Project NY 500 002-19

Final Technical Memorandum M-048

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EVALUATION OF THE STANDARD 40-FT BY 100-FT, ARCH RIB, METAL UTILITY BUILDING

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SUMMARY

The Standard Arch Rib, 40-ft by 100-ft, Prefabricated Metal Utility Building was evaluated by the Laboratory to determine its adequacy for advanced base use. The evaluation included a study of packaging and crating, ease of erection, and structural adequacy for the specified snow and wind loadings. It was found by test and observation that the building can be packaged for overseas shipment in 386.80 cu ft with a gross weight of 21,853 lbs; erection of the building was accomplished in 398 manhours, using a six man crew, a crane with operator, and a large pneumatic tired fork lift as an erection aid. The building was found structurally inadequate under the structural loadings specified by Military Specifications[1].

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INTRODUCTION

To facilitate construction of Military activities, a need exists for a standardized, prefabricated, 40-ft by 100-ft, utility 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-erection. As part of a program to develop suitable utility buildings, present standard and 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 the standard arch-rib 40-ft by 100-ft utility building. Studies and tests to determine the ease of erection, structural adequacy, and adequacy of packaging and crating were made by the Laboratory under Project NY 500 002-19, authorized by the Bureau of Yards and Docks[2].

DESCRIPTION OF BUILDING

The standard-arch-rib utility building, as shown in Figure 1, is an arch-rib, prefabricated steel structure with over-all dimensions of 41-ft by 100-ft 2-in., an outside height of 20-ft 8-in. at the crown of the arch. The building is designed to be erected on a concrete footing or slab. Anchor bolts are furnished with the building and are to be preset in the footing or slab.

The building, as furnished for test, consisted of 26 field assembled arch-ribs. Each rib is assembled from three (3) identical members. The assembled sections form half of a true circle. The ribs have an I-cross-section made by spot welding 14 gage specially shaped high tensile steel angles to the back of a 14 gage specially shaped high tensile steel channel. This forms a nailing groove down the center line of each member. The end wall studs and purlins have this same nailing groove down their center line. They are formed by spot welding two (2) 16 gage specially shaped channels back to back.

The building is erected by first bolting 6-in. channel-base shoes to the foundation anchor bolts which have been preset 4-ft center to center in the concrete foundation. Three (3) arch-rib segments are then bolted together with splice plates. The rib is then set in the base shoe and attached with sheet metal screws. The ribs are held on 4-ft spacing with seven (7) 18 gage 6-in. metal channel sections, bridging between the ribs. The bridging is spaced at definite intervals around the arch with the bridging in each adjacent bay staggered to obtain the maximum supporting effect. The location of the bridging in any two adjacent bays form the sequence for the pattern in the rest of the building. Seven (7) purlins are symetrically spaced about the crown of the arch and are attached to the top flange of the ribs with sheet metal screws. They run the longitudinal direction of the building and spaced over a 55 degree segment of the arch. The end-bulkhead frame consists of vertical studs, on 4-ft centers; door frame; wind girt, which fastens to the studs above the door frame and at each end to the curved rib; and wind braces, which are attached to the wind girt at one end and to the third arch from the end of the building at the other end.

All sheeting and flashing is 26 gage galvanized steel. The sheeting is standard corrugated sheeting and the flashing is shaped to fit its particular location. The side wall sheeting is run longitudinally up to the lower purlins, from there on curved sheeting is applied transversely over the purlins. Special cut sheeting with the corrugations running horizontal is applied to the end bulkheads. Flashing is used at the junction of the end bulkhead and side wall sheeting.

Interior natural light is provided by two (2) windows in each end bulkhead. A corrugated plastic sheet, 26-in. by 100-in., is used in place of the regular corrugated metal sheeting at window locations. A wiring harness is provided for artificial interior lighting and electrical power. Access is provided by a 12-ft 6-in. by 14-ft double inside sliding door in each end bulkhead. Natural ventilation is furnished by a 4-ft by 4-ft louvered ventilator above each door.

The building, as received, was packaged for export shipment in 20 crates weighing 21,853-lbs. and cubing 386.80 cu ft.

TEST FACILITIES

A special facility, see Figure 2, was built for testing 40-ft by 100-ft buildings. The facility consists of two test locations, one for erection and skylighting studies and one for structural tests.

At the first location, the erections are made on a steel foundation consisting of 12-in. by 3-in., 25.3-lb. structural channels welded end-to-end to form a simulated footing. The channels were leveled with blocking and were placed with flanges down to give a 12-in. wide work surface. Openings were provided at the location of the end wall doors to give accessibility. Studs were welded to the channel to simulate anchor bolts. The foundation was located with the longitudinal axis running east and west to expose one side of the roof of a test building to the northern light for skylighting studies.

At the structural test location, wide concrete footings, 40-ft on centers and 105-ft long, provide reactions against uplift caused by wind loadings. Structural WF beams were embedded transversely 5-ft on centers in the top of these footings to act as tie-downs for 12-in. by 3-in., 25.3-lb channel foundations for the side walls of buildings under test. These 105-ft channel foundations, placed with the flanges down, were bolted to the WF beams and are adjustable for building width. Studs were welded to the channels to simulate anchor bolts. Simulated wind and snow loads are applied through hydraulic cylinders. For the snow loads, the cylinders are attached to pontoons located on the inside of the building under test. These pontoons are filled with water for added reaction. For supporting and reacting the hydraulic cylinders in the application of wind loads, pontoon walls were placed along the outside of each concrete footing and connected across the top with transverse beams made up of 24-in. by 9-in., 80-lb WF beams reinforced with 12-in. by 3-in., 25.3-lb channels welded to the top flange for protection against lateral buckling. A special hydraulic pump and control panel is used to actuate the cylinders. Actual loads are applied through whiffletree systems, see Figure 3.

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 developed at the Laboratory.

TEST PROCEDURE

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

Five 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 civilian personnel of varied skills were used for all erections. A crane with operator and a large pneumatic tired fork lift were used to aid the first three (3) erections. Scaffolding and the large pneumatic tired fork lift were used in the last two (2) erections. The last two erections also required two additional men in the erection team. 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 the erection manual; to establish the tool requirements for efficient erection; and to record the manhours required to assemble the frame, exterior, sheeting, and end bulkheads. After the first erection, the building was completely disassembled. The second, third, fourth, and fifth erections and disassemblies were made to determine the suitability of the design for repeated assembly; to record the erection manhours required when using a crane and large pneumatic fork lift or scaffolding and large pneumatic fork lift as an erection aid, and as the team became more familiar with the building.

Structural tests were made to determine the adequacy of the building under 20-psf snow load and a 70 mph wind load as stipulated in the Military specifications[1]. The design snow load conditions, diagramed in Figure 4, were applied hydraulically to the roof through whiffletrees, as shown in Figure 5. The design wind load conditions diagramed in Figure 6 were applied hydraulically to the building through whiffletrees as shown in Figure 7. In each case, loadings scheduled to reach 185 per cent of design load were applied in 10 per cent increments. Location of the strain and deflection gages used in these tests are shown in Figures 8 and 9.

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 packaging and crating study is shown in Table 1. It is seen from this table that the standard arch rib utility building's weight is 21,853 lbs. and has a shipping cube of 386.80 cu ft.

The packaging scheme was found acceptable for handling and site layout. Further, the building parts were properly grouped for minimum weight and cube. In general, the crating was found adequate and complied with the Military specifications for crating and packaging[1].

Erection

A summary of the manhours required to erect the building in each of the first three erections using a crane and a large pneumatic tired fork lift as an erection aid and the fourth and fifth erections using scaffolding and a large pneumatic tired fork lift as an erection aid are shown in Table 2. The table gives a breakdown of the third erection and shows that a team using a crane and large pneumatic tired fork lift can erect the standard arch-rib building in 398 manhours.

During the first erection it was found that a 6 man crew using a crane and a pneumatic tired fork lift were required for efficient erection. It was found during this erection that a fork lift is very desirable when attaching the bridging and the purlins to the ribs, as the workmen can be elevated to the exact point of attachment. The erection manual furnished with contract N160-2495-9720 was found easy to follow, adequate in content, and superior to that dated August 1950. The tools furnished with the building were adequate.

The second and third erections using the crane and fork lift and the second erection using the scaffolding and fork lift became progressively easier as the crew became familiar with the parts and the proper method of handling them.

Squaring and plumbing the frame after four ribs have been erected seemed to cause difficulty in all erections. If some type of template or squaring members were provided for this operation, squaring and plumbing the frame would be simplified and easily accomplished.

Considerable difficulty was experienced in disassembly of the building because of the large number of nailed connections, the exterior sheeting suffered extensive damage during the disassembly process. The flanges of the structural members suffered damage during handling and disassembly. The light gage metal used in the framing members bends out of shape very easily.

Structural Adequacy

The stresses and deflections recorded in the snow-, and wind-loading tests are summarized in Table 3. All structural members in this building are made from high-strength, low alloy steel. The steel used in the purlins and end-bulkhead framing is 40,000 psi minimum yield and that used for the ribs is 50,000 psi minimum yield.

The snow loading was taken to a maximum of 185 per cent (37psf) of design. At design load, a maximum stress of 18,600 psi compression was recorded at point 5 on the rib as compared to 27,000 psi allowable, see Table 4, and at 185 per cent of design a maximum stress of 38,000 psi compression was recorded at the same point, see Table 4. The maximum recorded deflection at design load was a vertical movement of 1.78-in. at point 5, see Table 6. The maximum deflection at 185 per cent of design load was a vertical movement of 3.44-in. at the same point, see Table 6. During the snow load tests several of the ribs started to buckle between the channel bracing at the crown of the arch, see Figures 10 and 11. This condition appeared at about 145 per cent of design load. The severity of the buckling progressed with each increment of increased loading;

failure finally occurred just above 185 per cent of design load.

In arch-shaped buildings, the 70 mph wind load governs the design of the arch ribs. The wind load test scheduled for 185 per cent of design load was discontinued at 170 per cent (91 mph) of design load because an over-stressed condition resulted in a rib failure. At design load, a stress of 34,500 psi tension, as compared to 27,000 psi allowable, was recorded at point 5, see Table 5, and at 170 per cent of design, a maximum stress of 57,000 psi tension was recorded at the same point, see Table 5. The maximum recorded deflection at design load was a horizontal movement of 3.30-in. at point 3, see Table 6; and the maximum deflection at 170 per cent of design load was a horizontal movement of 5.40 at the same point, see Table 6. At 170 per cent of design load a rib buckled between support points about 9-ft 4-in. above the foundation on the windward side of the building, see Figures 12 and 13. A small dent or deformation in the outer flange was believed to have induced the rib to buckle at this point. Small dents or deformations are very common in the flanges of the ribs because of the light gage metal. They can occur both during shipping or erection of the ribs.

CONCLUSIONS

As designed, the standard arch-rib, 40-ft by 100-ft, prefabricated metal utility building does not meet the structural requirements set forth by Military specifications[1] (185 per cent of design load is maximum loading required). It was found during both the snow-, and wind-load structural tests that before 185 per cent of design load had been reached, failure or near failure conditions existed in the arch ribs. These conditions can be attributed to two main causes:

(1) The distance between the support points around the ribs is too great. This permits the compression flange of the rib to buckle under load.

(2) The 14 gage steel used in the ribs makes the flanges of the structural members vulnerable to damage either during shipping or erection of the building. If a dent or deformation happens to fall at a point where the flange is under compression the member is considerably weakened.

In addition, several inherent deficiencies exist in this design:

(1) Nailing the exterior sheeting to the frame does not lend itself to easy disassembly and re-erection.

(2) The arch-shape does not give the maximum usable space for the floor area.

(3) The lack of squaring members makes erection of the first few bays difficult. Squaring members would greatly assist this operation.

REFERENCES

1. MIL-B-16606(Docks) Military Specifications, "Building, Prefabricated Metal Arch-Rib, 40-Ft by 100-Ft, 31 August 1951.

2. RDB Project Card NY 500 002 Prefabricated Advanced Base Building, 15 May 1949.

Table 1. Summary of Packages, Cubes and Weights

Item	Units	Quantity
Packages	no.	20
Cubes of crated bldg.		
Maximum crate	cu ft	94.60
Minimum crate	cu ft	2.14
Total cubage	cu ft	386.80
Weight of crated bldg.		
Maximum crate	lb	1,732
Minimum crate	lb	185
Total weight	lb	21,853
Steel weight		
Frame	lb	8,885
Exterior sheeting	lb	8,554
Miscellaneous steel (all fasteners, doors and door hardware, etc.)	lb	1,827
Total steel weight	lb	19,266

Table 2. Summary of Erection Time Studies

Item	Crew Size	Equipment Used	Manhours
Entire building			
1st erection	6	*	445
2nd erection	6	*	420
3rd erection	6	*	398
4th erection	8	+	463
5th erection	8	+	419
Components (3rd erection)			
Frame			120.5
Sheeting (Horizontal side)			149.0
Sheeting (Curved roof)			51.5
Sheeting (Endwall & doors)			77.0
Total			398.0

* Crane with operator and large pneumatic tired fork lift.
\+ Scaffolding and large pneumatic tired fork lift.

Table 3. Summary of Structural Tests

The recorded values given in this table represent only liveload values. Allowable unit stress at design load = $\frac{50,000}{1.85}$ = 27,000 psi.

Item	Units	Snow	Wind
Design loading	as noted	20 psf	70 mph
Maximum loading	as noted	37 psf	91 mph
Load increment	% design	10	10
Stress (design load)			
Tension			
Magnitude	psi	15,600	34,500
Location	no.	15	5
Compression			
Magnitude	psi	18,600	22,800
Location	no.	5	29 & 30
Stress (maximum load)			
Tension			
Magnitude	psi	30,775	57,000
Location	no.	15	5
Compression			
Magnitude	psi	38,700	39,000
Location	no.	5	29
Deflection (design load)			
Vertical			
Magnitude	in.	1.78	2.00
Location	no.	5	6
Horizontal			
Magnitude	in.	1.09	3.30
Location	no.	9	3
Deflection (maximum load)			
Vertical			
Magnitude	in.	3.44	3.24
Location	no.	5	6
Horizontal			
Magnitude	in.	2.12	5.40
Location	no.	9	3

Table 4. Recorded Live-Load Stresses on the Rib for Simulated Snow Loadings

Allowable stress at design load $\frac{50,000}{1.85} = 27,000$ psi. Units: psi, + tension, - compression, $E_s = 30 \times 10^6$ psi for converting strain to stress. See Figure 8 for location of strain gages.

Point	SNOW LOADING			
	Per Cent Design Load			
	50	100	150	185
1	- 1,200	- 1,200	- 1,200	- 1,490
2	- 4,800	- 8,250	-12,900	-15,645
3	- 6,300	-12,900	-19,200	-22,620
4	- 7,350	-15,600	-25,500	-32,275
5	- 8,850	-18,600	-30,000	-38,700
6	- 6,450	-13,350	-21,150	-26,780
7	- 5,250	-10,800	-16,500	-20,615
8	- 3,150	- 6,000	- 9,000	-10,600
9	- 900	- 1,200	- 4,800	- 5,690
10	+ 1,500	+ 3,000	+ 5,400	+ 6,725
11	+ 3,600	+ 8,100	+13,500	+16,660
12	+ 5,550	+11,400	+18,000	+22,260
13	+ 6,750	+14,400	+23,400	+28,835
14	+ 7,050	+15,000	+23,850	+29,130
15	+ 7,500	+15,600	+24,600	+30,775
16	+ 6,450	+13,200	+21,000	+26,595
17	+ 5,400	+11,400	+16,500	+20,610
18	+ 4,200	+ 8,250	+12,900	+15,495
19	+ 600	+ 1,050	+ 1,500	+ 1,905
20	- 900	- 2,100	- 3,000	- 3,510
21	- 3,300	- 6,900	-10,500	-12,470
22	- 5,100	-10,650	-16,500	-20,275
23	- 7,200	-15,000	-24,000	-30,330
24	- 6,450	-13,050	-20,800	-24,310
25	- 8,100	-16,650	-25,800	-31,930
26	- 5,550	-11,400	-17,850	-20,660
27	- 3,750	- 7,950	-12,000	-14,675
28	0	0	+ 600	+ 750
29	+ 5,700	+11,400	+17,400	+21,360
30	+ 5,400	+10,800	+16,500	+20,615
31	+ 3,750	+ 7,950	+12,150	+14,825
32	+ 1,200	+ 1,950	+ 3,000	+ 3,470
33	- 3,750	- 8,400	-13,500	-17,035
34	- 6,600	-14,250	-22,500	-28,050
35	- 6,900	-14,400	-21,600	-26,585
36	- 5,250	-11,250	-18,000	-23,125
37	- 1,200	- 2,400	- 2,850	- 3,580

Table 5. Recorded Live-Load Stresses on the Rib for Simulated Wind Loadings

Allowable stress at design load = $\frac{50,000}{1.85}$ = 27,000 psi. Units: psi, + tension, - compression, E_s = 30 x 10^6 psi for converting strain to stress. See Figure 8 for location of strain gages.

Point	WIND LOADING			
	Per Cent Design Load			
	50	100	150	170
1	+ 9,600	+15,900	+23,700	+27,000
2	+10,500	+21,300	+31,500	+36,000
3	+15,600	+31,800	+48.000	+54,000
4	+15,900	+32,400	+48,000	+54,600
5	+17,100	+34,500	+51,000	+57,000
6	+12,900	+26,400	+38,400	+43,500
7	+ 9,600	+19,200	+28,200	+31,500
8	+ 4,800	+10,500	+15,000	+16,500
9	+ 1,200	+ 3,000	+ 4,500	+ 5,100
10	+ 900	+ 2,400	+ 2,700	+ 2,700
11	- 1,350	- 3,000	- 3,900	- 4,500
12	- 2,400	- 4,800	- 6,600	- 7,800
13	- 4,200	- 8,400	-12,000	-14,100
14	- 4,500	- 9,000	-12,900	-15,000
15	- 4,500	- 9,000	-12,000	-13,800
16	- 5,100	- 9,600	-13,800	-15,300
17	- 4,650	- 9,000	-12,600	-15,000
18	- 6,300	-10,200	-18,000	-20,700
19	- 3,600	- 6,600	- 9,000	- 9,600
20	- 2,100	- 4,500	- 6,000	- 6,000
21	- 2,400	- 4,500	- 6,000	- 6,000
22	- 900	- 1,200	- 900	0
23	+ 450	+ 600	+ 2,400	+ 3,000
24	+ 1,200	+ 3,000	+ 4,800	+ 6,000
25	+ 2,400	+ 5,400	+ 8,700	+ 9,600
26	+ 2,100	+ 4,500	+ 7,500	+ 9,000
27	+ 2,250	+ 4,500	+ 7,500	+ 9,000
28	+ 1,950	+ 600	+ 3,600	+ 4,500
29	-11,700	-22,800	-33,300	-39,000
30	-11,400	-22,800	-33,000	-36,600
31	- 6,300	-12,000	-18,000	-19,500
32	- 1,800	- 3,000	- 3,600	- 3,900
33	+ 3,150	+ 6,000	+ 9,600	+11,100
34	+ 4,200	+ 9,000	+13,800	+15,300
35	+ 6,150	+13,500	+19,800	+22,500
36	+ 6,300	+13,500	+19,500	+21,600
37	+ 6,000	+13,200	+18,300	+20,400

Table 6. Recorded Live-Load Deflections on the Rib for Simulated Snow and Wind Loadings

Point	SNOW LOAD				WIND LOAD			
	Per Cent Design Load				Per Cent Design Load			
	50	100	150	185	50	100	150	170
1	-0.40	-0.82	-1.22	-1.54	+1.38	+2.90	+4.20	+4.70
2	+0.06	+0.13	+0.24	+0.30	+0.18	+0.28	+0.31	+0.31
3	-0.26	-0.58	-0.87	-1.07	+1.55	+3.30	+4.84	+5.40
4	+0.55	+1.17	+1.87	+2.29	-0.31	-0.73	-1.18	-1.35
5	+0.88	+1.78	+2.78	+3.44	-0.85	-1.86	-2.75	-3.10
6	+0.50	+1.02	+1.60	+1.97	-0.90	-2.00	-2.90	-3.24
7	+0.40	+0.92	+1.50	+1.84	+0.75	+1.85	+2.58	+2.90
8	+0.02	0	-0.03	-0.06	+0.55	+1.32	+1.91	+2.12
9	+0.50	+1.09	+1.77	+2.12	+0.40	+0.85	+1.20	+1.31
10	-0.08	-0.21	-0.41	-0.50	+1.10	+2.20	+3.18	+3.52

Figure 1. Standard Arch-Rib, 40-Ft by 100-Ft, Utility Building.

Figure 2. Test facilities for 40-ft by 100-ft Buildings.

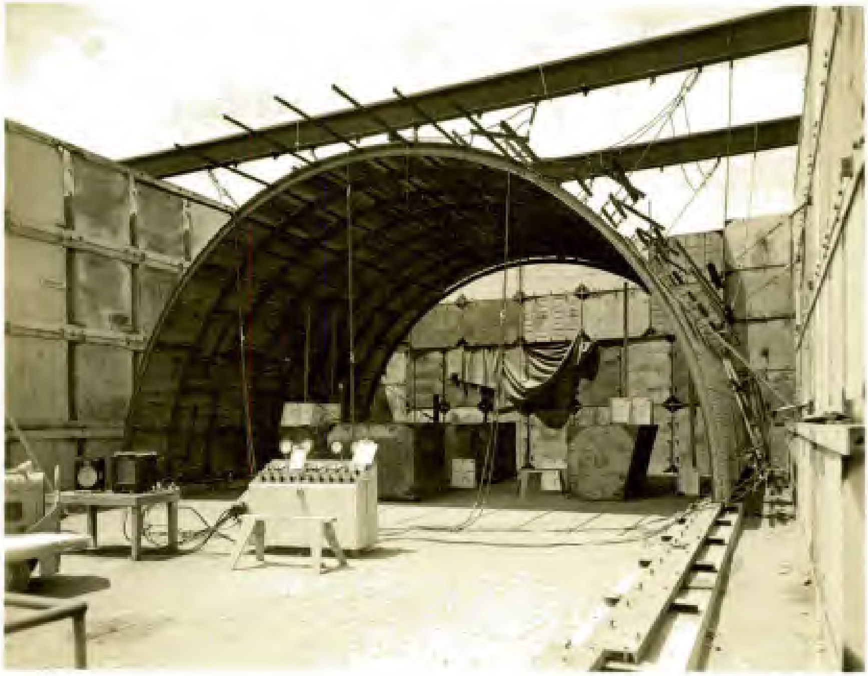

Figure 3. Test gear for applying a simulated wind load hydraulically using whiffletrees.

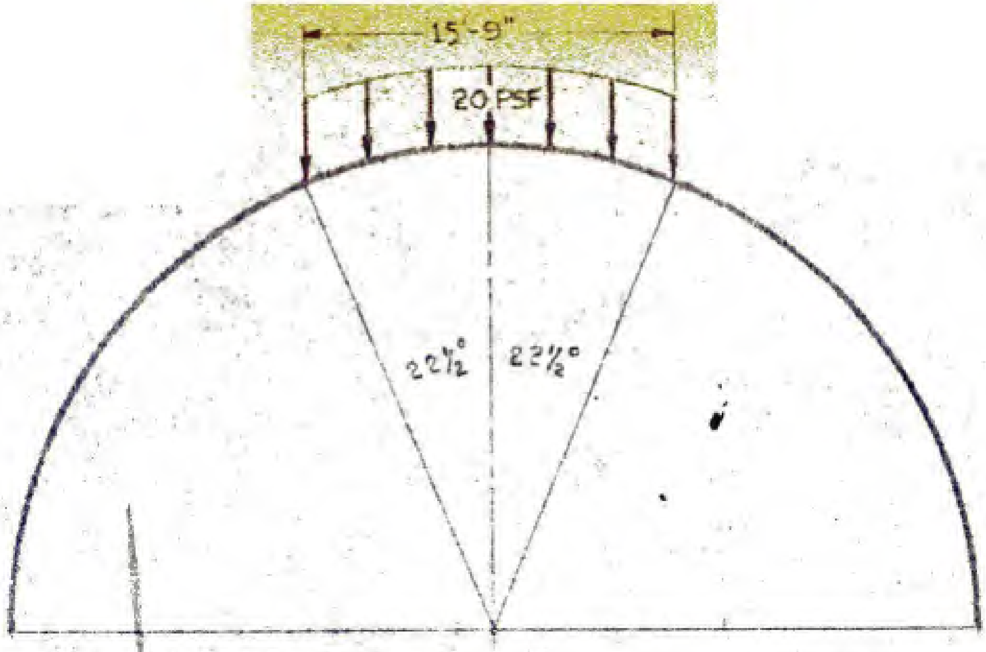

FIGURE 4. DESIGN SNOW LOADING CONDITION.

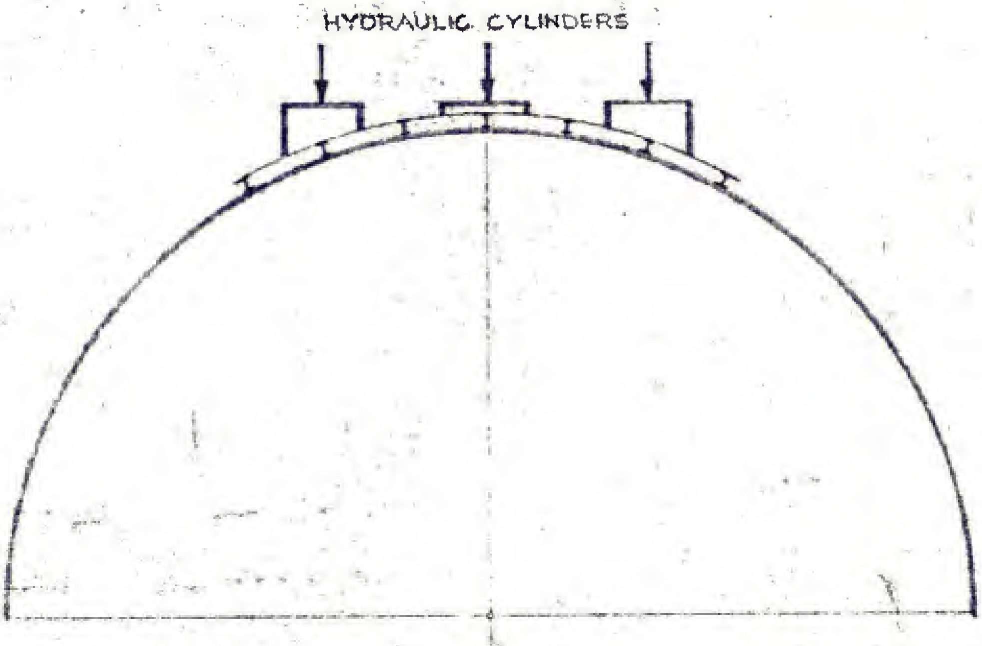

FIGURE 5. METHOD OF APPLYING SIMULATED SNOW LOADING.

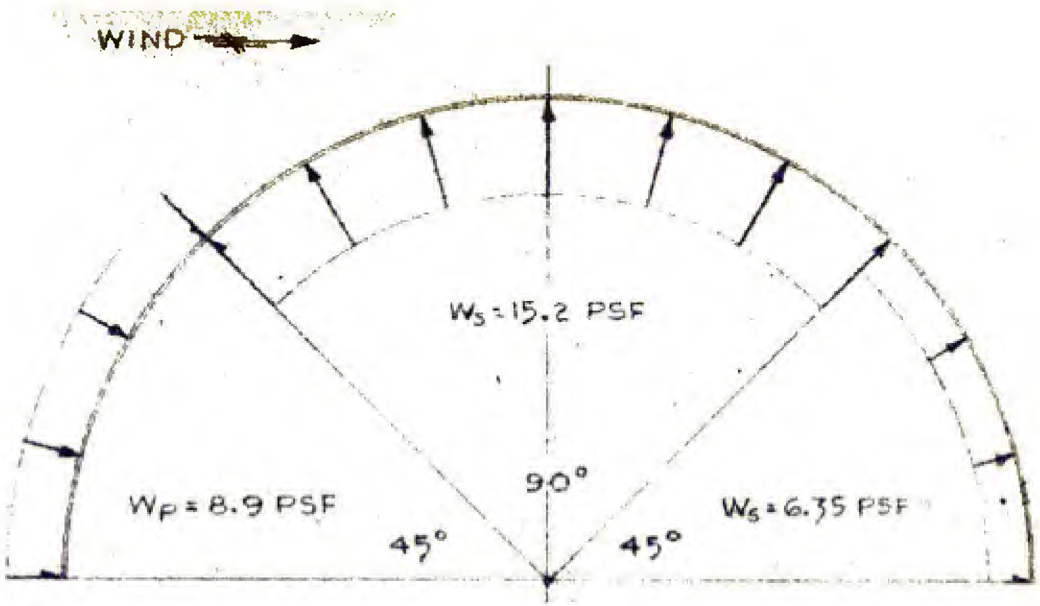

FIGURE 6. DESIGN WIND LOADING CONDITION FOR 70 MPH WIND.

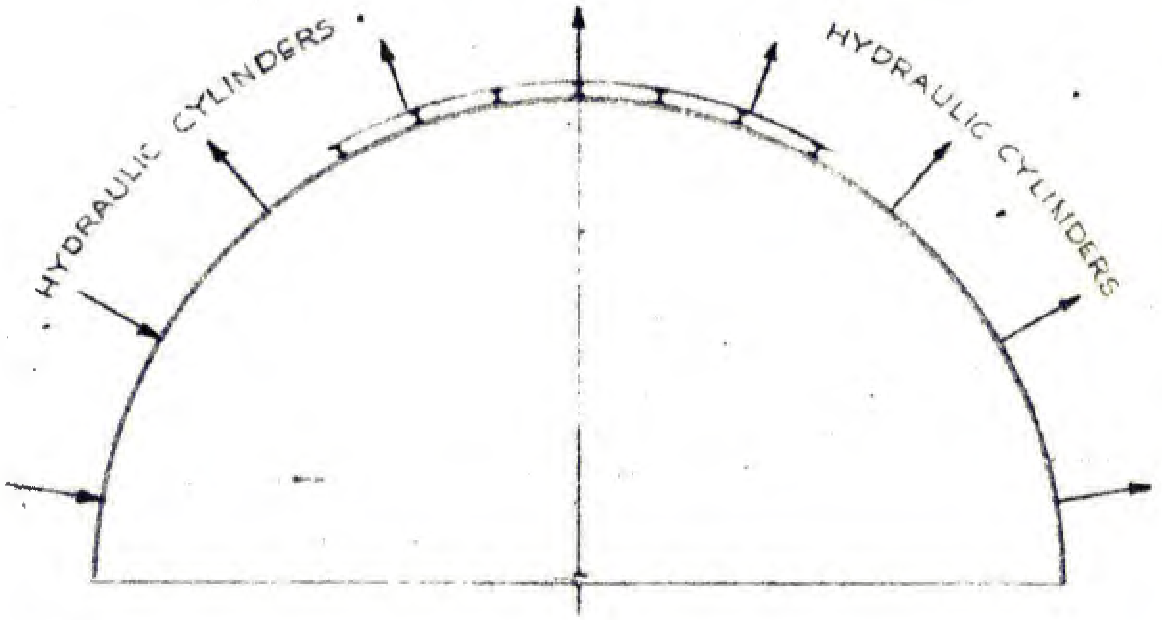

FIGURE 7. METHOD OF APPLYING SIMULATED WIND LOADING HYDRAULICALLY USING WHIFFLETREES.

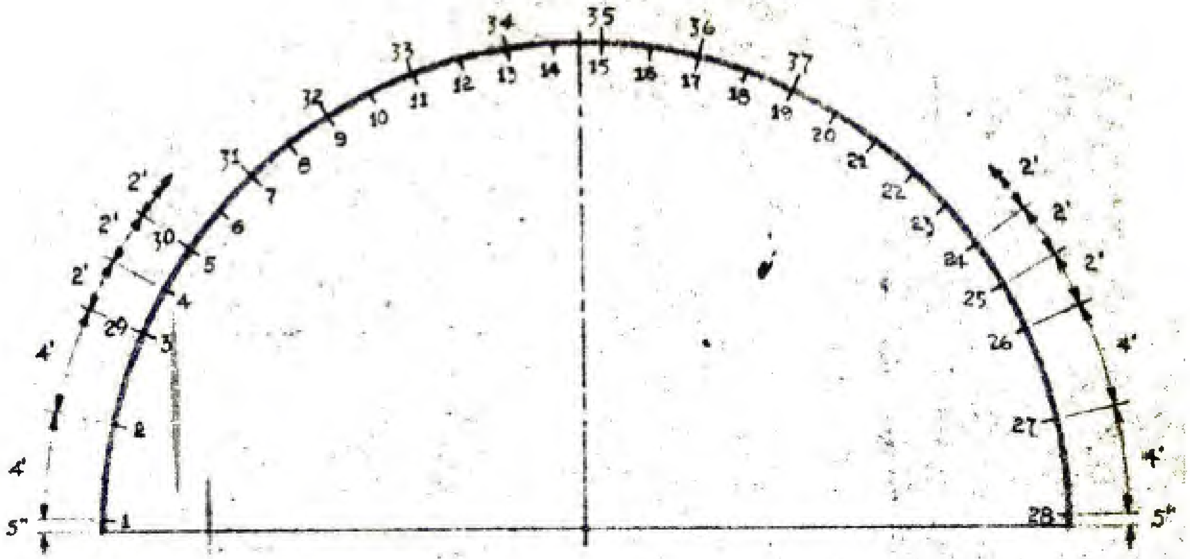

FIGURE 8. LOCATION OF STRAIN GAGES ON ARCH RIB.

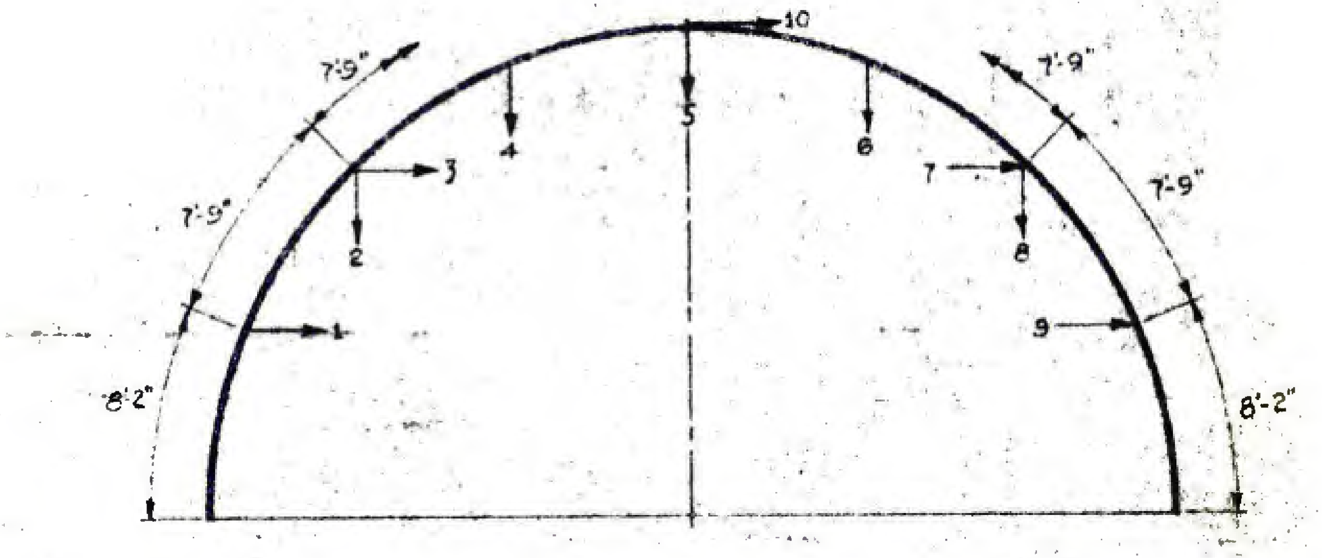

FIGURE 9. LOCATION OF DEFLECTION GAGES ON ARCH RIB.

Figure 10. Ribs buckled at the top of the arch. The condition shown is at 185% of design load.

Figure 11. Top view of one of the ribs shown in Figure 10. The sheeting was removed to take picture.

Figure 12. Base shoe pulled 1/2-in. away from foundation at 120% of wind load.

Figure 13. Outside flange of rib buckled at 170% of design wind load.