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EVALUATION OF A 20-FT BY 48-FT STRAIGHT-SIDED, ARCH ROOF, PREFABRICATED METAL BUILDING, NORTHERN TYPE, MANUFACTURED BY THE GREAT LAKES STEEL CORPORATION, STRAN-STEEL DIVISION

W. R. Mason and J. E. Schroeder

U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California

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Approved
J E Lorraine

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SUMMARY

The Great Lakes Steel Corporation, Stran-Steel Division of Detroit, Michigan, 20-ft by 48-ft straight-sided, arch roof, prefabricated metal building, northern type was evaluated by the Laboratory to determine its adequacy for advanced-base use. The evaluation included a study of packaging and crating; erection; weathertightness; and structural adequacy for specified snow, wind, and floor loadings.

It was found by test and observation that the building can be packed for overseas shipment in 230.4 cu ft with a gross weight of 10,010 lb; erection of the building was accomplished in the high erection time of 164.9 manhours, using a five man team; the building was essentially watertight; and was structurally adequate for the specified loading, high tensile steel was used in the structural framing members. The high erection time was due to several design deficiencies.

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ILLUSTRATIONS

INTRODUCTION

To facilitate the rapid construction of Naval Shore Establishments, a need exists for a standardized prefabricated, 20-ft by 48-ft, northern type building that is easily manufactured, economical in the use of materials, structurally adequate, lightweight, low in shipping cube, rapidly erected by unskilled labor and suitable for re-erections. As part of a program to develop a suitable building, commercial designs meeting the requirements are being evaluated by the U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California.

This technical memorandum covers the evaluation of a 20-ft by 48-ft straight-sided arch roof prefabricated metal building, northern type, manufactured by the Great Lakes Steel Corporation, Stran-Steel Division, Detroit, Michigan. Studies and tests to determine the adequacy of packaging and crating, ease of erection, weathertightness, structural adequacy were made by the Laboratory under Project NY 500 002-22 authorized by the Bureau of Yards and Docks.[1]

DESCRIPTION

The Great Lakes Steel Corporation, Stran-Steel Division, straight-sided arch roof, as shown in Fig. 1, is a prefabricated steel structure, 20-ft by 8-1/4 in. wide, 48-ft long, and 11 ft-2 in. high at the peak of the arch. The building can be placed either directly on the ground on a level site or on five rows, five feet on centers of posts or blocking spaced within the rows six feet on centers running longitudinally the length of the building.

The prototype, as furnished for test, consisted of a floor system, composed of five longitudinal sills spaced 5-ft on centers and twenty-five transverse joists spaced 2-ft on centers; a frame composed of thirteen rigid frames and six purlins; exterior corrugated sheeting; insulation interior fiberboard liner; and a plywood floor.

The sills are 3-5/8 in. special I-beam type members formed in one piece by a Yoder mill each 48-ft long sill is composed of four sections connected together with splice plates. The transverse 20-ft 8-in., 2-in. deep joists are fastened to the sills with sheet metal screws. Channel plates are provided for attachment of the rigid frames, these channels run the entire length of the building along each side and are attached to the top of the joists with sheet metal screws. The rigid frames consisting of two straight columns and a 20-ft radius arch roof beam are jointed together at the eaves by means of channel splice plates and bolts, and are attached to the channel plates with sheet metal screws. These frames are spaced four foot on centers and connected together with six purlins spaced across the arch roof. Sheet metal screws are used to attach the purlins to the frames. The joist, frames and purlins are I-beam type sections made by welding two special shaped channels back to back, providing a nailing space between channels, the frame members are made of high tensile steel (50,000 psi minimum yield). The end walls are formed with four studs with a head buck placed between the two center studs. Crating lumber is used between the frames and between studs to form window headers and sills and girts for the support of the wall board. The floor joists are covered with thirty 4-ft by 8-ft by 1/2-in. plywood panels held in place with metal splines and hook nails. The plywood panels are laid in a new pattern, two sheets running transversely with one longitudinal panel in the center. The exterior is sheeted with 26 gage mild-steel corrugated sheeting placed horizontally on the sidewalls and curved sheets transversely across the purlins on the roof. This sheeting is secured with 6d double-headed galvanized nails and lead washers. Special sheet metal flashing is used to close the joints at the eave and end walls. The interior of the building is finished with precut fiberboard held in place with metal splines along all horizontal joints and special nails and fiberboard battens along all transverse joints. Blanket-type fiberglass insulation is placed between the exterior sheeting and the interior liner, and is held in place with crating lumber.

Interior natural light was provided by three fixed translucent, corrugated plastic windows along each side wall and two in each end wall. Access is provided by a 3-ft by 6-ft 8-in. door located in the center of each end wall. Natural ventilation is furnished through screened louvers in the lower half of each door and a field assembled 20-in. diameter ventilator. Vertical sliding panels are located on the outside of each door for closing the door ventilators. Provisions for heating are provided with one 6-in. diameter smoke stack located on the roof. A wiring harness is provided for artifical interior lighting and electrical power.

The building as received was packaged for export shipment in 10 crates, the crates have a gross weight of 10,010-lb and cubes in 230.4 cu ft.

TEST FACILITIES

A special facility (see Figure 2) was built for testing 20-ft by 48-ft buildings.[2] The facility consists of three test stations, one for erection studies, one for structural tests, and one for weathertightness tests. For movement of buildings between these stations, a 330-ft long, 20-ft gauge, I-beam track was built to connect the stations; and a 21-ft by 50-ft flatdeck steel transfer cart was mounted on the track.

At station one, erection studies are made on the transfer cart with blocking placed on the deck of the cart to simulate foundation conditions. A 10-ft wide work platform was built along the track to simulate a natural ground height around the cart.

At station two, simulated wind and snow loadings are applied through hydraulic cylinders attached to a bent of Navy pontoon strings framed over the track. A special hydraulic pump and control panel are used to actuate the cylinders. Loads are applied to a building by the cylinders through pads, whiffletree systems, (see Figure 3), or combinations of both.

Floor loading tests are made on a special floor-testing jig using hydraulic cylinders in a manner similar to the wind and snow loadings. All strains are measured with SR-4 gages, type A-1, and observed in microinches on a 48-channel Foxboro scanner recorder. Deflections are measured in hundredths of inches with deflectometers, direct-reading dials connected to the building with a flexible wire, developed at the Laboratory.

At station three, an overhead sprinkling system is used to simulate a 2-in.-per-hour rainfall over the entire roof of a building. A portable Buffalo turbine sprayer-duster is used to simulate a 35-mile-per-hour wind driven rain along one side wall. For a windbreak, a high wall of pontoons was built around three sides of the station.

The test facilities permit simultaneous testing of three buildings.

TEST PROCEDURE

Each package of the building as received from the manufacturer was weighed, cubed, and reviewed for adequacy of crating and for compliance with naval storage requirements. Further, the packaging scheme was studied to determine possible combinations of packages, cubage, and weight reductions, and the suitability of packaging for convenient distribution around the erection site.

Three erection studies were made to determine the suitability of the building for assembly under advanced-base conditions. To simulate these conditions, a team comprised of naval enlisted personnel was used for all erections. The first erection was observed to determine the ease of erection with personnel unfamiliar with the procedure; to establish the most efficient size for the erection team; to check the accuracy of fabrication; to study the adequacy of the erection manual; to establish the tool requirements for efficient erection; and to record the man-hours required to assemble the floor, frame, exterior sheeting, end walls, insulation, and interior wallboard.

Following the weathertightness and structural tests, the building was completely disassembled, and a second and third erection and disassembly were made to determine the suitability of the design for repeated assembly and to record the erection manhours required as the team became more familiar with the building.

Upon completion of the first erection, the building was weather-tested. The overhead sprinkling system was turn on, and observations were made on the inside of the building to determine leaks resulting from a direct downpour. When sufficient time had elapsed to locate any possible leaks in the roof, the portable Buffalo turbine sprayer-duster was turned on and moved slowly along the side of the building at different elevations. The direct downpour fell all during the wind-driven rain test. Observations were made to locate air and water leakage along the side walls.

Structural tests were made to determine the adequacy of the building to withstand the 20-psf snow, and the 70-mph wind, and the 40-psf floor design loadings stipulated in the Military specifications.[3] A unit stress 27,000 psi was allowed by the specifications at design load for the lightgage, high tensile, 50,000 psi minimum yield steel framing members used in this building. To test the framing members under the most unfavorable condition and to observe them under load, all structural tests were performed without the interior wallboard.

Previous tests on framed structures have shown that the strains and deflections occurring in the center portion of the building are independent of the end walls. Because of this knowledge, a part of each end of the building was removed; and only a 16-ft center section, which spanned 5 of the frame assemblies, was used for the simulated snow and wind-loading tests. This design snow-loading condition diagrammed in Figure 4 was applied hydraulically to the roof through pads as shown in Figure 5. The design wind-loading condition diagrammed in Figure 6 was applied hydraulically to the roof and side walls through pads as shown in Figure 7. In each case, loadings scheduled to reach 200 per cent of design load were applied in 10 per cent increments until excessive strains and deflections were observed. Location of the strain and deflection gages used in these tests are shown in Figures 8 and 9.

For the simulated floor-loading test, a 16-ft by 20-ft section of the floor was selected for test. This section of floor covered the entire 20-ft width and a 16-ft length of the building. The design floor-loading condition was applied hydraulically to the floor through pads and whiffletrees. The loadings were applied in 10 per cent increments up to 200 per cent of design load. Location of strain and deflection gages used in this test is shown in Figure 10.

RESULTS AND OBSERVATIONS

The test results and general observations made during the evaluation of this building are covered in the following paragraphs.

Packaging and Crating

A summary of the packages, cubes, and weights is given in Table 1. The manufacturer's packaging scheme was found acceptable for handling and site layout. Further, the building parts were properly grouped to secure minimum weight and cube for the crated building.

In general, the crating was adequate and complied with the military specifications for crating and packaging,[3] with the exception of crates 4 and 8. A conference between the Laboratory and the Advanced Base Storage Depot, Port Hueneme, California, resulted in an agreement that these crates should be redesigned to afford more protection to the arch ribs and curbed sheeting. Further, it was observed that the battens on crate 7 should be moved to within 6-in. of each end to stabilize this crate for stock piling.

Erection

A summary of the manhours required to erect the building in each of three erections and a breakdown of the manhours required in the third erection is given in Table 2. The table shows that in the third erection a team erected the building 164.9 manhours.

The type of construction, nailed attachment of the exterior sheeting and the employment of many small sheet-metal screws, made the first erection relatively slow, requiring 220.0 manhours. During this erection, it was found that a five-man team was required for efficient erection. Since no erection manual was furnished with the building, the construction drawings for this building and the erection manual for the Standard Arch Rib Building were used making deviations in the manual where necessary. This slowed the first erection to a large degree. The same difficulty was encountered with the erection of this building as with the standard Arch Rib Building, namely, the difficulty in locating the nailing slot and driving the nails in the plywood floor and the exterior sheeting. Additional difficulty was encountered in nailing both the exterior sheeting and the interior liner at the eave connection as the splice plates and bolts drew the nailing slot closed making it impossible to drive nails in this area. Considerable difficulty was also encountered in attaching the eave purlin, the bolts holding the purlin to the rigid frames being held in position by the splice plates between column and roof beam (see Figure 11) were easily bent and broken and turned when the nut was tightened. Difficulty was also encountered in matching the corrugations of the roof sheets with the corrugations in the eave flashing, a common occurrence, since the corrugations of commercial corrugated sheets are made to varying dimensions by various fabricators. The new plywood floor pattern, although saving two rows of metal splines, was very flexible especially at the edges near the side walls.

It was also observed that the 45 foundation points required to support the sills on blocking or posts makes preparation of the foundation time consuming and the leveling of the large number of posts difficult.

Considerable difficulty was experienced in disassembly because of the large number of nailed connections, the exterior sheeting suffering extensive damage during the disassembly process. The flanges of many of the structural shapes became bent during handling, thereby decreasing their structural load carrying capacity.

Weathertightness

The roof proved watertight and did not leak after 30 minutes of simulated direct downpour rain. The side walls also proved watertight except at the eave where the flashing and roof sheets fitted poorly due to the mismatching of corrugations. The black mastic, although adequate for weatherproofing, is messy and difficult to apply and is very difficult to remove during disassembly.

Structural Adequacy

The stresses and deflections recorded in the snow, wind, and floor loading tests are summarized in Table 3. The rigid frame members are made from a high tensile steel having a minimum yield of 50,000 psi. All other members were made of mild steel.

The snow loading was taken to 200 per cent 40 psf of design and then discontinued, as this was the maximum for all loadings scheduled for this building. At design load, a maximum stress of 17,400 psi tension, as compared to the 27,000 psi allowable, was recorded at point 8 (see Table 4); and at 200 per cent of design, a maximum stress of 32,400 psi tension was recorded at the same point. The maximum recorded deflection at design was a vertical movement of 0.95 in. at point 5 (see Table 5); and at 200 per cent of design, a maximum vertical deflection of 1.91 in. was recorded at the same point.

The wind loading was taken to 200 per cent (100 mph) of design. At design load, a stress of 18,600 psi tension as compared to the 27,000 psi allowable, was recorded at point 3 (see Table 4); and at 200 per cent of design, a maximum stress of 33,600 psi tension was recorded at the same point. The maximum deflection at design load was a horizontal movement of 1.15 in. at point 5 (see Table 5), and the maximum horizontal movement at 200 per cent of design was 2.82 in. at the same point. No damage occurred to the building during either the snow or wind loadings.

The floor loading test was taken to 200 per cent (80 psf) of design. At design load, a maximum stress of 15,400 psi, compared to the 21,500 psi allowable, was recorded at point 4; and at 200 per cent of design a maximum stress of 27,800 psi was recorded at the same point. The maximum recorded deflection at design load was a vertical deflection of 0.11 in. and at 200 per cent a maximum deflection of 0.24 in. was recorded at the same point. The deflections at design load on both the sills and joist were below the allowable of 1/240.

CONCLUSIONS

The design of the building has undesirable features which include the following:

1. The use of arch shaped roof members requires additional cube for roof beam and exterior roof sheet packages. Adaptability of this design for interchangeability with wood is difficult.

2. The nailable section does not lend itself to easy demountability.

3. The building is difficult to erect.

The following deficiencies should be corrected.

1. The unsatisfactory connection between the purlin and the rib at the eave.

2. The unsatisfactory nailability of the column and roof beam at the eaves, due to the splice plates squeezing together the web of the column and roof beam.

3. The unsatisfactory eave flashing sheet on which the corrugations do not match the corrugated sheeting.

4. The unsatisfactory plywood floor pattern (two sheets running transversely and the one in the center running longitudinally).

5. The unsatisfactory method of supporting and attaching the 1/8" hard pressed fiberboard interior liner.

REFERENCES

1. RDB Project Card No. NY 500 502 Prefabricated Advance Base Buildings, 15 May 1949.

2. R. K. Steele, "Production-line Tests Maul Prefabs," Engineering News-Record, vol. 148, no. 1, 3 January 1952, p. 30-32.

3. MIL-B-15969 (Docks) Military Specifications "Building, Prefabricated Metal Arch Rib 20-ft by 48-ft Northern Type," 4 January 1951.

Table 1: Summary of Packages, Cubes and Weights

Item	Units	Prototype 3 Great Lakes St. - Sided	Standard 3 Arch Rib
Packages	no.	10	10
Cubes of Crated Building			
Maximum Crate	cu ft	62.00[2]	51.7[2]
Minimum Crate	cu ft	9.00	8.9
Total Cubage[1]	cu ft	230.40	226.0
Weight of Crated Building			
Maximum Crate	lb	1,650.0	1,624.0
Minimum Crate	lb	350.0	178.0
Total Weight[1]	lb	10,010.0	9,268.0
Steel Weight			
Floor System	lb	1,410.6	1,410.6
Frame	lb	2,214.0	1,243.7
Exterior Sheeting	lb	2,125.0	1,942.2
End Bulkhead	lb	510.0	441.9
Misc.	lb	160.0	109.5
Total		6,419.6	5,147.9

1. Operable windows add 485 lbs to total weight; 25.59 cu ft to the total cube.

2. Cubes, based on nesting 10 buildings for arch roof members and arch roof sheets, add 10.3 cu ft for the straight-sided building and 8.8 cu ft for standard arch rib building.

3. Non-operable windows.

Table 2: Summary of Erection Time Studies

Item	Prototype[1] Great Lakes Str.-Sided	Standard[2] Arch Rib
Entire Building		
1st Erection	220.0[3]	148.3
2nd Erection	180.0	130.0
3rd Erection	164.9	120.1
Components		
Floor System	10.0	5.6
Frame	22.0	5.8
Exterior Sheeting	66.5	56.2
End Walls	20.0	10.0
Insulation	12.0	10.2
Interior Wallboard	28.1	26.0
Vent & Stacks	1.3	1.3
Wiring	5.0	5.0
Total	164.9	120.1

1. Operable Windows.

2. Fixed Windows.

3. The first erection was accomplished from the drawings only, due to the lack of an erection manual.

Table 3

Summary of Structural Tests

The recorded values given in this table represent only live-load values. Allowable units stress = 27,000 psi

Item	Units	Snow	Wind	Floor
Design Loading	As noted	20 psf	70 mph	40 psf
Maximum Loading	As noted	40 psf	100 mph	80 psf
Load Increment	per cent design	10	10	10
Stresses (design load)				
Tension				
Magnitude	psi	17,400	18,600	15,400
Location	No.	8	3	4
Compression				
Magnitude	psi	10,950	9,300	----
Location	No.	12	10	----
Stresses (Maximum Load)				
Tension				
Magnitude	psi	32,400	33,600	27,800
Location	No.	8	3	4
Compression				
Magnitude	psi	19,800	20,100	----
Location	No.	12	10	----
Deflections (Design Load)				
Vertical				
Magnitude	in.	0.95	1.15	0.11
Location	No.	5	5	4
Horizontal				
Magnitude	in.	0.58	0.87	----
Location	No.	8	2	----
Deflections (Maximum Load)				
Vertical				
Magnitude	in.	1.91	2.82	0.24
Location	No.	5	5	4
Horizontal				
Magnitude	in.	1.19	2.00	----
Location	No.	8	10	----

Table 4

Recorded Liveload Stresses on the Rib for Simulated Snow and Wind Loadings

Units: psi, + tension, - compression, Es = 30 x 10^6 psi for converting strain to stress. See Figure 8 for location of strain gages.
(+ = tension, - = compression)

Point	SNOW LOADING Per Cent Design Load				WIND LOADING Per Cent Design Load			
	50	100	150	200	50	100	150	200
1	- 600	- 900	- 900	- 900	+ 900	+ 1800	+ 2700	+ 3000
2	-3300	- 7050	- 9600	-12,300	+4800	+12,000	+15,600	+23,700
3	-4500	-10,350	-14,250	-18,150	+8400	+18,600	+27,000	+33,600
4	-2400	- 6000	- 8150	-10,500	+2700	+ 6300	+ 9300	+13,500
5	+2400	+ 5700	+ 7650	+10,200	-3000	- 6400	- 9300	+14,200
6	-1500	- 2700	- 4200	- 4500	+4050	+ 9750	+14,400	+18,900
7	+5700	+12,900	+17,550	+22,500	- 900	- 1200	- 2250	- 3900
8	+7200	+17,400	+24,600	+32,400	-3750	- 8550	-13,200	-18,600
9	+4500	+10,800	+15,300	+20,250	-3900	- 9150	-13,950	-19,500
10	+1200	+ 3750	+ 5700	+ 8250	-4200	- 9300	-14,400	-20,100
11	-2850	- 5700	- 7650	- 9300	-2550	- 5550	- 7500	- 9000
12	-4350	-10,950	-15,450	-19,800	+ 600	+ 1200	+ 2700	+ 5700
13	-1950	- 4800	- 6600	- 8400	+ 900	+ 1800	+ 3300	+ 5700
14	+1950	+ 4500	+ 6150	+ 8400	- 450	- 1200	- 1850	- 3000
15	-3600	- 9300	-12,900	-16,500	+ 900	+ 2100	+ 3900	+ 6900
16	-2250	- 6000	- 8250	-10,500	+ 450	+ 900	+ 1800	+ 3300
17	0	0	0	0	0	0	0	0
18	+ 600	+ 1650	+ 2400	+ 3000	0	- 600	- 1200	- 1500

Table 5

Recorded Liveload Deflections on the Rib for Simulated Snow and Wind Loadings

Units: inches. See Figure 9 for location and positive direction of deflection gages.

Point	SNOW LOAD Per Cent Design Load				WIND LOAD Per Cent Design Load			
	50	100	150	200	50	100	150	200
1	0	0	0	0	+0.30	+0.63	+1.12	+1.47
2	-0.04	-0.13	-0.18	-0.25	+0.40	+0.87	+1.60	+2.08
3	+0.37	+0.80	+1.11	+1.70	-0.31	-0.82	-1.36	-1.98
4	-0.11	-0.30	-0.41	-0.57	+0.27	+0.60	+1.05	+1.35
5	+0.38	+0.95	+1.38	+1.91	-0.48	-1.15	-1.95	-2.82
6	+0.21	+0.55	+0.75	+1.09	+0.15	+0.42	+0.81	+1.11
7	+0.13	+0.32	+0.50	+0.69	-0.30	-0.76	-1.34	-1.90
8	+0.19	+0.58	+0.80	+1.19	+0.09	+0.21	+0.57	+0.75
9	+0.20	+0.43	+0.61	+0.83	+0.01	+0.12	+0.29	+0.36
10	--	--	--	--	+0.27	+0.60	+1.24	+2.00

Table 6

Recorded Liveload Stresses and Deflections on Floor Framing for Simulated Floor Loadings

Units: psi, + tension, - compression, Es = 30 x 10^6 psi for converting strain to stress. See Figure 10 for the location of the strain and deflection gages.

Point	STRESSES Per Cent Design Load				Point	DEFLECTION Per Cent Design Load			
	50	100	150	200		50	100	150	200
1	+5400	+10,420	+15,400	+20,500	1	0.02	0.04	0.12	0.20
2	+3000	+ 5680	+ 8450	+11,400	2	0.08	0.10	0.14	0.21
3	+2400	+ 5980	+ 9750	+13,900	3	0.07	0.10	0.14	0.21
4	+8180	+15,400	+21,500	+27,800	4	0.07	0.11	0.17	0.24
5	+ 540	+ 1740	+ 2550	+ 3420					
6	+2650	+ 5710	+ 9100	+12,800					

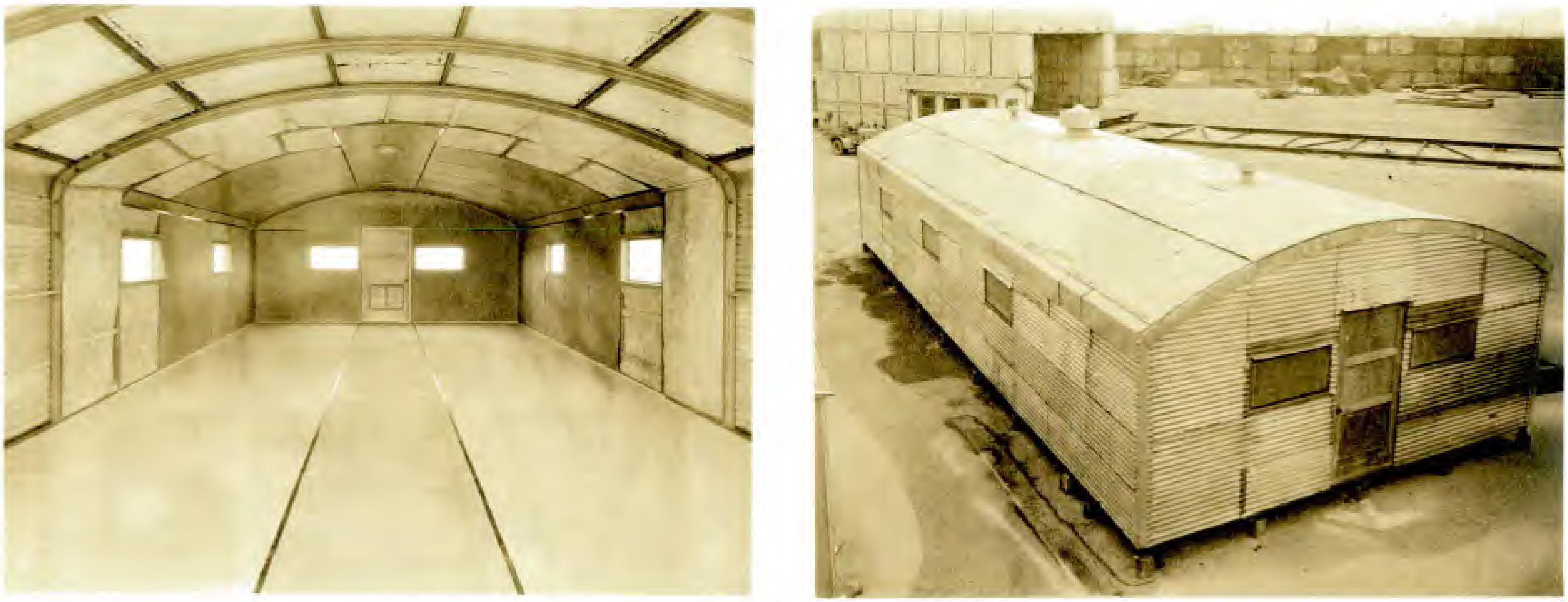


Figure 1. Exterior and interior view of the Great Lakes Steel Corporation, Stran-Steel Division, 20-ft by 48-ft straight-sided, arch-roof prefabricated metal building, northern type.

Figure 2. Test facilities for 20-ft by 48-ft buildings.

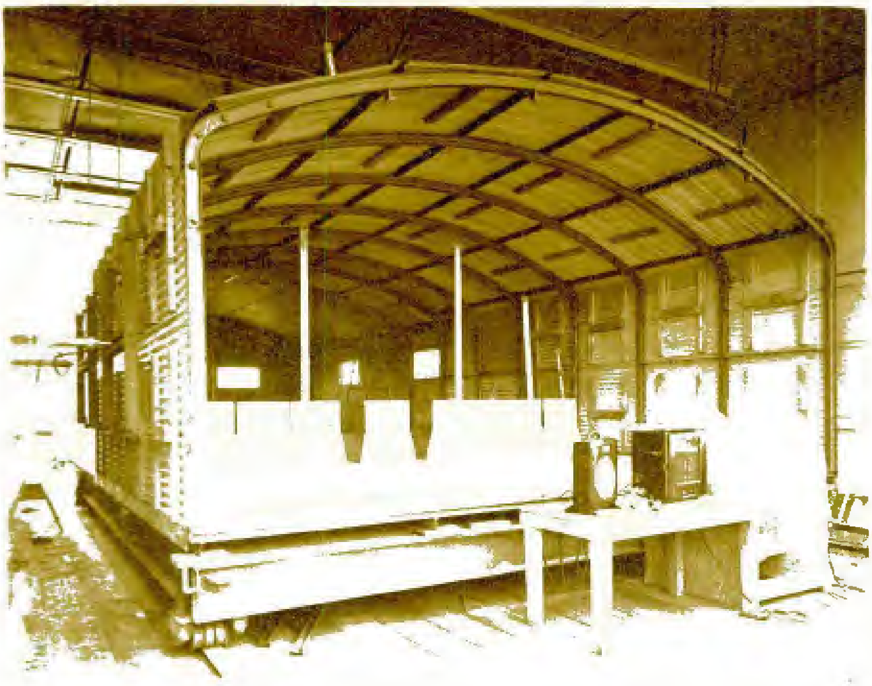


Figure 3. Test gear for applying a simulated wind loading.

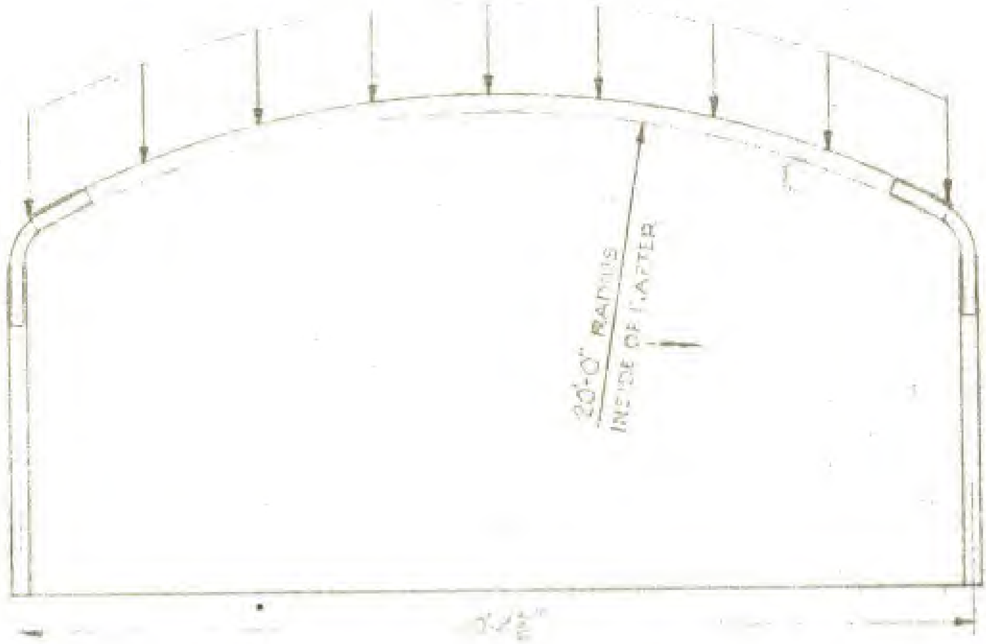


FIGURE 4. DESIGN SNOW LOADING CONDITION

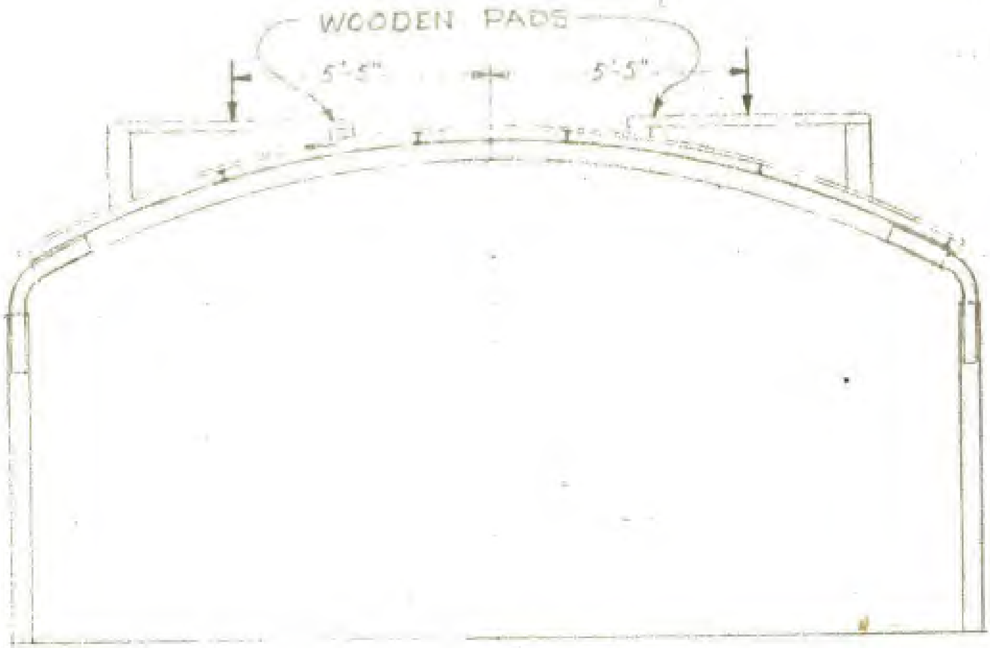


FIGURE 5. METHOD OF APPLYING SIMULATED SNOW LOADING

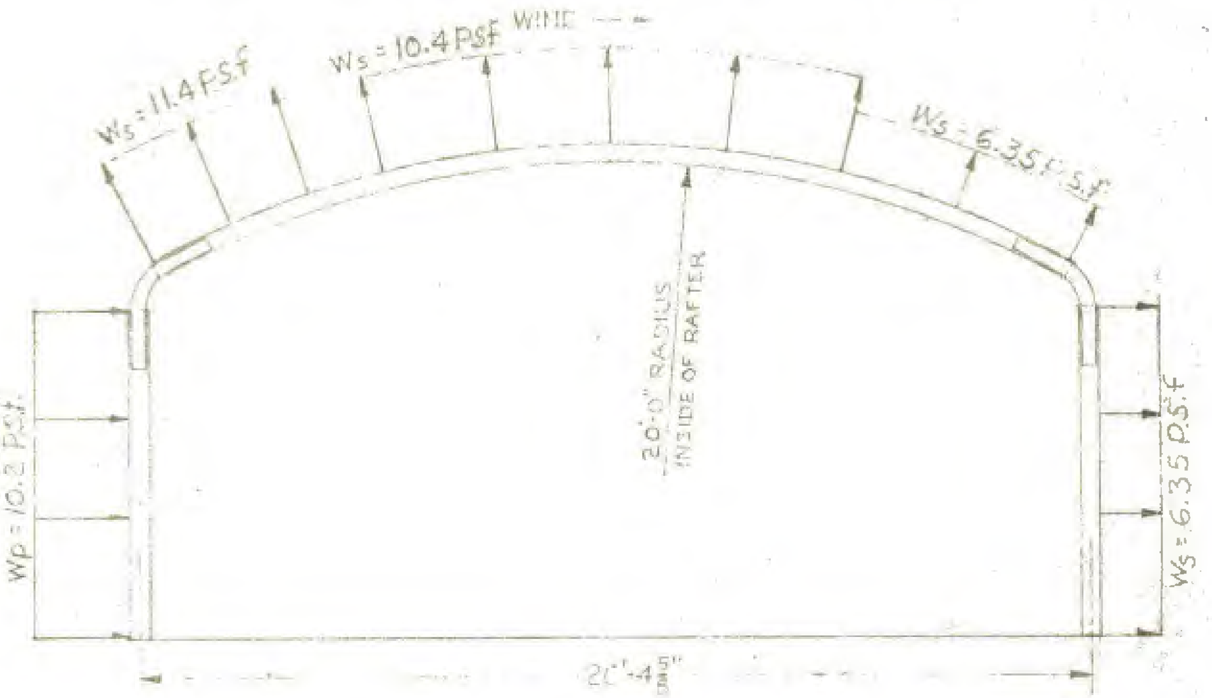


FIGURE 6. DESIGN WIND LOADING CONDITION FOR 70 MPH WIND

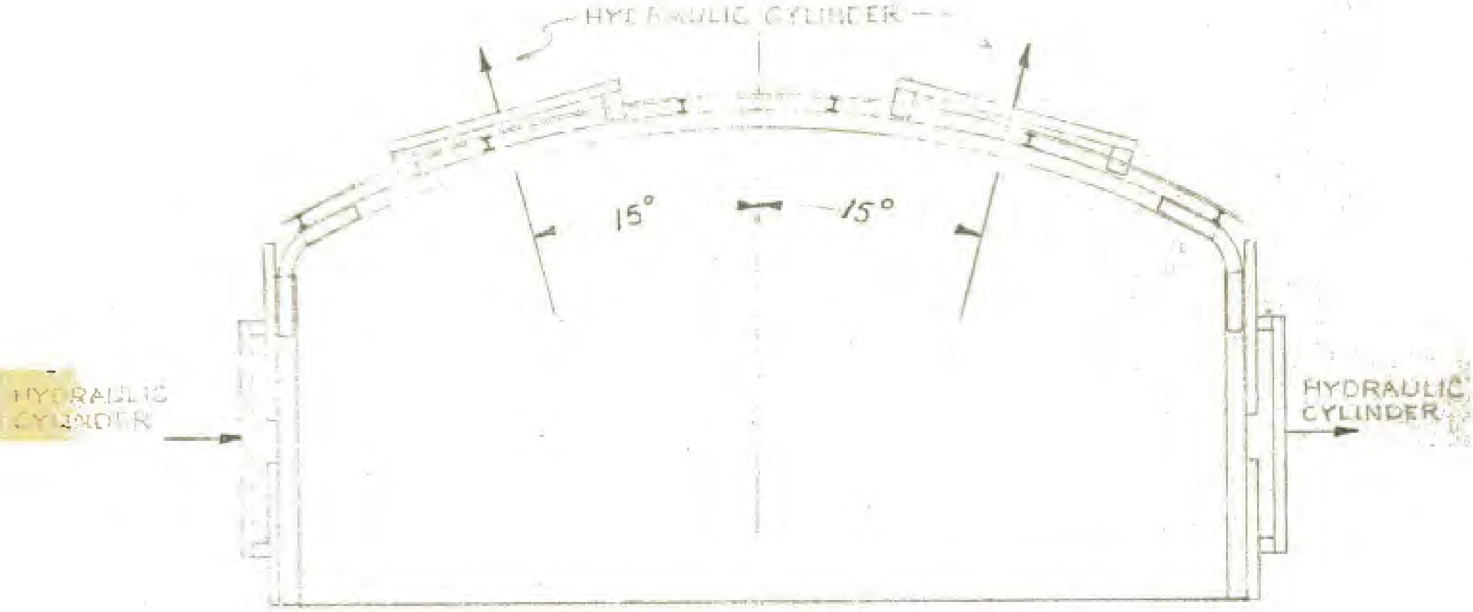


FIGURE 7. METHOD OF APPLYING SIMULATED WIND LOADING

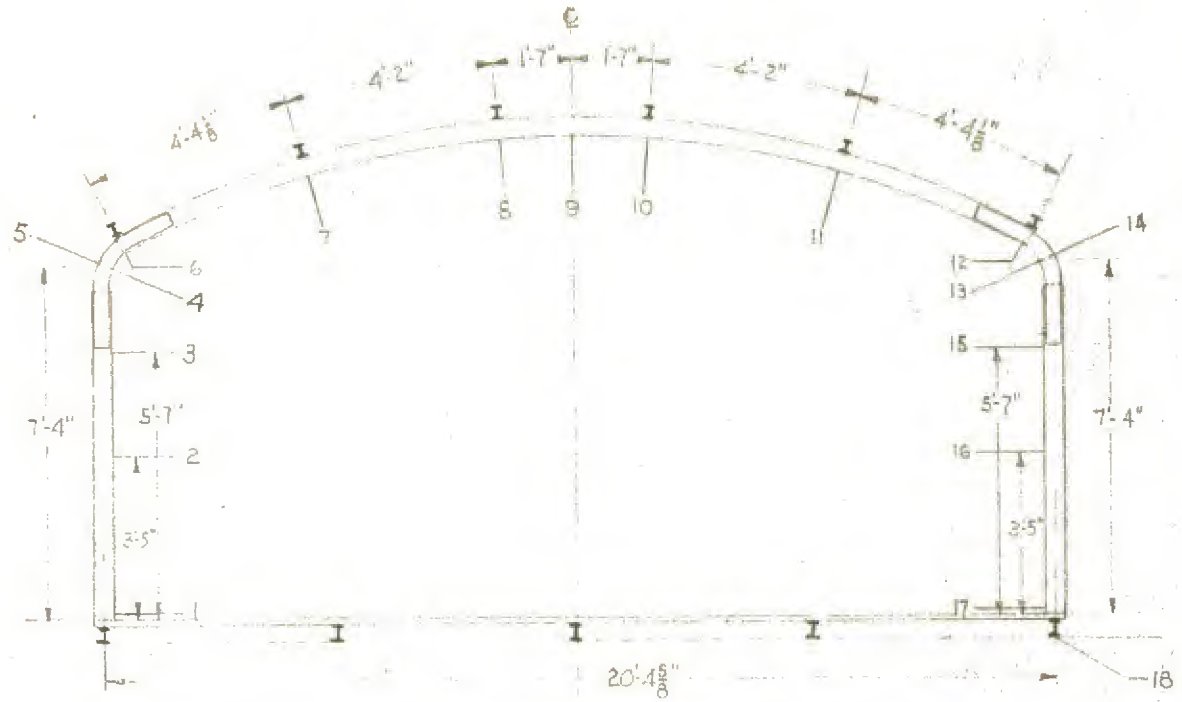


FIGURE 8. LOCATION OF STRAIN GAGES ON RIGID FRAME

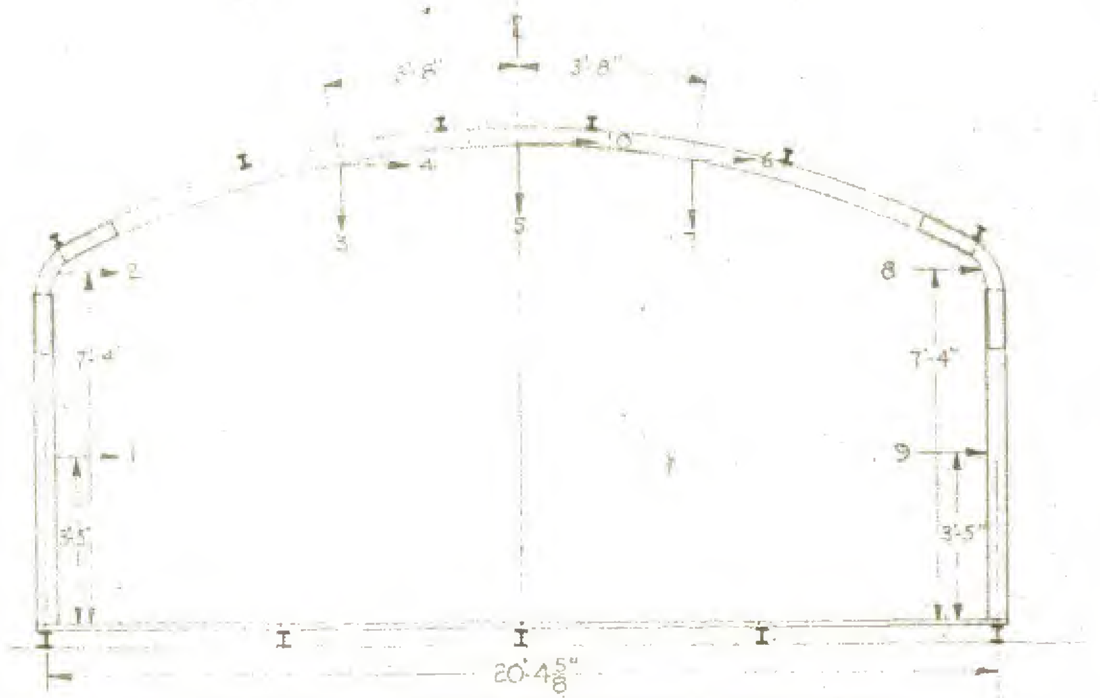
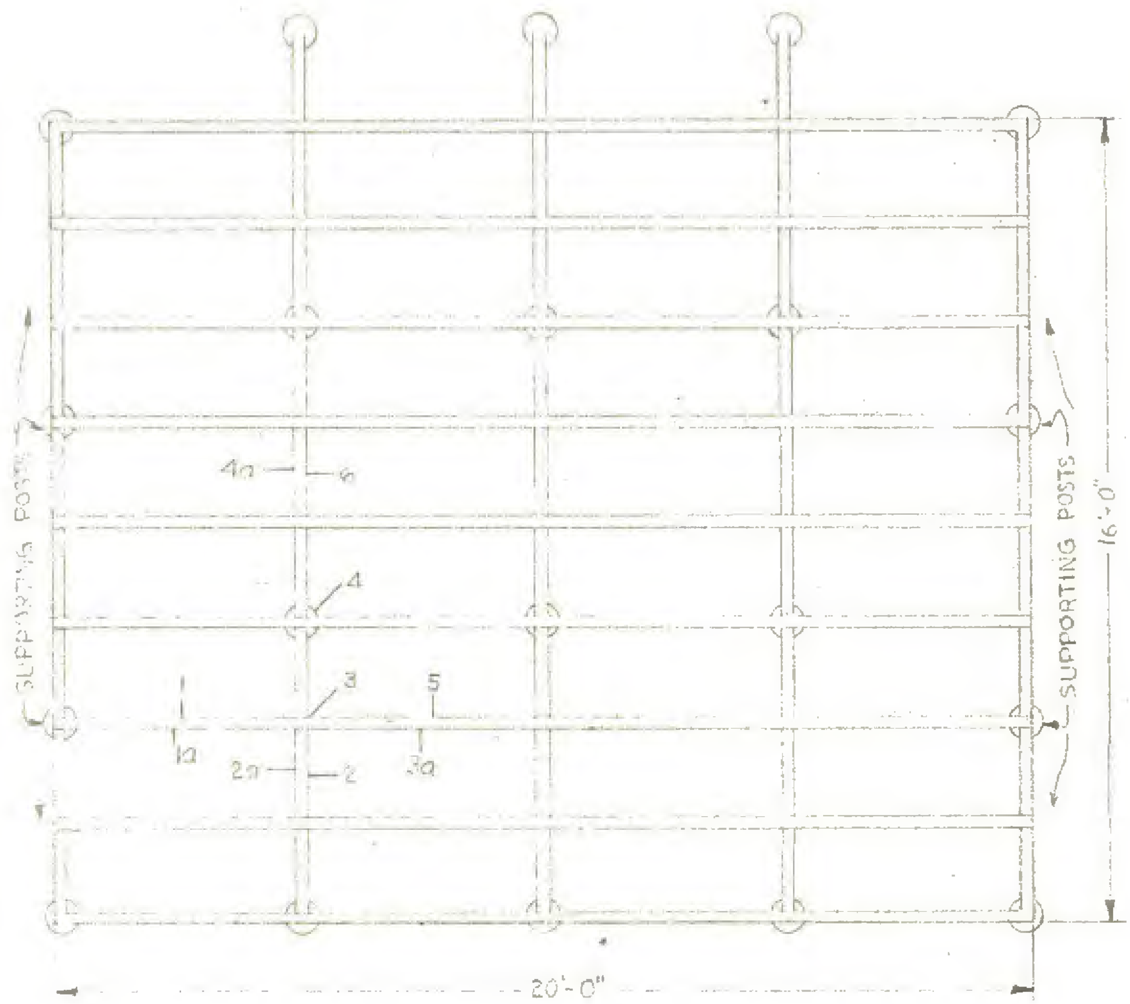


FIGURE 9. LOCATION OF DEFLECTION GAGES ON RIGID FRAME



STRAIN GAGES

1. JOIST-BOTTOM
2. SILL - BOTTOM
3. JOIST - TOP
4. SILL - TOP
5. JOIST- BOTTOM
6. SILL - BOTTOM

DEFLECTION GAGES

1a. JOIST - END SPAN
2a. SILL - END SPAN
3a. JOIST - INT SPAN
4a. SILL - INT SPAN

FIGURE 10. FLOOR AREA TESTED AND LOCATIONS OF STRAIN AND DEFLECTION GAGES

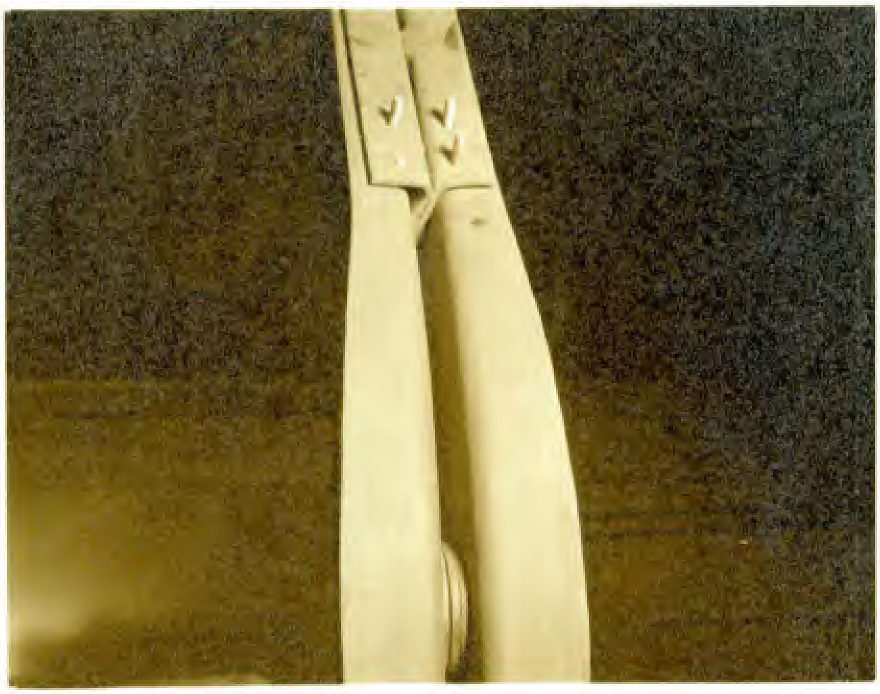


Figure 11. Close-up of eave connection and bolts used for attachment of eave purlin.