A study of the braced rafter roof

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A STUDY OF THE BRACED RAFTER ROOF

by

George E. F. Pickard

A Thesis Submitted to the Graduate Faculty for the degree of

MASTER OF SCIENCE

Major Subject Agricultural Engineering

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JUSTIFICATION FOR THE STUDY

Statistical study of losses due to wind damage of farm buildings has shown very definitely the need for more wind resistant construction. Data on constructional losses have been gathered through study of the experience of the Iowa Mutual Tornado Insurance Association. For the three year period 1930 to 1932, constructional damage to barns represented 53% of the total losses in farm buildings. (5)

Of this damage to barns, 60% was described as complete demolition. A considerable percentage of this loss may be attributed to failure of some roof member, throwing stress into other parts of the structure which resulted in the final collapse of the building.

Five and one-half percent of windstorm loss was paid for roofs blown off. This is almost always due to the lack of proper anchorage and wind bracing of the roof.

The gambrel shape roof as used so widely for barns has three features which contribute much to its popularity. First is its utility. It provides a large mow space in proportion to the roof area and the shape is well suited to the use of all hay handling equipment. Secondly, the gambrel shape lends
itself well to efficient use of building materials and the labor involved in construction is not excessive. The third commendable feature of the gambrel shape is its pleasing appearance. In its lines it harmonizes reasonably well with the rest of the farm buildings.

During the past few years the gothic roof has enjoyed wide popularity. This is partly due to the novelty of its shape but it is very definitely deserving of favor on account of its pleasing appearance and the clear mow space it provides. Many gothic roofs have been unsatisfactory because they have sagged out of shape. This can usually be traced to faulty construction rather than to any weakness in the design. However, the usual types of gambrel roof construction are not affected so critically by slighting of materials or faulty workmanship.

The braced rafter type of construction is very desirable for gambrel shaped roofs. Its ease of erection, reasonable economy of materials and fulfilment of the service requirements have made it the most popular type of gambrel roof construction. The rafters are self-supporting and light in weight making them easy to raise. After the first two rafters at the end of the structure have been raised into place, braced and tied together, a man can easily climb to a position near the ridge and, by means of light boards, tie the succeeding rafters in place as they are raised into position. Use of standard length timbers
eliminates excessive cutting and waste. The service require-
ments of a roof are very well satisfied by the braced rafter
construction. Objectionable purlin support columns are
avoided and none of the braces project far into the mow space.
Adequate height is provided and there are no obstructions to
the use of hay handling equipment.

The Objects of the Study

The purposes of this study are to investigate the strength
requirements of the braced rafter roof and to develop a more
wind-resistant type of construction. This improved rafter
should compare favorably in cost with present recommended de-
sign. Observations on wind damage to buildings indicate that
this improvement may be realized through; (1) improved method
of joining members; (2) more secure anchorage of the rafter to
the wall; and (3) more efficient and economical use of bracing
material.
HISTORICAL

The Project

This study is a part of the general project of the Agricultural Experiment Station, "An Investigation of the Wind and Fire Losses to Farm Buildings in Iowa". The object of this division of the project is to make a study of wind losses and building design in Iowa in order to secure more wind resistant construction.

This work has been pursued in five fields of activity. Briefly, these are

1. Field observation of wind damage.
2. Statistical study of losses.
3. Meteorological studies.
4. Aerodynamics.
5. Laboratory study.

The first three have been discontinued for the time being at least.

In the absence of conclusive and entirely dependable experimental work on the subject of wind pressures on farm buildings, the aerodynamic study has become a matter of adapting the results of other investigations to the problem in
hand. The work of such authorities as Dr. Dryden (6) of the Bureau of Standards has been of particular value.

The laboratory study has centered around the testing of rafters and trusses. Loads are applied to models of the structures to approximate wind loading conditions. Thus the strength, rigidity, and points of weakness of the various designs are determined.

Studies have been made on Clyde truss construction (2) and on laminated bent rafters. (5) Now this study of the braced rafter roof takes its place as an integral part of the general investigation.

Review of Literature

In his bulletin on Research in Farm Structures, Henry Giese (9) states the following:

"The losses due to fire and wind are truly economic. While the individual may be partly protected by insurance, the loss must be paid ultimately by the agricultural industry."

"Studies should be made as to the reasons for these losses and as to improved construction methods for reducing future losses."

In the bulletin Modern Connectors for Timber Construction,
prepared jointly by the National Committee on Wood Utilization and Forest Products Laboratory, the authors (10) point out the need for a stronger joint in wood construction. In reviewing the work done by early European engineers, they state,

"The timber joint, long recognized as the critical link in every wood structure, was the logical point for improvement."* With reference to Modern Connectors, these investigators state, "Obviously the experimental stage has long since been passed. Through the application of these devices, wood has assumed a new structural importance."

In his book Wood in Aircraft Construction, George W. Trayer (14) devotes considerable space to the composition, testing, and use of casein glue. He writes of this glue, "It is the best all-around glue for air-craft construction." This statement of the value of casein glue gives a suggestions of its probable adaptability to farm building construction.
Preliminary Considerations and Investigations

Analysis of the problem

In the consideration of any problem involving the design of a building frame, it is necessary to have a clear conception of the various requirements of that structure. These requisites of a barn roof will be dealt with in order.

Service requirements of the barn roof. There are a number of roof characteristics which are common for practically all general purpose barns and dairy barns. The roof must be such as to provide adequate mow space for hay storage. This mow space should be reasonably free from obstructions. Purlin columns are objectionable and heavy trusses which extend far into the mow space are to be avoided. The clear height should be sufficient for convenient storage and handling of hay when using a fork or slings. Trusses with low-hanging cross-ties or collar beams are not suitable for this reason. The width of the roof near the ridge should be great enough for the passage of the hay carrier without lowering the track far from the peak. This essential is satisfied by all of the roof slopes in general use.
Structural requirements. The structural demands upon a barn roof have respect mainly to strength, rigidity and stability. The supporting frame must be sufficiently strong and rigid to carry all probable loads without excessive temporary or permanent deflection. Pressures exerted by high wind velocities may cause failure of the roof frame or permanent deformation of the structure. The gusty nature of winds produces a vibratory deflection of the roof which tends to loosen the joints of the frame. In addition the roof structure must be sufficiently rigid to avoid objectionable deflection under its own weight.

With regard to stability it is evident that a roof should be in a stable condition of equilibrium under customary dead loads. That is the bending moment and resultant forces should be zero at all points on the roof where deflection is liable to occur.

Economic requirements. The economic demands upon any building construction have regard to the first cost and the subsequent charges for repair and depreciation. The barn roof design should provide economy of material and labor in construction. Standard sizes and lengths of lumber should be used. Excessive labor and the need for special skilled workmen should be avoided.

A permanent type of construction means small annual depreciation. The roof covering must be durable itself and must
provide protection for the supporting structure. Therefore, whether it be wood shingle or composition roofing the material should be of good quality. Sufficiently strong rafter construction and adequate bracing do much to insure years of satisfactory service.

Aesthetic requirements. Since the farmstead presents such a heterogeneous group of architectural designs, it is hardly feasible to require harmony of line between the barn and other buildings. Rather, as an individual unit, the barn should be pleasing to the eye, giving the impression of permanence and stability. Great difference in the length of lower and upper rafter members makes a gambrel roof barn appear ungainly or squatty. Such proportions should be avoided.

Selection of a standard barn shell

Dimensions. The width of barn to be used in this study is 34 feet. Observations of barns and barn plans made by Barre (4) show that; (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. From a study of various combinations of interior dimensions determined by the stall arrangements, it was found that the width of 34 feet gives the nearest to the average and optimum dimensions. In view of the facts revealed
by this study, it would appear that either the 34 or 36 foot width is desirable. Since the narrower barn is more economical and warmer, due to the smaller interior surface area, that width of 34 feet is to be preferred.

The height of the ridge above the mow floor is limited primarily by the mow space required. However, there are some other factors which should be considered in determining this height. The ridge should be sufficiently high for efficient use of hay handling equipment. If the hay carrier is high the hay does not have to be mowed back so much. On the other hand, excessive height is to be avoided. Barns with a high ridge are more costly, are more susceptible to wind damage and are not so pleasing in appearance as those of medium height.

In a study of barn dimensions Schweers (11) found that the average hay storage space required per cow under Iowa conditions is 1080 cubic feet. Eighty square feet of floor area per cow with a barn width of 34 feet gives a barn length of 2.35 feet per cow. The volume of hay storage space divided by the length per cow gives the new mow cross-sectional area required. This is 460 square feet. Schweers allowed 38 square feet for the space taken up by rafters, braces, etc., giving a gross mow area of 498 square feet. A mow cross-sectional area of 500 square feet will be used in this study.

Economic use of materials. In determining the shape and
dimensions of a barn shell careful consideration should be given to the effect of the design on the use of materials. Standard length timbers should be used wherever possible. Odd-length or oversize members cost more and increase the amount of waste. Care should be taken to avoid over-designing and unintentional slighting of materials.

The shape of roof to provide stability. A roof that has no tendency to sag at the ridge is very desirable. The essential condition for such a roof is that the applied forces and the internal stresses acting at the rafter splice be in equilibrium. The applied load at the rafter splice and the axial stresses in the rafters must intersect at a point which coincides with the intersection of the rafters. The rafter splice will then have no tendency to move in any direction and the forces will have no rotational effect.

If the rafter splice has no tendency to move inward or outward, the roof cannot sag except through bending of the rafter members. Such a shape may be selected that the rafters will carry all the dead load. The braces will carry only wind and hay load stresses.

Rafter lengths of 14 and 12 feet for lower and upper members respectively were chosen. These are the most popular lengths for 34 foot barns and give a roof shape that is pleasing in appearance.
In this study it is assumed that the weight of the rafters, sheathing and shingles acts at the ends of the rafter members. The weights used are shown in Figure 1. These were calculated for a design of the type recommended by the Midwest Plan Service (1) using a 2\"x6\"-12'-0" rafter-to-studding brace; 1\"x6\" rafter-splice braces and a 2\"x6\" collar beam at the ridge.

The procedure of the method used to determine the stable shape is illustrated in Figure 1. Three different positions of the members were taken. First the rafter splice was located on the semi-circle as shown. Next the lower member was inclined at an angle of 60 degrees to the horizontal. The third position located the rafter splice midway between the first two points.

As each position of the members was drawn, the corresponding stress diagram was made. Various directions of the reaction R2 were obtained when the polygon for the forces at the rafter splice was made to close. But, of course, R2 must be horizontal. However, it was found that the positions of the point e in the stress diagram described a smooth curve. This was carefully plotted by taking additional positions of the members. The position of e required to make R2 horizontal was then located. The positions of eB and eC so established give the inclinations of the rafter members. Thus, the shape of
the stable rafter was determined and is shown in Figure 1.

In this position the stress in member CD is the resultant of the two applied loads at D and therefore really represents the path of the line of force to the support. That is, members CD and BC represent the position of the equilibrium polygon. Since the polygon passes through all the points of the roof, there can be no tendency for the roof to move in any direction. That is, there is no tendency for it to sag.

The lower members are inclined 62 degrees and the upper members 31 degrees to the horizontal. The gross mow area above the plate is 396 square feet. Four feet of studding above the mow floor is required to give a net area of 494 square feet as desired. The height of the ridge above the mow floor is then 22 feet, 6 inches.

The conclusions that may be drawn concerning this determination of a desirable shape are:

1. For any combination of rafter lengths there is one position in which the sum of the forces (external and internal) acting at the rafter splice, and the moments at that point, are zero for customary dead loads.

2. Such a shape of roof has no tendency to sag at the ridge.

3. For a roof using 14 foot lower and 12 foot upper rafters, this condition is secured when the lower and upper
Figure 1. Determination of Shape of Roof to Provide Stability
rafters are inclined to the horizontal at angles of 62 degrees and 31 degrees respectively.

4. A wall height of 4 feet above the mow floor is required to provide sufficient hay storage space when this roof shape is used.

5. The height of the ridge above the mow floor is 22 feet 6 inches.

Wind-load distribution

The problem of accurately determining the pressures exerted upon structures by wind is still in its infancy. In the past many tests have been made on plane surfaces and of late considerable work has been done on models of structures. Much of this work has been found to be of little value due to inaccurate apparatus or to disregard of certain effects of the wind and of the shape of the model under test. Most of the wind loads used in practice are based upon theoretical computations which disregard the shape of the structure and take into account only the impact pressure of the wind against the windward surface or surfaces of the structure.

The old methods of calculating wind loads (7)

Numerous formulae have been used from time to time to show the relation between wind velocity and the pressure
exerted by the wind on any given object in its path. For calculating the pressure on a body which presents a flat surface perpendicular to the wind, the formula has been \( p = kv^2 \), where \( p \) is the pressure in pounds per square foot; \( v \) is the velocity of the wind in miles per hour; and \( k \) is a constant. It is in the selection of this value for \( k \) that there have been great variations.

Newton and Rankine, by purely theoretical calculation and disregarding the suction created on the leeward side of any object, recommended values for \( k \) of 0.0027 and 0.0054 respectively. From experimental results, Smeaton chose the empirical value for \( k \) of 0.005 which has been used for over 150 years. Although this determination of the constant takes into account the suction on the leeward side of the apparatus used by Smeaton, it does not recognize the effect of the shape of the object and the value of \( k \), thus obtained, has no true value for general application in practice.

For the calculation of wind pressures 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. However, this formula has been found to be widely erroneous and has been little used. The empirical
relation \( p_n = (p \sin \theta + 1.84 \cos \theta - 1) \) suggested by Button has had wide use, as has also the more recently accepted formula of Duchemin, which gives \( p_n = p \left( \frac{2 \sin \theta}{1 + \sin^2 \theta} \right) \). Neither of these formulae takes into account the suction on the leeward side of the surface, nor the reduced pressure on the windward side caused by the increase in wind velocity parallel to the surface. Therefore, these formulae cannot be considered adequate for the required determination of wind loads on structures, and it becomes necessary to adopt some method of calculating wind pressures that will take cognizance of the conditions neglected by the earlier investigators.

**A discerning method of calculating wind loads**

Considering any point on an object against which the wind is blowing, the pressure at that point can be assigned a value represented by \( p \). Now this pressure \( p \) can be considered to consist of \( p_s + p_w \), where \( p_s \) is the static pressure and \( p_w \) is that pressure caused by the presence of the object in the airstream. If the object is not there, \( p = p_s \) and the wind pressure \( p_w \) is equal to zero. Now in consideration of a structure such as the gambrel roof barn, with all doors and windows closed, the pressure inside is assumed equal to the static pressure \( p_s \). Therefore, the resulting unit pressure, tending to cause impact breaking or bursting of the structure is equal
to the difference in pressure on the opposite surfaces of the walls or roof. This will be $p - p_g$, since $p = p_g + p_w$.

Now, $p_w = f(V \frac{L \epsilon}{\nu})$

where $q$ is the velocity pressure $(c \frac{v^2}{2})$, $c$ the air density, $V$ the wind speed, $\nu$ the viscosity of the air, $f$ a constant and $L$ a linear dimension indicating the scale. (6) This formula applies only to geometrically similar bodies because it has been found that for many shapes the factor $V \frac{L \epsilon}{\nu}$, which is known as Reynolds number, varies very rapidly with variation in wind speed and size of object. The variation is known as scale-effect and when present makes very difficult the use of models to indicate pressures on full size structures. However, for bodies without curved surfaces and having sharp corners, the expression $p_w$ is practically independent of the wind speed and size of object. (6) That is, $f(V \frac{L \epsilon}{\nu})$ is constant for any given wind direction. Dr. Dryden of the Aerodynamics Laboratory of the Bureau of Standards in a communication to Professor Giese states, "It is believed that the scale-effect on the gambrel roof barn is small."

When the scale-effect is considered to be small, $p_w = f(V \frac{L \epsilon}{\nu})q = cq$, where $c$ depends only upon the location of the point on the structure. Therefore, if the value of $c$ is found for any point on a model, that value will apply to any size of the model at any wind speed. $V \frac{L \epsilon}{\nu}$ is a pure number,
as is \( f \), so \( c \) is independent of the units used as long as the pressures are all measured in the same units.

**Wind-load distribution**

It is required to find the distribution of wind loads, for various wind speeds, over the surface of a structure having little or no scale-effect. In doing this we can make valuable use of the coefficient \( q \), which is a pure number and constant for any station on the structure for any given direction of the wind. Charts may be prepared to show the value of \( q \) for all points on the building for various wind directions and then by means of tables of velocity pressures the wind load at any point or over any given area may be directly calculated.

Where possible, these values of \( q \) over the surface of the structure may be established by means of wind-tunnel tests performed upon a model of the building. In these tests the value of \( q = \frac{p_w}{d} \) is directly measured at a number of points over the surface of the model and from the data so obtained the required charts are prepared. Unfortunately no dependable tests have been made on models of gambrel roof barns. However, tests have been made on some structures, such as mill buildings and airplane hangars from which estimates can be made for the shape we require. The advice of Dr. Dryden was sought and on the basis of his recommendations the charts shown in Figures
2 and 3 were prepared.

The chart of Figure 2 shows the distribution of the values of the coefficient \( \alpha \) for a wind blowing at right angles to the ridge. That of Figure 3 indicates the condition that is produced by a wind parallel to the ridge and blowing in the open mow door. The pressure inside is practically equal to that at the opening on the windward face which will be about equal to the velocity pressure, giving a value of the coefficient equal to one. This must then be added to the value of \( \alpha \), equal to about -.6 which is developed on the outer surface by the movement of the wind parallel to it. This gives a value for \( \alpha \) of -1.6 over that portion of the structure which is above the mow floor.

This wind pressure distribution was compared to that determined by Stapleton (12) and was found to check very closely with his pressure contours. This pressure distribution also agrees closely in general characteristics to that found by Bailey (3) in his tests on a large shed.

Investigation of reactions and bending moments

Objects of the study. The purpose of this section of the preliminary investigations was three-fold. For probable wind loads it was proposed to determine.

1. The magnitude and direction of the reactions at the
points of support of the lower rafters and between the upper members at the ridge.

2. The magnitude, direction and location of the maximum bending moment in the members of the roof.

3. The magnitude and direction of the bending moment at the rafter splices.

**Stress analysis.** The following principal assumptions were made in the stress analyses: that the rafter used is a three hinged arch, i.e. hinged connections at lower supports and at ridge; that each rafter used is rigid from support to ridge and is to be considered as a beam; that the supports are so completely rigid as to prevent any horizontal or vertical movement at these points.

The wind load assumptions used in these analyses are given in Figure 2 and Figure 3. For the first condition the wind is blowing at an angle of 90 degrees to the side-wall with a velocity of 70 miles per hour. The second condition represents a wind of the same velocity blowing against the end of the barn where the large hay mow door is open into the wind. This wind velocity was chosen because it is close to 68 miles per hour, the highest wind ever recorded in Iowa. (5)

The dead and wind loads were assumed to act at the quarter points and ends of each member. The wind loads are assumed normal to the roof surface.
Figure 2. Roof Stress Analysis; Wind at 90° to Side Wall
Figure 3. Roof Stress Analysis. Wind in End of Barn.
The dead and wind loads for each point on the rafter were combined into resultants, the direction and magnitude of which are given at the respective points on the diagrams of Figures 2 and 3.

Conclusions. The reactions and bending moments for the assumed conditions are given in Figures 2 and 3. From these results the following conclusions of particular significance have been drawn:

1. The end wind produces by far the most critical conditions as to both reactions and bending moments.

2. For the side wind the reactions at the supports have large horizontal components. The vertical components at both supports are small.

3. The vertical components of the reactions at the supports are very large when the wind is in the end of the barn.

4. There is a very large horizontal reaction between the rafters at the ridge for the end wind.

5. In all cases the maximum bending moment is located in the lower rafter about 3\(\frac{1}{2}\) feet below the rafter splice. For the end wind this maximum value is about 19,300 inch lbs. For the side wind it is about 12,650 inch lbs.

6. The bending moment in the upper members is low for the side wind condition.
Experimental

In this report the heading "Experimental" includes all of the laboratory study. This work was concerned chiefly with the construction and testing of models and full-size specimens of roof members and details. This work has been headed "Experimental" to distinguish it from the preceding investigations which were of a more theoretical nature.

Tests of sheathing

In the consideration of barn roof design from the viewpoint of an investigator, the natural place to start is at the outside. The character of sheathing and surfacing very definitely affects the type of rafter or truss construction. In barn construction the sheathing may determine the allowable spacing of the rafters.

In the past the spacing of rafters has been limited by the fact that it is necessary to have joints of shiplap sheathing come on the rafters. In the use of end matched sheathing it is desirable to make the joints between the rafters. This makes possible the use of any rafter spacing and justifies the question, "What is the best spacing for rafters?"

Object of the study. The purpose of this investigation was to determine, if possible, the most desirable spacing of
rafters to meet service, structural and economic requirements.

Analysis of the problem. Limiting factors in the spacing of rafters are:

1. Strength and stiffness of sheathing.
   a. The distance between rafters should not be so great as to permit the development of dangerous stresses in the sheathing. These stresses may be produced by the forces exerted by wind or may be caused by a concentrated load such as the weight of a man standing on the roof.
   b. Excessive deflection of the sheathing must be avoided because it will cause tearing of the asphalt shingles so generally used. This deflection may be, (1) a gradual permanent deflection due to the dead weight of the roof; (2) a steady temporary deflection due to the weight of a man or to wind loads; or (3) a vibratory deflection caused by the wind.

2. The rafter spacing adopted must be such that the load on each rafter will not be too great. The size of members for practical use is limited by, (1) the type of truss or rafter to be used; (2) the economical means available for joining members; and (3) the weight of truss that can be conveniently erected.
with generally available men and equipment.

**Apparatus and method of procedure.** It was thought at first that the problem of sheathing stresses and deflections might be attacked by a theoretical or mathematical analysis as well as by an experimental investigation of these conditions. Some time was spent in an endeavor to calculate the stresses and deflections that would be produced by the weight of a man on the end-matched roof sheathing for various rafter spacings. This condition was chosen as being one of the most severe to which the sheathing might be subjected. However, difficulty was found in arriving at the probable distribution of the load from one board to those adjoining; also in estimating the proportion of the load carried by the cleavage, bending and shear stresses in the timber. It was felt, therefore, that more valuable results might be obtained from experimental investigation of model sections of roof sheathed with this end-matched lumber and subjected to the various loading conditions.

Concurrently with this preliminary investigation, a testing procedure was outlined and tests were begun. Observations of certain old barn roofs showed that there was an objectionable deflection of the sheathing between the rafters. These observations raised two questions, (1) was this large deflection due to excessively wide spacing of the rafters? and
(2) was this deflection progressive from the time the barn was built? It was desired, therefore, to obtain the relation, if such exists, between the span length, time and deflection. Since it is impractical to wait twenty or thirty years for these results it was thought that an indication of them might be secured by means of tests using an accelerated deflection. If the loading were increased to produce a rapid deflection, it might be possible to obtain the required relation.

Accordingly, a set-up was evolved to measure the deflection on specimens of yellow pine sheathing in lengths of 24, 30, 36 and 42 inches. These were supported at the ends as simple beams with a concentrated load at the center. The loadings adopted were one, two and four times the weight of the sheathing and shingles.

The instruments used to read deflections was a Starret Dial Test Indicator mounted on a small board which carried a pin at each end. For zero deflection the pins and the stem of the indicator were at the same height and the dial was set to read zero for this condition. The heads of tacks driven into the under side of the specimens were used as bearing surfaces upon which to take the readings of deflection.

Readings were taken once a day for the first seven days and once a week thereafter. The tack heads did not provide a sufficiently dependable point upon which to take readings.
Consistent results could not be obtained so the tests were discontinued.

This study of the roof sheathing was not continued since it was felt that the rafters demanded more urgent consideration. In this study of rafters the conventional spacing of 2'-0" will be used.

Tests of joints

Introduction. The joint has long been recognized as the weak point in wood construction. It has always been very difficult to get a joint to transmit a high percentage of the load that the gross area of the members will carry. Since the old framing method of cutting and fitting was forsaken in favor of nails and bolts, the need has always been for some device to offset their undesirable stress distribution. Nails, particularly, are a very ineffective way of joining wood members of even the small dimensions used in farm building construction. Observations of wind damage to buildings show that much of this destruction was due to failure of joints or to weakening of the members at the joints.

A number of new methods of joining wood members has been developed in the last two decades. Prominent among these are the steel ring connectors and wood dowels known collectively as Modern Connectors. (10) The commercial manufacture of
casein glue has broadened the field of usefulness for this method of fastening wood members and suggested its use in general farm building construction.

The specific objects of these tests were,

1. To obtain a comparison of the strength of joints made with Modern Connectors, casein glue, bolts and nails.

2. To obtain the strength relations for full size and model joints for use in the interpretation of the rafter test results.

3. To collect any other pertinent information relative to the manipulation and adaptability of these various devices used in wood construction.

Method of testing. The type of specimens used in these tests are shown in Figure 5. These are of the form used in the tests conducted by the Forest Products Laboratory. (8) In the first joint the load was applied to the center timber and the force acted parallel to the grain of all members of the joint. In testing the second joint the load was applied to the side pieces so that the force acted perpendicular to the grain of the center timber.

Douglas Fir timber was used to make the specimens. The wood used in making the full size joints and the center timbers
Figure 4. Southwark-Emery Testing Machine

Figure 5. Joint Test Specimens
of the models was Common lumber while that used for the outside pieces of the models was graded as Fir Finish. Compression tests of these two classes of wood showed ultimate strengths of about 6000 pounds per square inch for the Common and about 9000 pounds per square inch for the Finish lumber. While the scope of these tests does not justify any mathematical adjustment of the results, this difference in strength must be born in mind in their interpretation.

Four general types of fastener were tested, namely, Glue, Modern Connectors, bolts, and nails. Three classes of glued joints were used. The first of these had all surfaces flat and a good contact was obtained between all members. The outside pieces of the second group were warped so that a convex surface was presented to the center member as shown in Figure 6. The large reduction in glued area is evident. The outside members of the third set of glued joints were also warped but in this case the concave surface was glued. The tightening of the bolt split the side pieces and drew the whole inner surface into contact with the center member.

The "Tecco" brand of Modern Connectors was used in these tests. Two types of these were tested and are pictured in Figure 8. These are known as Split-Ring and Toothed-Ring connectors. The split-ring is used with a pre-cut groove as shown in Figure 9, while the toothed-ring is imbedded by
Figure 6. Specimen of Warped Glued Joint

Figure 7. Failure of Glue Joints
forcing the two adjacent timbers together.

Four $\frac{3}{4}$ inch machine bolts were used in each joint for the bolt tests. Extending through all three members the bolts were in double shear. This joint is shown in Figure 10.

For the nailed joints eight 16d nails were used in single shear. That is, the nails did not pass completely through the center timber from the outside piece.

The scale used in the construction of the models was $6'' = 1'-0''$. This applies to all parts of the specimens including wood members, connectors, bolts and washers.

The glue used in these tests was the Casco brand of casein glue. This is a very convenient glue for all construction purposes because it is a cold-water-mix glue which is used cold. The glued area of the full size joints was approximately 80 square inches while that of the models was about 20 square inches. A single bolt in each joint was used to hold the members together while drying and this bolt was left in place during the test. In the full size joints $\frac{1}{2}$ and $\frac{3}{4}$ inch bolts were used with square plate washers. The dimensions of washers for these joints were $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times 3/16''$ inches and $3\times 3\times 3/16''$ inches, respectively. The bolts and washers in the models were to scale.

Six different ring connectors were tested; the $2\frac{1}{2}$ and 4 inch split rings and the 2, 2 5/8, 3 3/8 and 4 inch toothed
Split Rings

Toothed Rings

Figure 8. Modern Connectors
Assembly of Split-Ring Joint

Assembly of Toothed-Ring Joint

Figure 9. Assembly of Ring Joints
Figure 10. Bolted Joint in Testing Machine

Figure 11. Failure of Nailed and Bolted Joints
rings. These are all shown in Figure 8. The sizes are based on the inside diameter. The depths of the 2½ and 4 inch split rings are ¾ inch and 1 inch respectively. The toothed rings are all 1 inch in depth.

The size of wood members for these joints was determined largely by the end and edge margins required for each ring as specified by the manufacturers. These specifications in regard to margins, sizes of bolts and of washers were followed closely in the construction of the test specimens. For the toothed-ring joints the bolt holes were drilled the same size as the bolts while those for the split rings were drilled 1/16 inch oversize to accommodate the mandrel of the groove cutting tool. The bolt holes of the model joints were all the same size as the bolts used.

The margins used in construction of the bolted joints were those recommended by the Forest Products Laboratory. (8) The sizes of the bolts and nails and their type of loading have already been given.

The method of construction of the test specimens was similar to that which would be used in field fabrication. To insure a good testing condition for each joint, a special frame was built to hold the specimens square while they were being assembled. Figure 12 shows this frame being used while a toothed-ring joint is being assembled under the loading of
In the construction of the glued joints no great care was taken to coat the whole contact surface since in practice time would not permit this application of the glue to be a painstaking operation. When the glue had been spread, the joints were immediately assembled, squared and tightened up. They were then given a setting period of seven days before being tested.

The grooves for the full size split-ring connectors were cut by means of the groove cutting tool purchased from the manufacturers of the rings. This cutting tool is shown in Figure 14. In this picture it is set to cut the grooves for the 2½ inch rings. The cutters and the sleeve for the mandrel to be used for the 4 inch ring are shown beside the tool.

Figure 15 shows the tool being used with an electric drill. The grooves for the models were cut with the circular saw shown in the same picture. A split-ring joint in process of assembly is shown in Figure 9.

The model split rings were made to the proper scale from mild sheet steel. These were cut and bent to shape without heating.

In the assembly of the toothed-ring joints the rings were first placed in position and secured there by small nails as shown in Figure 9. A strong bolt is provided by the
Imbedding Toothed Rings With Testing Machine.

Using High-Strength Bolt to Imbed Toothed Rings.

Figure 12. Imbedding Toothed Rings
Figure 13. Cutting Grooves for Split Rings

Figure 14. Groove-cutting Tools
manufacturers of the rings for the purpose of imbedding the rings in the timbers. The use of this bolt is illustrated in Figure 12. Most of these joints were assembled in the testing machine to save time and labor. The load required to imbed two rings was found to vary from about 5000 to 8000 lbs. depending upon the size of ring and density of the timbers.

Testing procedure. In these tests of joints the term "slip" is to be taken as meaning the total relative movement of the center and outside members when under load. If slip is thought of as being the "play" in a joint, or the amount that it will move without the application of a load, then the slip of these tests is a combination of slip and deformation. The use of the word "slip" in these tests is the same as in the tests outlined in the bulletin Modern Connectors for Timber Construction. (10)

The method used to read this slip was very simple. A scale marked in tenths of inches was laid off on tracing linen. Beside this scale a suitable direct vernier scale was marked off. A line separated the two scales. Black line prints of this scale and vernier were made. Now before testing the joints this scale was pasted over the crack between the center and outside timbers as shown in Figure 5. When the cement had dried, the scale and vernier were separated by making a clean cut down the line between them with a razor
blade. With this device the movement of the vernier relative to the scale could be read to one-hundredth of an inch which is sufficiently close for this type of test. Check of this method of reading was made by placing scales on opposite corners of a joint. These scales were then read concurrently during the test and were found to agree very closely.

Loads were applied to the test specimens by means of the Southward-Emery 300,000 lb. testing machine shown in Figure 4. This picture shows the loading of a joint where the load is perpendicular to the grain of the center timber. The center timber is here supported on a block which fits between the outside pieces and rests upon the bed of the machine. The outside members bear against a spherical bearing block in the stationary head of the machine. Some of these joints were supported by means of a block under each end of the center timber. However, this type of support was considered unsuitable for the glued joints on account of the stresses produced in the glue by the bending of the center timber.

The load was applied to the joints continuously and readings of the load were taken at each .01 inch of slip up to .1 inch and thereafter at .12 and .15 inch slip. The rate of ascent of the movable bed of the machine was .05 inch per minute. The load at the first apparent slip, at first and subsequent failures and the ultimate load were noted and
character of failure and other significant details were also noted.

**Discussion of results.** The results of these tests are shown in the graphs of Figures 15, 16 and 17. Figure 15 gives a comparison of the various joints and also shows the comparative strength of joints when loaded parallel to and perpendicular to the grain. These curves are averages of those plotted for individual joints. Three tests of each type of joint were made in most cases.

The glued joint performed particularly well when the load was applied parallel to the grain. In most cases failure occurred due to shearing of the timbers rather than the glue, as illustrated in Figure 7. This picture shows on the right the failure of a warped specimen. In spite of the small contact surface of this joint, it carried a load of 32,800 pounds. The ultimate strength of the glued joint perpendicular to the grain is about 1/3 of the strength when the load is parallel to the grain. This is due to the nature of these joint failures. The fibres of the center timber can be torn apart perpendicular to their axes much more easily than in the line of the axes.

Under parallel loading the slip of the bolted joint at proportional limit load was much lower than that of the 4-inch split ring but the ring carried 22,000 pounds at proportional
Figure 15. Load-slip Curves for Joints

<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Force Applied Parallel to Grain of Center Timber</th>
<th>Force Applied Perpendicular to Grain of Center Timber</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Glued joints, area 30 sq. inches</td>
<td>0.02</td>
<td>0.04</td>
</tr>
<tr>
<td>2. 4-1/2 inch split rings</td>
<td>0.02</td>
<td>0.04</td>
</tr>
<tr>
<td>3. 2-4 inch toothed rings</td>
<td>0.02</td>
<td>0.04</td>
</tr>
<tr>
<td>4. 2-2/3 inch toothed rings</td>
<td>0.02</td>
<td>0.04</td>
</tr>
<tr>
<td>5. 2-2/3 inch split rings</td>
<td>0.02</td>
<td>0.04</td>
</tr>
<tr>
<td>6. 8-20d nails</td>
<td>0.02</td>
<td>0.04</td>
</tr>
</tbody>
</table>

(Glue Load in lb.) vs. Load (thousand lb.)
Figure 16. Strength Comparisons of Glued Joints.

1. Glued joint, parallel to grain, full size.
2. Glued joint, parallel to grain, full size.
3. Glued joint, perpendicular to grain, full size.
4. Glued joint, perpendicular to grain, full size.
5. Split-ring joint, parallel to grain, full size.
6. Split-ring joint, perpendicular to grain, full size.
7. Split-ring joint, perpendicular to grain, full size.
8. Split-ring joint, perpendicular to grain, full size.
9. Same as 7, but surfaces flat.
10. Same as 7, but surfaces flat.

Figure 17. Strength Comparisons: Full Size and Model Joints.

- Glued joint, parallel to grain, full size.
- Glued joint, parallel to grain, full size.
- Glued joint, perpendicular to grain, full size.
- Glued joint, perpendicular to grain, full size.
- Split-ring joint, parallel to grain, full size.
- Split-ring joint, perpendicular to grain, full size.
- Split-ring joint, perpendicular to grain, full size.
- Same as 7, but surfaces flat.
limit compared to the 11,000 pounds of the bolted joint. The slip in the ring joint was nearly proportional until very close to the point where the cores in the joint sheared off as marked by the sudden drop of the load from 22,500 at a slip of .085 inches. Pictures of the failure of bolted joints are shown in Figure 11.

The 2 ½-inch split ring compared rather closely to the larger ring in behavior. The slip was rapid during the initial period of loading due to the loose fit of the rings in the grooves. The proportional limit load was about 5000 pounds at a slip of .05 inch. This would indicate a working load of approximately 2800 pounds which is very much lower than that recommended by the Forest Products Laboratory. (8) Failure of a 2½-inch ring joint parallel to the grain is shown in Figure 18.

The 2 5/8 and 4-inch toothed rings behaved very similarly to the bolted joint. The slip at proportional limit was between .01 and .02 inch. Figure 19 shows the failure of a 4-inch toothed ring joint parallel to the grain.

The low strength of the nailed joints is shown by curve No. 7. The slip increased rapidly from approximately zero load. The strength of these joints checks quite closely with the results obtained by Wells (15) in his tests of nailed joints conducted at Stanford University. A typical failure
Figure 18. Failure of Split-Ring Joints
Load Parallel to the Grain

Load Perpendicular to the Grain

Figure 19. Failure of Toothed-Ring Joints
of nailed joints is shown in Figure 11.

When tested perpendicular to the grain, the joints showed quite a different strength relationship. As before mentioned, the strength of the glued joint decreased to about 1/3 of its value when loaded parallel to the grain. This decrease placed it down among the other types of fasteners as shown by the curves of Figure 17.

The bolted and nailed joints showed very little difference in strength for the two directions of loading.

The curves of the split-ring joints for the perpendicular loading show some similarity to those for the parallel loading. However, the strength of the proportional limit load of the 4-inch split ring changed from 22,000 to about 11,000 pounds placing it equal to that of the bolted joint.

The strength of the toothed-ring joints was reduced about 33% by the change in direction of loading.

Examples of failures of these joints are given in Figures 18 and 19.

Figure 16 shows some significant strength comparisons obtained by using warped members in the construction of glued joints. It is of particular interest to note that the ultimate strength of glued joints having the small contact area shown in Figure 7 is 75% of the strength of flat-surfaced joints. The joints having concave surfaces crushed down by
the bolt showed a higher ultimate strength than the ones with flat surfaces. This relation should be born in mind when using warped material in glued-joint construction of any kind.

The warped glued joints having the load perpendicular to the grain, gave a higher strength than the regular flat ones did. Also, the character of the failure was rather different. The warped joints showed a steady slip while the flat ones slipped suddenly in a manner similar to that of the parallel joints. No reason for the unusual behavior of these joints has been discovered.

Figure 17 illustrates the relation between the strength and slip of full size and model joints. The model glued joints loaded parallel to the grain had an ultimate strength 35% of that of the full size joints. When the load was perpendicular to the grain, the ultimate strength of the models was 42.6% of that for the full size joints.

The behavior of the $2\frac{3}{4}$-inch split-ring joints was almost identical for full size and model specimens during the first .03 inch of slip. This similarity is shown in the curves of Figure 17. When loaded perpendicular to the grain, this resemblance continues to a slip of .06 inch after which the curve of the full size ring rises rapidly. The slip of the joint at the point of divergence of the curves is evidently that amount of slip which takes place during the period of
adjustment of the rings under load.

The proportional limit load of the model, parallel to the grain, was about 50% of that of the full size joint. A similar reduction is shown for the perpendicular loading.

**Conclusions.** Due to the variation in properties of most wood used for general construction, it would be difficult and hazardous to work up any general design equations or to give any recommended working loads based upon the results of these tests. Time did not permit the testing of an adequate number of specimens to get entirely satisfactory checks and no adjustment has been made for variations in quality of test materials.

The following is a statement of certain significant results of the tests together with some conclusions related to the outlined objectives of the study:

1. The glued joints gave the greatest strength of all joints tested with the load parallel to the grain of the timbers.

2. The strength of the glued joint loaded perpendicular to the grain is about 1/3 of the strength parallel to the grain.

3. The 4-inch split ring has the highest proportional limit load of all the metal fasteners.

4. The split rings have the greatest slip at proportional limit load.
5. There was no case in which the slip of the split rings was greater than .05 inch at the probable maximum working load.

6. In spite of its larger slip, the split ring appears to be the better ring for farm building construction. With equipment that is usually available, this ring has an advantage in the matter of labor and time required for fabrication.

7. A study of casein glue-connected rafters is justified by the results of these tests.

8. With the materials used in these tests, the glued joints made to a scale of 6" = 1'-0" have an ultimate strength about 35% of that of the full sized joints. For 2½-inch split-ring joints this proportion is about 50%.

9. In using warped timbers for glued construction, the concave surfaces should be placed together to secure maximum strength.

10. The relative ineffectiveness of nailed joints was clearly demonstrated by these tests.

**Preliminary designs of rafters**

**Introduction.** It was suggested in the introduction that improvement in rafter design might be achieved by more secure method of joining members; more efficient use of bracing material; and adequate anchorage of the rafter to the walls.
The tests of joints suggested the probable value of modern connectors and glue for joining rafter members. These might also be used in attaching the rafters to the wall to provide secure anchorage for the roof.

The bending moment diagrams of Figures 2 and 3 show that the maximum moment occurs in the lower member about $3\frac{1}{2}$ feet below the rafter splice. The type of brace used in conventional design comes just below this point of critical stress. This means that the rafter does not receive the stiffening and strengthening effect of the brace over the section where it is most in need of reinforcing. Furthermore, this brace extends a considerable distance along the upper member where it is needed least on account of the low bending moments prevailing there. These conditions suggest that the brace might be more effective if shifted downward to those parts of the rafter where the moments are largest.

The question then arises as to how far down the lower member the brace should extend. Also, how far along the upper member must the brace reach to provide adequate strength.

Method of testing. With a view to obtaining some indications of the answer to these questions, a number of models of proposed designs were made and tested. These test specimens are shown in Figure 20. The scale used was 1 inch equals 1 foot.
Rafter No. 1 consists of a 2 x 4 lower member and 2 x 6 upper member with a 2 x 4 brace. This brace extends from a point one foot above the support to a point in the upper member 3 feet from the rafter splice. This rafter was designed to use 2½-inch modern connectors of the split-ring type at all major joints. These connectors were to be used also to fasten the rafter to the wall studdings and to secure the upper members at the ridge. This method of anchorage and fastening at the ridge was chosen as offering the possibility of far greater strength than the conventional practice of toe nailing at these points.

Tests of this rafter suggested modifications which are included in No. 2. The 2 x 4 lower member was replaced by a 2 x 6. The 2 x 4 brace was shortened to reach only to a point 4 feet above the support in the lower member. Thus, the cost of the two rafters was kept about equal.

It was then felt that these designs might not provide sufficient bracing of the upper member, particularly for the end wind condition. Accordingly, rafter No. 3 was designed. Approximately the same amount of material was used but the brace was shifted upward closer to the conventional position. This brace was made in two parts, as shown, to keep it from projecting too far into the mow space.

The models of these rafters were tested with the
The Testing Apparatus

The Test Specimens

Figure 20. Preliminary Designs of Rafters
apparatus pictured in Figure 20. Loads to approximate the wind loading conditions as shown in Figures 2 and 3 were applied by means of heavy rubber bands. The model load for each point was calculated for wind velocities of 70, 100, 120 and 150 miles per hour to develop fibre stresses equal to those for the full size rafter. The rubber bands were then calibrated to find the length to which it was necessary to stretch each in order to exert the required force. These calibrations were later checked to verify their accuracy and test the dependability of this manner of load application. An additional check of this method was made during the tests. Certain wind loads were maintained for a period of an hour or more. A change in deflection of no more than one thousandth of an inch indicated that the bands sustained a reasonably constant load.

Measurements of deflection at the rafter splice were made by means of a Starrett Dial Test Indicator mounted as shown in Figure 20. A steel rule was used to read the other deflections which were taken.

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

Discussion of results. All of these rafters were designed to use split-ring connectors at all the main joints, at the supports and at the ridge. Small bolts were used in the models to approximate ring connector affect. The use of these joints
# TABLE I

Tests of Small Rafter Models  
Wind at 90°, (Windward Side)

<table>
<thead>
<tr>
<th>Number</th>
<th>Velocity</th>
<th>Splice Deflection</th>
<th>Deflection At Point 11</th>
<th>Deflection At Point 22</th>
<th>Deflection At Ridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 1</td>
<td>70</td>
<td>1.112</td>
<td>outward 0.05&quot;</td>
<td>outward 0.22&quot;</td>
<td>Up 0.25&quot;</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.078</td>
<td>inward 0.015&quot;</td>
<td>outward 0.23&quot;</td>
<td>Up 0.31&quot;</td>
</tr>
<tr>
<td>No. 2</td>
<td>70</td>
<td>0.125</td>
<td>outward 0.06&quot;</td>
<td>outward 0.25&quot;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.06</td>
<td>outward 0.19&quot;</td>
<td>Up 0.32&quot;</td>
<td></td>
</tr>
<tr>
<td>No. 3</td>
<td>70</td>
<td>0.035</td>
<td>inward 0.17&quot;</td>
<td>outward 0.31&quot;</td>
<td>Up 0.32&quot;</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td>In 0.25&quot;</td>
</tr>
</tbody>
</table>
TABLE II

Tests of Small Rafter Models
Wind in End of Barn

| Number: | ity | (inches) | (inches) | (lbs.) |
| 70 : | 0.104 | 0.12 | 449 |
| 100 : | 0.2 | 0.19 | 973 |
| 120 : | 0.33 | 0.36 | 1790 |
| 150 : | 0.62 | 0.53 | 2820 |
| 70 : | 0.088 | 0.09 | 470 |
| 100 : | 0.183 | 0.11 | 985 |
| 120 : | 0.34 | 0.31 | 1700 |
| 150 : | 0.62 | 0.5 | 2745 |
| 70 : | 0.08 | 0.09 | 492 |
| 100 : | 0.18 | 0.2 | 1000 |
| 120 : | 0.375 | 0.44 | 1455 |
| 150 : | 0.81 | 0.875 | 2460 |
involved lapping the members. This lapping of the members made the rafter unsymmetrical and raised the problem of its behavior under wind load. In these tests all of the rafters began to buckle at a load corresponding to a wind velocity slightly over 100 miles per hour. However, when the rafters were held straight as they would be under the restraint of the sheathing, the buckling was very slight even at a wind velocity of 150 miles per hour.

The deflections for rafters No. 1 and No. 2 were approximately the same for both wind conditions as shown by Tables I and II. The use of a 2 x 6 lower member in Rafter No. 2 appeared to be justified by reduced deflection in this member. The shortening of the brace had no apparent ill effects.

Rafter No. 3 did not give as good results as the first two. Deflections became much greater when the wind velocity exceeded 100 miles per hour.

Bending of the upper rafter members of No. 1 and No. 2 was rather too large in the end wind condition. This indicates that the brace should be attached farther from the rafter splice in order to give this member more support.

None of these rafters showed signs of failure at a wind velocity of 150 miles per hour. This provides a factor of safety of 4 in wind load and 2 in velocity over the maximum recorded velocity of 68 miles per hour.
Conclusions

1. The performance of Rafters No. 1 and No. 2 seems to justify this arrangement of braces.

2. Upper and lower rafter members should be of 2 x 6 stock.

3. The brace should extend from a point about 3 feet from the support of the lower member to a place near the midpoint of the upper member.

4. Lateral bracing of the rafters may be necessary to avoid excessive buckling under heavy wind loads.

5. Two struts should be used between the brace and lower member to reduce the bending of the latter.

Tests of rafters

Introduction. The fourth step in the experimental study was the design, construction and test of large models of barn rafters. Six different designs were tested. All followed closely the service requirements discussed earlier in this report. Four of the rafters were new designs. These involved modifications of the bracing used in the preliminary designs, combined with various forms of connecting devices. The remaining two rafters were conventional in design and method of construction.

The purposes of these rafter tests were; (1) to study the
behavior of each rafter to discover its points of strength and weakness; (2) to obtain a comparison between various conventional rafters and the proposed designs; and (3) to discover means of improving new or existing types of construction.

Selection of scale for models. The ideal procedure would be to build rafters to full scale to test. However, this is hardly feasible on account of the large and expensive equipment required for such a set-up. Therefore, a scale of 6" = 1'-0" was used in the construction of test specimens. The conventional system of using bolts, connectors and nails to half scale of those in the specifications was followed. That is, where ½ inch bolts and 2½ inch connectors are specified, the bolts and connectors used were ⅛ and ⅜ inch, respectively. Table III shows the full size and scale size of these materials used.

Determination of loads on rafters. Two classes of loads were applied to the rafters under test. The dead loads are constant and the ones calculated in the analysis of the roof shape were used. The magnitude of these was given in Figure 1.

The wind load varies as the square of the velocity so the loading at each point of application must be calculated for each succeeding increment of wind velocity. These loads were applied at the quarter points and ends of each member of the rafter and at right angles to the member. These loads are
## TABLE III

Sizes of Materials Used in Rafter Models

<table>
<thead>
<tr>
<th></th>
<th>Full Size</th>
<th>Scale Dimensions</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Diameter</td>
<td>Length</td>
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<tr>
<td>Nails, 16d</td>
<td>.161</td>
<td>3 1/2&quot;</td>
</tr>
<tr>
<td>Nails, 8d</td>
<td>.131&quot;</td>
<td>2 3/4&quot;</td>
</tr>
<tr>
<td>Nails, 6d</td>
<td>.113&quot;</td>
<td>2 &quot;</td>
</tr>
<tr>
<td>Bolts, 1/2 inch</td>
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<td></td>
</tr>
<tr>
<td>Connectors</td>
<td>2 1/2&quot;</td>
<td>1/2&quot;</td>
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<tr>
<td>Glued Joints</td>
<td>Approximate area = 80 sq. in.</td>
<td>Approximate area = 20 sq. in.</td>
</tr>
</tbody>
</table>
shown in Figures 2 and 3 for a wind velocity of 70 miles per hour. The loads apportioned to act on the lower members at the support were not applied in the tests since they do not produce any stress in the rafter.

In calculating the wind loads to be applied at each of the points on the rafter, the formula employed was \( P_w = c q = C \left( \frac{0.00189 \cdot (22)^2 \cdot V^2}{15} \right) \); where \( P_w \) is the pressure in pounds per square foot, \( c \) is the pressure coefficient at the point considered (see page 25), 0.00189 is the air density at 15°C and 760 mm. absolute pressure and \( V \) is the wind velocity in miles per hour. The value of \( C \) used for each load point was the average value for the length of rafter represented. These values were obtained from the pressure distribution curves given in Figures 2 and 3. The load applied therefore was \( W = P_w A \), where \( A \) is the roof area represented by the point.

In this way the load at each point of the full size rafter for each wind velocity was calculated for side and end wind conditions.

It is required to apply such loads to the model as will develop the same fibre stress as the given load would produce in the full size rafter. Since the stress in a beam varies as the third power of the depth, the bending moment produced in the model need be only \( 1/3 \) of that in the full size rafter. But the length is reduced one-half so the load required to
produce this bending moment is 1/4 of the full scale load. Accordingly all loads were divided by 4 to obtain the model load for each condition of wind velocity or direction. Each load was then reduced by the amount of the preceding load to give the increment of load necessary to represent the increase of wind velocity. These loads and increments with their derivation are given in Tables IV and V.

The test specimens. A detailed description of each rafter tested will be given in the discussion of results. Drawings of these rafters appear in Figure 21.

In the construction of these rafters Western Douglas Fir finish lumber was used. This lumber has an average ultimate compressive strength of about 9000 pounds per square inch which is considerably higher than for common timber which averages around 6000 pounds per square inch. This difference must be taken into account in the interpretation of the test results. This lumber was used because of its dependable quality and fine grain which are very important in models for test purposes.

The sizes of modern connectors, bolts, nails and glued surfaces used were given in Table III. These modern connectors were of the "Teco" split-ring type as pictured in Figure 8. The metal used to make the models was mild sheet steel and the method of construction was the same as for those rings
<table>
<thead>
<tr>
<th>Load Point Number</th>
<th>Lower Left (Windward) Member</th>
<th>Upper Left (Windward) Member</th>
<th>Upper Right (Leeward) Member</th>
<th>Lower Right (Leeward) Member</th>
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Table IV. Wind Loadings; Wind at 90° to Side Wall.
<table>
<thead>
<tr>
<th>Load Point Number</th>
<th>Lower Left Member</th>
<th>Upper Left Member</th>
<th>Upper Right Member</th>
<th>Lower Right Member</th>
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<td><strong>Area of P (m^2)</strong></td>
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</table>

**Table V. Wind Loadings; Wind in End of Barn.**
used in the tests of models of joints. The circular saw shown in Figure 14 was used to cut the grooves for these rings also.

One-quarter inch machine bolts were used in ring-connected and glued joints. Square steel plate washers of \( \frac{1}{2} \) the size specified by the manufacturers of the rings were used on all bolts.

The same "Gasco" brand of casein glue was used for the glued rafters as was used in the tests of joints.

**Apparatus and method of procedure.** The apparatus used in these rafter tests is shown in Figure 22. The loads were applied to the rafters by means of sand bags of the proper weight to give the required increments of load. These bags were carried in the wooden boxes or baskets which may be seen at both ends of the testing frame. The cables carrying each basket passed through special ball bearing sheaves to a set of eveners which distributed the load in the proper proportions to the various points on the rafter. In this way four loads were used to apply the required forces at 15 points on the rafter. This loading is a closer approximation to actual wind loading conditions than when loads are applied only at the center or ends of the rafter members. The location, proportions and direction of action of these load points and eveners are shown in Figure 23.
Test of Rafter No. 5; Loading for Wind at 90° to Side Wall.

Test of Rafter No. 5; Loading for Wind in End of Barn.

Figure 22. Rafter Testing Apparatus
Figure 23. Rafter Loading Diagrams
The friction loss in the cable and pulleys of this equipment, as determined by Clark (5), is about 1.5%. This is well within the percentage of experimental error and the wind loads are only estimates. Therefore, no correction for friction loss was made on the loadings applied.

The weight of the loading equipment such as the baskets and eveners was taken into account in calculating the loads required. The dead loads were applied at the ends of the members by hanging sandbags of the necessary weight at these points. The supports for the rafters were heavy stub timbers bolted securely in the frame to prevent any deflection at these points in the rafter. These supports are shown in Figure 22.

Deflections at the rafter splices and ridge were taken by a self-recording apparatus mounted on the rafter at these points. This device consisted of a pencil carried in a copper tube which was secured to the rafter by means of screws. This tube contained a compression coil spring whose purpose it was to hold the pencil against the sheet of graph paper which was mounted on the testing frame behind the rafter. In this way the progressive deflection of the rafter was traced out during the test. The position of the pencil point was marked for each increment of 10 miles per hour of wind velocity.

In the performance of each test a definite procedure was
followed. First the rafter was placed in the testing frame with supports and ridge secured in the manner required for construction practice. The eveners were then attached to provide the desired wind load condition and the deflection-recording equipment was put in place. Whenever lateral bracing of the rafters was used, these ties and braces were fastened before any load was applied.

When everything was in readiness to proceed, the positions of the pencils on the graph papers were marked. The loading for a 40 mile per hour wind was applied by putting the proper sandbags in the baskets of the apparatus. This wind load was arbitrarily chosen to start the test since a certain wind is required to overcome the effects of the dead loads and to take up any play in the apparatus.

When load "1" had been applied (corresponding to a 40 mile per hour wind load) a line was drawn on the deflection charts where the pencil stood and marked "1". Loads "2" and "3" (corresponding to 45 and 50 miles per hour) were put in the baskets and the deflection charts were then marked "3". Thus, the loading continued in regular order until failure of the rafter occurred or until it was desired for some reason to discontinue the test. The behavior of the rafter under test was carefully noted throughout and a record was made of all noticeable incidents during the run.
Results of rafter tests. The results obtained from these rafter tests are shown in Figures 24, 25 and 26, and Table VI. Figures 24 and 25 show the actual deflections at the rafter splices and ridge for the two wind directions. In Figure 26 these deflections are plotted against the square of the wind velocity. This method of plotting was used on account of the relation between wind velocity and pressure or load. Due to the nature of the wind loading, it is impossible to plot the deflection against the load. If the deflection were plotted against wind velocity, part of the curvature of the curves obtained would be due to the nature of the deflection and part would be due to the relation between velocity and load. This latter factor was eliminated by plotting against the square of the velocity and the curves so obtained are virtually load-deflection curves. The locations of the corresponding velocities were plotted to facilitate study of the graphs.

The results of the tests on each rafter will be discussed separately and then an analysis of all the tests will be made by a discussion of Table VI and the composite graphs of Figure 26. A brief description of each rafter will accompany the discussion to augment the drawings of Figure 21. In this discussion all references to dimensions are in terms of the full size rafters.
Figure 24. Recorded Deflections of Rafters; Wind at 90° to Side Wall.
Figure 25. Recorded Deflections of Rafters; Wind in End of Barn.
Figure 26. $V^2$-Deflection Curves for Rafters Tested.
Rafter No. 1

As indicated in Figure 21, this rafter used 2½-inch split-ring connectors at all joints. The same fasteners were used to secure the lower rafter to the studding and to join the upper members at the ridge.

The brace used was 16 feet long of 2 x 4 stock. It was attached to the upper member 5 feet from the rafter splice and to the lower member 3 feet from the point of its attachment to the studding.

Test in side wind. Rafter No. 1 gave very good results in this test. This rafter was loaded to the equivalent of 150 miles per hour and then unloaded in order to find how much it would recover. The amount of this recovery is shown in the graphs of Figure 26. The loads were then applied until the rafter failed at a loading corresponding to a wind velocity of 245 miles per hour. The lower member on the windward side broke in bending at a point slightly above the point of attachment of the brace. This failure is shown in Figure 27. The leeward rafter began to buckle badly after a wind velocity of about 190 miles per hour had been reached.

Test in end wind. This rafter was tested twice in end wind and the deflection charts are shown in Figure 25. For the first run the rafter was braced laterally to simulate the restraint of the sheathing but the right rafter buckled
Failure of Lower Windward Member
Under Side-Wind Loading.

Failure of Upper Left Member
Under End-Wind Loading.

Figure 27. Failures of Rafter No. 1
excessively and the test was discontinued at a velocity of 120 miles per hour. For the second run a new rafter was constructed and additional lateral bracing was used as shown in Figure 21. Failure occurred at 185 miles per hour by breaking of the upper left member between the brace and the ridge. This failure is pictured in Figure 27.

**Rafter No. 2**

In this design an attempt was made to eliminate some of the lack of symmetry of Rafter No. 1. The upper and lower members were of laminated construction; made of two 1 x 6's nailed together and joined at the rafter splice as shown in Figure 21. A 1 x 6 brace was used at the joint to increase its strength.

**Test in side wind.** In test this rafter buckled excessively due to its laminated type of construction. At a wind velocity of 160 miles per hour it was found that the lateral bracing used was carrying part of the wind load. When these were loosened the rafter deflected as shown in Figure 24. Failure occurred at a wind velocity of 200 miles per hour by breaking of one lamination of the upper leeward member at the point of attachment of the brace. Further testing of this rafter was considered inadvisable since its performance was inferior to that of Rafter No. 1 and its construction is more costly.
Rafter No. 3.

This rafter is shown in Figure 21. It was of a type similar to those recommended by the Midwest Plan Service, an example of conventional design and good construction.

Test in side wind. The windward rafter began to lift off the plate at a wind velocity of 80 miles per hour. At 130 miles per hour the rafter had lifted \( \frac{1}{2} \) inch accompanied by splitting of the brace at the plate and the 1 x 8 pieces at the leeward rafter splice had begun to split. Ultimate failure occurred at 175 miles per hour by splitting of the brace and rafter at the windward plate.

Test in end wind. Under end wind loading the weakness of conventional methods of joining members was revealed very clearly. Both rafter splices had begun to fail at a wind velocity of 110 miles per hour. The nature of this failure is shown in Figure 28. The ultimate failure of the rafter occurred at the ridge under a wind velocity of 140 miles per hour. Failure was due to pulling of the nails in the collar beam and at the ridge.

Rafter No. 4

This rafter was the same as No. 3 except that no brace was used at the plate. Five 16d nails were used to toe-nail the foot of the rafter to the plate.

Test in side wind. The first failure of this rafter
Failure of Rafter No. 5 Under End-Wind Loading.

Failure of Rafter Splice of Rafter No. 3 Under End-Wind Loading.

Figure 28. Failures of Rafters
occurred at a velocity of 100 miles per hour when the 1 x 8 pieces of the leeward rafter splice began to split. At 120 miles per hour the windward plate began to lift at the outer edge and at 125 miles per hour the plate pulled off the studding completely. The nature of this failure showed that the rafter was held rather securely to the plate and it was felt that the use of five nails at this point was hardly representative of common practice.

**Rafter No. 5**

This rafter was identical to No. 4 but three 16d nails instead of five were used to toe-nail the rafter to the plate.

**Test in side wind.** This rafter failed at a wind velocity of 110 miles per hour by pulling of the toe nails holding the rafter to the windward plate. Deflections to this point were about the same as for No. 3 and No. 4.

**Test in end wind.** In the test of this rafter for the end wind condition, the rafter began to lift from the plates at a wind velocity of 65 miles per hour. The complete separation of rafter and plate at 75 miles per hour is shown in Figure 28.

**Rafter No. 6**

In this rafter the split-ring connectors of No. 1 were replaced by casein glue. Otherwise the design is identical. As shown in Figure 21 bolts were used at all joints except in the struts between the lower member and the brace.
Test in side wind. In this test the rafter was provided with sufficient lateral bracing to prevent excessive buckling even at high wind loads. Loading was continued until failure of the glue joint between the windward rafter and the studding. Shearing of the glue occurred there at a wind velocity of 235 miles per hour.

Rafter No. 7

Rafter No. 7 resembles the conventional design in the matter of bracing and differs but slightly in the construction of the rafter splice. Casein glue and nails were used in all joints except at the studdings and ridge where the construction of Rafter No. 6 was used. The weakness of the usual type of rafter splice was revealed in the foregoing tests. In No. 7 the usual 1 x 8 splice plates were replaced by longer pieces of the same dimension placed parallel to the main brace as shown in Figure 21.

Test in side wind. The change in the rafter splice was justified by the results obtained. Failure of the glued joint at the studding occurred at a wind velocity of 180 miles per hour. The rafter splices showed no signs of failure at this load in contrast to splitting of the conventional rafter splice at velocities as low as 100 miles per hour.

Discussion of results. In a comparison of these rafters tested, there are three factors to be born in mind. These are
ultimate breaking strength, rigidity, and economy of construction.

The comparative strength of the rafters tested is shown in Table VI. The only rafter which even approached No. 1 in ultimate strength was No. 6. This seems to indicate some degree of superiority of the bracing used in these rafters. Moreover, the tests of No. 6 and No. 7 support this new type of bracing. These two rafters differed only in the bracing and splice used. During the test the splices of neither showed any signs of failure and both failed by shear of the glue at the support. However, No. 6 failed at 235 miles per hour and No. 7 at 180. During the test of No. 7 it was noted that the lower rafter member bent a great deal more than that of No. 6, thus producing a larger stress in the glue at the studding. This excessive bending appeared to be due to the inferior support given by the conventional type of brace.

The type of failure of Rafter No. 1 is worthy of particular note. This was the only rafter in which the joints carried sufficient stress to cause bending failure of the members.

The very low ultimate strength of the conventional rafter with no brace at the plate (No. 5) is significant in view of the fact that this type of construction is in common use.

In drawing conclusions from these test results as to the
### TABLE VI

**Comparative Strength of Rafters**

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<tr>
<th>Rafter No.</th>
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<th>Wind in End of Barn</th>
<th>Veloc.</th>
<th>Ultimate Strength Based Upon</th>
<th>Veloc.</th>
<th>Ultimate Strength Based Upon</th>
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### TABLE VII

**Cost of Materials for Rafters**

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probable performance of full size rafters, certain adjustments would have to be made. In the selection of the scale for the models, the assumption was made that the model connectors and glued joints would carry one-quarter the load of the full size specimens. The tests of joints showed that this assumption was not true. However, this variation from the assumed relation is thought to be due mainly to the quality of the wood in the specimens. Therefore, the nature of the failures would probably be the same for a full size rafter since the strength in bending is correspondingly higher for the wood of the models. However, the ultimate wind load for the full size rafter would probably be lower and the amount of this difference would have to be estimated upon the basis of the quality of timber used.

A study of the deflection curves of Figure 26 reveals the following general relationships:

1. With the wind at 90° to the side wall, the deflections of the conventional rafters were from 3 to 4 times the deflections of Rafters Nos. 1, 6 and 7.

2. Rafter No. 6 shows the least deflection of all rafters tested in side wind. No. 1 is very nearly as rigid.

3. Under the end-wind condition the deflections of Rafter No. 1 were from 2 to 3 times as great as the deflections of the conventional design, No. 3.
4. The deflections of Rafters Nos. 1 and 5 were about equal under end wind loading.

5. The deflection of the rafter splices of Rafter No. 3 increased rapidly after failure of these joints began. Time did not permit the testing of Rafters No. 6 and No. 7 under the end wind loading.

A comparison of the cost of materials for the rafters tested is given in Table VII. Estimates of the labor required for fabrication were made upon the basis of the various operations involved. These estimates indicate labor charges that are very nearly equal for the five rafters.

Rafter No. 1 contains less timber than any of the other rafters of different design but the cost is 30% greater than that of No. 3. This discrepancy in cost will be narrowed by the probable reduction in price of the split rings contingent upon the expansion of their market.

Future investigations of the strength and stiffness of roof sheathing may justify using a larger spacing of rafters. This would favor the No. 1 type of rafter and make possible a more efficient use of its superior strength.

The design of Rafters Nos. 1 and 6 fails to meet the established requirements concerning the use of standard length members. However, this objection may be overcome by decreasing the center-to-center length of the members to make possible the
use of standard lengths. Moreover, even when using the lengths given, the "waste" may be used to good advantage in the struts between the lower member and the brace.

Conclusions. The following are a number of significant conclusions which appear to be justified by the observations and results of these rafter tests:

1. The system of bracing used in Rafters No. 1 and No. 6 appears to be superior to the conventional method.

2. None of the joints in conventional rafters of the type tested are strong enough to carry the ultimate load of the gross cross-sectional area of the members.

3. The use of split rings appears to be the most effective method of joining rafter members and of securing anchorage to the wall.

4. Observations and results of these tests indicate that the 2\(\frac{1}{2}\)-inch split ring is entirely suitable for braced rafter construction.

5. The results of these tests justify and call for further investigation into the use of casein glue in farm building construction.

6. The large deflection of Rafter No. 1 under end-wind loading is not thought to be a serious characteristic on account of the very rare occurrence of this condition.

7. Upon the basis of Iowa's maximum recorded wind
velocity of 68 miles per hour, Rafter No. 1 has a factor of safety of 3.6 in velocity and 13 in load for the side wind condition. In end wind these are reduced to 2.72 and 7.4 respectively.

8. The corresponding factors of safety of the conventional Rafter No. 5 are 1.57 and 2.63 for the side wind, and 1.1 and 1.22 in end wind.

9. The investigation in rafter spacing should be continued.
SUMMARY

1. This study of the braced rafter roof is justified by; (a) the data on wind damage; and (b) the popularity and acceptability of this type of rafter.

2. The objects of the study were to investigate the strength requirements of the braced rafter roof and to develop a more wind resistant construction.

3. The service, structural, economic, and aesthetic requirements of the barn roof were discussed.

4. The size and shape of barn shell to be considered in this study were established.

5. Wind pressures on the gambrel roof were investigated and the distribution to be used in this study was selected.

6. The reactions and bending moments in a roof of the shape selected were determined for a wind velocity of 70 miles per hour.

7. The problem of the effect of roof sheathing upon rafter spacing was analyzed.

8. The strength relations of glued, ring-connected, bolted and nailed joints were investigated.

9. Rafter models of new and conventional designs were tested to obtain a comparison of their performance and to locate the weak and strong points of each design.
CONCLUSIONS

1. Fourteen foot lower and twelve foot upper rafter members are desirable for a 34 foot gambrel roof span. The lower members should be inclined 62° and the upper members 31° to the horizontal. The roof shape so established has no tendency to sag at the ridge under customary dead loads.

2. Tests of joints and rafters indicate the probability of extensive use of casein glue and modern connectors in farm building construction.

3. The material in the braces of conventional designs of braced rafters is not being used to the best advantage.

4. The system of bracing developed during this study appears to be more effective than the conventional method.

5. Rafters in which casein glue or modern connectors were used with the new system of bracing showed ultimate strength from 2 to 5 times the strength of various conventional designs.
LITERATURE CITED


ACKNOWLEDGMENT

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