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Masonry Barn Design and Construction

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SUMMARY AND CONCLUSIONS

In 1913 an investigation was begun with the object of developing an all masonry barn which could be constructed at a reasonable cost and yet have the advantages of permanent and fire resistant construction. The studies which have been conducted pertain chiefly to the roof structure, with particular emphasis on the method of construction.

In addition to a number of design studies, models of roof sections were built to develop a method of roof construction. Strength tests were made on roof models to check the reliability of the designs. The information obtained served as the basis of the design and method of constructing an experimental barn, which was built at Iowa State College in 1926-27. Common overall dimensions and a desirable roof shape were established to make the roof forms usable for a number of barns; wind load assumptions were adapted from reliable wind pressure investigations to permit a more intelligent and efficient roof design.

The results of the design studies, construction and tests on models and roof sections, and the construction of the experimental barn, together with other related experiences, seem to warrant the following general conclusions:

1. The masonry arch is a very stable type of roof structure as shown by the tests on sections, which check closely the design calculations.

2. The construction of the roof is difficult and involves a large amount of labor because of:

   a. The use of heavy steel forms to carry a large part of the roof weight.

   b. The manipulation of the forms in erection, moving, dismantling and transporting.

   c. The handling and placing of roof materials.

3. The additional cost of the roof over a wood frame type construction is due not so much to the cost of materials, as to the cost of the unproductive labor in handling the materials and in manipulation of the steel forms. The overhead cost of the forms becomes a large item in the first cost if they are used for only one or a few barns.

4. Experiments in the methods of making a roof watertight have not as yet indicated an entirely successful method. A heavy fibered asphalt has been found the best of the waterproof coatings which have been used. Leaks appear to be due to slight openings in the joints and to the development of fine cracks.

5. The construction of the roof should be directed by one who is familiar with masonry construction.

6. A roof with a span of 34 ft. and a height of 20 ft. provides enough storage space for most conditions.

Masonry Barn Design and Construction[1]

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This bulletin reports the results of a study, initiated in 1913,[2] of the design of an all masonry barn which might be constructed at reasonable first cost with the advantages of permanent and fire resistant construction. This investigation is only a step in the development in this type of barn, and since studies were confined largely to the roof, the solutions to a number of other problems remain incomplete. The basic principle of arches—the placing of all of the materials in compression—was used in the design of the roof. Reinforced concrete ribs were added for additional stability and to resist eccentric loads such as those caused by wind.

BARN DESIGN

PRELIMINARY CONSIDERATIONS

In most types of masonry roof construction economy depends largely on the use of forms which may be used for a large number of barns, thus reducing the form cost per structure. A roof of standard dimensions in cross section is highly desirable, so that this economy in use of forms may be obtained.

SIZE OF BARN

A study of the size of barns to establish roof dimensions which may satisfy a large percentage of farm needs involves first a study of barn widths, since the principal dimensions of the roof, with respect to its cross section, are determined by the width of the first floor structure and the amount of storage space to be provided in the upper story.

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[1] Project 24 of the Iowa Agricultural Experiment Station.

[2] The following contributions to the study of the masonry barn are acknowledged: W. W. Ashby and J. B. Kelley, who studied designs of an all masonry barn for their senior theses in Agricultural Engineering; W. G. Kaiser, A. W. Clyde and L. J. Fletcher, who designed and constructed a full sized roof section of hollow clay blocks; the Structural Clay Tile Association for its support of a research fellowship held by Bruce Russell, who designed the experimental barn for partial fulfillment of the requirements for a master's degree.

Two general methods were used in attempting to determine a barn width which is generally accepted in practice and therefore likely to be used, namely: (a) Plans of barns recommended by the United States Department of Agriculture and state experiment stations (13), together with observations of barns in use; and (b) a study of interior dimensions of both the "face in" and "face out" arrangements. The observations on actual barns were taken from the following: "Dairy barns from a manufacturing point of view" (20), "An economic study of farm buildings in New York" (9), and the dairy barn survey made by the Structures Division of the American Society of Agricultural Engineers (1).

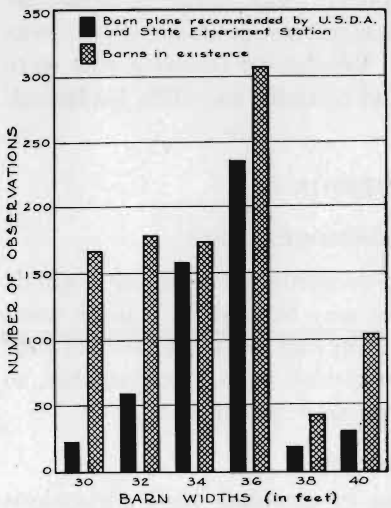
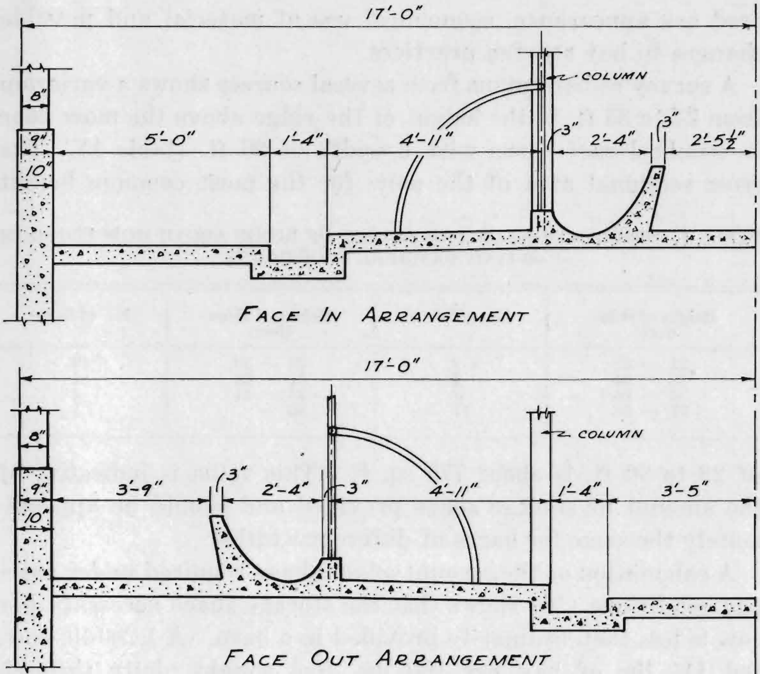


Fig. 1. Number of barns by widths.

Observations of barns and plans by common widths are shown in fig. 1. These show that 34 and 36-foot widths are generally recommended for dairy and general purpose barns and that there is in existence a large number of barns of widths narrower than those generally recommended. The average of the 30, 32, 34, and 36-foot widths is 34.3 ft., while the average of all barns included in the plans is 33.9 ft. A barn of 34 ft. gives ample work space and conforms well to present practice.

Interior dimensions affecting barn widths for both the "face in" and "face out" arrangement are given in fig. 2. The largest variations occur in the length of the stall and width of litter and feed alleys, the latter depending on the type of arrangement used.

From the above and previous studies a choice of either the 34 or 36-foot barn would seem satisfactory. Because a narrower barn requires less material per cow housed and is warmer due to less exposed wall area, a width of 34 ft. was selected.



DETAIL	ARRANGEMENT	MIN.	MAX.	AVERAGE	34 FT. WIDTH
Feed Alley	Face In	5'-0"	6'-0"	5'-6"	5'-0"
	Face Out	3'-6"	5'-0"	4'-3"	3'-9"
Litter Alley	Face In	4'-0"	6'-0"	5'-0"	4'-9"
	Face Out	5'-0"	10'-0"	7'-6"	6'-10"
Stall		4'-8"	5'-2"	4'-11"	4'-11"
Manger		2'-0	2'-8"	2'-4"	2'-4"
Gutter				1'-4"	1-4"

Fig. 2. Barn cross sections showing the near average dimensions of the details obtained by a width of 34 ft.

The amount of storage space necessary for feed for the live-stock housed in the barn may be a determining factor in establishing the height of the barn roof. Other factors to be considered are appearance, economical use of material and possible changes in hay storage practices.

A survey of barn plans from several sources shows a variation from 21 to 35 ft. in the height of the ridge above the mow floor in gambrel roof barns with a width of 36 ft. (table 1). The cross sectional area of the mow for the most common height

TABLE 1. OBSERVATIONS OF THE HEIGHT OF RIDGE ABOVE MOW FLOOR IN 36-FOOT GAMBREL ROOF BARNS.

Height of ridge (feet)	No. of barns	Height of ridge (feet)	No. of barns
21 — 22	2	29 — 30	19
23 — 24	4	31 — 32	4
25 — 26	7	33 — 34	2
27 — 28	12	35 —	1

of 29 to 30 ft. is about 770 sq. ft. This value is indicative of the amount of storage space provided and should be approximately the same for barns of different widths.

A calculation of the amount of roughage required under average conditions (10) shows that the storage space necessary per cow is less than ordinarily provided in a barn. A 1,200 lb. cow, fed 1½ lbs of hay per 100 lbs. live weight, daily through a feeding period of 220 days, would consume 3,960 lbs. or 990 cu. ft. of loose hay, with silage fed in ample quantities. The barn length chargeable to a cow is approximately 2.35 ft. The necessary mow area in cross section required per cow would be 422 sq. ft.

Additional mow space, above the amount required, may be desirable in many instances for surplus hay, grain, equipment or bedding. The mow should provide the necessary space for loose hay and still not have much waste space when chopped or baled hay is used . For good appearance, stability, and the above requirements, a roof height of 20 ft. would be a reasonable choice. It gives a cross sectional area of 462 sq. ft. for a curved roof conforming to the shape of an inverted catenary.

SHAPE OF ROOF

The shape of the barn roof should conform to some symmetrical curve suitable for arches, especially when masonry materials are used. The selection of such a curve involves the consideration of the roof with respect to its structural stability, appearance and economical use of material for the amount of mow storage space provided.

A light high arch roof differs from a low massive arch used for bridges, chiefly in the loading and the stresses. The design in the latter is governed largely by the direct stresses in compression from its own dead weight, whereas that of the former is governed almost entirely by the bending stresses due to eccentric live loads produced by the wind. Reinforcing must be supplied in the roof structure to resist these bending stresses, either in the roof slab itself or in arch ribs spaced at intervals along the roof. Since masonry materials are comparatively weak in tension, a roof shape, which would be stable under its own weight (that is, subjecting all materials to compressive loads only) would offer some advantage in design.

Of the curves presented in fig. 3 for the proposed shapes, the parabola and the inverted catenary most nearly satisfy this condition. The catenary is a curve formed by a flexible, inextensible cord or chain of uniform weight per unit of length when suspended at the ends. The parabola is similar with the exception that the weight is distributed uniformly along the span, rather than along the length of the cord or chain. The inverted catenary satisfies the condition of stability for a light and high arch under its own dead weight as shown in fig. 16. It provides slightly greater cross sectional area than the parabola for a given height and presents a more pleasing appearance.

The semi-ellipse, with its major axis vertical, is suitable from the standpoint of mow space, but not desirable from the standpoint of stability because the resultant of the dead loads deviates considerably from the arch axis and subjects certain portions of the arch to tensile stresses.

WIND LOADS

Although the conventional method of wind load assumptions, by which only impact pressures are considered, was used in the design of the model arches, an attempt was made in succeeding designs to extend information obtained from results of numerous wind pressure investigations on structures for load assumptions which approximate actual conditions. Wind load formulas do not take into account "negative" or outward pressures,

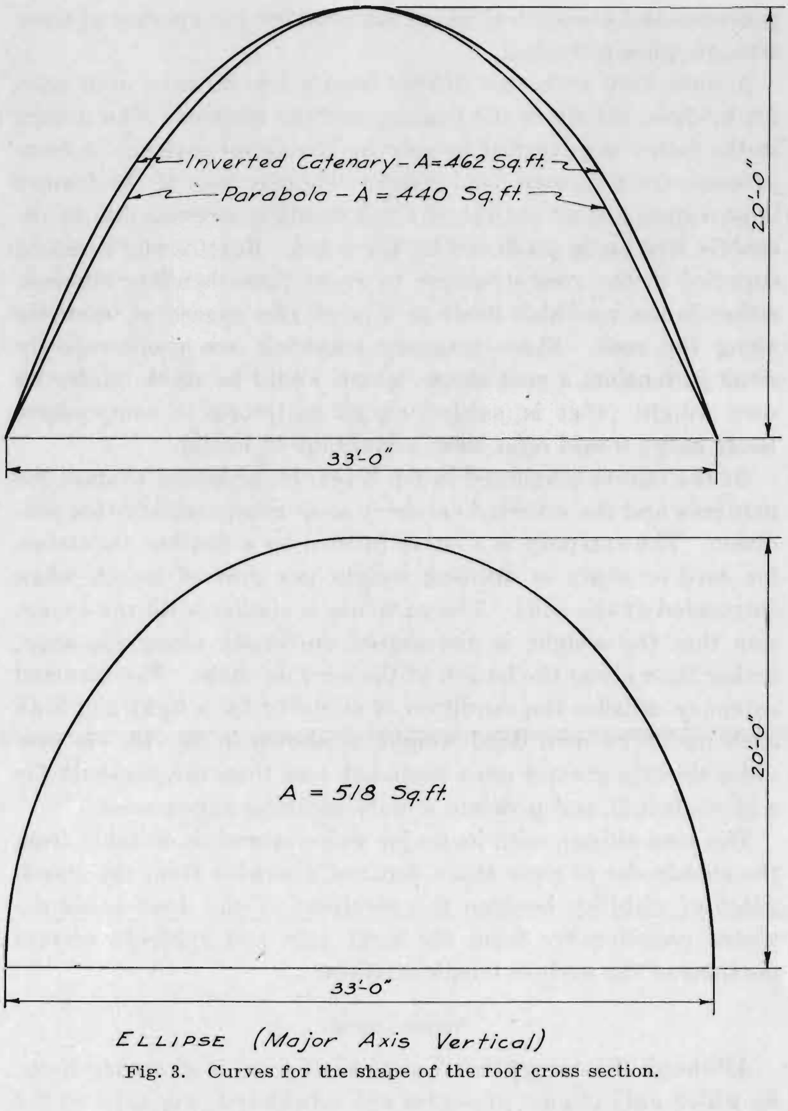


Fig. 3. Curves for the shape of the roof cross section.

nor are they comprehensive enough to account for variation of pressure distribution and form variation of structures.

A few of the studies in this connection are mentioned briefly.

Prof. A. Smith (15) at Purdue University conducted, in 1913, a series of tests in natural winds on a model of a building with a roof semi-circular in section. The results are shown graphically in fig. 4. The positive or impact pressure constituted a very small part of the total wind load.

Similar results were obtained in Carl Arnstein's (19) wind tunnel experiments on the model of the large airship hangar at Akron, Ohio. (See fig. 4.) The positive or impact pressure constituted a very small part of the total wind load.

Dryden and Hill of the United States Bureau of Standards (7), in their tests on circular cylinders, found that the larger pressures were directed outward. Similar results were found in their tests on a model mill building (6). In many instances the loads on appreciable areas of a face were often as great as twice the average over the entire face.

The most important phenomenon noted by Sylvester (16), in his wind tunnel tests on model airship hangars, was the large negative pressures. He explains: "The extensive area over which these pressures are exerted is surprising, but the extent to which their consideration has been neglected is even more surprising."

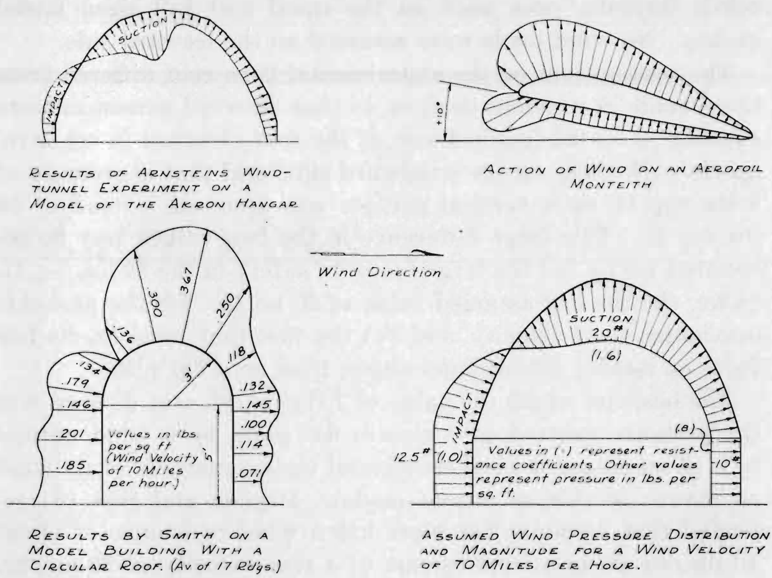


Fig. 4. Wind pressure distribution on models of buildings and the distribution and magnitude of wind pressures on the barn roof.

The Building Research Board (4) of London, England, in their attempt to determine how results obtained from small scale models may be applied to full sized buildings, found that pressure on the leeward side of a full sized building is greater, on the whole, by about 50 percent.

Experiments on aerofoils in aerodynamics indicate that 60 to 70 percent of the total lifting force is due to a reduced pressure on the upper side (fig. 4).

Other investigations have shown that negative pressure constitutes a large part of the total force acting and that wind pressure distribution for a structure varies considerably with its shape, and must, in many cases, be determined experimentally.

The wind load assumptions which were used in connection with this investigation and the recommendations set forth for roof structures of similar shapes are described in the following paragraphs.

The conventional method of assuming a pressure of 30 lbs./sq. ft. on the vertical surface and reducing the normal pressure on the inclined surface on the windward side by Duchemin's formula,[3] was used on the small and full sized model arches. No wind loads were assumed on the leeward side.

The assumptions on the experimental barn roof differed from those made in previous designs, in that outward pressures were considered on the leeward side of the roof identical in other respects with those on the windward side, and that a pressure of 7 lbs./sq. ft. on a vertical surface was assumed instead of 30 lbs./sq. ft. This large difference in the two values may be accounted for by (a) the large factor of safety in the 30 lbs./sq. ft. value, (b) the low assumed value of 60 mi./hr. for the probable maximum wind velocity, and (c) the fact that wind exerts less force on a three dimensional object than on a flat plate.

The basis on which the value of 7 lbs./sq. ft. was derived was the pressure exerted on a square flat plate by a wind normal to it of an assumed wind speed, and the comparison of its wind resistance to that of a roof model. Dryden and Hill (8) reported that a square flat plate has a wind resistance of about 10 lbs./sq. ft. in a wind stream of a true velocity of 60 mi./hr.

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[3] $P_n = P \dfrac{2 \sin\theta}{1 + \sin^2\theta}$

A comparison of the wind resistances was obtained by constructing a small model of the roof, equal in shape and area to the vertical projection of the model. They were made equal in weight and when suspended as pendulums in a wind stream, it was found that the displacement of the model was about two-thirds that of the plate. On the basis of these displacements, the value of 7 lbs./sq. ft. was established as the pressure on a vertical surface; the normal pressures were determined by Duchemin's formula.

In view of more recent data on wind pressures and weather records, the above assumptions can be improved and a better set of recommendations set forth for wind load assumptions on a roof of this shape.

The assumed wind velocity of 60 mi./hr. appears to be too low, especially since the maximum wind velocity recorded by the United States Weather Bureau stations in Iowa is 68 mi./hr. (18). This value, unquestionably, is exceeded for short intervals of time, since the Weather Bureau anemometer records enable one to determine only an average velocity over the period necessary for a mile of wind to pass the anemometer. The maximum velocity of gusts in high winds should be considered also. There is very little precedent to make an assumption to take gusts into account, but a value of 70 mi./hr. would seem to be a reasonable assumption for maximum velocity.

Figure 4 shows the magnitude and distribution of pressures for a wind velocity of 70 mi./hr., as determined on the basis of results of the above described experiments. Note that pressures differ radically from those assumed on the experimental barn roof, especially at the crown.

Attention is called to the so-called "resistance coefficients" which appear on the diagram (fig. 4). These coefficients are the ratios of the actual pressure to the theoretical or "velocity" pressure, which is expressed by the relation

$$P = .001189\left(V \times \frac{22}{15}\right)^2,$$

where P is in lbs./sq. ft., V the true wind speed in mi./hr., and .001189 is the density of air in lbs./cu. ft. corresponding to 15° C., 76 cm. Hg. These coefficients are the same for different wind velocities, and once these have been determined for various points on the structure, the pressures for different wind velocities can be determined conveniently.

STRUCTURAL DESIGN

The studies reported here pertained largely to the design of the roof structure. It has been the aim to simplify technique of construction and to observe possible economies in labor and materials, as well as to obtain a structurally sound roof.

The procedure in designing the roof structure is essentially the same as that for arches, a description of which is given in Turneaure and Maurer, "Principles of Reinforced Concrete" (17). A preliminary design of the roof is assumed similar to that in arches, which are made either by the aid of past designs or empirical formulae. An exact analysis is then made and the results used in correcting the design. By successive designs and analyses the correct roof section for a particular load condition may be determined.

In the tile roof design, reinforced concrete arch ribs were supplied at intervals to resist the bending moments caused by the eccentric wind loads. (See fig. 12.)

Lightness in weight of roof and saving of form work would be obtained by placing hollow clay blocks between the ribs. In addition to the reinforcing in the supporting ribs, wire reinforcing would be placed in each mortar joint, extending from

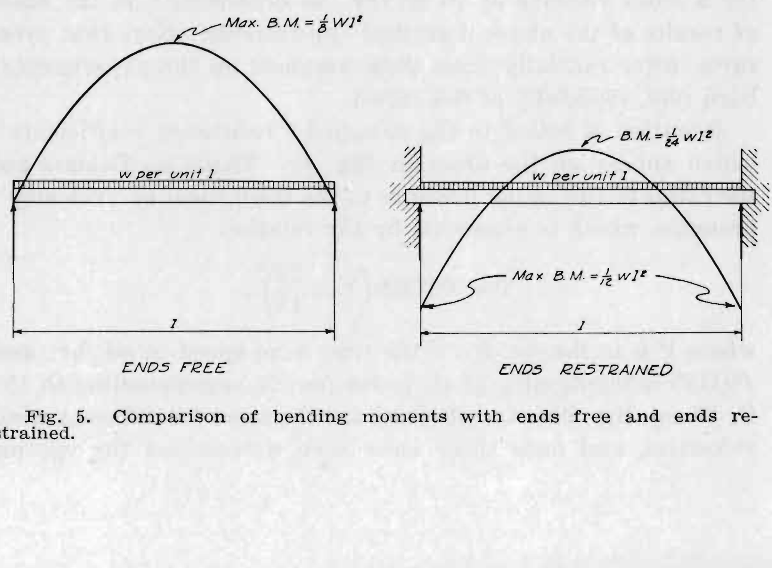


Fig. 5. Comparison of bending moments with ends free and ends restrained.

rib to rib. The arch supports would frame in at the mow floor and be high enough to give ample hay mow capacity.

The method of calculation used in this, as well as the other types of construction, assumes fixed ends; that is, the abutments of the arch are so massive or rigid that they will not yield when the arch is loaded. This condition may be approximately fulfilled in massive arches, but in the barn roof it is probable that the part below the spring line will yield considerably. Such yielding of the abutments would make the bending stresses at the spring line less than the calculated values, and increase them at some point higher up on the arch. This can be illustrated roughly by comparing the arch to a beam. A certain load on a beam with fixed ends will produce bending moments at the ends and at the center of the beam. If the ends of the beam are not fixed, however, the moment at the ends will be zero, while the moment at the center will be increased under the same load. (See fig. 5.)

Figures 6 and 7 show the design and some of the important results of the analysis of the model arch (3), which was intended for a barn 30 ft. wide and a roof 20 ft. in height above the mow floor. The shape of the roof conformed to an inverted catenary. The arch ribs reinforced with 4 3/8-in. round bars were spaced 6 ft. 6 in. on the centers. The ribs were 6 in. wide and varied in depth from 13 in. at the supports to 4 in. at the crown. Four-inch hollow clay blocks were placed between the ribs. In the stress analyses a positive wind pressure of 30 lbs./sq. ft. on a vertical surface was assumed, other loads including the weight of the blocks being neglected. A check of the stresses for the loads assumed, showed the roof structure to be amply strong.

Two weeks after the arch had been constructed, 23 sacks of sand, each weighing 100 lbs., were placed on one side of the roof to simulate wind load conditions (fig. 8). No cracks or severe deflections could be detected at any point along the arch ribs.

To obtain a larger load application, a system of levers was used, since it was difficult to add sacks of sand (fig. 9). Under a concentrated load of 2,200 lbs., the steel on the inside of the rib split away from the concrete. This shows the importance

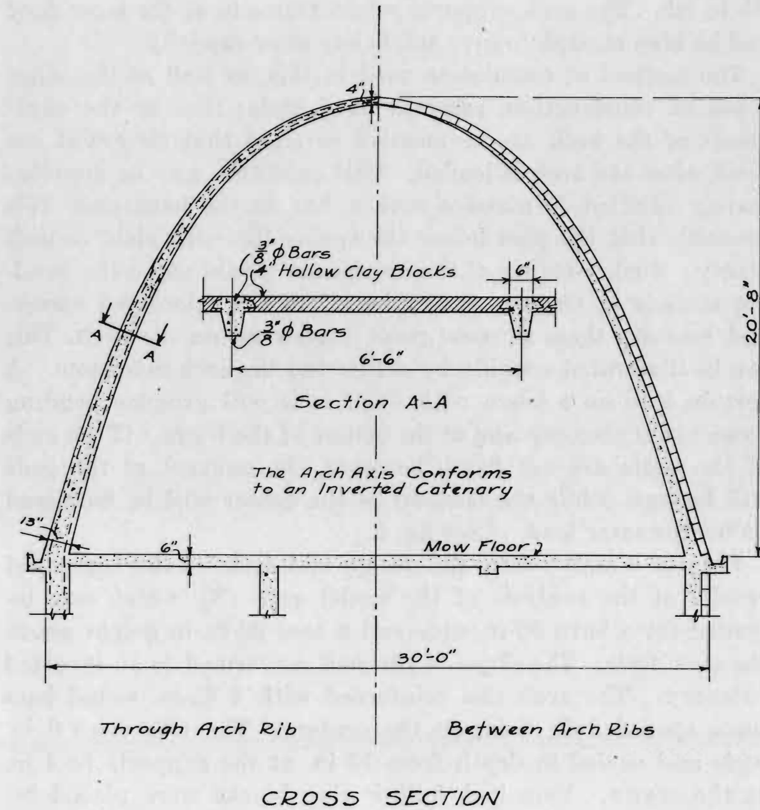


Fig. 6. Design of masonry arch roof represented by the model arch.

of wiring the reinforcing bars in the top and bottom sides together. Had this been done and the concrete permitted to set at least 2 weeks longer, the roof would probably have resisted double the load at failure, which was equivalent to a wind load of 84 lbs./sq. ft. on a vertical surface.

After further failure had occurred in the ribs through the application of a greater load, the roof still remained in an upright position, although the crown had been displaced a few inches. A 100-pound sack of sand dropped from the top floor of the adjacent building, a height of about 30 ft., broke only 2 clay blocks.

These results indicate that the arch was designed and con-

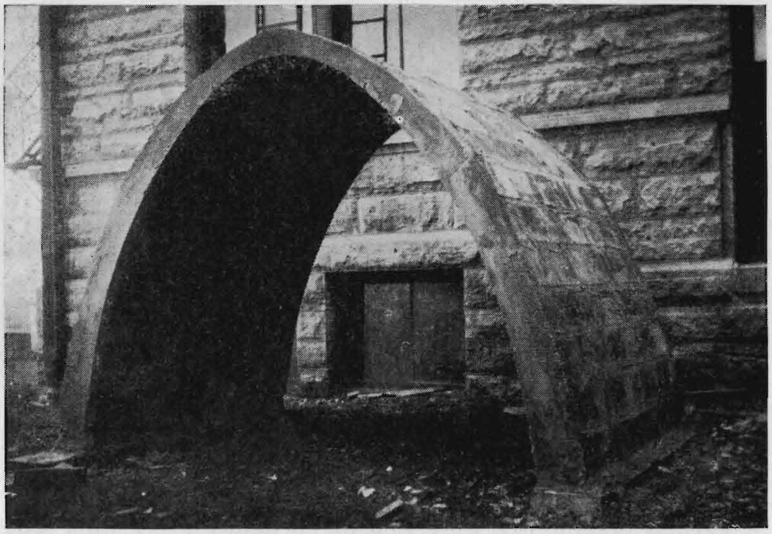


Fig. 7. The model arch roof.

Fig. 8. Manner of loading the model roof for the strength tests.

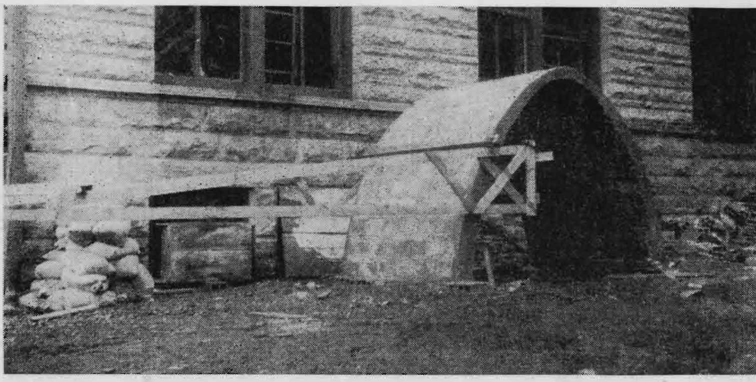


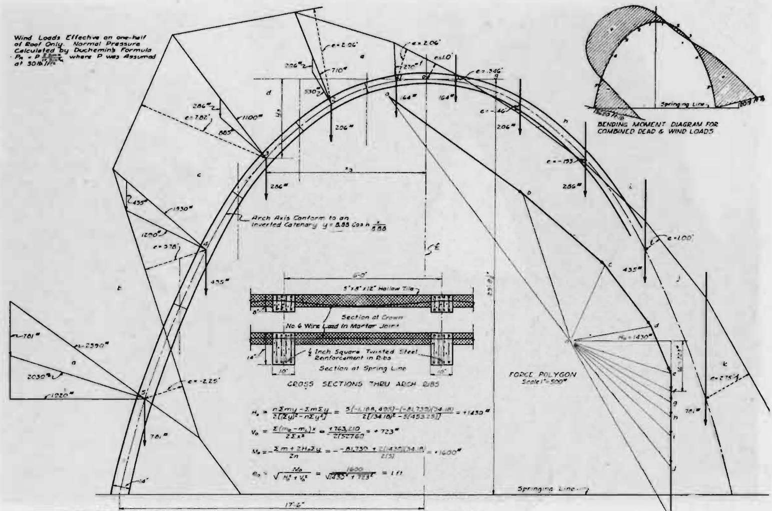
Fig. 9. Apparatus for larger load application.

structed with a satisfactory factor of safety, and that there were no apparent weaknesses in the arch.

Various possibilities were considered before proceeding with the design of the full sized arch (11). A design similar to that of the model arch appeared to be most promising. A span of 36 ft., with a rise of 24 ft. 7 in. above the mow floor, was chosen for the test section. The arch ribs, spaced 6 ft. on centers (fig. 10), were 10 in. wide and varied in depth from 8 in. at the crown to 14 in. at the springing line. They were reinforced with 8 ½-inch twisted bars, half of the bars being near the top and half near the bottom of the rib section. Clay blocks, 5 x 8 x 12-inch, with a no. 6 wire extending from rib to rib in each horizontal mortar joint provided the sheathing between the ribs.

Stress analyses were made by assuming a wind load of 30 lbs./sq. ft. of vertical surface. The weight of the blocks was neglected. The shape of the arch conformed essentially to the inverted catenary, with slight modification to bring it nearer the equilibrium polygon or the line of thrust under its own weight. The final design and the results of the analysis are given in fig. 10.

The tests on this arch were made more carefully than in the previous test on the model arch (5). One concentrated load was applied at the point of application of the resultant of wind loads, 12 ft. 9 in. above springing line and normal to the sur-



POINT	x(ft)	y(ft)	ds	I_T	$\frac{ds}{I_T}$	H_0Y	V_0X	m_L	m_R	M_L	M_R	H_L	H_R	e_L	e_R
1	1.70	0.15	3'-7"	299	.137	212	1230	0	0	+3042	+582	1480	1680	+.206	+.346
2	5.40	1.46	4'-5"	374	.137	2090	3910	-1020	-407	+6580	-827	1170	1800	+.563	-.460
3	9.10	4.59	5'-5"	476	.132	6560	6570	-5550	-1972	+9380	-382	1200	1980	+.782	-.193
4	12.52	9.73	7'-2"	650	.132	14000	9050	-16330	-5350	+8320	+2300	2200	2300	+.378	+1.000
5	16.00	18.25	11'-5"	1002	.134	26100	11580	-44200	-8010	-4950	+8110	2200	2950	-.225	+2.150
Σ		34.18				33900	12650								
Sprg L	17.50	23.70						-61770	-10835	-13620	+12015	4370	2950	-.312	+4.015

Fig. 10. Dead and wind load analysis of the full sized section of the arch roof.

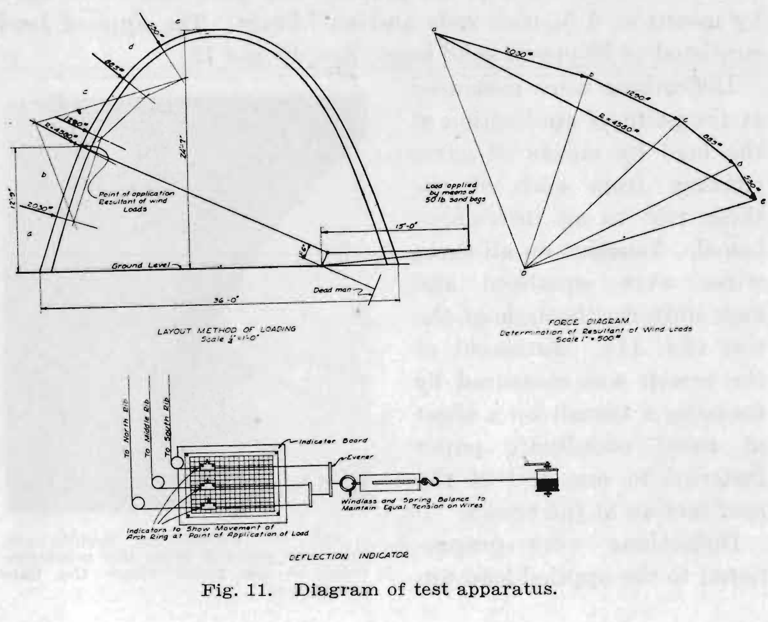


Fig. 11. Diagram of test apparatus.

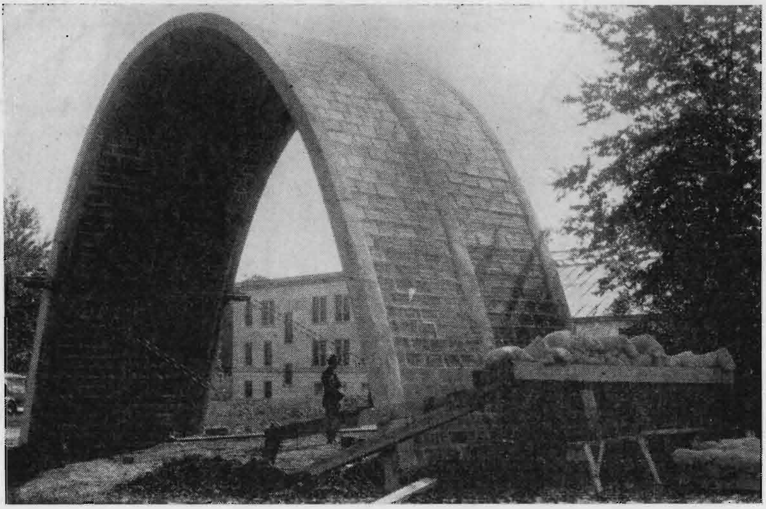


Fig. 12. Application of large loads on full sized roof section by system of levers.

face. The apparatus used for this purpose was a set of levers (ratio 10:1) anchored to the ground and connected to the arch by means of 4 ⅝-inch rods and an I-beam. The applied load consisted of 50-pound sand bags (figs. 11 and 12).

Deflections were measured at the point of application of the load by means of wires running from each of the three ribs to an instrument board. Tensions on all three wires were equalized and kept uniform throughout the test (fig. 11). Movement at the crown was measured by focusing a transit on a sheet of ruled coordinate paper fastened to one end of the roof section at the crown.

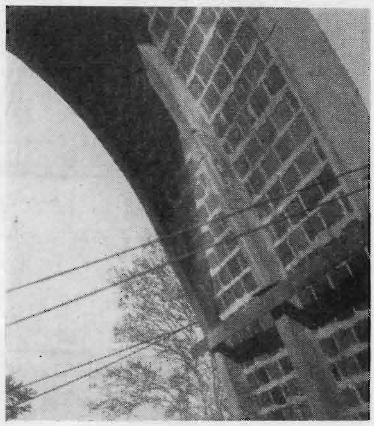


Fig. 13. Failure of middle rib. Concrete spalded from the reinforcement at the point where the bars overlapped.

Deflections were proportional to the applied load until the load reached 27,000 lbs., at which load the arch failed. Owing to the short distance of travel of the levers and the slack in the apparatus, it was necessary to unload, take up slack, and reload. This was repeated a number of times. After deflecting more than 2 in., the arch regained almost its original shape.

The failure which occurred slightly above the point of application of the load (fig. 13) was not typical of a reinforced concrete beam, but seemed due to the fact that all laps in reinforcing bars were made in one place, making such a mass of steel that the individual bars were not surrounded with concrete.

A slab of concrete approximately 3 ft. in length, as wide as the rib and as thick as the layer of concrete below the steel shelled off as a result.

The structure was so thoroughly bound together that destruction was accomplished only by crushing each piece individually with sledges (fig. 14). Dynamite could not be used on account of the structure being in close proximity to greenhouses.

Although an effort was made to salvage the tile, not one came out intact. Of particular interest was the quality of bond between the mortar and the hard burned clay blocks (fig. 15). In practically all cases, breaks would occur in the tile, in the mortar, or across both. Very few instances could be found in which the mortar had separated from the tile.

The breaking load of 27,000 lbs., which is nearly equal to six times the total wind load assumed, shows that the design was considerably heavier than that required in roof construction.

The experiences in the design and construction of the full sized section of the roof, together with the results of the strength tests, were very helpful when designing the roof of the experimental barn (14). The overall dimensions of the arch were very nearly the same as that of the latter; the size of the arch ribs was decreased considerably, because of lower values used in the wind load assumptions and the design of the full sized section was heavier than necessary. The ribs with a width of 6 in. varied in depth from 12 in. at the springing line to 8 in. at the crown, and were spaced 6 ft. 3 in. on centers, permitting 5½ 12-inch hollow clay blocks to be placed between the ribs. Two $\frac{1}{2}$-inch square bars placed in each of the top and bottom halves of the rib constituted the reinforcing.

In the dead load analysis the weights of the laid-up tile and the reinforced concrete rib were assumed to be 25 lbs./sq. ft. and 150 lbs./cu. ft., respectively. The results of the analysis presented in fig. 16 show that the line of thrust is very near the center line of the arch rib. The eccentricity, which is the

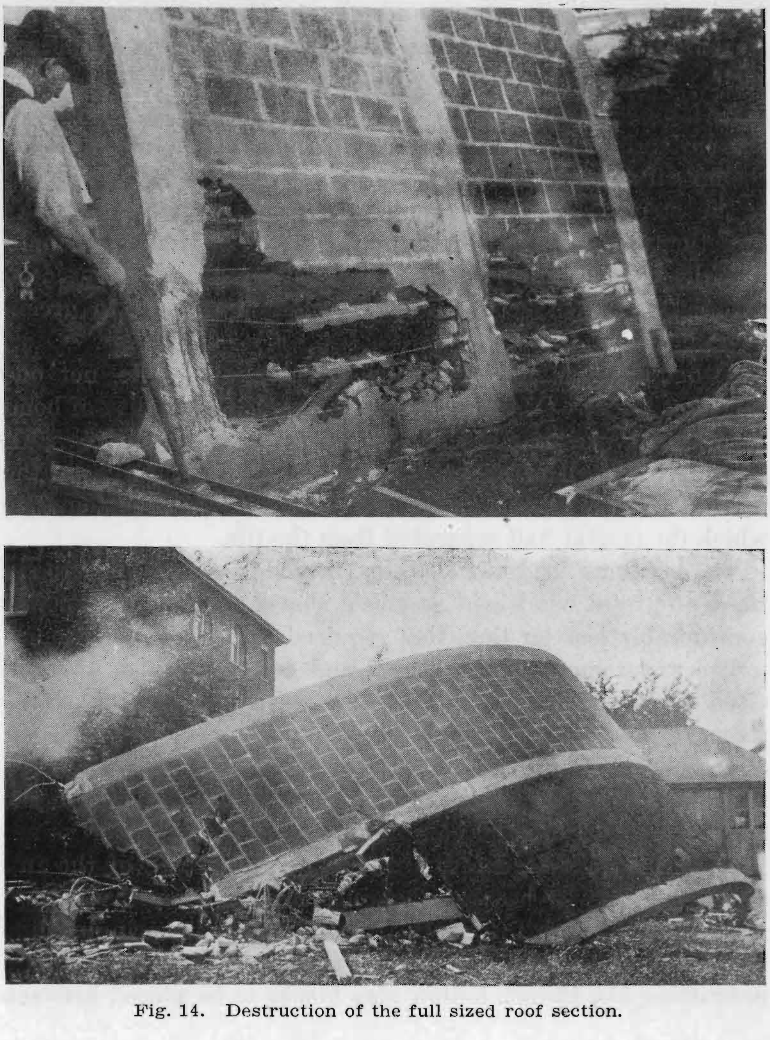


Fig. 14. Destruction of the full sized roof section.

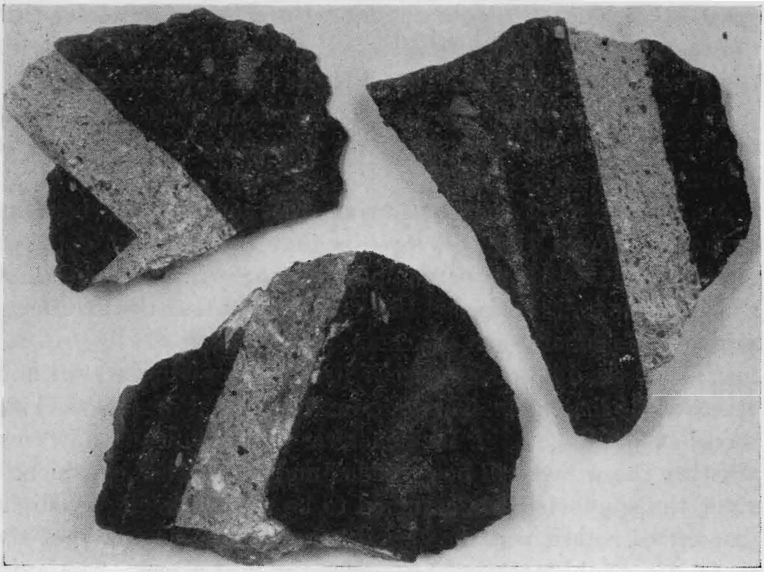
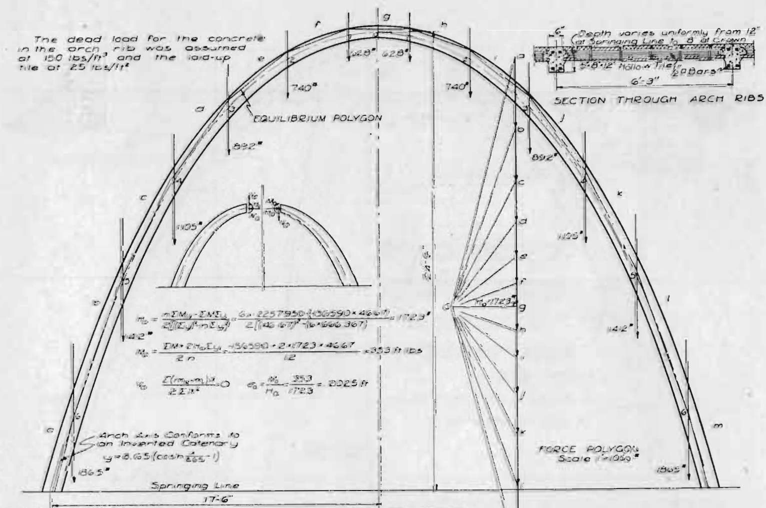


Fig. 15. The good quality of bond between the tile and the mortar is shown by these broken pieces.



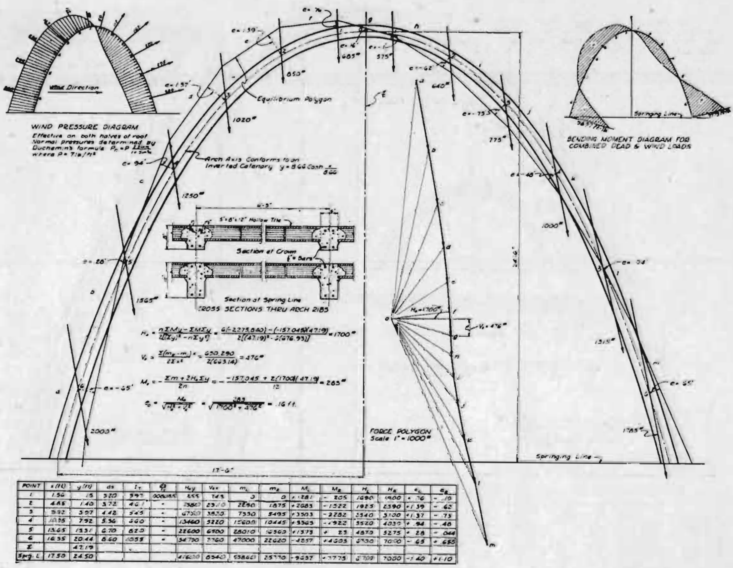
Point	x	y	ds	I	ds/I	m_L-m_R	m	my	M_0y	V_0x	M_L-M_R	M_L+M_R	e
	ft	ft	ft	ft⁴	ft⁻⁴	ft lbs	ft lbs	ft² lbs	ft lbs	ft lb	ft lbs	lbs	ft
1	1.60	.13	3.20	3.97	.00807	0	0	0	2214	0	577	1925	.316
2	4.85	1.35	3.72	4.61		-2040	-4085	-5550	2320		653	2173	.300
3	8.00	3.94	4.42	5.46		-6350	-12700	-50,100	4790		793	2950	.270
4	10.90	7.82	5.36	6.59		-12,905	-25,800	-201,800	13,150		898	3775	.234
5	13.66	13.16	6.70	8.21		-22,190	-44,380	-583,000	22,650		913	5100	.160
6	16.30	20.36	8.60	10.55		-34,810	-39,620	-1,417,900	35,050		593	6925	.086
Σ		46.67					-156,595	-2,257,950					
Spr L	17.50	24.50							42,800		-192	6925	.028

Fig. 16. Stress analysis for dead loads.

greatest at the crown, does not exceed 4 in. This indicates that the roof, conforming in shape to a catenary curve, is subjected to very small bending stresses under its own dead weight.

The wind loads used in the combined dead and wind load analysis have been discussed in a preceding section and are shown graphically in fig. 17.

An inspection of the equilibrium polygon and the bending moment diagram (fig. 17) reveals that the greatest external moment occurs at the rib support on the windward side. A check on the unit stresses at this point shows that the maximum stress in the concrete on the compression side is 838 lbs./sq. in., and the stress due to thrust is 77 lbs./sq. in., giving a total unit stress of 915 lbs./sq. in. This value exceeds the allowable working stress of 800 lbs./sq. in. for concrete. It is doubtful, however, whether this stress will be exceeded under the loads considered, since the supports were assumed to be rigidly fixed. A slight amount of rotation at the support due to the elasticity of the materials would materially decrease the unit stress. The cross sectional area of the concrete can be expected to be somewhat



POINT	x (ft)	y (ft)	ds	I_T	$\frac{ds}{I_T}$	$H_0 y$	$V_0 x$	m_L	m_R	M_L	M_R	H_L	H_R	e_L	e_R
1	1.56	.15	3.20	3.97	.000488	255	743	0	0	+1281	− 205	1690	1900	+ .76	− .10
2	4.65	1.40	3.72	4.61	-	2380	2310	2890	1875	+2083	−1522	1925	2390	+1.39	− .62
3	5.92	3.97	4.42	5.45	-	6750	3870	7330	5495	+3503	−2282	2540	3100	+1.37	− .73
4	10.35	7.92	5.56	5.60	-	13460	5220	15600	10445	+3565	−1922	3520	4030	+ .94	− .40
5	13.65	13.31	6.70	6.20	-	22600	6500	28010	16560	+1373	+ 23	4870	5275	+ .28	− .044
6	16.35	20.44	8.60	10.55	-	34790	7760	47000	22620	−4557	+4603	5700	7000	− .65	+ .655
Σ		47.19													
Sprg. L	17.50	24.50				41600	6540	55860	25770	−9457	+3775	5700	7000	−1.40	+1.10

Fig. 17. Stress analysis for combined dead and wind loads of the roof for the experimental barn.

larger due to the concrete which runs into the open ends of the tile. Hence, the arch was considered to resist the loads with the proper degree of safety, and a check on the stresses of the steel and other points along the arch shows the materials to be under much smaller stresses.

The lower structure of the experimental barn, including the mow floor, was designed to carry much heavier loads than the usual hay loads for which most barns are designed, because of the contemplated use of the barn as a grain storage building. The ceiling height was fully 2 ft. greater than the usual height of 7½ to 8 ft. Although the barn is considered a tile building, reinforced concrete was used for the important structural members. The details in fig. 18 show that the type of construction for the building other than the roof is quite conventional.

In view of studies conducted subsequent to the design and construction of the experimental barn, an analysis was made

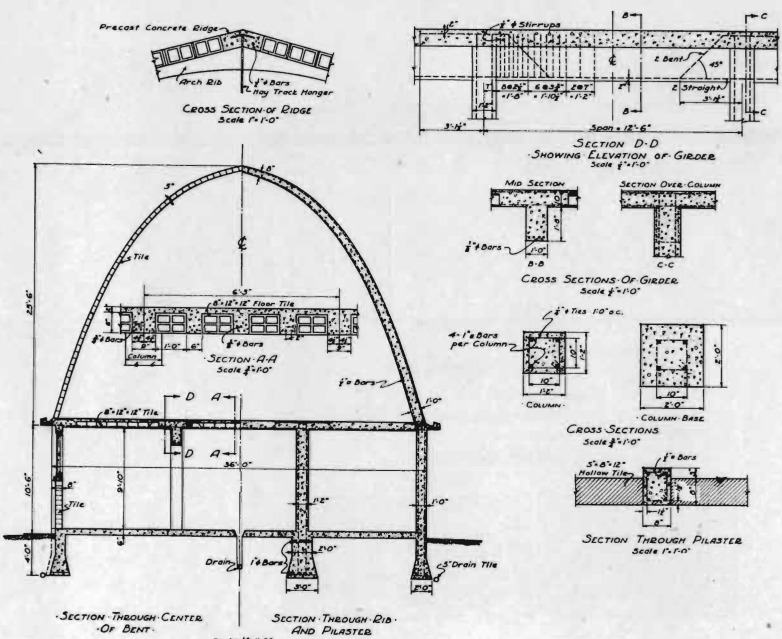


Fig. 18. Cross section and typical details of the experimental barn.

to determine the reduction in stresses with the recommended wind load assumptions and span and height of roof set forth in preceding sections of this report. (See figs. 3 and 4.)

The roof which is for a barn 34 ft. in width has a span of 31 ft. 6 in. at the mow floor, because the arch rib is continuous through the mow floor to the top of the foundation. The spacing, size and reinforcing of the arch ribs are the same as that of the experimental barn roof.

The results of the analysis for the combined dead and wind loads and other related data are given in fig. 19. The bending moment diagram shows that the greatest stress occurs at the support on the windward side. The calculated unit stresses of 356 lbs./sq. in. for concrete and 8,018 lbs./sq. in for steel are very low in comparison with the allowable unit stresses. The assumed section of the arch rib at this point is capable of resisting over twice the external bending moment of 82,400 in. lbs.

It appears, therefore, that the arch rib could be decreased

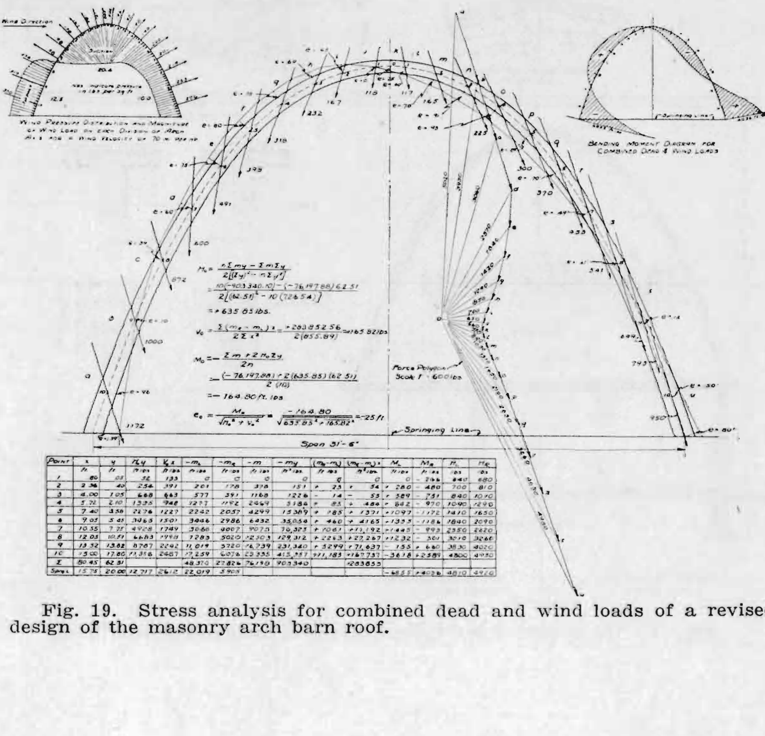


Fig. 19. Stress analysis for combined dead and wind loads of a revised design of the masonry arch barn roof.

considerably at the supports for the loads assumed. The same is true for the arch rib at the haunch and crown, since the bending moments at these points are less than one-third of that at the left support.

CONSTRUCTION PROBLEMS

The method of constructing a masonry barn, especially the roof, has been a very important consideration in the design with respect to initial cost, manipulation, transportation of forms and necessary scaffolding and handling of materials. The essential difference in the initial cost of a masonry and a wood barn is in the cost of the roof construction, rather than in the cost of materials.

ROOF FORMS

The design of this type of construction presupposes that form work will be used for casting the arch ribs and for supporting the tile. A large part of the weight of the roof must be carried by the forms until the roof is completed and all of the concrete and mortar have hardened.

Roof forms should have the following essentials: Light weight, ease of manipulation, ease of transportation and durability. The cost of constructing forms for the entire barn roof is prohibitive. Economy of roof construction might be accomplished if the forms could be used in successive units in a barn, and be moved and used in other barns.

Two types, termed the "closed" and "open" forms (fig. 20), were considered as possibilities in the construction of the experimental barn. The following description pertains to the design of these forms. Changes made in those used in the construction of the barn are indicated in the description of the construction of the model and the roof section.

The closed form has one notable advantage in that it separates into only two sections. These sections, when moved, make for speed in construction without the laborious task of handling a large number of pieces. This advantage is offset, however, by the disadvantages of moving large and very heavy pieces, not only from section to section, but from barn to barn. This type of form does not permit convenient scaffolding.

The open forms, although requiring considerable unproductive labor in erecting and dismantling in moving each unit, have a number of advantages over the other type. Scaffolding is easily accomplished by placing planks on the horizontal members of the forms. (See fig. 20.) The necessary support for laying of the tile is furnished by laying boards just ahead as the tile is being laid. The forms can be transported easily by entirely dismantling them. The amount of labor involved in moving the forms is partially overcome by leaving them in large units. These advantages seemed to warrant the choice of the open over the closed type.

The magnitude and direction of the combined dead and wind loads which either half of the forms may be expected to carry are shown graphically in fig. 21. An approximate check of the stresses in the members, which was obtained by considering either half of the forms as a truss with a span of 29 feet, shows them to be well below the allowable working stresses.

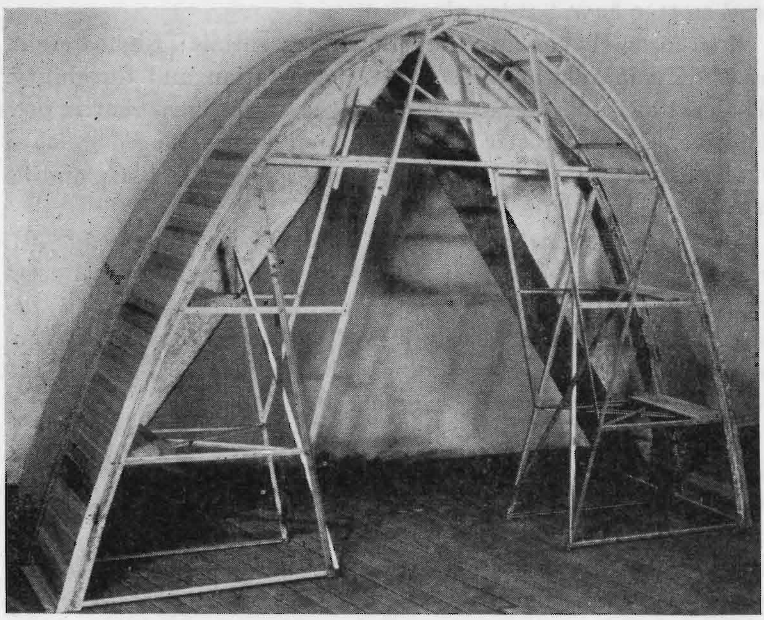


Fig. 20. The closed and open type of roof forms.

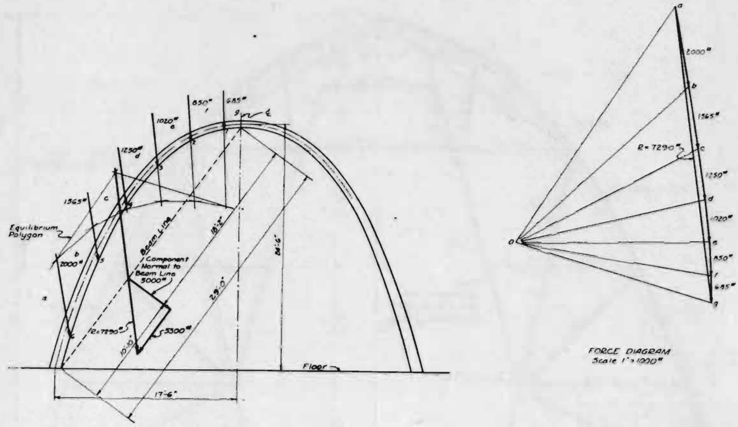


Fig. 21. Magnitude and direction of the loads to be carried by the roof forms.

Figure 22 gives a detailed elevation of the forms with the sizes of the members as indicated. The accompanying detail shows that the side plates for forming the concrete rib have been given a batter of 2 degrees to make the removal of forms easier. Although these plates are in several sections, they constitute a continuous form for the arch rib. The auxiliary framing members give added rigidity to the plates, act as supports when the forms are being moved and serve as scaffolding supports. The joint in the forms at the crown is inclined sufficiently to permit the clearing of the right half of the form as it is being removed.

Several minor changes were made in the design of the forms before they were constructed. A few of the angles in the lower part of the truss were made heavier and a few additional angles and struts were supplied to increase the rigidity as well as the strength of the forms. These changes necessitated the use of a gusset plate at one of the joints.

The total weight of the steel of the forms for one roof section is about 3,028 lbs. An estimate of the total cost, including the necessary lumber and the labor for assembling, is about $265.

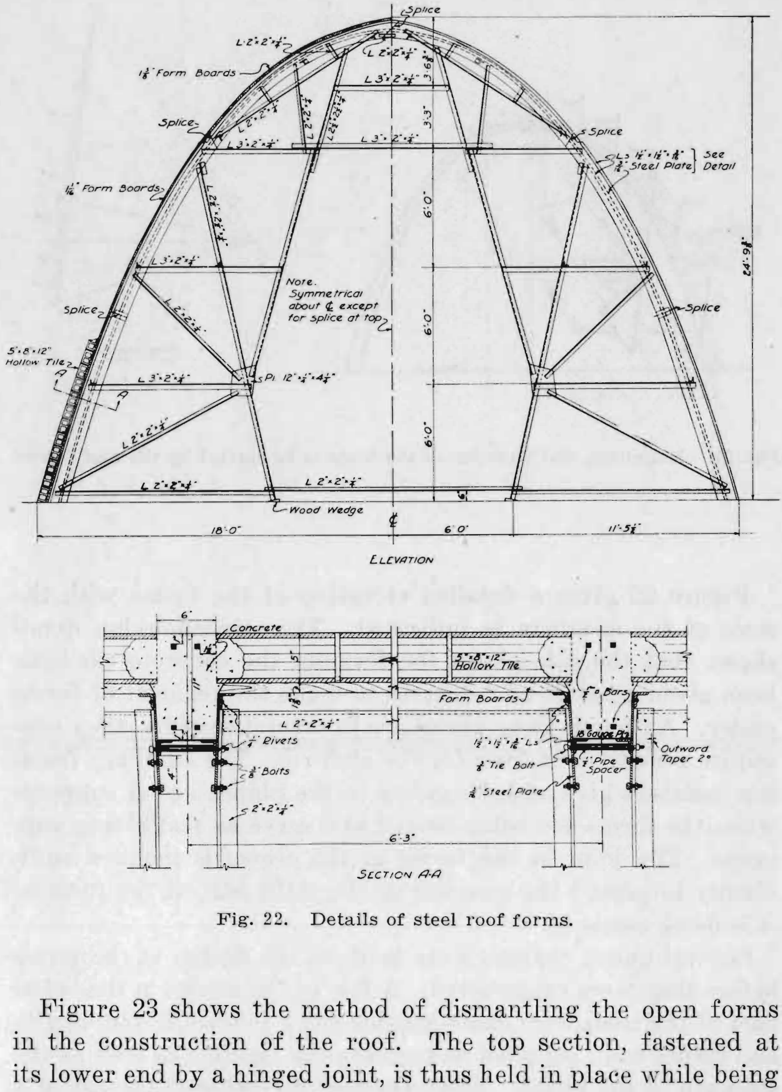


Fig. 22. Details of steel roof forms.

Figure 23 shows the method of dismantling the open forms in the construction of the roof. The top section, fastened at its lower end by a hinged joint, is thus held in place while being removed. By removing the two temporary longitudinal braces near the top, the entire form can be moved through the central opening of the adjacent form and again erected. This particular feature gives it the advantages of the closed form. A more detailed description of the manipulation of the forms is given under the construction of the roof of the experimental barn.

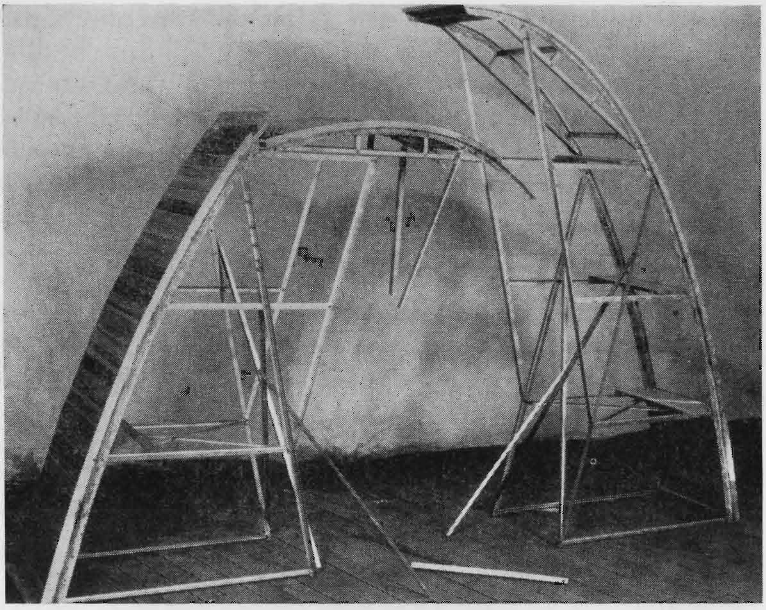


Fig. 23. The "open roof forms" partially dismantled. The boards supply the necessary scaffolding and supports for laying the tile.

MODEL ARCH

A model of one section of the roof of the proposed design was constructed to a scale (4 inches=1 foot). (See fig. 7.) In recommending a method for the construction of the roof, the following problems seemed to deserve the greatest consideration: (1) The cost of the materials and labor for the construction of the forms; (2) the necessary scaffolding for the mason; and (3) placing the materials within the mason's reach.

As a solution to the first two problems, the combined forms and scaffolding were recommended. These consisted of a truss frame work made in two sections conforming to the shapes of the roof, and kept in place at the top by a pair of jack screws. Boards of 1-inch thickness were to be placed, one at a time, as the mason worked over them from a scaffold supported by the inside framework of the forms. After completing a section of the roof, the forms would be tipped down at the top, moved to the next position, and the adjacent section would be constructed in like manner.

It was suggested that the materials be placed within the mason's reach from a carrier suspended from a cable supported by two gin poles, one on each side of the roof. The carrier could be manipulated with two ropes, one for hoisting and the other for pulling it along the cable.

FULL SIZED ARCH

The forms for the construction of the full sized arch were quite complicated, and the necessary lumber cost over $100. An attempt was made to build them in such a manner as to permit easy manipulation when erecting and disassembling, and to

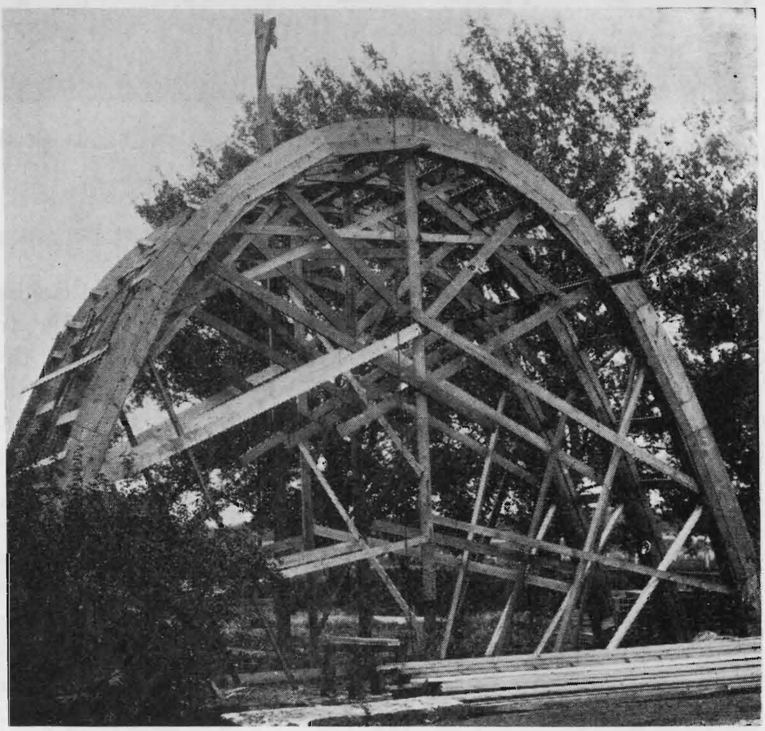


Fig. 24. Wood forms for the construction of the full sized arch.

make them usable for roofs of the same overall dimensions as well as other sections of the same roof.

Good bracing was necessary to withstand the heavy loads without objectionable deflection (fig. 24). Up until the time that a section of the arch is completed, a large part of the weight must be sustained by the form work.

The tile were laid in each section, keeping both sides of the arch about the same height in order to better distribute the loading on the forms (fig. 25). The concrete for the ribs was

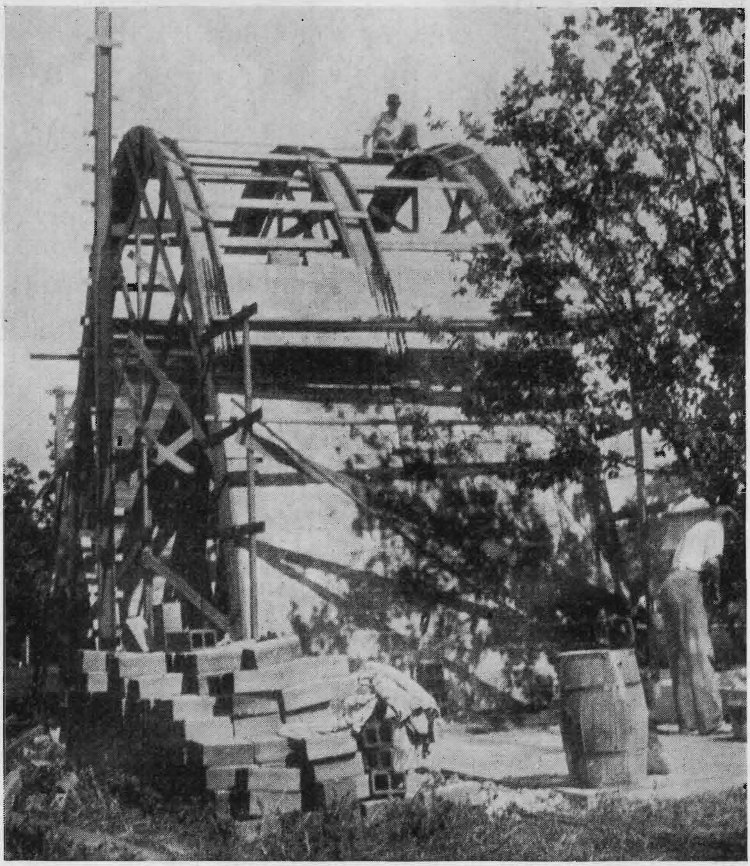


Fig. 25. Construction of the full sized arch roof section.

Fig. 26. The completed section of the full sized arch.

poured after the sections of blocks were laid. This ran back into the openings of the tile, securing a good bond between the rib and the hollow clay block sheathing. The completed section is shown in fig. 26.

EXPERIMENTAL BARN

An experimental barn was constructed on the college farm in 1926-27 (fig. 27).

The discussion of the construction which follows is confined mainly to the roof, since that of the first story does not differ materially from regular construction practices.

The roof was built in sections, beginning at one end and progressing along the length of the barn, building one section per day. By this method the roof could be built for any length of barn with two sets of forms. The following general steps were necessary in building the roof: (1) Assembling of the forms, (2) laying of the tile, (3) pouring of the arch ribs, and (4) the disassembling of the forms.

The lower trusses of the forms were assembled individually on the floor of the barn before placing them upright in position. The separate trusses were bolted together with struts before any of the parts of the upper truss were added. The scaffolding on the lower section of the trusses could be used to advantage in erecting the upper truss. (See figs. 28 and 29.)

The forms were placed at one end of the barn floor and the ends of the barn roof, with its windows and doors, were constructed at the same time that the adjacent sections were being built (fig. 30). Scaffolding could be placed easily on the forms for the construction of the ends. Wood forms were used for the pilasters and cornices.

With the forms in place, the tiles were laid. Boards, 2x6-inch, were laid in the forms ahead of the tile to allow laying operations from the scaffolding on the inside. A piece of straightened no. 7 wire about 7 ft. long was laid in every mor-

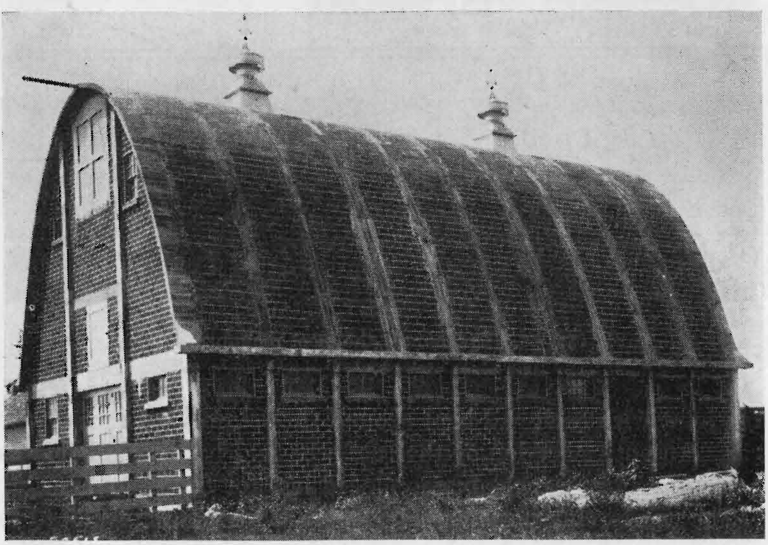


Fig. 27. The completed experimental barn.

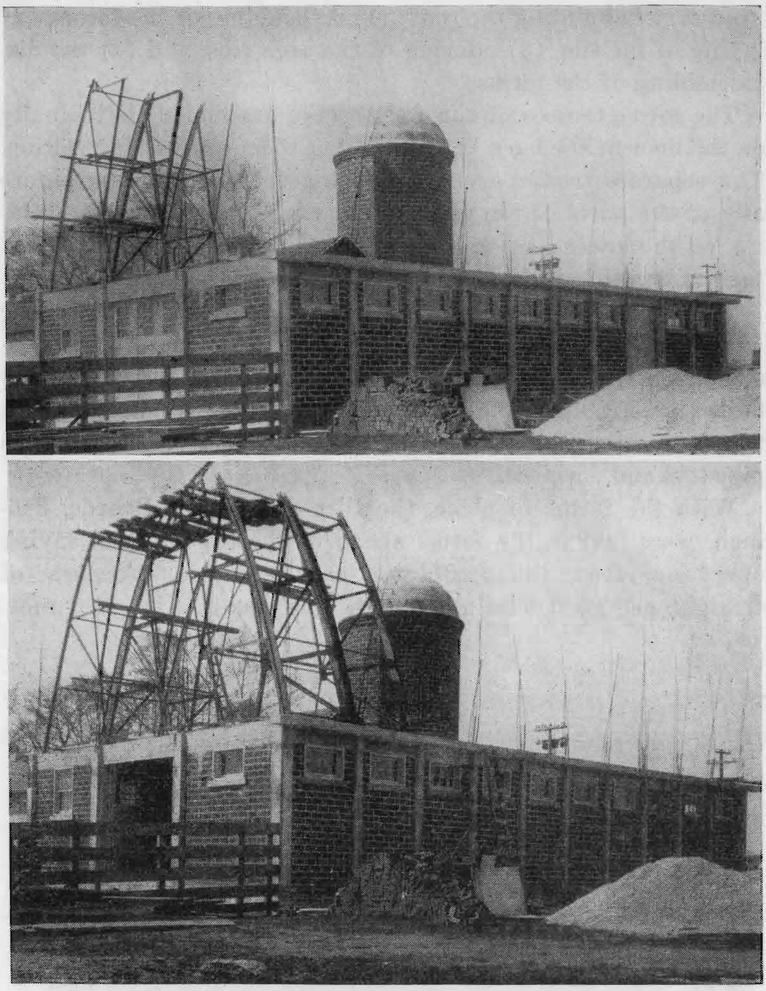


Fig. 28. Erection of the steel roof forms.

tar joint with each end bent around the reinforcing rods of the rib. An entire section of the roof was laid before the rib was poured. A concrete slab was placed last for the ridge.

The reinforcing bars of the arch rib were bent and placed to conform to the shape of the roof at the same time that the tiles

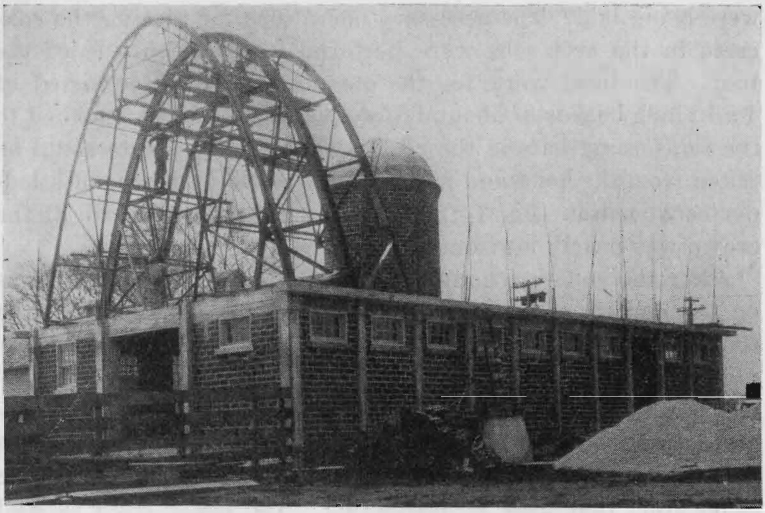


Fig. 29. The steel forms in place.

Fig. 30. Beginning of the construction of the roof.

were being laid. The necessary operations for placing the concrete in the arch ribs were performed on the outside of the roof. The form work for the outside of the rib consisted of 1 x 10-inch boards of about 14-foot lengths, the ends fastened to the reinforcing bars in the rib by wire. The concrete could be taken from the hoist and poured into the rib by means of a ladder construction (fig. 31). Since the portion of the rib at the crown was nearly horizontal, no forms were necessary.

After the concrete had been permitted to harden, the forms on the outside of the ribs and the steel forms supporting the roof from below were removed. Since the materials in a completed section will be self-supporting, the forms could be removed after a short time of hardening of the mortar and concrete without subjecting the structure to severe stresses. In general, the forms were removed by proceeding in the reverse order from that used in assembling. The lower truss as well as the upper was disassembled, piece by piece, instead of being removed as a unit. The parts so removed were then placed in immediate readiness for use in the construction of the next adjacent section.

Considerable difficulty was experienced in removing the forms from the ribs adjacent to the first section constructed, because of the bonding of the concrete to the flange of the steel forms. This difficulty was overcome by cutting off a part of the flange. The concrete was then permitted to run against the ends of the 2 x 6-inch planks which support the "laid-up" tile. (See fig. 32.)

WATERPROOFING PROBLEMS

The roof of the experimental barn, as well as the full sized arch, has been found to leak rather freely in several places, due to the development of cracks where the tile joins the reinforced concrete ribs. Frequent expansion and contraction due to changes in temperature cause cracks and make it necessary to provide a waterproof coating which will be sufficiently elastic to avoid breaking of the coat. Some attempts have been made throughout this investigation to devise a practical as well as economical waterproofing, which will either preserve the

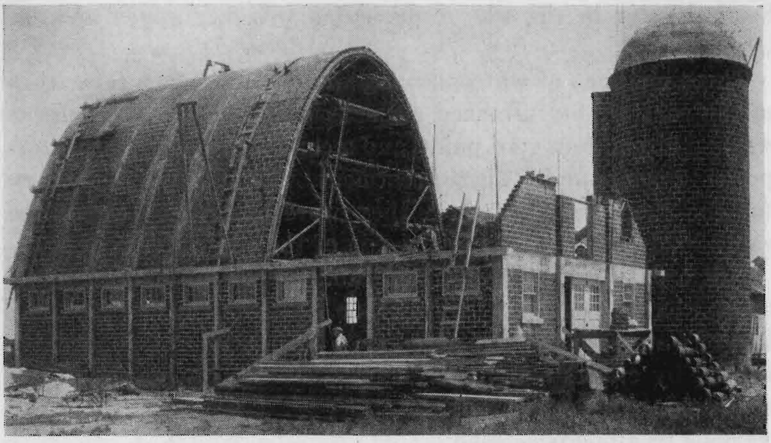


Fig. 31. The masonry arch roof under construction.

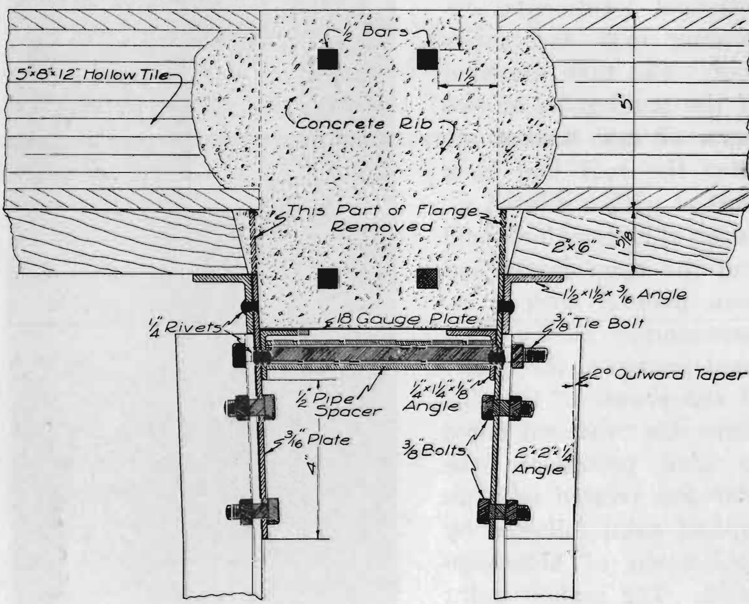


Fig. 32. Section of roof form of arch rib in place.

natural color of the tile or present a pleasing appearance in itself.

Three methods of waterproofing were tried on separate areas of the roof of the full sized masonry arch, namely: A cement wash, a bituminous roof paint, and a commercial integral waterproofing compound. These treatments were fairly satisfactory for the short time in which they were observed. Dampness could be detected on the lower side of the roof after rains.

Numerous leaks were observed under the roof of the experimental barn after rains, and it soon became evident that some sort of waterproofing was necessary. The bad leaks, apparently caused by cracks along the ribs, were more numerous on the south side. A majority of the leaks which occurred after the various treatments seemed to result from cracks, exposed by the failure of the waterproof coatings.

The roof received two principal treatments, extending over the entire roof. The first consisted of the application of two coats of raw linseed oil, after the roof had been washed with a diluted solution of hydrochloric acid and the open joints had been pointed with a cement mortar. Such a treatment preserved the natural appearance of the tile. Since this treatment failed to give protection, the roof was treated with an asphalt paint followed by application of aluminum paint. The asphalt paint was of a consistency which permitted it to be

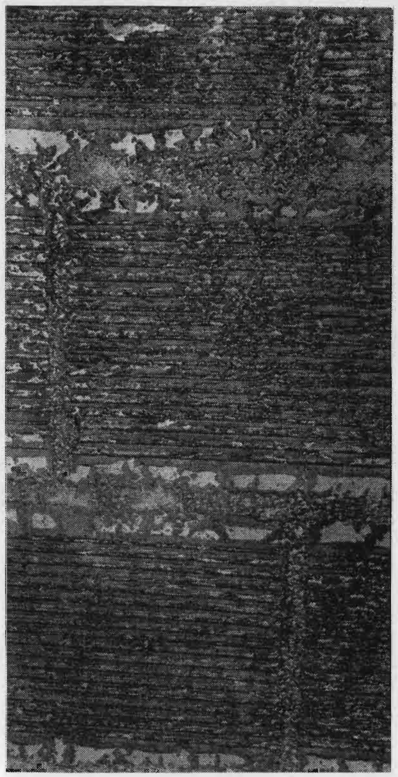


Fig. 33. Poor condition of the asphalt and aluminum paint.

Fig. 34. Treating of the leaky areas with fibered asphalt and aluminum paint.

applied with a paint brush. It dried rapidly upon application and failed to exhibit the elastic properties of asphalts. Consequently, the complete treatment failed to give proper protection (fig. 33).

In furthering the attempts to seal the cracks and otherwise make the roof watertight, a heavier and fibered asphalt was used in treating the leaky areas (fig. 34). Its consistency was such as to permit application with a brush. The leaky areas which had been noted and other visible cracks were treated with one coat of this material. A coat of aluminum paint was applied after sufficient drying.

After almost a year's exposure, further leaks, some of which appeared after the failure of the previous treatment, were in evidence. These were temporarily repaired with the same fibered asphalt and aluminum paint.

The linseed oil and the asphalt paint have proved to be unsatisfactory as waterproof coatings for the barn roof. The condition of the coatings on the south side was much worse, apparently due to the more severe sun exposure. Both treatments after a year's exposure failed to give the necessary protection. The linseed oil was largely gone and that which remained had curled and was valueless. Similarly the asphalt had hardened quickly in drying and later had curled. Nearly all of the asphalt and paint were removed over a large part of the roof on the south side.

The condition of the fibered asphalt was fairly good after a year's exposure. In a few places it had become firm and dry and had partially cracked, exposing former cracks.

Raw linseed oil and asphalt paint are not satisfactory for roof waterproofing, since they are not able to withstand, without breaking, the heat of the sun and the expansion and contraction of the joints. A heavier and a fibered asphalt seems to be more satisfactory for sealing the cracks.

COST OF MATERIALS FOR THE ROOF

An estimate of the cost of the materials used in the construction of the masonry roof of the experimental barn is presented in table 2. Table 3 gives an estimate of the cost of materials in the part of the structure above the mow floor of a typical gambrel roof barn of braced rafter construction, which is very nearly the same in overall dimensions and cross sectional area as the roof of the experimental barn. Plan no. 72121 of the Midwest Farm Building Plan Service Catalogue (2) was selected as a typical gambrel roof barn.

The unit prices used in both tables are those for which ma-

TABLE 2. ESTIMATE OF MATERIAL COST OF THE ROOF OF THE EXPERIMENTAL BARN 36'x64'-6", HEIGHT 24'-6", CROSS SECTIONAL AREA 632 SQ. FT.

Kind of material	Quantity	Unit price	Cost
1. Structural clay tile 5x8x12"	7,248	$85.00/M	$616.08
2. Portland cement (sacks)	290	.59	171.10
3. Sand (yds.)	20.4	1.50	30.60
4. Gravel (yds.)	23.8	1.50	35.70
5. Lump lime (bbls.)	1.5	2.75	4.13
6. Reinforcing bars (lbs.)	5,679	.03	170.37
7. Drawn steel wire No. 7 (lbs.)	620	.05	31.00
8. Dimension lumber (bd. ft.)	100	55.00/M	5.50
9. Waterproofing			
Asphalt paint (gals.)	40	.75	30.00
Aluminum paint (gals.)	5	4.95	24.75
10. Ready mixed paint (gals.)	2	3.30	6.60
11. Doors			
5'-0" by 5'-9" panel doors	2	4.00	8.00
8'-0" by 10'-0" panel hay door	1	12.00	12.00
12. Windows			
2'-10½" by 4'-6" by 1 3/8" 12 lt. 2 piece	5	2.76	13.80
13. Roof ventilators, 24-inch flues	2	54.00	108.00
14. I-beam, 3-inch St. 7.5#/ft., 20 ft. (lbs.)	150	.04	6.00
15. Wire for tying reinforcing No. 11 (lbs.)	25	.06	1.50
16. Hardware			
14-inch strap hinges (prs.)	2	.90	1.80
10-inch T-hinges (prs.)	2	.65	1.30
6-inch hooks and staples	4	.10	.40
Nails (lbs.)	10	.04	.40
		Total cost	$1,279.03

TABLE 3. ESTIMATE OF MATERIAL COST OF THE PART OF THE STRUCTURE ABOVE THE MOW FLOOR IN A GAMBREL ROOF BARN (MIDWEST PLAN NO. 72121) 36'x64', HEIGHT 23'-6", CROSS SECTIONAL AREA 637 SQ. FT.

Kind of material	Quantity	Unit price		Cost	
1. Dimension lumber No. 1 com. (bd. ft.)	7,746	$50.00/M	______	$387.30	______
2. Rough boards No. 1 com. (bd. ft.)	342	55.00/M	______	18.81	______
3. Surfaced boards No. 1 com. (bd. ft.)	103	53.00/M	______	5.46	______
4. Drop siding, 1x6" No. 2 com. (bd. ft.)	2,510	55.00/M	______	138.05	______
5. Roof sheathing 1x8" shiplap No. 3 com. (bd. ft.)	4,610	42.00/M	______	193.62	______
6. Wood shingles No. 1—16"—5/2" (sqs.)	39	5.40	______	210.60	______
7. Matched lumber 1x6" No. 1 com. (bd. ft.)	88	55.00/M	______	4.44	______
8. Windows 9 lt. —9x12"single sash	4	2.70	______	10.80	______
9. Ready mixed paint (3 coats) (gals.)	10	1.10	______	11.00	______
10. Roof ventilators 24-inch flues	2	54.00	______	108.00	______
11. Galvanized ridge roll (ft.)	70	.05	______	3.50	______
12. Hardware					
Bolts 1/2x10"	16	.05	______	.80	______
Steel track for hay door (set)	1	6.80	______	6.80	______
Guard rail for hay door (ft.)	20	.15	______	3.00	______
Nails (lbs.)	615	.04	______	24.60	______
		Total cost		$1,126.78	______

terials would be delivered on the job in Ames, Iowa, in the summer of 1935. Blank spaces are supplied in the table for substitution of different unit prices and costs of each item, since these vary with different localities.

The total cost values presented in the tables show that the estimated cost for materials of the masonry roof is nearly $1,280, which is 13½ percent greater than the estimated cost for materials of the structure above the mow floor in a gambrel roof barn of wood type construction. This shows that the additional initial cost including labor, of a masonry roof is due, not so much to the cost of material, but rather to the cost of additional labor and the amount chargeable to equipment.

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