EXPERIMENTAL POLE-TYPE STRUCTURE: initial evaluation

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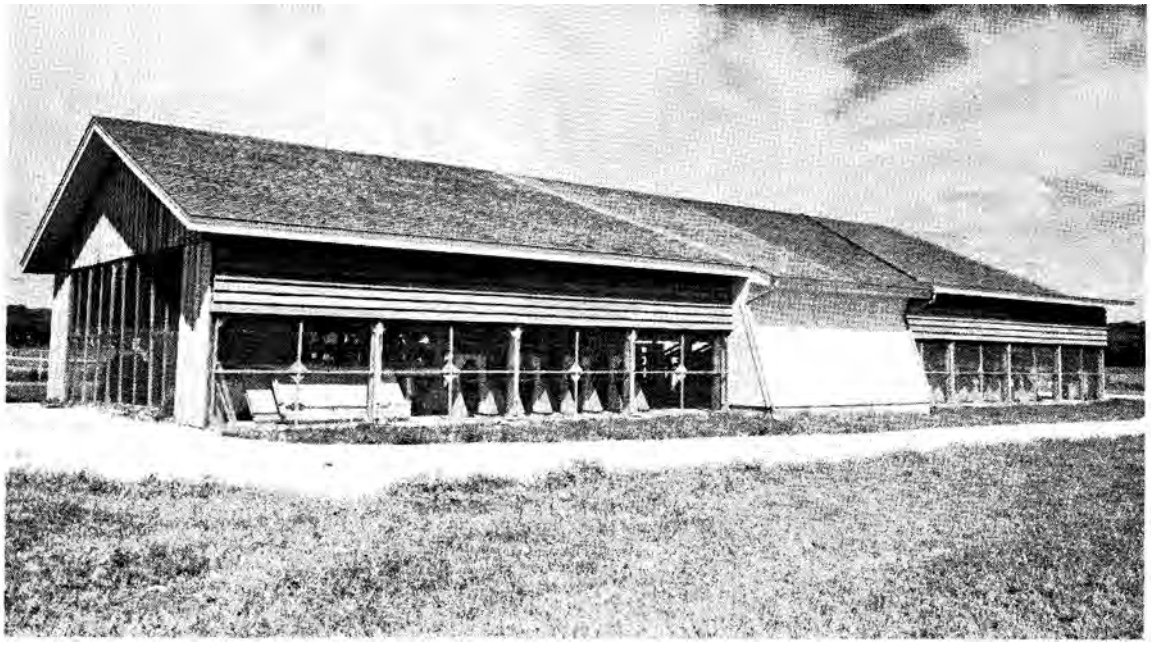

ABSTRACT

These are the results of an initial evaluation on a pole-type building constructed at the Forest Products Laboratory. The objectives of this long-term study are threefold: To appraise the rigidity contributed by the constituent parts of a 36-foot, clear-span, pole-type building; to evaluate the strength and stiffness of the trussed rafters; and to investigate the performance of protective decorative finishes and fire-retardant treatments on wood panels and shingles.

Results of testing during the erection of the building showed that the addition of the constituent parts increased building rigidity. The initial tests of trussed rafters gave center deflections of 0.22 to 0.56 inch under a design load of 140 pounds per lineal foot of truss.

After 2 years of exposure, one of two types of protective decorative finishes sprayed on wood shingles and siding retarded mildew and the normal graying of wood. Results of tests on fire-retardant-treated decks showed the treatments to be somewhat effective but not equal in effectiveness to protection afforded by a full-cell pressure-type treatment.

Subsequent observations and tests to evaluate the performance and durability of the various building components, protective finishes, and fire retardants will provide data on the practicableness and the limitations of the various building parts and treatments under long-term service conditions.

EXPERIMENTAL POLE-TYPE STRUCTURE:[1] initial evaluation

By D. V. DOYLE[2] Engineer

Forest Products Laboratory[3]
Forest Service
US. Department of Agriculture

INTRODUCTION

A full-size, clear-span building was constructed in the Building Research Park at the Forest Products Laboratory. Construction began in the fall of 1965 and was completed in the fall of 1966. The building, planned to provide shelter for experimental research studies including long-term load tests of wood trussed rafters, also afforded an opportunity to include a number of research investigations of its design. These included evaluations of (1) the rigidity contributed to the building by the constituent parts as they were added to the building, (2) the relative stiffness of the trussed rafters constructed of different wood species with different types of joint fastenings, and (3) the performance of protective decorative finishes and fire-retardant treatments on wood shingles and siding. Performance is evaluated in terms of loads and conditions that normallywould he associated with this type of building. Additional observations and tests will be scheduled from time to time to evaluate the performance and durability of the various building components.

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[1] Information presented at the Winter Meeting of the American Society of Agricultural Engineers, Chicago, Ill., December 10-13, 1968.

[2] Acknowledgment is gratefully extended to the following staff members of the Forest Products Laboratory: L. O. Anderson for the design and supervision of construction of the building; T. L. Wilkinson for assistance in testing the trussed rafters; J. M. Black for development of the finishes; H. W. Eickner for development of the fire-retardant treatments; E. W. Kuenzi for the mathematical analysis of pole deflections; and to others of the staff who assisted with construction of the building and the investigations.

[3] Maintained at Madison, Wis., in cooperation with the University of Wisconsin.

DESCRIPTION OF BUILDING

The building, a clear-span type, is 36 by 100 feet (outside to outside), as shown in figure 1. It is an open pole-type structure except for the center 20 feet of length that is framed with slant-leg rigid frames; these widen the span to 44 feet at the base. The rigid frame section is enclosed on the outside walls. The pole-type sections of the building extend 40 feet each side of the rigid frame section and consist of woodtrussed rafters supported by preservative-treated southern pine poles. The architectural drawings of elevation and plan views of the building, as well as construction details, are given in the Appendix.

The poles average about 7.4 inches in top diameter and are spaced 10 feet on center. They are embedded 5 feet in clay soil and extend 10 feet 10 inches above the finished floor. They are slabbed on their outer face to provide a continuous flat surface for the attachment of sheathing or framing, as well as to facilitate the alining and the plumbing of the poles. The slabbing was performed in a sawmill and consisted of a minimum saw cut from groundline to tip of the intermediate poles and two saw cuts at right angles to each other from groundline to tip of the corner poles.

Two 2- by 12-inch Constructiongrade Douglas-fir members (headers) were placed along each long side of the building to support the trussed rafters. The members are bolted to the tops of the poles, one on each side, with two 1/2-inch bolts.

The roof framing consists of 36-foot W-type wood trussed rafters of 5 in 12 slope, spaced 4 feet on center. The trussed rafters are anchored to the headers with a special light-gage metal fastener. Their configuration and location in the building are shown in figure 2. They are designed for a vertical load of 140 pounds per lineal foot on the upper chord and are constructed of nominal 2-inch dimension Construction grade Douglas-fir, No. 1 southern pine, No. 1 (nonstress-rated) oak, and No. 1 (nonstressed-rated) aspen. The materials of construction and stiffness data for each trussed rafter are listed in table 1.

The upper chords of all except the aspen trussed rafters are 2 by 8's and the lower chords and web members are 2 by 4's. Double 2 by 8's and 2 by 4's, nail-glued together, were used for the aspen trussed rafters. The upper chords of the Douglas-fir and southern pine trussed rafters are continuous, and the lower chords are spliced near their midpoint. The upper chords of the oak and aspen trussed rafters are spliced near their midpoint, and the lower chords have two splices.

The gussets and splice plates are of 1/2-inch A-C grade exterior-type plywood and 1/4-inch tempered hardboard. They are fastened to the

Figure I.--Experimental pole and rigid-frame 36- by 100-foot building constructed on the U.S. Forest Products Laboratory grounds. After rigidity tests were completed, side panels and screens were added to the building. (The end of the building, at left, faces south.) (M 135 255)

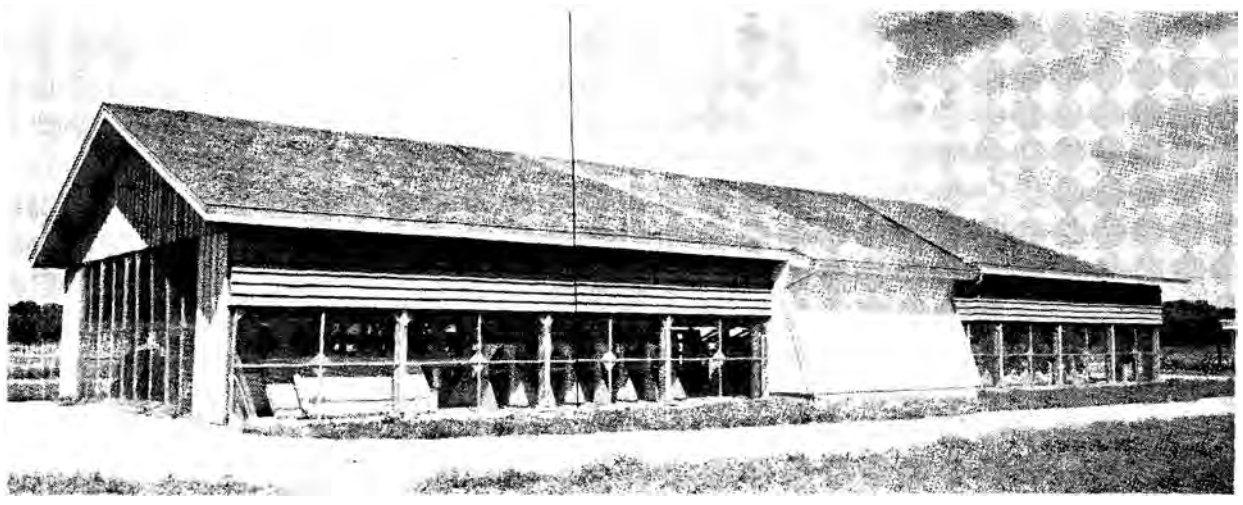


Table 1.--Construction materials and stiffness values for trussed rafters used in a pole-frame building

Truss No.	Species of members	Gusset material	Connection system	Mid-span deflection at design load[1]	Span deflection ratio[2]	Residual deflection after first loading	Maximum load
				In.		In.	Lb.
10	Southern pine	1/2-inch plywood	Sixpenny common nails	0.53	804	0.07	.....
[3]21		1/2-inch plywood	Sixpenny common nails	.36	1,183	.05	.....
12		1/2-inch plywood	1/4-inch bolts	.47	906	.09	.....
23		1/2-inch plywood	Phenol-resorcinol glue	.36	1,183	.03	.....
24		1/2-inch plywood	Casein glue	.39	1,092	.02	.....
11		1/4-inch tempered hardboard	Sixpenny common nails	.56	761	.13	.....
22		1/4-inch tempered hardboard	Phenol-resorcinol glue	.40	1,065	.01	.....
1	Douglas-fir	1/2-inch plywood	Sixpenny common nails	.42	1,014	.07	.....
[3]6		1/2-inch plywood	Sixpenny common nails	.44	968	.05	.....
17		1/2-inch plywood	Square serrated nails	.37	1,151	.06	.....
16		1/2-inch plywood	Annular-grooved nails	.41	1,039	.11	.....
2		1/2-inch plywood	1/4-inch bolts	.46	926	.05	.....
3		1/2-inch plywood	Phenol-resorcinol glue	.46	926	.01	.....
7		1/2-inch plywood	Casein glue	.47	906	.01	.....
9		1/4-inch tempered hardboard	Sixpenny common nails	.50	852	.10	.....
8		1/4-inch tempered hardboard	Phenol-resorcinol glue	.43	991	.00	.....
20	Red oak	1/2-inch plywood	Sixpenny common nails	.42	1,014	.10	.....
19		1/2-inch plywood	Sixpenny common nails	.44	968	.09	[4]16,080
18		1/2-inch plywood	1/4-inch bolts	.45	947	.10	.....
15		1/2-inch plywood	1/4-inch bolts	.40	1,065	.07	16,150
14		1/2-inch plywood	Phenol-resorcinol glue	.42	1,014	.00	.....
13		1/2-inch plywood	Phenol-resorcinol glue	.41	1,039	.03	14,950
26	Aspen	1/2-inch plywood	Phenol-resorcinol glue	.22	1,936	.01	.....
25		1/2-inch plywood	Phenol-resorcinol glue	.22	1,936	.03	12,790

[1]Deflection values are average values obtained from second and third loading of trussed rafters to the design load of 5,000 lb.

[2]Tested over a 35-ft. 6-in. span.

[3]Installed at gabled ends of the pole-type building.

[4]Upper chord buckled at this load without breaking member.

truss members with either (1) sixpenny common nails, (2) 1-1/2-inch-long square serrated nails, (3) 1-1/2-inch-long annular-grooved nails, (4) 1/4-inch-diameter bolts, (5) phenol-resorcinol glue, or (6) casein glue (table 1).

The legs and rafters of each of the rigid frames consist of pairs of 2- by 12-inch Douglas-fir or southern pine dimension lumber pieces glued and nailed with phenol-resorcinol glue and sixteen-penny common nails. The ridge and haunch connections are plywood gusset plates of 3/4-inch A-C grade exterior-type Douglas-fir nail-glued to the outer faces of the double members. The frames are spaced 4 feet on center; they are fastened to a wood plate anchored to a concrete foundation. The sloping legs are sheathed on their outer edges with 3/4-inch paper-overlaid plywood.

The entire roof is sheathed with 3/4-inch-thick 4- by 8-foot sheets of A-C grade exterior-type Douglas-fir plywood. Two metal clips equally spaced between the trussed rafters and rigid frames join the edges of the adjoining plywood pieces. The plywood is nailed to the trussed rafters with eightpenny nails spaced 5 to 7 inches on center.

Cedar shingles, 16 inches in length and laid 5 inches to the weather, are nailed to the plywood sheathing with threepenny galvanized nails. The roof was divided on each side of the ridge into five sections. The shingles in four of the sections each received a particular type of protective decorative treatment or fire-retardant treatment (fig. 2).

A 6-inch-thick reinforced concrete floor slab forms the floor of the structure. It was recessed about 9 inches from the wood poles (Appendix, fig. A2).

Building Rigidity

The rigidity contributed by the constituent parts of the pole-type section of the building was determined at various stages of construction, as listed in table 2. In all, eight independent tests identified "a through h" and representing six stages of construction were made by laterally loading six poles, three on each side, at the south end of the building. Figures 3, 4, and 5 depict the construction stages when tests b, c, and e were conducted.

Rigidity determinations made with headers, trussed rafters, roof sheathing, soffit, trim, and gable-end paneling in place on a portion of the building (d) were repeated when the ground was

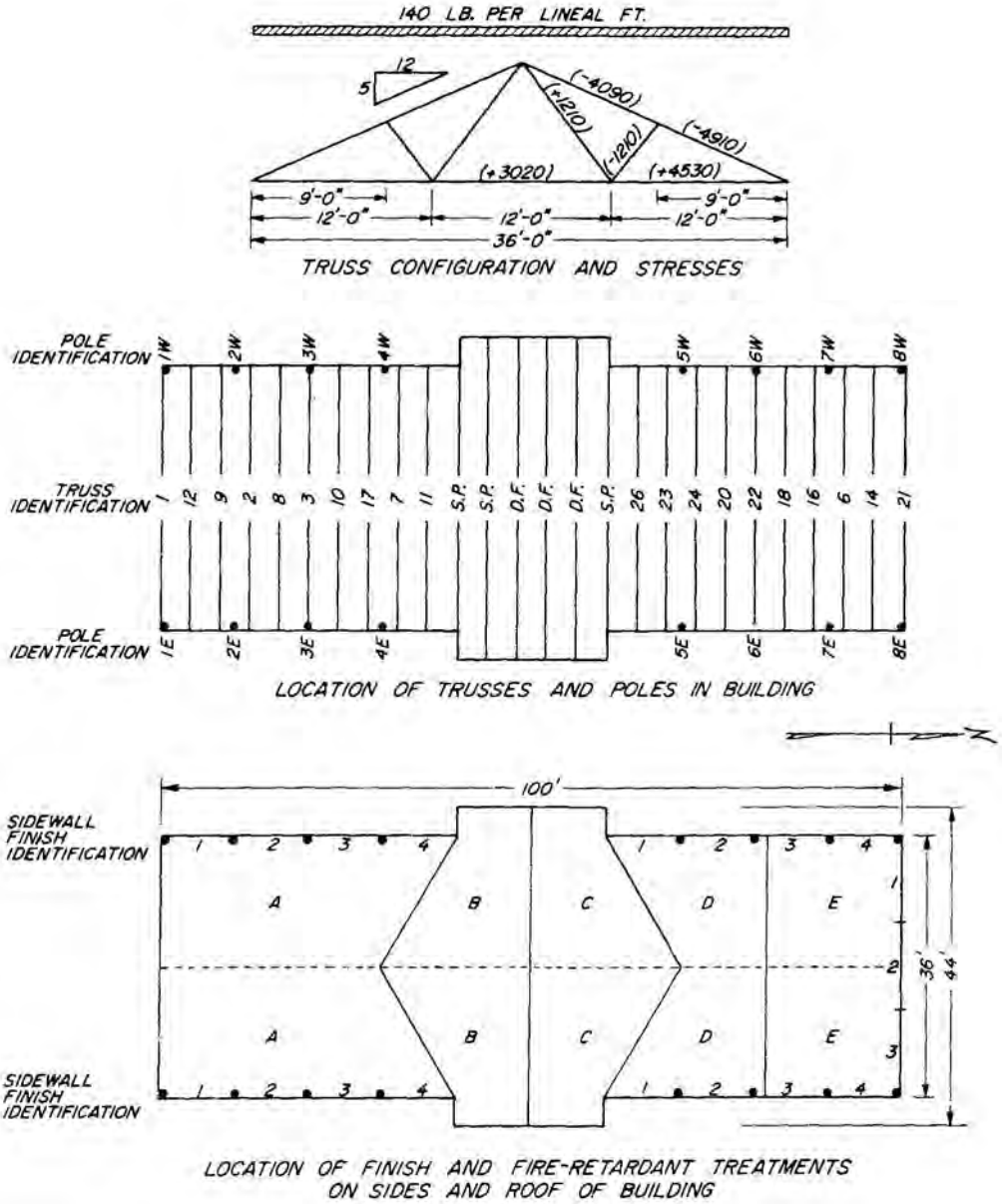


Figure 2.--Schematic details of trusses and building layout. (M 135 612)

frozen to a depth of about 30 inches (e) and again when the ground was not frozen and the entire building has reached this same state of construction (f).

Load-deflection tests were conducted by applying a load near the top of each of three poles (one at a time) along each side at the south end of the building. The load was applied through a steel cable attached near the top of the pole (9 ft. above the floor) and to a load cell and screw-type loading mechanism (fig. 6) anchored to the floor at the opposite side of the building. The deflection of the pole was measured by reading the location of the crosshair of an engineer's transit on a scale attached to the pole at a point 9 feet above the floor. Permanent reference points were used in the setting of the transit and the scales for each series of tests. Load was measured at deflection intervals of 0.1 inch.

The loading at each interval began with the corner pole and progressed to the second and third pole. Poles on the west side of the building were loaded first: those on the east side were loaded about 24 hours later. In the tests with the trusses

Table 2.--Test sequence for rigidity contributed by pole-building parts

Test identification	Date	Constituent parts affixed to building
a	Nov. 3, 1965	Poles
b	Nov. 5, 1965	Headers
c	Nov. 22, 1965	Trusses
d	Dec. 3, 1965	Roof sheathing, soffit, trim, and gable-end paneling (only south end of building erected - ground not frozen)
e	Mar. 8, 1966	Same parts as for "d" (ground frozen)
f	May 10, 1966	Same parts as for "d" (entire building erected - ground not frozen)
g	June 7, 1966	Shear panels in end wall
h	Oct. 17, 1966	Shingles

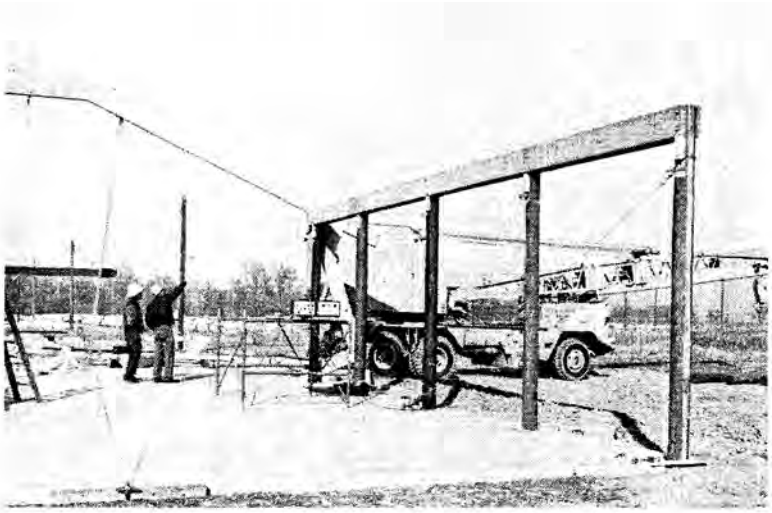


Figure 3.--Poles with headers attached; stage of construction for test b. Corner pole is being loaded parallel to the long axis of the building.

(M 129 994)

Figure 4.--Building with trussed rafters in place; stage of construction for test c. The poles are being loaded perpendicular to the long axis of the building. (M 129 997)

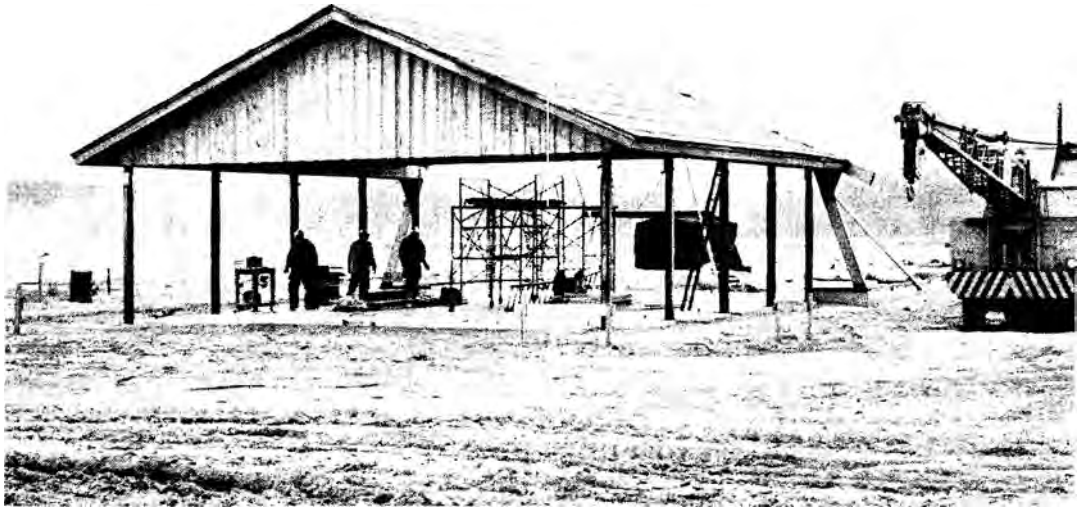


Figure 5.--Building with roof sheathing and gable-end paneling in place; stage of construction for tests d and e. (M 132 135)

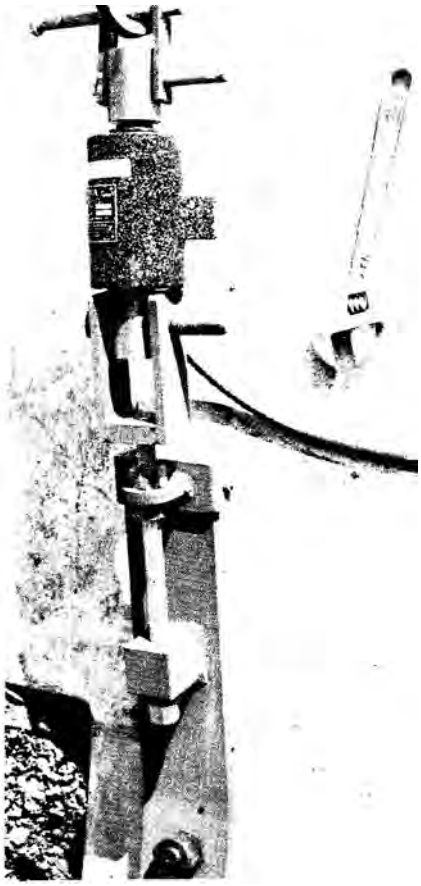


Figure 6.--Load-cell and screw-type loading mechanism used in the building tests. (M 129 995)

in place ("c" through "h") the loading of the poles on one side of the building caused the poles on the opposite side to deflect and move in the earth. The deflection recorded for the poles on the east side of the building therefore included any pole displacement that may have resulted from the prior loading of the poles on the west side. No attempt was made to retamp the earth around the poles after each loading. Alapse of time, however, between some of the tests was sufficient for the voids around the poles to be refilled by the wind and the rain. In most instances, the loading of the poles was stopped at about 4,000 pounds or when the deflection had reached about 1 inch for each pole.

Tests were also made at a few of the stages of construction by loading each of the two corner poles in a direction parallel to the length of the building (fig. 3).

Load deflection data obtained during the erection of the building are given in table 3 and are shown by the bar chart in figure 7. The deflections of the poles resulting from concentrated test loads applied 9 feet above the floor are listed in table 3 for horizontally applied loads of 250, 500, and 1,000 pounds. For the corner poles loaded parallel to the long axis of the building, the deflections are for horizontally applied loads of 450, 900, and 1,730 pounds. The test loads correspond to loads resulting from wind at velocities of about

Table 3.--Deflection near top of poles at loads equivalent to approximate 30-, 42-, and 60-mile-per-hour winds[1]

Test identification	Deflection with--									
	Load perpendicular to building axis for pole No.--							Load parallel to building axis for pole No.--		
	1W	2W	3W	1E	2E	3E	Average	1W	1E	Average
	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.
	LOADS EQUAL TO 30-MILE-PER-HOUR WIND									
	(250 pounds)							(450 pounds)		
a	0.67	1.23	1.10	0.97	1.11	0.73	0.97	1.35	1.66	1.50
b	.61	.74	.77	.88	.73	.62	.72	.16	.14	.15
c	.27	.43	.20	.61	.77	.34	.44	.....	.....	.....
d	.....	.....	.....	.26	.28	.15	.23	.....	.....	.....
e	.13	.03	.13	.05	.12	.06	.09	.....	.....	.....
f	.10	.47	.17	.09	.12	.07	.17	.12	.07	.09
g	.02	.10	.06	.03	.27	.09	.09	.....	.....	.....
h	.02	.09	.06	.02	.06	.03	.05	.02	.02	.02
	LOADS EQUAL TO 42-MILE-PER-HOUR WIND									
	(500 pounds)							(900 pounds)		
a	1.50	.....	.....	.....	.....	.....	.....	.....	.....	.....
b	1.30	1.70	1.53	1.83	1.51	1.39	1.54	.35	.37	.36
c	.80	.90	.44	1.18	1.40	.56	.88	.....	.....	.....
d	.....	.....	.....	.67	.57	.25	.50	.....	.....	.....
e	.24	.07	.30	.09	.23	.11	.17	.....	.....	.....
f	.30	.61	.29	.37	.44	.14	.36	.23	.13	.18
g	.03	.25	.11	.06	.60	.17	.20	.....	.....	.....
h	.03	.16	.12	.04	.19	.06	.10	.05	.05	.05
	LOADS EQUAL TO 60-MILE-PER-HOUR WIND									
	(1,000 pounds)							1,730 pounds)		
a	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
b	.....	.....	.....	.....	.....	.....	.....	.93	.93	.93
c	.....	.....	1.11	.....	.....	1.17	.....	.....	.....	.....
d	.....	.....	.....	1.93	1.65	.78	1.42	.....	.....	.....
e	.48	.38	.....	.53	.46	.20	.41	.....	.....	.....
f	.62	.....	.51	.97	1.17	.28	.71	.40	.25	.32
g	.07	.....	.23	.12	.86	.28	.31	.....	.....	.....
h	.06	.37	.23	.08	.53	.14	.23	.08	.09	.08

[1]The load was applied and the deflections were read at a point 9 ft. above the floor.

30, 42, and 60 miles per hour.[4] These wind loads are for a building with three sides enclosed and the windward side open.

The magnitude of the load that caused a deflection of 1/2 inch near the tops of the poles at the different stages of construction is illustrated in figure 7. In some of the tests on the completed building (h), the loading was stopped at a deflection of about 0.4 inch, and the load-deflection curves were extrapolated for the data at 1/2-inch deflec-

[4] Computed on the basis of the information presented in R 288.1, "Designing Buildings To Resist Snow and Wind Loads," 1968 Agricultural Engineers Yearbook, American Society of Agricultural Engineers, St. Joseph, Mich.

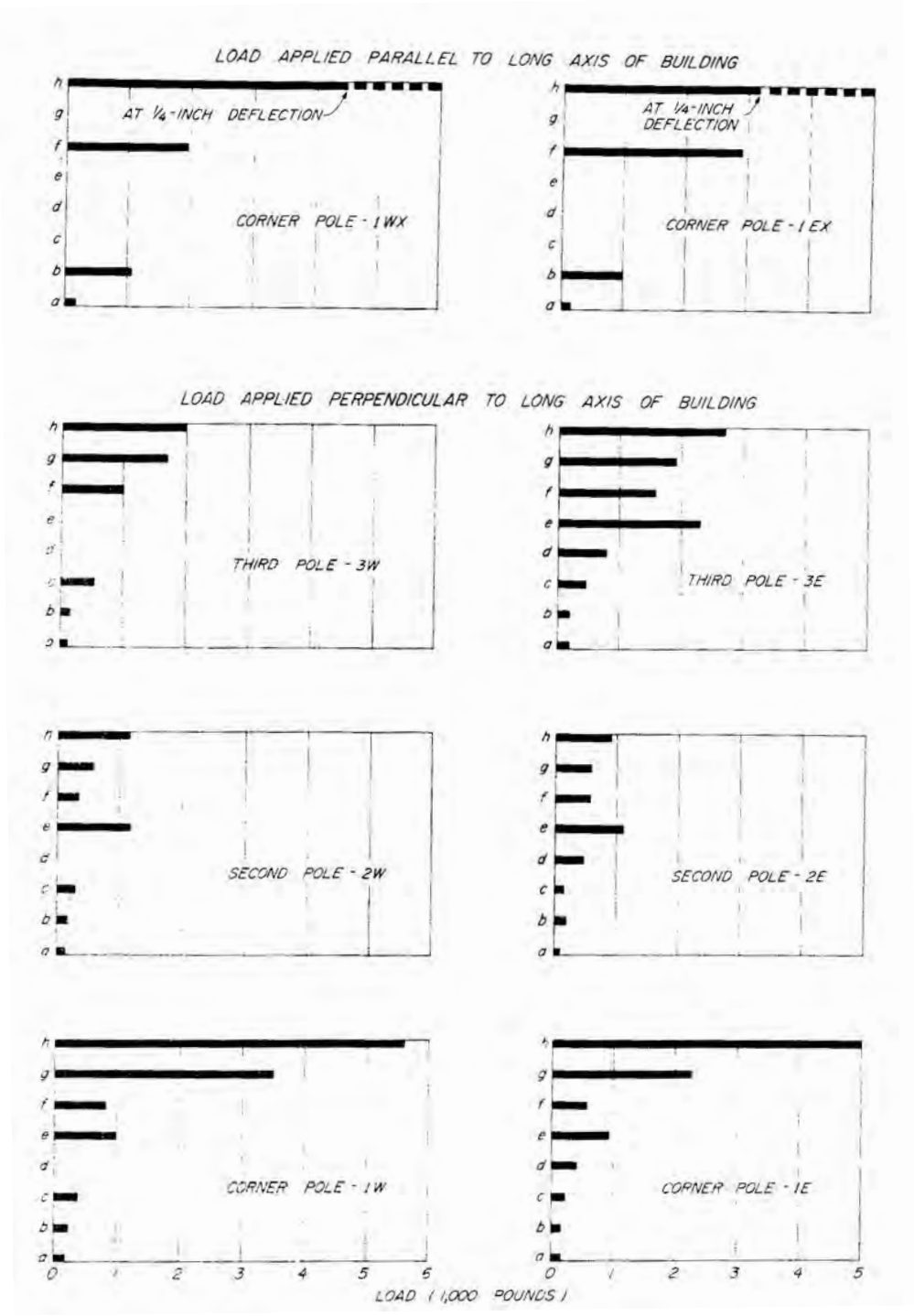


Figure 7.--Chart showing the loads required to deflect the poles 1/2 inch at the different stages of building erection. (Erection stages a through h are defined in table 2.)

(M 135 510)

tion shown in figure 7. The load at 1/4-inch deflection is shown for the corner poles loaded parallel to the long axis of the building (h).

The experimental data show that the rigidity of the building increased at each stage of construction. In the completed building (h), the deflection near the tops of the poles ranged from 0.06 to 0.53 inch for a horizontal load of 1,000 pounds (60 m.p.h. wind).

In the initial tests, only, the individual poles were loaded, whereas in subsequent tests the purlins and trusses transmitted some of the load to the adjacent and opposite poles. During the loading of the poles, particularly in the earlier tests, the poles moved at the groundline. An approximation of the amount of movement of the pole in the ground may be noted in table 3 and figure 7 by comparing the deflection obtained at the same stages of construction when the ground was frozen (e) and unfrozen (d) For loads up to 1,000 pounds (60 m.p.h. wind), the deflection of the poles in frozen ground was roughly one-half that of the poles in unfrozen ground. The rigidity obtained from the tests conducted when the building was partially erected (d) was somewhat less than that obtained from tests conducted when the entire building had reached the same stage of construction (f).

The attachment of a 4- by 10-foot by 5/8-inclu plywood panel at each corner at the front end of the building (g) substantially improved the lateral rigidity of the building, particularly at the corner poles. The panels, in addition to being fastened to the corner poles, were attached to the end roof truss and to the concrete floor. Rough boards and battens were nailed to the plywood (fig. 1). Shingles also added to the rigidity of the building. The shingles overlapped the joints of the plywood sheathing and apparently caused the roof to act as a large rigid diaphragm.

Mathematical Analysis of Pole Deflection

A mathematical analysis of the deflection of the individual poles was made with equations developed for beams on an elastic foundation.[5] The equations given in figure 8 evaluate the movement of the top of the pole due to the movement of the pole in the ground, the bending of the pole in the ground, and the elastic bending of the pole above the ground. The test load that was required to move the top of each pole 1/2 inch was used to compute the deflections associated with the movements of the pole and the soil foundation modulus values listed in table 4. The modulus of elasticity of the individual poles was determined by a nondestructive flexure test prior to their placement in the ground. The soil foundation modulus obtained for the individual poles ranged from 145 to 375 pounds per square inch per inch of deflection, with an average of 210 pounds per square inch per inch of deflection for the six poles. These values are within the expected range of soil modulus normally associated with clay soil.[6]

The computed pole deflection at the top of the pole resulting from the elastic bending of the pole above ground averaged 0.243 inch for the six poles (table 4). This value is almost equal to the computed deflection resulting from the combined movement of the pole in the ground (0.222 in.) and the bending of the pole in the ground (0.035 in.). This comparison tends to substantiate the difference in the amount of pole deflection obtained in the tests conducted with frozen (e) and unfrozen ground (d).

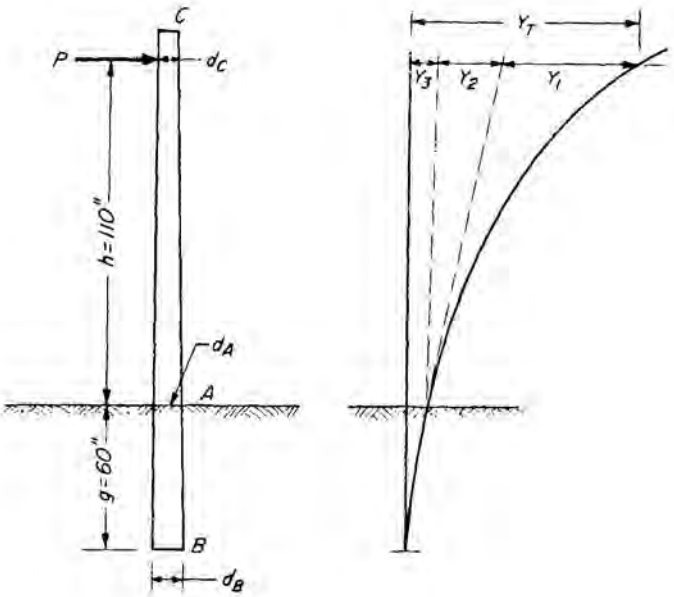
Approximate formulas for determining the movement of the pole in the ground and the movement resulting from the bending of the pole in the ground are also given in figure 8. These formulas give values that are about 10 percent less than those computed by the more exact formulas.

Trussed Rafters

The trussed rafters were evaluated for stiffness prior to their placement in the building. They were symmetrically loaded at eight points over a 35-foot 6-inch span through a system of cables and sheaves similar to that shown in figure 9. Midspan deflection was measured relative to the points of support by reading the location of a taut wire on a scale at uniform increments of load. Each trussed rafter was loaded three times to 5,000 pounds (the assumed design load for these tests,

[5] Heteriyi, M, Beams on Elastic Foundation. University of Michigan Press, Ann Arbor, Mich., 1946.

[6] Terzaghi, Karl. Evaluation of Coefficients of Subgrade Reaction. Geotechnique 5: 297-326(Dec.) 1955.



$$Y_3 = \frac{2P\lambda}{k(\mathrm{Sinh}^2\lambda g - \sin^2\lambda g)}\left[\mathrm{Sinh}\lambda g\ \mathrm{Cosh}\lambda g - \sin\lambda g\ \mathrm{Cos}\lambda g + h\lambda(\mathrm{Sinh}^2\lambda g + \sin^2\lambda g)\right]$$

$$Y_2 = \frac{2P\lambda^2 h}{k(\mathrm{Sinh}^2\lambda g - \sin^2\lambda g)}\left[\mathrm{Sinh}^2\lambda g + \sin^2\lambda g + 2h\lambda(\mathrm{Sinh}\lambda g\ \mathrm{Cosh}\lambda g + \sin\lambda g\ \mathrm{Cos}\lambda g)\right]$$

$$Y_1 = \frac{Ph^3}{3EI_A\ (d_C/d_A)}$$

$$Y_T = Y_1 + Y_2 + Y_3$$

$$\lambda = \left(\frac{k}{4EI_A}\right)^{1/4}$$

$$k = Fd_A$$

Approximate formulas for determining Y_2 and Y_3,

$$Y_2 = \frac{2P\lambda^2 h}{k}\left[1 + 2h\lambda\right]$$

$$Y_3 = \frac{2P\lambda}{k}\left[1 + h\lambda\right]$$

Figure 8.--Mathematical expressions for computing the movement of pole set in the ground and loaded as a cantilever beam. F is foundation modulus, in p.s.i., required to compress soil 1 inch. Constant cross section is assumed between A and B, and tapered section is assumed between A and C.

35 lb. per sq. ft. times 36 ft. times 4 ft.). The residual deflection or set, was measured immediately, following the removal of the initial design load.

The span-deflection ratios, table 1, for the trussed rafters ranged from 761 for a southern pine truss with 1/4-inch tempered hardboard gussets applied with sixpenny common nails, to 1,936 for the double-member aspen truss with nail-glued (phenol-resorcinol) plywood gussets.

The deflection of the trussed rafters at midspan ranged from 0.22 to 0.56 inch at design load (5,000 lb). Two of the trussed rafters deflected 0.32 and 0.56 inch at design load, which exceeded the 1/2-inch deflection criterion considered for trusses to be used in houses of similar span. The design load assumed for the trusses was somewhat higher than that normally assumed for agricultural buildings.[4]

In addition to the trussed rafters for the building, three trussed rafters of oak with plywood gusset plates were constructed. One was fastened with sixpenny nails; a second with 1/4-inch-diameter bolts; and a third was nail-glued with phenol-

Table 4.--Computed deflection near top of pole

Pole	Diameter: Top d_C	Diameter: Ground d_A	Diameter: Bottom d_B	I_A[1]	Modulus of elasticity[2]	P[3]	Pole movement[4]: Y_1	Pole movement[4]: Y_2	Pole movement[4]: Y_3	Pole movement[4]: Y_T	F[5]
	In.	In.	In.	In.[4]	1,000 p.s.i.	Lb.	In.	In.	In.	In.	P.s.i. per in. defl.
1W	7.39	8.21	8.70	223	1,740	196	0.249	0.219	0.034	0.502	250
2W	7.28	7.59	7.77	163	1,460	124	.241	.227	.036	.504	165
3W	7.31	8.10	8.57	211	1,470	148	.234	.226	.038	.498	165
1E	7.84	8.07	8.21	208	1,450	145	.219	.239	.041	.499	145
2E	7.04	7.74	8.16	176	1,290	134	.288	.189	.023	.500	375
3E	7.54	8.32	8.79	235	1,530	167	.227	.232	.040	.499	165
Av.	7.40	8.00	8.37	203	1,490	152	.243	.222	.035	.500	210

[1] Moment of inertia at groundline.

[2] Modulus of elasticity determined from nondestructive bending test.

[3] Test load required to move top of pole 1/2 in.

[4] Computed movement of pole (see fig. 8).

[5] Foundation modulus computed on basis of pole properties and test load.

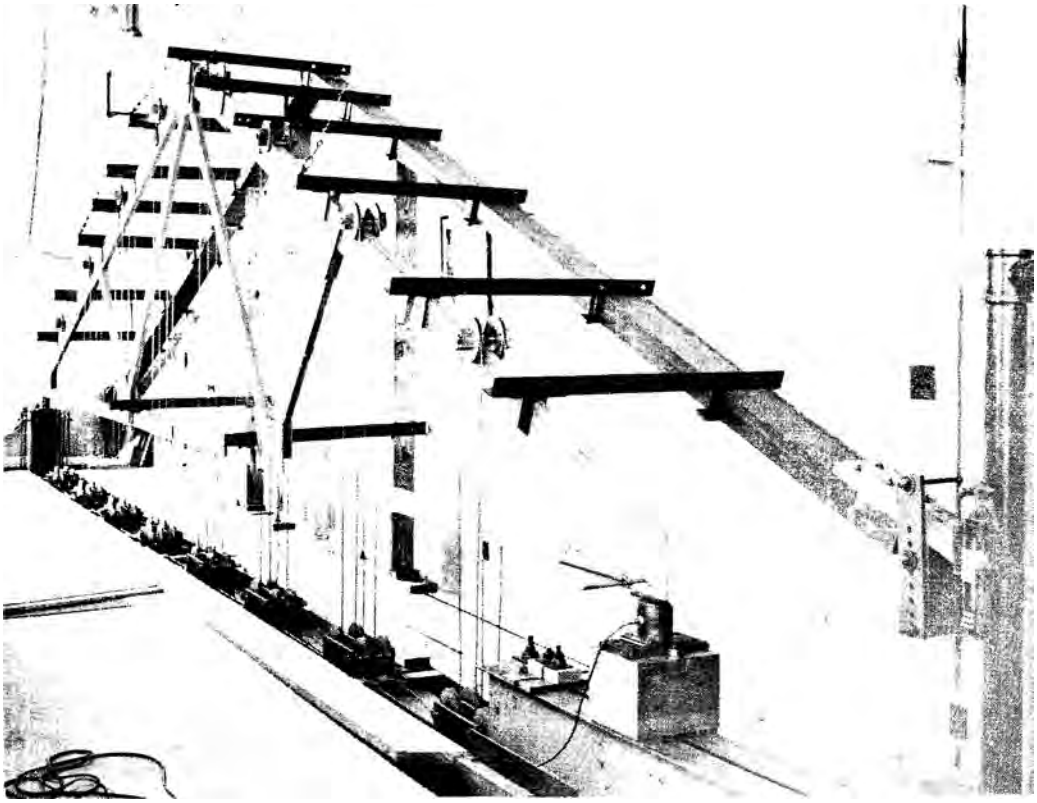


Figure 9.--Trussed rafter of oak with nailed plywood gussets ready tor test. The load is applied through a system of cables and sheaves and is measured at each reaction by a calibrated load cell. (M 130 881)

resorcinol glue. A double-memeber aspen truss with plywood gusset plates, nail-glued with phenol-resorcinol glue, was also constructed. These four trussed rafters, tested in the same manner as the trussed rafters used in the building, were loaded to destruction.

The criterion established for trusses in houses is that the ultimate load should be 2-1/2 times the design load. The aspen and the three oak trussed rafters that were tested to failure gave loads that were in excess of this value. Two of the oak trussed rafters failed in the upper chord by lateral buckling of the chord member between the restrainers (fig. 9) and the third failed at a knot cluster in the upper chord. The aspen trussed rafter failed in rolling shear in the heel gusset.

The residual deflection remaining at the center of the trussed rafters after the removal of the design load ranged from 0 to 0.13 inch. The maximum residual deflection for the trussed rafters with glue-nailed gasset plates was 0.03 inch.

The performance of the wood trussed rafters in the building will be visually observed periodically and, when the building is dismantled, the trussed rafters may again be tested in the laboratory to evaluate their stiffness.

Finish and Fire-Retardant Applications on Shingles

When protecting shingles against decay or fire, it is normally most efficient and economical to treat bundles of wood shingles using a full-cell pressure process. However, in some situations the shingles are in place in a roof, and it is practical to apply only a surface treatment.

In September 1966, protective decorative finishes were applied to the shingles on sections B and C of the roof, and fire-retardant treatments were applied on sections D and E as shown in figure 2, Section A received no treatment. Finishes were also applied to the various wood coverings on the ends and sidewalls of the building. A visual evaluation of the performance of the treatments was made after 2 years of exposure on the building. The condition and appearance of the treatments as observed are described here in general terms.

One protective decorative treatment of a cobalt-phosphate solution was sprayed on the shingles of section 2 with a 3-gallon hand-operated pressure sprayer. A 10-gallon solution, consisting of 10 percent colbalt phosphate, 13 percent phosphoric acid, and 77 percent water by weight, was sprayed on the shingles in 1/2 hour at the rate of about 60 square feet per gallon. This treatment, after 2 years, was ineffective in retarding weathering and mildew on the surface of the shingles.

A second protective-decorative treatment consisting of a copper-chromate solution was sprayed on the shingles of section C in a similar manner. An 8-gallon solution, consisting of 6.0 percent copper sulfate, 6.0 percent sodium dichromate, 0.2 percent citric acid, 0.2 percent concentrated ammonium hydroxide, and 87.6 percent water by weight, was applied in less than 1/2 hour at the rate of about 75 square feet per gallon. This treatment significantly retarded mildew and the normal graying of the shingles. Weathering by photodegradation was retarded, and retreatment was considered unnecessary after 2 years of exposure.

The fire-retardant treatment placed on section E consisted of five spray coats of a water solution of equimolar parts of zinc chloride and zinc silicofluoride (total concentration of 10 pct. zinc). The total coverage was about 30 square feet per gallon of solution. Following 3 days of drying, the section of roof was covered with polyethylene film and anhydrous ammonia gas applied under the film to approximately 0.03 pound per square foot of roof surface. This was intended to convert the soluble compounds to water-insoluble basic zinc compounds.

The fire-retardant treatment placed on section D was a water-soluble formulation of 15 percent solution of a conventional water-soluble fire-retardant for mulation (AWPA Standard P-10, Type D) containing zinc chloride, ammonium sulfate, boric acid, and sodium dichromate. It was sprayed on the roof at the rate of 33 square feet per gallon. After the shingles dried, they were sealed with three spray coats of a commercial water-repellent preservative with 20 percent tricresyl phosphate as a fire-retardant additive.

Separate small test decks or panels were also given the same fire-retardant treatments used on the roof sections. Representative samples were evaluated in the Class C burning-brand test of ASTM Standard E 108-58 and in the modified Schlyter-panel flame-spread test. The results showed some effectiveness but not equivalent to that to be obtained by full-cell pressure treatment. Observations and fire tests will be made

periodically on the test decks exposed to the weather as well as on shingles removed from the roof areas.

Finish Applications on Siding

The rough-sawed spruce boards on the gable ends and end-wall panels on the north and south ends of the building were sprayed with two applications of a commercial water-repellent preservative. The first application was made in the fall of 1965 and the second in the spring of 1966. In the fall of 1968, evidence of mildew was noted only on the south end-wall panel. It was refinished with one brush coat of water-repellent preservative (formula for panel No. 2 of table 5).

Table 5.--Four formulas for a water-repellent preservative for louvered panels

Formulation	Panel No.			
	1	2	3	4
	Grams	Grams	Grams	Grams
Metasol 57	1,200	--	--	--
Penta concentrate	1,200	1,200	1,200	1,200
Sylgard 184	--	--	--	600
Boiled linseed oil	600	600	600	--
Paraffin wax	60	60	60	60
Mineral spirits	2,900	4,140	--	4,140
No. 2 fuel oil	--	--	4,140	--

The center section of the building, which was enclosed with paper-overlaid plywood, was painted in the summer of 1966. The plywood received a spray coat of Woodlife,[7] a brush coat of T-T-P-25a primer and a brush finish coat of M22 Scotch Laddie white lead and oil with yellow tint. The paint coating was in good condition after 2 years, and it is anticipated that repainting will not be necessary for at least 3 or 4 years.

Louvered panels, approximately 4 by 10 feel, of beveled redwood and cedar siding were placed along the upper portion of all sidewalls except for the end wall facing south (fig. 1). Four different experimental formulations of a water-repellent preservative were prepared at the Laboratory and applied to the louvered panels on each of these wall sections in the summer of 1967. The formulations are listed in table 5. There was no evidence of a need for refinishing any of the panels after 1 year.

SUMMARY

A 36-foot, clear-span, pole-type building erected on the grounds of the Forest Products Laboratory to provide shelter for a prolonged research study afforded an opportunity of including a number of research objectives in its design. These included evaluation of (1) the interaction of the constituent parts of the building in increasing the overall rigidity of the structure, (2) the strength and stiffness of several constructions of trussed rafters, and (3) the performance of protective decorative finishes and fire-retardant treatments on wood shingles and panels.

Load-deformation tests conducted by applying a lateral load near the top of supporting poles at diiferent stages of construction during the erection of a pole-type building showed that the additionof each constituent part increased the rigidity of the building. While the trusses, headers, roof sheathing, and shear walls made major contributions to the rigidity of the building, even the shingles also added some rigidity. In the completed building, the deflection at the tops of the test poles ranged from 0.06 to 0.53 inchunder a lateral load equivalent to that of a 60-mile-per-hourwind.

Trussed rafters constructed of four wood species and with various types of glued and mechanically fastened gusset plates of plywood and tempered hardboard gave center deflections of 0.22 to 0.56 inch under a design load of 1-10 pounds per lineal foot of truss. The trussed rafters of oak and aspen that were loaded to destruction sustained ultimate loads of 12,790 to 16,150 pounds (design load was 5,000 lb.).

After 2 years of exposure, observations on the appearance of the protective decorative finish formulations that were sprayed on the wood shingles showed that mildew and the normal graying of the wood had, for the most part, been retarded. Only the cobalt-phosphate solution was considered an ineffective treatment for the shingles.

The burning-brand and the flame and tests conducted on small test decks of wood shingles, which were treated in the same manner as the shingles on the roof of the building, showed that the fire-retardant formulations that were sprayed

[7] Mention of trade names and commercial enterprises or products is solely for necessary information. No endorsement by the U.S. Department of Agriculture is implied.

on were somewhat effective in retarding fire but their effectiveness was not equivalent to that to be obtained by a full-cell pressure-type treatment.

The finish treatments with water-repellent preservatives sprayed on the side- and end-wall panels showed no need for refinishing after 1 and 2 years, respectively, except for the south end-wall panel which showed evidence of mildew and was retreated after 2 years. The paint on the paper-overlaid plywood was in good condition after 2 years.

APPENDIX

Architectural drawings of the elevation and plan views of the experimental pole-type structure, and construction details, are given here in figures A1 to A5.

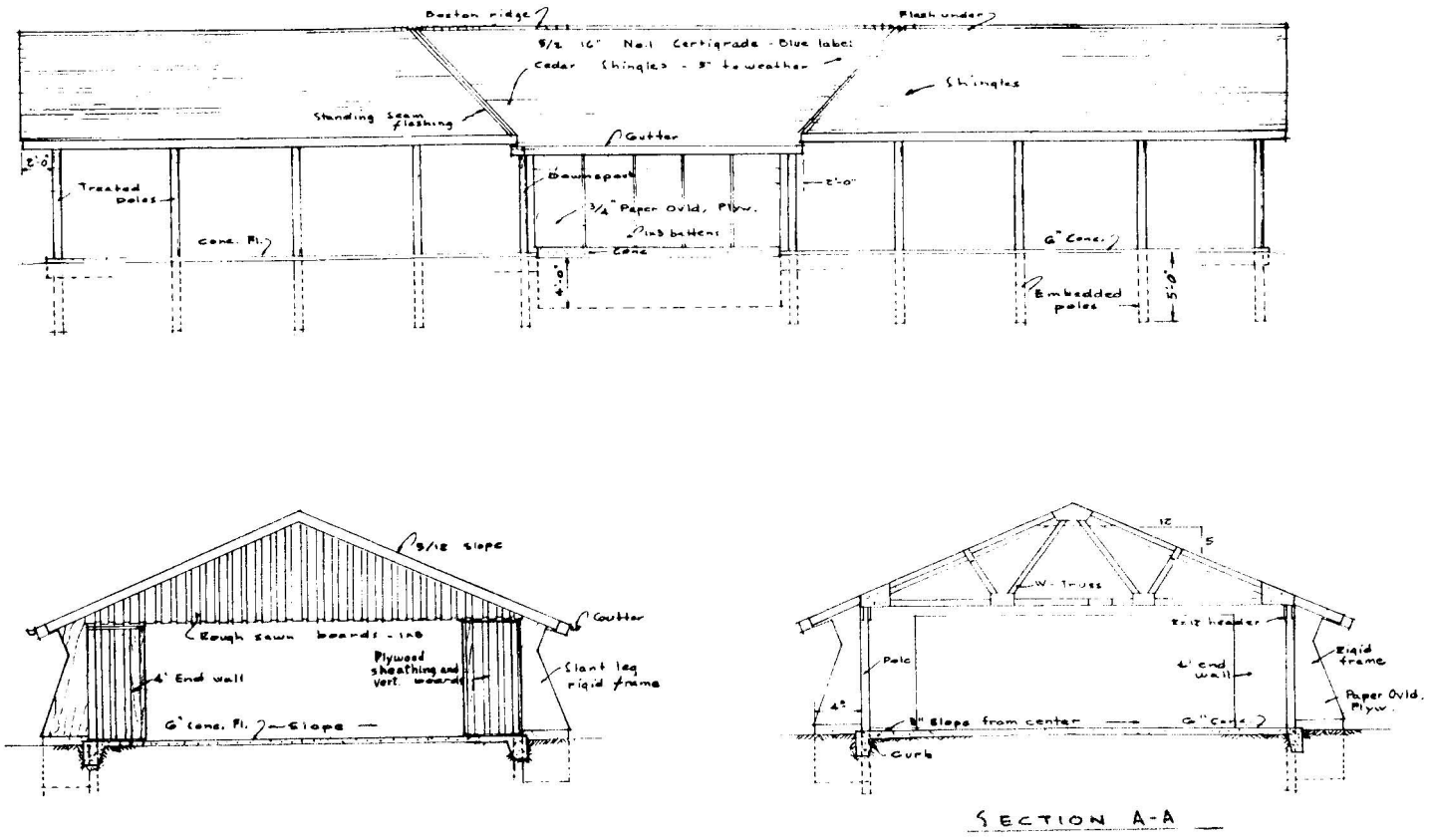


Figure A1.--Elevation and section views of pole-type building. (M 136 707)

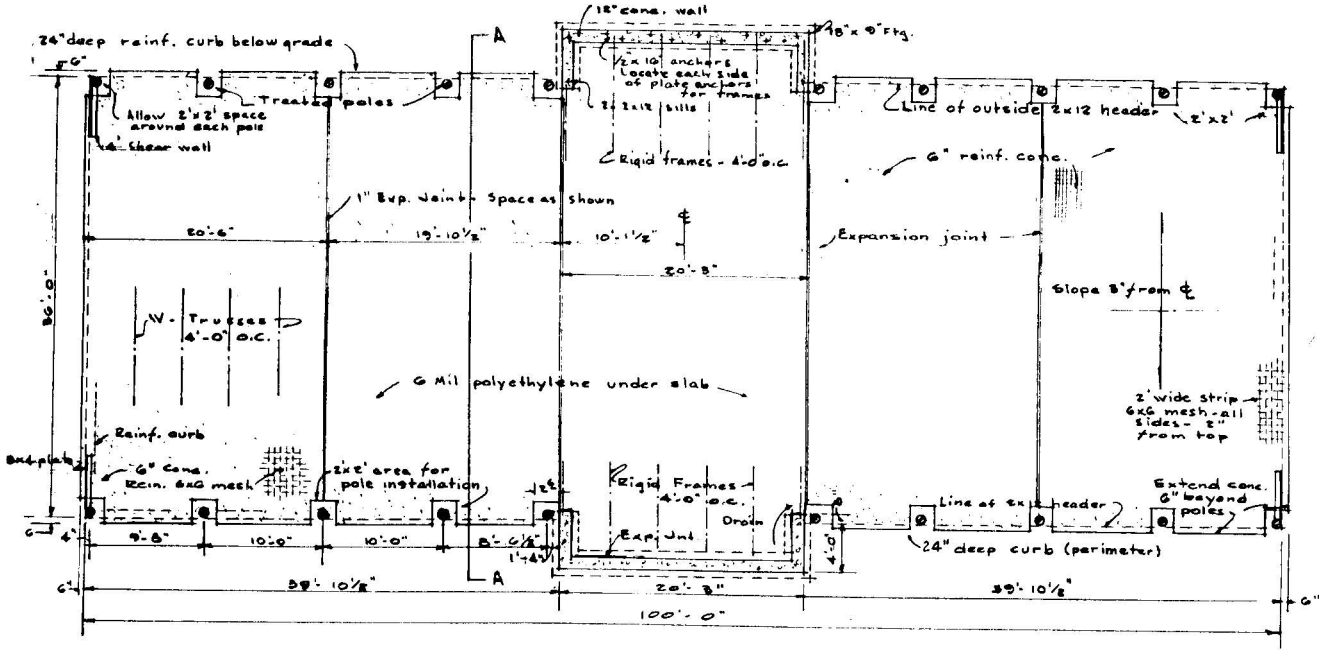
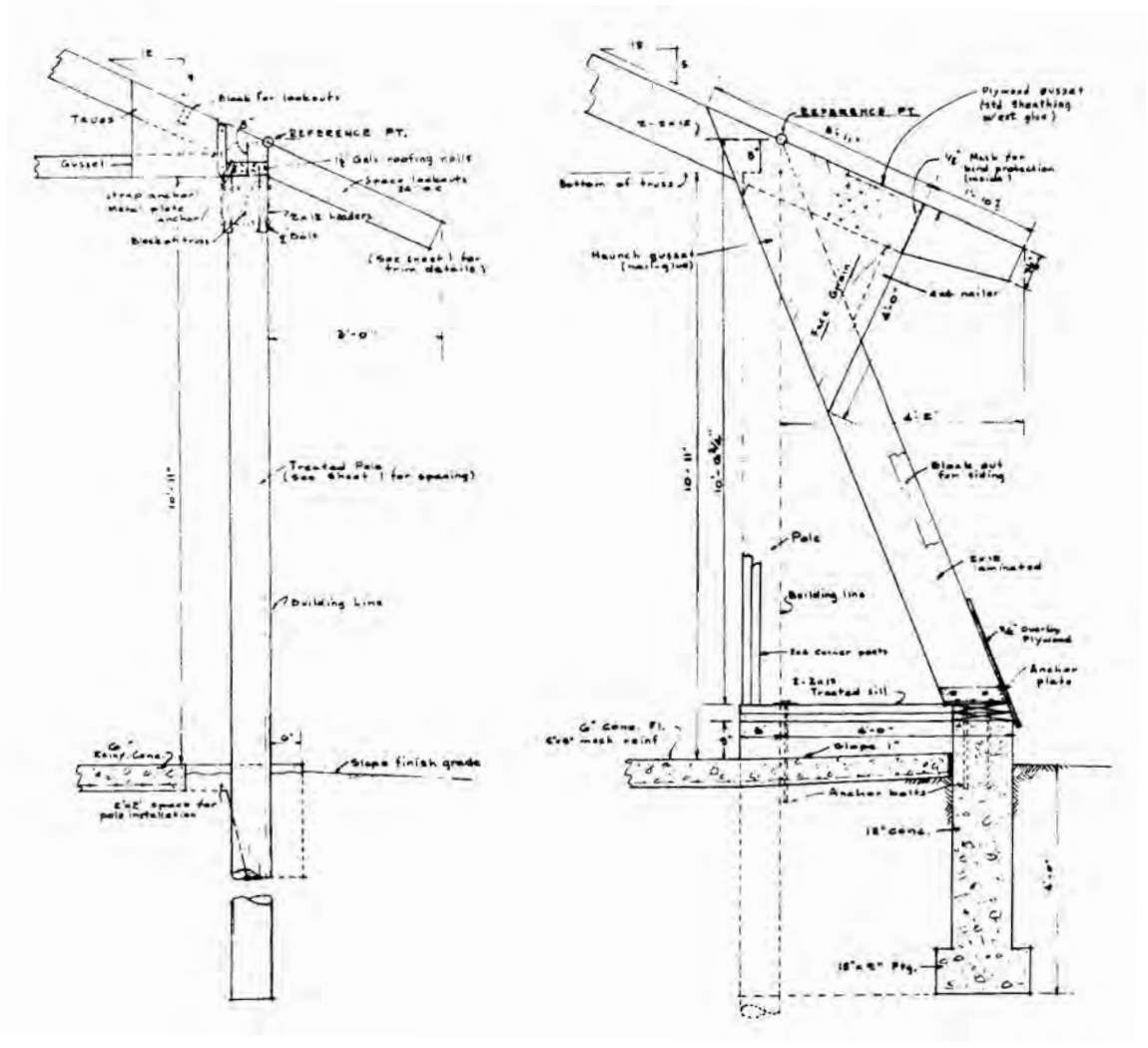
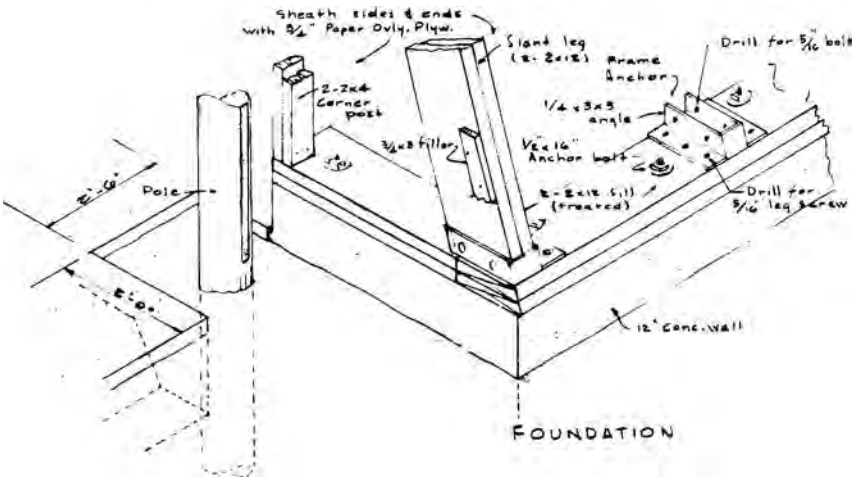


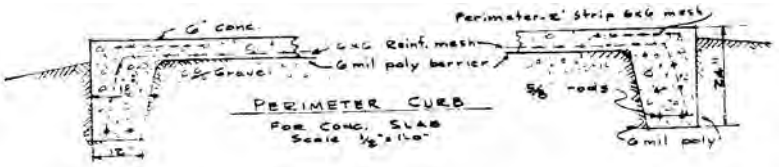
Figure A2.--Plan of pole-type building. (M 136 708)



(M 136 709)

Figure A3.––Construction details of wall section at trusses (left) and at rigid frames (right).





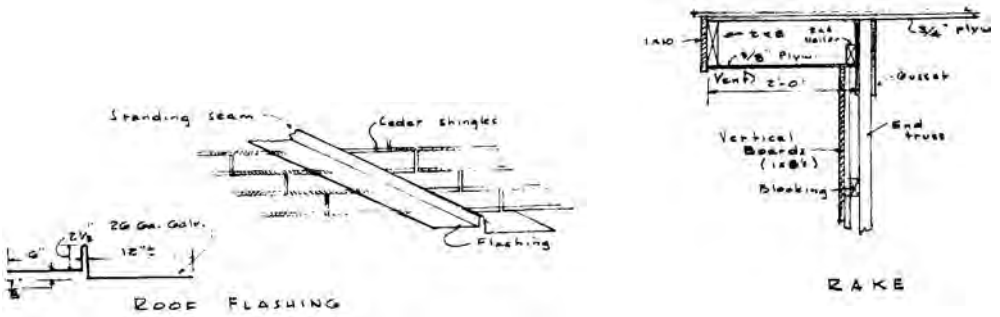


Figure A4.--Construction and fastening details for foundation, rake, and shingles. (M 136 706)

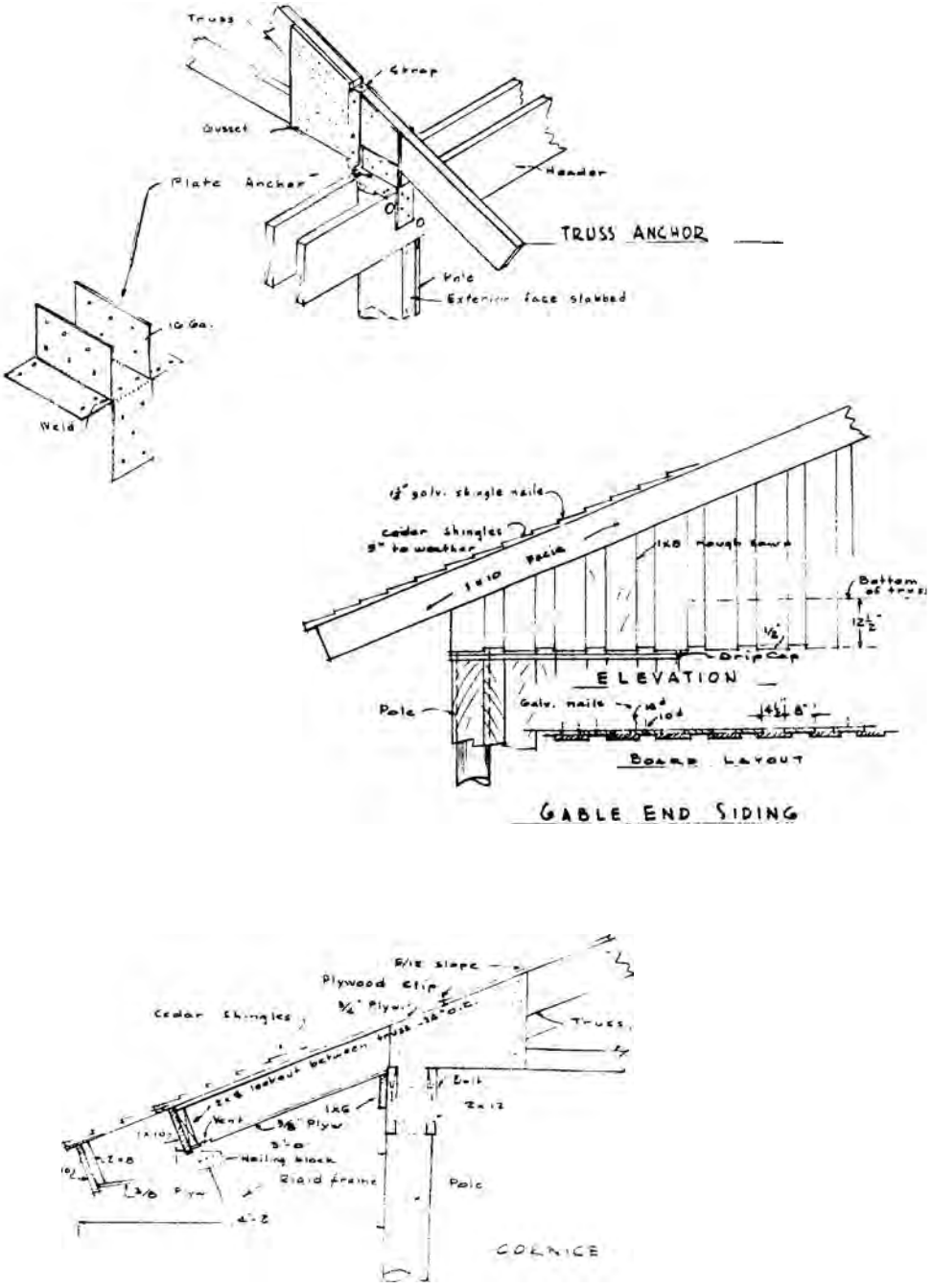


Figure A5.--Construction and fastening details for roof and gable end (M 136 705)

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* Conducting forest and range research at over 75 locations ranging from Puerto Rico to Alaska to Hawaii.

* Participating with all State forestry agencies in cooperative programs to protect, improve, and wisely use our Country's 395 million acres of State, local, and private forest lands.

* Managing and protecting the 187-million acre National Forest System.

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