td>
<td>101.3</td>
<td>Top lamination failed in tension at 600 lbs.</td>
</tr>
<tr>
<td>2</td>
<td>2840</td>
<td>2.367</td>
<td>9600</td>
<td>220.0</td>
<td>Failure occurred suddenly, accompanied by a loud report.</td>
</tr>
<tr>
<td>3</td>
<td>1960</td>
<td>2.339</td>
<td>5200</td>
<td>153.0</td>
<td>Joint loosened at 920# and failure extended to 2nd lamination.</td>
</tr>
<tr>
<td>4</td>
<td>2780</td>
<td>2.298</td>
<td>9420</td>
<td>217.0</td>
<td>Failure similar to rafter No. 2.</td>
</tr>
<tr>
<td>5</td>
<td>1800</td>
<td>2.651</td>
<td>8370</td>
<td>164.0</td>
<td>Top lamination failed in tension.</td>
</tr>
<tr>
<td>6</td>
<td>2220</td>
<td>3.186</td>
<td>7550</td>
<td>173.0</td>
<td>Joint loosened at 960# and top lamination separated from second lamination.</td>
</tr>
<tr>
<td>7</td>
<td>1440</td>
<td>1.981</td>
<td>4880</td>
<td>150.0</td>
<td>Joint loosened at 940# and failure extended into fifth lamination.</td>
</tr>
<tr>
<td>8</td>
<td>2240</td>
<td>3.113</td>
<td>10400</td>
<td>204.0</td>
<td>1700# tension failure in top lamination.</td>
</tr>
<tr>
<td>9</td>
<td>2860</td>
<td>2.468</td>
<td>9700</td>
<td>223.0</td>
<td>Tension failure following 30° diagonal grain.</td>
</tr>
<tr>
<td>10</td>
<td>3020</td>
<td>3.083</td>
<td>10240</td>
<td>235.0</td>
<td>Compression failure from stress concentration about knot near center of span.</td>
</tr>
</tbody>
</table>

(Continued)
<table>
<thead>
<tr>
<th>Rafter No.</th>
<th>Max. Load (Lbs.)</th>
<th>Max. Deflection (Inches)</th>
<th>Max. Stress (Lbs./sq. in.)</th>
<th>Max. Horizontal Shear (#/sq.&quot;</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>2780</td>
<td>3.365</td>
<td>9425</td>
<td>234.0</td>
<td>Compression failure at center. Beam buckled; not carried to destruction. Still capable of carrying load.</td>
</tr>
<tr>
<td>12</td>
<td>2120</td>
<td>1.967</td>
<td>7186</td>
<td>165.0</td>
<td>Tension failure in top lamination; glue failure between second and third laminations.</td>
</tr>
<tr>
<td>13</td>
<td>820</td>
<td>1.194</td>
<td>2780</td>
<td>64.0</td>
<td>Glue failure.</td>
</tr>
<tr>
<td>14</td>
<td>2100</td>
<td>3.390</td>
<td>9765</td>
<td>191.0</td>
<td>Tension failure in top lamination; also long glue failure between top and 2nd laminations.</td>
</tr>
<tr>
<td>15</td>
<td>2180</td>
<td>2.079</td>
<td>7390</td>
<td>169.0</td>
<td>Tension failure in top lamination. Glue to both sides of laminations. Short glue failure in 2nd lamination.</td>
</tr>
<tr>
<td>16</td>
<td>1720</td>
<td>3.277</td>
<td>5830</td>
<td>133.0</td>
<td>Failure at joint. Fracture going into third lamination. Not glue failure.</td>
</tr>
<tr>
<td>17</td>
<td>1520</td>
<td>3.446</td>
<td>7008</td>
<td>137.0</td>
<td>Tension failure in top lamination and second lamination; long break is not glue failure.</td>
</tr>
</tbody>
</table>
Figure 16. Load Deflection Diagrams for Rafter Test Sections
excessive deflection with each load application. The deflection when failure occurred was .607 inch. However, the rafter continued to take load up to 1200 pounds, when complete failure occurred. Figures 17 and 18 illustrate the nature of the failure. When the load reached 860 pounds, the failure in the top lamination extended through to the second lamination. The effective depth of the rafter was reduced to 5-25/32" laminations, or to a total of 3.9 inches. As loading continued, the failure followed along the line between the second the the third laminations.

The results of the first test were not as was expected. The joint developed sufficient strength in this particular case. However, the failure was not typical of what could be expected. When the formula $S = \frac{MC}{I}$ is used, the fiber stress developed with the load of 600 pounds is approximately 2000 pounds per square inch. This fiber stress should not cause failure when applied under the conditions of the test; failure was no doubt caused by a defect in the top lamination. However, careful inspection of the failure did not reveal any apparent defects.

Rafter No. 2

This test section was composed of 7-1 3/4" x 25/32" laminations with the top lamination continuous from end to end. The positions of the joints are shown in Figure 15. The load deflection diagram indicates that the deflection is
Figure 17. Failure of Rafter Section No. 1

Figure 18. Failure of Rafter Section No. 1

Figure 19. Failure of Rafter Section No. 1

Figure 20. Failure of Rafter Section No. 2
proportional to the loads up to about 2400 pounds. At this loading the recorded deflection was 1.927 inches. The loading was continued up to 2640 pounds, where failure occurred, as shown in Figures 19 and 20.

Failure occurred all at once. A loud report accompanied the failure and the rafter was virtually destroyed. Figure 19 illustrates the beam at the time of failure. The compression failure shown in Figure 20 is no doubt a combination of compression failure and bearing failure. The glue did not fail during the test. The wood fibers were torn apart in every case, and that condition indicated a good bond between laminations. The fiber stress developed was calculated to be 9600 pounds per square inch.

Rafter No. 3

Rafters No. 1 and No. 3 are similar in that each had the joint in the top lamination occur in the center of the 10'-0" span. Each rafter was composed of 7-1 3/4" x 25/32" laminations. Rafter No. 3 was loaded in the manner previously described. When the loading reached 920 pounds, the top lamination gave away at the joint. Failure did not follow along the line between the first and second laminations, but passed into the second lamination. When the load reached 1000 pounds, the second lamination gave away completely. The failure reduced the effective depth of the rafter from seven laminations to five laminations. Loading was continued until complete failure occurred at 1960 pounds. The deflection
corresponding to this loading was 2,339 inches. After the 1000-pound load was applied, the deformation was more or less uniform with each addition until complete failure occurred. Figures 21 and 22 illustrate the nature of the failure and show that when the first lamination failed it carried away part of the second lamination. Since no previous tests had been conducted the nature of the failures could not be predicted. However, the interesting fact about the test seemed to be that the strength of the rafter was reduced by more than one lamination as a result of the joint's being in the center. The glue caused the failure to extend down into the second lamination and left only five laminations to carry the load.

Rafter No. 4

This specimen and Rafter No. 1 were identical. The rafter section was composed of 7- 1 3/4" x 25/32" laminations. The top lamination was continuous from end to end. The results of the test were identical with those of Rafter No. 2. Failure occurred at the load of 2780 pounds with a corresponding deflection of 2.298 inches. Figures 23 and 24 indicate that the nature of the failure was similar to the failure of Rafter No. 2. The main difference in the two failures can be accounted for by the fact that Rafter No. 4 did not fail in bearing. Both rafters gave evidence of compression failure. In each case the glue held and the
wood fibers were separated.

Rafter No. 5

The top lamination of Rafter No. 5 was removed by means of a plane. Since the tests of the rafters with the joint in the center indicated that the strength was reduced by at least one lamination, it was decided to remove the top lamination, leaving the second lamination continuous from end to end, and compare the elastic properties with the previous tests. Figure 15 illustrates the spacing of the remaining joints. The deflection was proportional to the load up to the point of failure. Since the depth of the beam was reduced by 25/32", the rigidity was reduced. The load-carrying capacity was superior to those of Rafter No. 1 and Rafter No. 3. Removing the top lamination does not increase the load-carrying capacity, but removing the joint in the top lamination does remove the weakness. As has been stated previously, when the joint fails, the strength is reduced by more than one lamination. Figures 25 and 26 illustrate the method of fracture. The failure occurred by the top lamination failing in tension. Here again the glue did not fail, but the wood fibers were pulled apart. This fact emphasizes the effectiveness of glue as a means of joining wood members.

Rafter No. 6.

This test section was an exact duplicate of Rafters No. 1 and No. 3. As in the previous tests, failure occurred in
the joint with a comparatively small amount of load. When loading reached 960 pounds, the top lamination loosened and the glue seemed to fail by allowing the top lamination to separate.

Figures 27 and 28 picture the nature of the failure. Rafters No. 2 and No. 6 both failed in compression or a combination of bearing and compression. Since the laminations were securely glued together, very little movement could take place between members. This lack of movement permits a concentration of stress which in the case of Rafters No. 2 and No. 6 occurred at the center bearing plate. The loading necessary to cause failure in Rafter No. 6 was 2220 pounds, a load that corresponds to an extreme fiber stress of approximately 7500 pounds per square inch, with a deflection of 3.186 inches.

Figure 16 gives the load deflection curve for the rafter. The deflection was proportional to the load until the joint failed, thereby permitting a certain amount of horizontal displacement. The deflection was again proportional to the load. At 1940 pounds the bottom lamination failed in compression, resulting in almost one-half inch horizontal movement with very little increase in load.

Rafter No. 7

From an inspection of Figure 16 it can be seen that the results of test Rafters No. 1, No. 3, and No. 6 did not agree
very closely. Rafter No. 7 was selected with a joint in the center of the top lamination and was used as a check for the three previous rafters.

The joint failed at a load of 940 pounds. Rafters No. 3 and No. 6 check closely with the results of Rafter No. 7. Rafters No. 3 and No. 6 required an ultimate load of approximately 2000 pounds, while Rafter No. 7 required an ultimate load of only 1420 pounds. This difference can be attributed to the difference in materials. Rafter No. 1 may be disregarded in this comparison.

Figure 29 illustrates the nature of the failures and shows that the grain of the wood in laminations No. 2 and No. 3 were inclined in the same direction. As soon as failure started, it carried across the two laminations. The remaining members were not able to withstand the stress, and complete failure occurred. As in the case of almost all rafters, failure occurred all at once and was accompanied by a loud report.

A shear failure of the glue took place between laminations No. 3 and No. 4. This failure was one of the very few glue failures observed during the tests.

Rafter No. 8

As previously stated, eight test specimens could be obtained from the four full-size rafters. The top lamination of Rafter No. 8 was removed to make the test specimen
Figure 29. Failure of Rafter Section No. 7

Figure 30. Failure of Rafter Section No. 8

Figure 31. Failure of Rafter Section No. 9

Figure 32. Failure of Rafter Section No. 10
similar to Rafter No. 5. The removal left this section composed of 6- 1 3/4" x 25/32" laminations with the top lamination continuous from end to end.

The deflections for Rafter No. 8 were consistently less than those for Rafter No. 5. Again difference in material could account for the variation.

Figure 30 shows that the rafter failed in both tension and compression. The top lamination failed in tension at a load of 1660 pounds. However, loading was continued to 2240 pounds, when complete failure occurred.

Rafters No. 9, No. 10, and No. 11

These sections were solid members. No. 9 and No. 10 were taken from a 3" x 12" plank and planed down to the exact size of a laminated rafter composed of 7- 1 3/4" x 25/32" laminations. The actual dimensions of such a rafter are 1 3/4" x 5 1/2". Section No. 11 was a 2" x 6" member purchased as No. 1 Douglas fir with actual dimensions of 1 5/8" x 5 1/2".

The solid members were included in the series of tests for the purpose of comparing the strength of the laminated sections with the strength of solid members. The advantages of laminated, glued construction are many, provided that strength equal to that of a solid member can be obtained.

The diagrams of load deflection for the solid members are shown in Figure 16 along with the same diagrams for the
laminated sections. Rafter No. 9 failed at a load of 2860 pounds which resulted in an extreme fiber stress in bending of 9700 pounds per square inch. The failure is pictured in Figure 31. The rafter was almost completely destroyed, with the fracture following along the diagonal grain.

Rafter No. 10 was subjected to a concentrated load of 3020 pounds and developed a fiber stress of 10,240 pounds per square inch. The failure is illustrated in Figure 32 and indicates a compression failure resulting from stress concentration about a knot near the center of the span. Figure 33 was intended to show the great amount of bulging of the wood fibers just adjacent to the knot and the tension failure in the top lamination.

Rafter No. 11 was the 2" x 6" member. This member was not carried to complete destruction because excessive lateral buckling occurred at a load of approximately 2200 pounds. Figure 34 indicates that failure would have resulted from a combination of bearing and shear.

Rafter No. 12

Rafters No. 12, No. 13, and No. 14 were made in the laboratory by the use of ordinary hand clamps. Glue was applied to one side of the laminations only. A few 18 gauge brads were used to hold the laminations in place until the clamps could be applied.

Figure 35 indicates a tension failure of the top lamination with a glue failure occurring between the second
and the third laminations. The rafter did not compare favorably with Rafters No. 2 and No. 4. The difference can be accounted for by the fact that the commercial rafters were made by using pressures of approximately 100 pounds per square inch, while the pressure applied to Rafter No. 12 was much less.

Rafter No. 13

Table V gives the ultimate load carried by Rafter No. 13 as 820 pounds. Glue failures caused an extremely low ultimate load. An inspection of Figure 36 indicates at least three major glue failures. The low stress carried by the rafter indicates that extreme care and great pressure were necessary for the construction of a dependable rafter.

Rafter No. 14

When the load applied to Rafter No. 14 was 2100 pounds, a fiber stress of 9765 pounds per square inch resulted. Figure 37 reveals a tension failure in the top lamination and a long glue failure between the top and second laminations.

Rafter No. 15

Rafters No. 15, No. 16, and No. 17 were made by applying glue to both faces of each lamination. The laminations were held in place with 18-gauge brads until the hand clamps could be applied.

The use of glue on both surfaces indicates an increase in the strength of Rafter No. 15 over that of Rafter No. 12.
The two rafters were of identical construction except in the use of glue. Figure 38 reveals a tension failure of the top lamination and a short glue failure between first and second laminations. However, particles of wood fibers were torn away with the glue.

Rafter No. 16

A joint was provided at the center of the top lamination of Rafter No. 16. Figure 39 illustrates the method of failure. The fracture occurred at the joint and extended into the third lamination, thus reducing the effective depth. No glue failure was recorded. The ultimate load of 1720 pounds equaled the ultimate load of Rafter No. 3.

Rafter No. 17

Rafter No. 17 was constructed of 6- 1 3/4" x 25/32" laminations. The fracture was a result of tension failure in the first and second laminations. The long break between the third and fourth laminations as shown in Figure 40 was not a result of glue failure.

Conclusions.

1. In bent rafter construction it is important that the best material be used in the extreme fibers, the outer lamination next to the sheathing, and the inside lamination. Lower grade material may be used in the intermediate laminations.
2. The horizontal shearing strength of glue exceeds the shearing strength of wood.

3. Glued members with the top lamination continuous throughout the 10'-0" length resulted in stiffness equal to a solid member of the same dimensions.

4. A joint in the center of the top lamination reduced both the strength and stiffness of members.

5. A section with the top lamination continuous is three times as strong as a section with a joint in the center of the top lamination.

6. A section composed of 6- 1 3/4" x 25/32" laminations is twice as strong as a section composed of 7- 1 3/4" x 25/32" laminations where the top laminations are joined at the center of the span. (Comparison based upon the load required to produce first failure.)

7. Laminations should be as long as possible in order to reduce the number of joints to a minimum.

8. Joints should not be close to the ends of a member. Where possible, they should be at points of minimum bending moment.

9. After failure of one or more laminations the rafter can be expected to carry increased load up to a certain point. However, excessive deflection may occur.
10. When rafters are constructed by the use of cold casein glue and clamped in a form under pressure of approximately 100 pounds per square inch, nails and bolts are not necessary.

11. From the results of these tests no comparison can be attempted as to the effect of initial bending stress upon the final strength of a rafter section.

12. Applying the glue to both faces of the lamination increased the strength of the glued joint.

Shape to provide stability under dead loads

Selection of loads. The loads classed as dead loads are those resulting from the weight of the rafters, sheathing and roof covering. These loads produce bending stresses in the rafters if their line of thrust falls outside the boundaries of the material. Whenever the bending stresses are of considerable magnitude the ridge has a tendency to sag. The weight of pine sheathing (16) is given as 4.0 pounds per square foot. Cedar shingles weigh about 2.0 (20) pounds per square foot, and the weight of the rafters constitute another 1.5 pounds per square foot. The total dead weight has been taken as 7.0 pounds per square foot for the purpose of the following calculations.

Glued, laminated, bent rafters of the latest design are made for two types of barn construction: (1) rafters continuous from the sill to the ridge, and (2) rafters
extending from the mow floor to the ridge. The latter type is intended for use on barns with a masonry wall extending to the mow floor. The stresses resulting from dead loads will be analyzed first for the type of rafter extending from the mow floor to the ridge, and secondly for the type of rafter extending from the sill to the ridge.

Dead loads are uniformly distributed loads, but for the purpose of calculation it is much better to consider them as concentrated loads acting at the center of gravity of a section. A rafter of the type under consideration is approximately 30'-0" in length. This rafter, divided into ten sections, gives a length per section along the rafter of 3'-0". By using a rafter spacing of 2'-0" and a weight of 7.0 pounds per square foot, a dead load of 42 pounds per section results. These loads are shown in Figure 41.

The second type of glued, laminated, bent rafter construction is the rafter made continuous from the sill to the ridge. These rafters are approximately 36'-0" in length. When divided into ten sections, the length along the rafter per section is 3.6 feet. When a rafter spacing of 2'-0" and a weight of 7.0 pounds per square foot were used, a dead load of 50 pounds per section resulted. The weights used are shown in Figure 42. These loads were calculated for a design of the type recommended by the Midwest Plan Service (2).
Resulting bending moment and fiber stress. A building such as a barn is an indeterminate structure. Each member is either nailed or bolted to some other member and the amount of stress transmitted between members is very difficult to determine.

Because of the indeterminate nature of a barn, certain assumptions regarding the behavior of the structure are necessary before a stress analysis can be attempted. The fewer the assumptions necessary the more exact the solution. Also, the closer the assumptions approach actual field conditions the more valuable the solution.

In Figure 41 the rafter was treated as a three-hinged arch. Since the rafters are usually fastened at the mow floor with modern connectors, these points can be assumed to be hinged. The third hinge is at the ridge. The force polygon was drawn for the dead loads and the equilibrium polygon passed through the three hinges as at these three points no bending could result. A hinge is capable of resisting only thrust and shear. The equilibrium polygon now represents the exact position of the line of thrust. By obtaining the magnitude of the thrust from the force polygon and scaling the perpendicular distance to the thrust, the resulting bending moment can be obtained at any section.

The greatest eccentricity occurs about five feet below the ridge and is equal to 1.4 feet. The fiber stress
Figure 41. Dead Load Stress Analysis. Rafter Extending from Mow Floor to Ridge
caused by this eccentricity is only about 265 pounds per square inch for the cross-section composed of 7-1 3/4" x 25/32" laminations. The fiber stress is not critical.

The type of construction shown in Figure 42 may be analyzed as a three-hinged arch. The usual method of toenailing the ends of the rafters to the sill makes it possible to treat the joint as a hinge. Ten concentrated loads of 50 pounds each were applied to each rafter section. The fiber stress resulting from the eccentricity is about 590 pounds per square inch. This second analysis was completed for the purpose of comparison. In actual conditions the mow floor is fastened to the side of the rafters and provides considerable support. The bending moments produced would be considerably less than that shown as a result of dead loads only. However, if the support provided by the mow floor permitted considerable movement of the rafters, the bending moments would approach those shown.

The conclusions that may be drawn from this part of the investigation are:

1. It is impossible to secure a shape such that the line of thrust falls entirely within the boundaries of the material when the rafter sections are composed of circular arcs.
Figure 42. Dead Load Stress Analysis. Rafter Extending from Sill to Ridge
2. When a 33'-0" radius is used, the fiber stress resulting from the dead loads is not critical and should not produce sagging at the ridge.

3. The mow floor contributes toward the reduction of bending moments in a rafter which is continuous from the sill to the ridge.

Investigation of reactions and bending moments

Object of the study. The purpose of this section of the investigation was to determine the magnitude and direction of the reactions and bending moments resulting from probable wind loads. A structure has been treated in the following manner:

1. As a solid block resting upon a foundation and anchored against horizontal movement, but not against vertical movement.

2. As a three-hinged arch extending from the mow floor to the ridge and subjected to a wind load of 70 m.p.h.

3. As a three-hinged arch extending from the sill to the ridge and subjected to a wind load of 70 m.p.h.

4. As a structure supported by a foundation of infinite cross-section with the support attached to the rafters at the mow floor.
Preliminary consideration. Since the wind loads produce the largest stresses in a barn frame, the question of what wind velocity to use in design calculations is a very important consideration. The highest recorded wind velocity in the state of Iowa is given as 68 m.p.h. This value is an average velocity over a five-minute period. No doubt for short periods of time this velocity is exceeded as a result of gusty winds. A wind velocity of 70 m.p.h. has been selected for the purpose of wind stress calculations. By using this wind velocity in the formula

\[ p = 0.001189 \frac{(22V)^2}{15} \]

a wind pressure normal to a vertical surface of 12.53 pounds per square foot is obtained. The pressure normal to a vertical surface is changed into a pressure normal to an inclined surface by the use of the wind pressure distribution diagram as shown in Figure 9. The rafter has been divided into ten sections of equal lengths and the velocity pressure multiplied by the average resistance coefficient over the section. Figure 43 gives the combined dead and wind loads for the rafter under consideration.

The combined loads result in an overturning moment. A vertical force of 64 pounds is necessary to hold the barn down on the windward side, while a vertical force of 280 pounds is necessary to hold the barn up on the leeward side. These values were obtained by taking simple moments
Figure 43 - Reactions at Sill for Combined Wind and Dead Loads
about each end of the rafter at the sill. A check was obtained by combining the reactions that result from dead loads only and wind loads only as shown in Figures 42 and 45. The forces necessary to resist overturning are very important from the standpoint of determining the conditions of end support for theoretical calculations.

Stress analysis. R. Gray (10) has outlined the following procedure for use in designing against wind loads:

"Design of any structure which is required to resist pressure or other forces due to wind falls naturally into four stages:

1. Estimating the speed of the wind and the forces exerted thereby on the structure
2. Calculating the moments, shears and thrusts induced in the members of the structure
3. Selecting suitable members to resist these forces
4. Designing connections between the members and such details as bases and, sometimes, foundations."

This outline applies to farm buildings as well as to steel structures. The speed of the wind has already been determined and the next step is to determine the moments, shears, and thrusts induced in the members as a result of combined wind and dead loads.

The rafter has been analyzed as a three-hinged arch
extending from the mow floor to the ridge. Several attempts were made to secure a solution to the problem by the use of combined loading, but the nature of the structure and the direction of the applied loads gave intersections that could not be handled on the ordinary sheet of drawing paper. Since dead load stresses had previously been determined, the decision was made to treat the wind loads separately and combine the results. Figure 44 illustrates the solution of the problem. The wind direction has been assumed to be from the left with a velocity of 70 m.p.h. The force polygon was plotted by the use of the applied loads, and the equilibrium polygon was made to pass through the three hinges. The equilibrium polygon now represents the manner in which the applied loads are transmitted to the supports. Since only one equilibrium polygon can be drawn through the three points, the component Oa represents the correct reaction at A while the component Ou represents the correct reaction at B. The magnitude of the bending moment at any section is calculated as before: by obtaining the amount of thrust in pounds from the force polygon, scaling the perpendicular distance from the section to the thrust in feet, and obtaining their product to give foot pounds bending moment. The bending moments at the sections are plotted in Figure 44. The direction and magnitude of the reactions resulting from the combined dead loads and wind loads reveal that very little lifting effect is exerted upon a
Figure 44. Wind Load Stress Analysis. Rafter Extending from Mow Floor to Ridge
rafter of this type at a wind velocity of 70 m.p.h. However, at any higher wind velocity considerable lifting effect would be exerted. At 70 m.p.h. the dead loads just balance the vertical component of the wind loads.

Figure 45 illustrates the resulting reactions and bending moments when the rafter is treated as a three-hinged arch extending from the sill to the ridge. The solution to the problem is the same as previously described. The wind loads and dead loads were treated separately. The reactions produced by the dead loads tend to equalize the reactions produced by the wind loads, but even so, the supports are required to hold the roof down as well as up during a wind storm where the wind reaches a velocity of 70 m.p.h. The bending moments produced by the dead loads are almost negligible when compared to the bending moments produced by the wind loads.

During this investigation of the glued, laminated, bent rafter, Test (17) states: "Analysis of the internal stresses in this type of rafter is somewhat involved, but it may be expected that the maximum bending moment will occur at the plate joint, since the action of the rafter is similar to that of a cantilever beam, although it is not rigidly fixed at that point."

Even though the stress analysis is somewhat involved, an attempt has been made to determine the actual stresses in a rafter section which extends from the sill to the ridge.
Figure 45. Wind Load Stress Analysis: Rafter Extending from Sill to Ridge
The principal assumptions used in this analysis are: (1) rafters are securely fastened at the mow floor and at the sill; (2) no horizontal or vertical displacement occurs at either point. These conditions provide rigid support at the mow floor. This rigid support can be maintained if sufficient bracing is supplied to the end of the barn. Once a condition of fixed ends is assumed, the elastic curved beam theory can be applied to the arch, and values can be obtained for the thrust and shear at the ridge. From the thrust and shear at the ridge the bending moments at any section can be obtained. The method and theory used is that outlined by Caughey (4) in his text, "Reinforced Concrete."

In the solution of problems that involve arches there are generally three quantities to be obtained, namely: thrust, shear, and moment. In order to solve for three unknowns, it is necessary to have three equations which may be solved simultaneously. The ridge joint in a glued, laminated, bent rafter is not sufficiently rigid to transmit bending moment; therefore this point must be considered hinged. The unknowns are now reduced to two, namely: the thrust and shear at the ridge. The table in Figure 46 presents the entire procedure used in obtaining the two unknown quantities. After this thrust and shear have been obtained, the common pole point can be established for the purpose of drawing the equilibrium polygon. Once established, the equilibrium polygon
can be used to determine the bending moment at any section. A bending moment diagram is shown in Figure 46 and should be compared with the bending moment diagram obtained when the rafter was considered as a three-hinged arch.

**Results of stress analysis.** The third step in the design of any structure as outlined by Gray (10) is the selection of suitable members to resist the forces imposed by the applied loads.

The results of the stress analysis as presented in Figures 41 to 46 include forces resulting from dead loads, wind loads, and combined dead and wind loads. The members, to resist these forces, must be capable of withstanding the maximum stresses developed.

As a result of the overturning effect, the sill support will first be called upon to resist a vertical lifting force of 64 pounds on the windward side and a vertical compressive force of 280 pounds on the leeward side. The ordinary toenailed joint using four 16d nails driven into a sill composed of two 2" x 6" members is capable of safely resisting a vertical pull of 450 pounds (19). For a 70 m.p.h. wind the ordinary joint has a factor of safety of seven.

The use of vertical siding is also suggested as an added factor of safety. Studs are usually spaced 2'-0" o.c. along the length of the sill. This spacing allows for at least six nails per rafter to aid in holding the stud to the sill. The
Figure 46-a. Application of Elastic Curved Beam Theory of Gothic Arch Rafter
Figure 4.6-b. Application of Elastic Curved Beam Theory to Gothic Arch Rafter
lateral resistance of six 12d nails would add another 384 pounds (19) to the strength of the joint at the sill.

In addition to the vertical component, each sill joint must resist its share of the horizontal components of the applied loads. Dead loads are assumed to act vertically, and no horizontal components result. In the case of the wind loads the conditions are different. The resulting forces have both horizontal and vertical components. As shown in Figure 41, the horizontal components of the wind load were very nearly 1000 pounds. This fact means that each support must provide resistance to at least 500 pounds for a wind velocity of 70 m.p.h. The safe lateral load for wire nails as suggested by "Wood Handbook" (19) can be expressed by the formula \( p = K D^{3/2} \). Four 16d nails are usually used to toenail a rafter to the sill. The safe lateral load for such a joint would be 300 pounds. However, a factor of safety between safe load and ultimate load of 6 to 1 for coniferous woods and 11 to 1 for hardwood is provided. Joints made from Douglas fir will have a safety factor of not over 3 to 1 in a 70 m.p.h. wind.

The analysis as outlined in Figure 44 applies only to those cases where the rafters are supported by a masonry wall at the mow floor. A modern connector is usually used to fasten the rafter to the side of the joist and provide a hinged joint. The reactions necessary to hold this roof on its foundation are shown for wind loads only and for combined
dead and wind loads. The combined dead and wind load forces are the ones of greatest interest, because the dead loads are always present to help counteract the lifting force of the wind.

For a wind velocity of 70 m.p.h. the dead loads almost cancel the vertical lifting force of the wind and the windward reaction is almost horizontal, while the leeward reaction must provide a vertical component of approximately 100 pounds. Each reaction must resist a horizontal component of about 350 pounds. This force can safely be resisted by four 16d nails.

The bending moment resulting from the wind loads is shown in tabular form in Figure 44 and the maximum bending moments are indicated on the bending moment diagram. The rafter section must be capable of resisting the maximum stresses. The fiber stress resulting from the bending moment of 2350 foot pounds is almost 3200 pounds per square inch. A factor of safety of at least two is provided in the commercial rafter of the type tested during this investigation.

The results shown in Figure 45 are similar to those shown in Figure 44 except that the reactions and bending moments are much greater as a result of the increased rafter length and an addition of wind loads between the mow floor and the sill. The reactions are the same as those discussed in connection with the overturning effect. The maximum bending moment is 4228 foot pounds. The fiber stress approaches the ultimate
strength of the commercial rafters tested.

When rafters are made continuous from the sill to the ridge, the mow floor must provide considerable support and reduce the bending stresses, as shown in Figure 45.

The elastic curved beam theory is one method of determining the stresses in the members of a Gothic barn roof. The assumption has been made that the mow floor is sufficiently rigid to transmit the horizontal forces to the ends of the barn. A separate heading has been reserved for the justification for this assumption.

The results of this analysis show a considerable redistribution of the bending moment in the rafter. The maximum bending moment of 2518 foot pounds occurs at the leeward support. This bending produces a fiber stress of over 3400 pounds per square inch. This provides a factor of safety of almost 3 for the prefabricated rafters.

Based upon the results of this theoretical analysis, the fiber stress causing failure of the rafter investigated by Test (17) was 8400 pounds per square inch.

Some consideration has been given to the matter of redistribution of the material in the rafter to the points of maximum bending moment. This redistribution would be desirable from the standpoint of economy of material. The rafter should be stronger and more rigid by changing the cross-section in accordance with the requirements of the bending
moment. At a point about 9'-0" above the mow floor the bending moment changes sign. The maximum bending moment from this point to the ridge is about 950 foot pounds. This reduced bending moment would allow for the removal of at least two laminations without materially reducing the stiffness.

As a final consideration, the horizontal shear developed at the neutral axis was investigated. Horizontal shear in long beams is not generally a controlling factor. Extreme fiber stress in bending usually controls. The general equation for horizontal shear is expressed by the formula $S_h = \frac{V_0}{I_t}$. The magnitude of the maximum vertical shear was obtained by scaling the perpendicular component of the thrust at each section. A maximum value for 400 pounds was used and substituted into the above equation. A maximum value for the horizontal shear of 40 pounds per square inch resulted. The safe allowable shearing stress for wood has been given as 120 pounds per square inch, and glue is capable of withstanding several times this amount. Horizontal shearing stresses are not of major concern.

**Conclusions.** The following conclusions seem apparent from the results of the stress analysis of a Gothic arch barn roof.

1. For a wind velocity of 70 m.p.h. the overturning moment is equal to 2200 foot pounds. The windward reaction must resist a vertical lifting force of 65 pounds.
2. The bending stresses produced by the dead loads are not critical and are almost negligible when compared with the bending stresses produced by the wind.

3. The maximum bending stress produced by the wind in a rafter extending from the mow floor to the ridge is 2350 foot pounds.

4. In this rafter the maximum bending occurs at a point 14'-0" above the mow floor on the leeward side.

5. When the rafter is assumed to be a three-hinged arch extending from the sill to the ridge, the maximum bending stress produced is 4228 foot pounds.

6. In this case the maximum bending occurs at a point 14'-0" above the sill on the windward side.

7. When the rafter is assumed to be rigidly supported at the mow floor the maximum bending stress produced by the combined wind and dead loads is 2518 foot pounds.

8. When the rafters are securely fastened to the mow floor and sill, a condition of rigid support results.

9. The maximum fiber stress induced in the rafter under these conditions is 3400 pounds per square inch for a rafter composed of 7-1 3/4" x 25/32" laminations.

10. A factor of safety of three is provided against such bending stress, provided the rafter does not
have a joint occurring in the outside lamination near the point of support.

ll. Horizontal shearing stresses are not of major importance in glued, laminated, bent rafter sections.

Support provided by the mow floor

The Gothic arch barn roof has rafters that extend from the sill to the ridge. Experience has shown that this type of construction welds the structure into a unit that is capable of resisting the ordinary stresses and strains that might normally be put upon it without any appreciable variation in alignment in any direction. Since the rafters are fastened to the mow floor, support is provided against horizontal displacement. The horizontal forces are carried to the ends of the barn where sufficient bracing can be installed to prevent any change in vertical alignment.

The ability of the mow floor to transmit horizontal forces will depend upon the direction of the flooring, the number and size of nails used, the holding power of each nail, variations in strength of the wood, defects in the wood, variations in workmanship, and variation in the load application. Since the strength of a simple beam varies as the square of the depth, the stresses imposed upon the mow floor will be exceedingly small. Because the stresses are small, the great number of variables is of minor importance.
The stress induced in the mow floor as a result of wind loads can be calculated on the basis of a solid beam whose length is equal to the length of the barn and whose depth is equal to the width of the barn.

For a wind velocity of 70 m.p.h. the velocity pressure is 16.2 pounds per square foot. This force distributed over a rafter length of 14'-0" results in a load of 227 pounds per foot of barn length. The bending moment produced at the center of the span would be equal to 116,224 foot pounds or 1,394,688 inch pounds. The fiber stress produced in a beam one inch wide, 34 feet deep, and 64 feet long would be:

\[ BM = SZ \quad S = \frac{M}{Z} \frac{6M}{bd^2} \]

\[ S = \frac{6 \times 1,394,688}{1 \times (\frac{34 \times 12}{2})^2} = 50 \text{ lbs. per sq. in.} \]

From the calculations given above, it would seem that the mow floor is sufficiently rigid to transmit all the horizontal forces to the ends of the barn.

Support provided by the sill joint

During the stress analysis the sill joint was assumed to be sufficiently rigid to resist all the forces imposed upon it. However, when the rafter is supported by the joist at the mow floor and fastened at the sill, the bending moment developed by the wind loads imposes another horizontal force at the sill which has not been taken into account.
The magnitude of this horizontal force may be obtained by dividing the bending moment at the mow floor by the distance from the sill to the mow floor. For a 70 m.p.h. wind this force would be:

$$2518 + 5.5 = 460 \text{ pounds}$$

The safe lateral load for a 2" x 6" stud toenailed to a 4" x 6" sill with four 16d nails is 300 pounds. A factor of safety of six was provided in this calculation. A load of 460 pounds upon the sill joint reduces the factor of safety from six to four. Since wind loads are imposed for comparatively short periods a factor of safety of four should be sufficient. However, since no actual test data are available, it was believed that laboratory tests on the sill joint should be conducted.

**Tests of sill joints**

**Introduction.** The fourth step in the design of any structure is to design connections between members. The common method of toenailing the rafter to the sill has received a great deal of criticism.

The final step in the experimental study was the design, construction, and testing of full-scale sill joints. Seven different joints were tested. While four of the joints were of new designs, the remaining three were the conventional toenailed joints.
The testing frame. The apparatus used was designed and constructed for the purpose of testing members, joints, and roof trusses in connection with the study of wind loss and grain storage problems. The two steel frames shown in Figure 47 were redesigned and set on the concrete foundation shown in Figure 48. Steel angle irons were embedded in the concrete and the distance between the frames was made adjustable. A number of 2" x 10" planks were placed between the frames for the purpose of holding the test specimen in position while the loads were applied by placing sandbags in baskets suspended from cables. Figure 49 shows the completed testing frame.

Method of procedure. The results of the stress analysis indicated that the windward sill joint must provide resistance to a vertical lifting force as well as to a horizontal force. The leeward sill joint must resist a vertical compressive force and a horizontal force.

Since a structure such as a Gothic arch barn can be no stronger than its weakest joint, the windward sill joint was selected for further investigation.

In ordinary barn construction at every three or four feet the sill is bolted to a concrete foundation. The rafters then rest upon the sill and are securely toenailed. The procedure used in this investigation was reversed. The rafter was securely fastened and held rigidly in place by
Figure 48. Foundation Details for Rafter Testing Frame
Figure 49: Rafter Testing Frame
a 2" x 6" brace 5'-0" in length, as shown in Figure 51. Load was applied in a horizontal direction by means of an evener and clevis attached to the sill. In actual practice the rafter tends to move away from the sill, while in the test setup the sill moved away from the rafter. The vertical component of the load was supplied by placing sandbags in a basket suspended from a cable and passing through a pulley placed directly over the joint.

Three types of full-size sill joints were constructed. The sill was composed of 2" x 6" members 2'-0" in length. Six rafters were made of 7- 1 3/4" x 25/32" laminations, as would be used in actual practice, while the seventh rafter was a solid 2" x 6" member. Figure 50 illustrates the construction details used in making the sill joints.

Results. The results of the tests of sill joints will be discussed separately.

Sill Joint No. 1

The design used in Joint No. 1 is new and was used in an attempt to secure more resistance to horizontal forces. Two triangular members made by sawing along a diagonal of a 2" x 2" were glued and nailed to the sill. The end of the rafter was shaped to fit into the trough formed by the triangular members. The joint was held together by means of two 16d nails driven at an angle through the rafter
Figure 50. Construction of Sill Joints
composed of 7-1 3/4" x 25/32" laminations. One nail was driven from each side.

A constant vertical lifting force of 60 pounds was applied to the sill while the horizontal force was applied in 50-pound increments until failure occurred.

Sandbags were added to the basket until a load of 1700 pounds was reached. Since the safe stress for the 1/8" nineteen strand flexible cable had been exceeded, loading was discontinued. Figure 51 shows the test equipment, and Figure 52 shows the effect upon the joint at the final loading. The sill had loosened from the rafter and lifted about 3/4 of an inch, but was still capable of resisting more horizontal load. The joint was pried apart for the purpose of inspecting the effect upon the nails and triangular bearing plates. In a 70 m.p.h. wind, this joint has a factor of safety of at least 3, the computation being based upon the results of the structural analysis.

Sill Joint No. 2

Joint No. 2 was also a new design created in an attempt to secure a construction capable of withstanding horizontal forces. One triangular bearing plate made from a 2" x 2" was glued and nailed to the sill. Glue was also applied to the sill over an area approximately equal to the area of the rafter. Two 16d nails were used to fasten the rafter to the sill and provide the necessary pressure for holding the
joint together until the glue had dried.

In the basket were placed 1600 pounds of sandbags. Apparently a good glued joint had been obtained because the glue was not broken and the joint was in perfect condition when removed from the frame. Figure 56 illustrates the condition of the joint after it had been subjected to the testing procedure.

Sill Joint No. 3

Joint No. 3 was made by toenailing a solid 2" x 6" stud to the sill. Four 16d nails were used, two on each side. Failure was first noticed at a loading of 950 pounds. The picture shown in Figure 53 was taken at a loading of 1190 pounds and complete failure, as shown in Figure 54, occurred at a load of 1350 pounds. The two nails towards the load were pulled through the end of the stud and can be seen in Figure 54.

Sill Joint No. 4

Six 16d nails were used to fasten the stud to the sill in Joint No. 4. Increased strength resulted from the use of the two extra nails because no noticeable movement occurred until a loading of 1050 pounds had been reached. The picture shown in Figure 55 was taken at a load of 1600 pounds. Loading was continued up to 1700 pounds when the spindle supporting one of the pulleys failed. After a careful inspection of the testing apparatus the decision was made
to use loads up to 1600 pounds. Rafter No. 4 had lifted about 1/2 inch on the side away from the load but was still capable of resisting more load.

Sill Joint No. 5

The construction used in Joint No. 5 was the same as that used in Joint No. 1, except that glue was applied to the sill over an area approximately equal to the area of the end of the rafter. Also one 6d nail was driven through the triangular bracing plate into the edge of the stud in an attempt to secure a good glued joint.

A vertical force of 100 pounds was applied to the last three joints in place of the 60-pound load which previously was used. In addition to this vertical load, the joint withstood a horizontal load of 1600 pounds and showed no signs of failure when removed from the testing frame. The glued joint was in perfect condition and from all appearances would withstand several times the load imposed upon it. Figure 56 shows the joint at the time it was removed from the testing frame.

Sill Joint No. 6

This joint was constructed in the same manner as that used in Joint No. 2 except that no glue was applied to the joint. Four 16d nails were used to toenail the stud to the sill. This joint was subjected to a horizontal load of 1600 pounds and a vertical load of 100 pounds. Figure 57 shows
the condition of the joint after the 1600 pound load was applied. The sill had lifted about 1/2 inch from the end of the stud. An additional load of several hundred pounds would be necessary to produce complete failure of this joint.

Sill Joint No. 7

The ordinary method of fastening a rafter to the sill was used in Joint No. 7. Four 16d nails were used. Failure was first noticed at a load of 1000 pounds. Figure 58 illustrates the condition of the joint after applying a load of 1500 pounds. One nail was pulled through the end of the stud.

The condition of the seven sill joints after having been subjected to test is shown in Figure 59. All but Joint No. 1 were just as taken from the testing frame. Joint No. 1 was pulled apart for the purpose of observing the condition of the nails. Figure 60 illustrates the loading basket after 1600 pounds of sandbags were placed in it. Table VI presents the results of these tests in tabular form.

Since the loads were applied by flexible wire rope and ball bearing pulleys, it was necessary to determine the friction in the rope and pulleys. A spring balance with a maximum capacity of 200 pounds was installed between the load and the point of application. Known loads were placed in the basket, and the readings of the spring balance were recorded. The spring balance registered 198 pounds when a
Figure 59. Condition of Sill Joints After Testing

Figure 60. Loading Basket
Table VI

Results of Tests on Sill Joints

<table>
<thead>
<tr>
<th>Sill Joint No.</th>
<th>Specifications</th>
<th>Start of Failure Lbs.</th>
<th>Ultimate Load Lbs.</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 Triangular bearing members placed flush with each edge of sill. 2-16d nails used.</td>
<td>1000</td>
<td>1900</td>
<td>Joint withstood load of 1700 lbs. without complete failure. Lifted about 3/4 inch.</td>
</tr>
<tr>
<td>2</td>
<td>1 triangular bearing member placed in center of sill. 2-16d nails and glue used at joint.</td>
<td>No failure</td>
<td>No failure</td>
<td>Joint in perfect condition after having been subjected to a load of 1600 lbs.</td>
</tr>
<tr>
<td>3</td>
<td>2&quot;x6&quot; solid stud toenailed to sill. 4-16d nails used.</td>
<td>950</td>
<td>1350</td>
<td>Two nails pulled through the end of the stud.</td>
</tr>
<tr>
<td>4</td>
<td>6-16d nails used to fasten stud to sill.</td>
<td>1050</td>
<td>1700</td>
<td>Sill lifted 1/2&quot; at final load.</td>
</tr>
<tr>
<td>5</td>
<td>Same as Joint No.1 except glue used in addition to nails.</td>
<td>No failure</td>
<td>No failure</td>
<td>Joint in perfect condition when removed from the frame.</td>
</tr>
<tr>
<td>6</td>
<td>1 triangular bearing member placed in center of sill. 4-16d nails used to toenail stud to sill.</td>
<td>1400</td>
<td>No failure</td>
<td>Sill lifted about 1/2 inch after a load of 1600 pounds.</td>
</tr>
<tr>
<td>7</td>
<td>4-16d nails used to toenail rafter to the sill.</td>
<td>1000</td>
<td>1500</td>
<td>Sill pulled from stud. One nail remaining in the stud.</td>
</tr>
</tbody>
</table>
load of 200 pounds was placed in the basket. The same set of pulleys and cables had been calibrated previously by Clark (5). The percentage of error was 1 1/2 per cent for a load of 500 pounds. This variation was well within the allowable error and no corrections were made.

**Conclusions.** The following statements of facts are justified from the results of these tests:

1. The ordinary method of fastening the rafter to the sill has a factor of safety of two for a 70 m.p.h. wind.

2. The use of a small quantity of glue increases the strength of the joint many times.

3. The sill joint has sufficient strength to resist the horizontal components of the wind loads.

4. The use of triangular bracing members increases the strength of the joint over that of the ordinary method of fastening.

5. The sill joint is capable of providing the necessary support to balance the bending moment induced in the rafter at the mow floor.

6. Six 16d nails did not add materially to the strength of the joint over that provided by four 16d nails.
SUMMARY

1. A review of the wind loss statistics was conducted to show the magnitude and distribution of wind damage.

2. The requirements of the barn roof were discussed.

3. The methods of determining wind pressure distribution were reviewed.

4. The study of the elastic properties of glued, laminated rafter sections is justified by: (1) the increased use of the glued, laminated, bent rafter; (2) the results of the statistical study.

5. The deterioration of glue under service conditions was investigated.

6. The effect of joints upon the elastic properties of a glued, laminated rafter section was determined.

7. The roof shape now in use was analyzed for bending stresses.

8. The magnitude of reactions and bending moments resulting from combined dead and wind loads were determined.

9. The support that the mow floor was capable of providing to the rafters was discussed.

10. The strength of the new and conventional designs of sill joints was tested to obtain a comparison of their structural performance.
CONCLUSIONS

1. The quality of the glue used in bent rafter construction will not be appreciably affected by eight years of service conditions. The results of the short beam tests and pure shear tests show the shearing strength of glue to be approximately 500 to 550 pounds per square inch after a period of eight years. Since the allowable shearing strength of wood as given by "Wood Handbook" (19) is about 120 pounds per square inch, the glue provides a factor of safety of 5. The results of the structural analysis show that the maximum horizontal shearing stresses will not exceed 40 pounds per square inch for a wind velocity of 70 m.p.h. The glue provides a factor of safety of 12 to such a shearing stress.

2. A joint in the outer lamination seriously affects the strength of a glued, laminated rafter section. The results of tests on full-size rafter sections 10'-0" in length show that the strength of a rafter section with the top lamination continuous is three times as strong as a similar section with a joint in the center of the span. These tests also show that a section with the outer laminations continuous throughout its length has a stiffness equal to that of a solid member of equal dimensions.
A section composed of six laminations with the top lamination continuous throughout the length of the rafter is twice as strong as a section composed of seven laminations but having the top lamination joined in the center. (Comparison is based upon the load that caused the initial failure.)

3. Rafters No. 1 to No. 8, inclusive, were taken from full-size rafters manufactured by a commercial company and sawed into two 10'-8" sections. The tests of these rafter sections show that prefabricated rafters made in a manufacturing plant under controlled conditions and by careful workmen should give years of satisfactory service to the users. Also, the results of testing rafter sections No. 1 to No. 8, inclusive, show that when rafters are constructed by the use of cold casein glue and clamped in a form under a pressure of approximately 100 pounds per square inch, nails and bolts are not necessary.

4. A shape in which the line of thrust falls entirely within the boundaries of the material is impossible when the sections are composed of circular arcs.

5. The results of the stress analysis show that a barn foundation is required to hold a building down as well as up during a 70 m.p.h. wind. For a rafter spacing of 2'-0" on center, the windward sill joint must withstand a vertical lifting force of 65 pounds, while the leeward sill joint must withstand a vertical compressive force of 280 pounds.
The ordinary method of fastening provides strength to withstand such forces.

6. The bending stresses produced by the dead loads are not critical and are almost negligible when compared with the bending stresses produced by the wind.

7. The bending stresses produced by a 70 m.p.h. wind are as follows:

   a. A maximum bending moment of 2350 foot pounds is produced when the rafter is considered to be a three-hinged arch extending from the mow floor to the ridge. The maximum bending moment occurs at a point 14'-0" above the mow floor on the leeward side.

   b. A maximum bending moment of 4228 foot pounds is produced when the rafter is considered to be a three-hinged arch extending from the sill to the ridge. The maximum bending moment occurs at a point 14'-0" above the sill on the windward side.

   c. A maximum bending moment of 2518 foot pounds is produced when the rafter is considered to be rigidly supported at the mow floor. The maximum bending moment occurs at the mow floor. A rafter composed of 7- 1 3/4" x 25/32" laminations is subjected to a fiber stress of
3400 pounds per square inch. The rafter is capable of resisting stresses of over 9000 pounds per square inch, or three times the stresses produced by a 70 m.p.h. wind, the figures being based upon the results of testing the full-size sections. Since the wind pressure varies as the square of the velocity, a full-size rafter can resist a wind velocity of 110 m.p.h.

8. A few facts are presented based upon the results of the tests on sill joints. The ordinary toenailed sill joint provides a factor of safety of two during a 70 m.p.h. wind. The use of a small quantity of glue greatly increases the strength of a sill joint as shown by the results of testing Joints No. 2 and No. 5. The design used in Joints No. 1, No. 2, No. 5, and No. 6 proved superior to the conventional toenailed joint. The sill joint can be made sufficiently rigid to resist the horizontal forces of a 70 m.p.h. wind. Six 16d nails did not add materially to the strength of the joints over that provided by four 16d nails.
LITERATURE CITED


ACKNOWLEDGMENTS

The writer wishes to express his appreciation for the invaluable help and suggestions of Professor Henry Giese, the leader of this project.

The constant interest and cooperation of Dr. J.B. Davidson are gratefully acknowledged.

The helpful suggestions and criticisms of the several members of the Agricultural Engineering Department were received with appreciation.

The author recognizes the value of his association with Professor R. A. Caughey of the Civil Engineering Department, whose suggestions and advice are greatly appreciated.

The writer also wishes to express his gratitude to the Iowa Mutual Tornado Insurance Association and to the Iowa Farmers' Mutual Reinsurance Association, whose support of this project is acknowledged with appreciation.