REPORT 19

Report 19.

Oct. 18, 1968.

Azimuth Towers for Homologous 300-Foot

S. von Hoerner, NRAO.

The present report mainly gives the design data for the complete azimuth structure with two towers. A detailed analysis of its performance will follow soon.

I. Requirements

1. Size

The size of the 300-ft dish structure is shown in Fig.1. For the towers we then obtain:

$$\left.\begin{array}{lll} \text{tower top above ground, point 4,} & 1990 \text{ inch} = 166 \text{ ft,} \\ \text{elevation drive above ground, point 6,} & 890 \text{ inch} = 74.2 \text{ ft,} \\ \text{distance between tower tops, } 4 - 4a, & 2400 \text{ inch} = 200 \text{ ft.} \end{array}\right\} \quad (1)$$

Point 5 must have clearance for the back-up structure of the dish, as shown in Fig.1.

The azimuth rails are standard railroad (no special foundation). For making the towers tetrahedral (they are shown in Fig.2), we need

diameter of azimuth ring 346.4 ft. (2)

2. Loads for Survival

We consider the weight of dish and snow, survival winds, and dumping snow, with the following data:

points 4:	weight of complete dish	2430 kip = 1100 tons	(3)
4:	maximum snow (20 lb/ft^2 whole area)	1454 kip = 660 tons	(4)
4:	wind on dish (85 mph, full side-area, $C_s = 1.3$)*	1050 kip = 476 tons	(5)
4,6:	additional force from torque (asymmetry)	334 kip = 153 tons	(6)
4,6:	dumping 6 inch of snow, plus 45 mph wind	269 kip = 118 tons	(7)

Since (7) is less than (6), it was omitted. Although (4) is larger than (5), it turned out that (5) always gives larger stresses, we thus omit (4) in the following. The wind force on the tower members is included, using $C_s = 1$ for pipes. The stress in each member then is calculated for three different load conditions:

1) Simultaneously a) dead load of tower, b) weight of dish, with 1215 kip each on points 4 and 4a in z direction. Call S_1. (8)

2) x-wind only; with simultaneously a) 693 kip each on points 4 and 4a,

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* C_s = shape factor

in x direction, b) 334 kip on point 6, in -x direction, c) wind force on all tower members, in x direction. Call S_x. (9)

3) Same as above, but replace x by y, and -x by -y. (= y-wind only) Call S_y. (10)

The maximum stress , for any wind direction, then is obtained as

$$S_m = |S_1| + \sqrt{S_x^2 + S_y^2} \;. \tag{11}$$

3. Loads for Wind Deformation

The velocity adopted is 18 mph (wind is below this value for 3/4 of all time); the total area, face-on, is used, and a shape factor of 1.56. In addition, the maximum torque is added. Two load conditions are calculated:

1) 64.9 kip each on points 4 and 4a, in x direction, and simultaneously 31.3 kip on point 6, in -x direction, plus wind force on all tower members. (12)

2) Same for y direction. (13)

The maximum deformation then is simply

$$\Delta_m = \max(\Delta_x , \Delta_y). \tag{14}$$

4. Restraints

We need five different restraints for the following five cases:

case	calculating	assuming
A	Δ_x	wheels fixed along rails, but soft perpendicular;
B	Δ_y	
C	S_x	wheels soft both x and y, all taken up by pintle;
D	S_y	
E	-	wheels fixed both x and y.

(15)

Actually, we calculate only one tower; we thus need some additional restraints for replacing the action of the second tower and for avoiding free rotation. All restraints used are shown in Table 1.

To be on the safe side, maximum stresses S_m were calculated according to (11) for survival wind in all five contraint conditions, and for each member the maximum of the five values S_m was then adopted.

Table 1: Five restraint conditions.

	A			B			C			D			E		
point	x	y	z	x	y	z	x	y	z	x	y	z	x	y	z
1	r	r	r	r	r	r	r	r	r	r	r	r	r	r	r
2	r		r	r		r			r			r	r	r	r
3	r		r	r		r			r			r	r	r	r
4				r						r					
5															
6		r		r				r		r				r	
7		r	r	r		r		r	r	r		r			r
8		r	r	r		r		r	r	r		r			r

5. Wind Deformations

It turned out that $\Delta_x > \Delta_y$. We thus give Δ_x only:

point	deformation
4	0.126 inch
6	0.074 inch
sum =	0.202 inch.

(16)

The vertical distance between points 4 and 6 is 1222 inch. The deformation thus yields a pointing deviation of

$$\alpha = (.202/1222)\ 2.06 \times 10^5 = 34 \text{ arcsec.} \tag{17}$$

Most of (17) stays constant and is corrected by the optical pointing system. For the remaining fraction of (17) we apply two reduction factors:

$$\left.\begin{array}{ll} \text{pressure fraction faster than 1 sec} & = .34 \\ \text{average of independent gusts over dish area} & = .51 \end{array}\right\} \tag{18}$$

and the resulting pointing error then is

$$\Delta\alpha = 5.9 \text{ arcsec.} \tag{19}$$

This result is acceptable. For the shortest wavelength, $\lambda = 2$ cm, the beamwidth is 54 arc sec, and (19) then is 1/9 of the beam, for integration times of 1 sec and shorter. For longer integration times, the errors average out. An integration time of 15 sec, for example, will have a pointing error of 1/20 beamwidth.

II. Design

The design of the towers is done the same way as that of the dish; we use long, pin-ended built-up members. This has two advantages. First, it is economical with respect to stiffness/weight as well as to maximum force/weight. Second, the analysis is broken down in two independent parts; the overall-structure is analysed considering each member as a single rod with given area and density, and the member can be analysed separately. Only two types of members will be considered here.

1. Data for Overall-Structure

The geometry of the towers is shown in Figures 1 and 2, and the coordinates are given in Table 2. Nominal bar areas (see following section) are given in Table 3, together with the length of each member. The last column gives the type.

Table 2: Coordinates (inch)

point	x	y	z
1	0	0	0
2	-1039	1800	0
3	1039	1800	0
4	0	1200	1990
5	0	810	1000
6	0	0	890
7	-2078	0	0
8	2078	0	0

Table 3: Members; A_n = bar area (inch^2), L = length (inch), t = type.

points	A_n	L	t
1-2	70	2078	2
1-3	70	2078	2
1-5	130	1286	1
1-6	30	890	1
1-7	70	2078	1
1-8	35	2078	2
2-3	35	2078	2
2-4	120	2324	2
2-5	50	1749	1
2-7	40	2078	2
3-4	120	2324	2
3-5	50	1749	1
3-8	35	2078	2
4-5	140	1064	1
5-6	40	817	1
6-7	70	2261	1

2. Data for Member Design

The built-up member is shown in Figures 3 and 4. Table 4 gives the length b_c of the center batten (thickness of member) in terms of the member length L, and the bar area A_p of the pyramid, area A_c of the chords, A_b of the battens, A_d of the diagonals, and A_t of the little triangles, all in terms of the nominal member area A_n as used in Table 3.

Table 4: Design of built-up members.

	b_c/L	A_p/A_n	A_c/A_n	A_b/A_n	A_d/A_n	A_t/A_n
type 1	.0557	.421	.2603	.0407	.0444	.0182
type 2	.0808	.411	.2603	.0527	.0424	.0182

Following this procedure then results in a table of bar areas for each bar in each member. All bars are pipes; the actual bar area is taken as the one from the Steel Manual which comes closest (up or down) to the one in the table.

3. Preliminary Data for Foundation

The maximum reactions at the tower legs (point 1 and 3) are about 650 tons downward and 130 tons uplifting. Each tower leg will stand on a support fixed on several cars; support and cars will provide already some counterweight, and the rest may be concrete (or just rocks) in the cars, for balancing the 130 tons. The maximum down force on each leg then is

$$780 \text{ tons.} \tag{20}$$

With a maximum load of 33 tons/axle (see Report 14 of Sept. 1966), we then need 24 axles, or six heavy gondolas of 4 axles each, for each of the four tower legs. They could be arranged on a double railroad as shown in Figure 5.

The maximum forces on the central pintle bearing are about

$$\begin{aligned} &900 \text{ tons down,} \\ &560 \text{ tons horizontal.} \end{aligned} \tag{21}$$

Changes for Tower

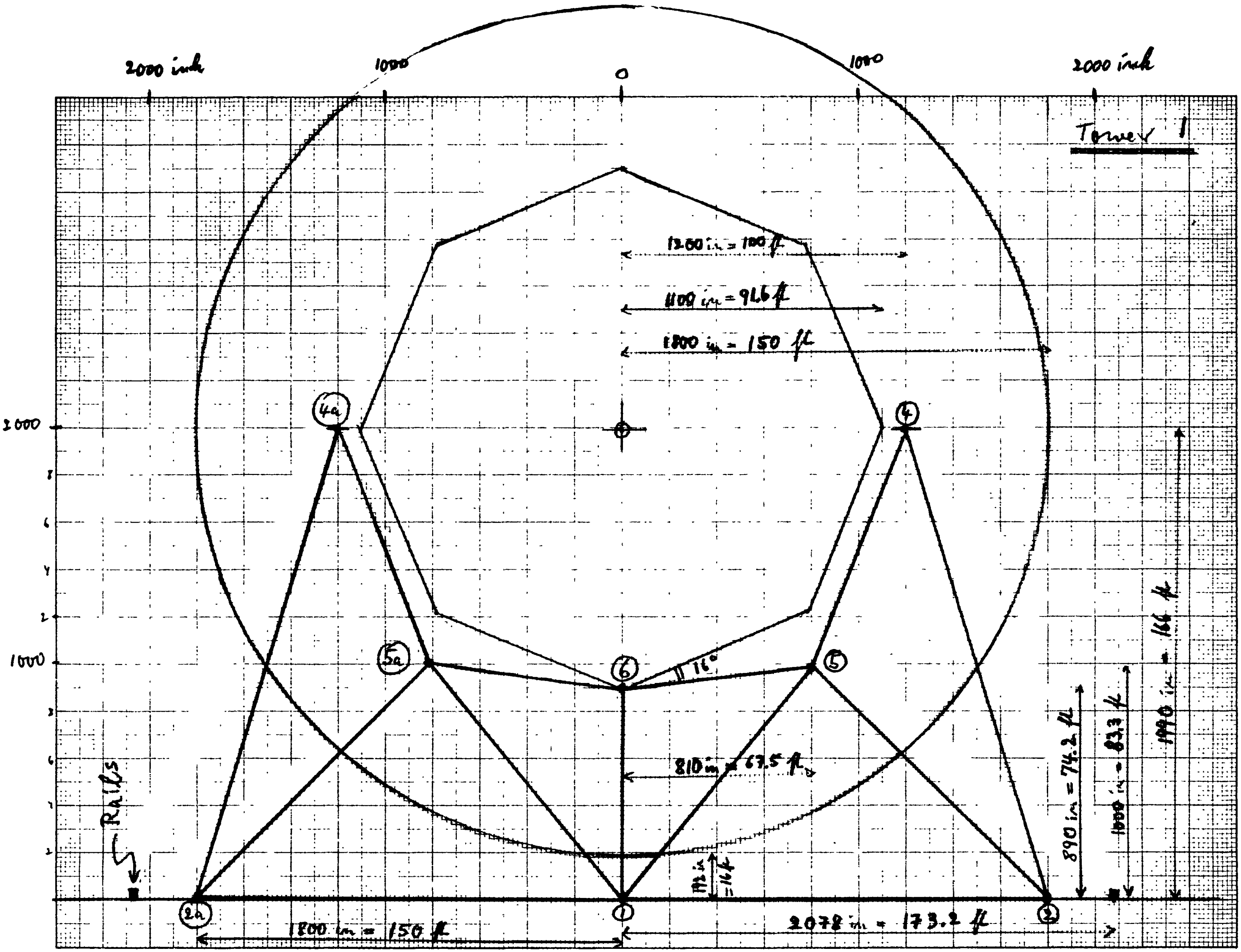
In Report 19 (Oct. 18, 68), please make the following changes:

1. In Table 3:

points	A_n	t
1-2	75	
1-6	20	
1-8	30	
2-3	30	
2-4	140	1
3-4	140	1
3-8	30	
5-6	30	

2. In Table 4:

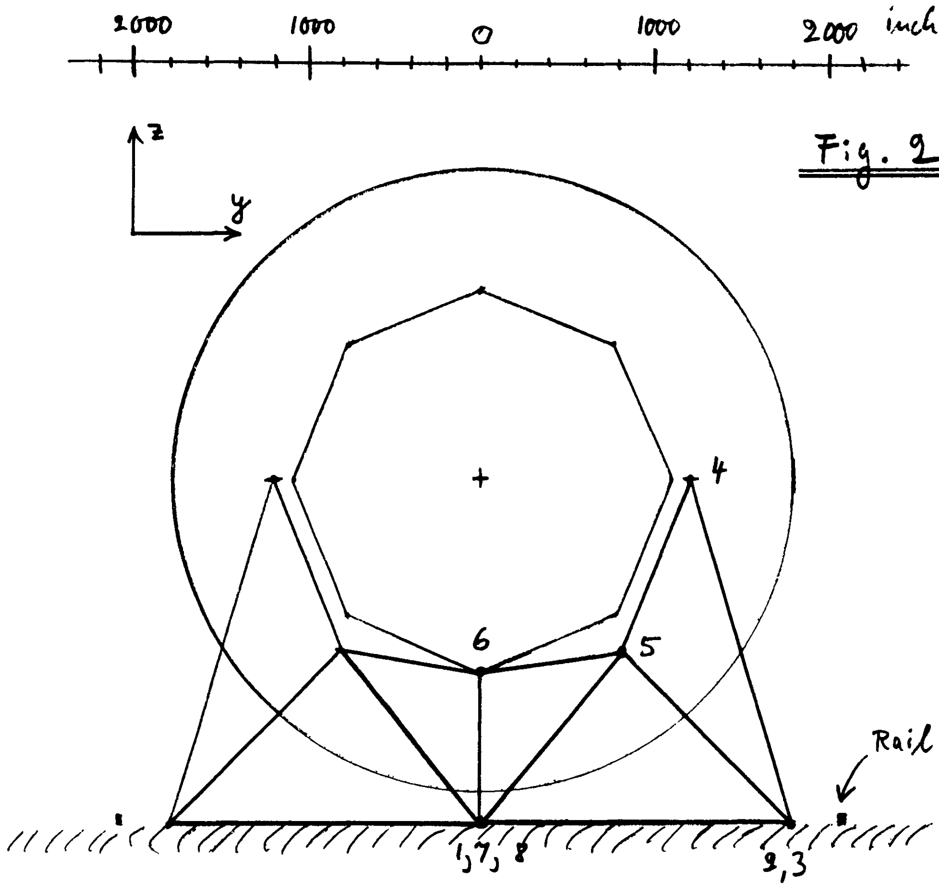
		A_p/A_n	A_c/A_n	A_b/A_n	A_d/A_n	A_t/A_n
type 1		.430	.2738	.0428	.0466	.0191
type 2		.455	.2906	.0589	.0474	.0203



2000 inch
1000
0
1000
2000 inch
Tower 1
1200 in = 100 ft
1100 in = 91.6 ft
1800 in = 150 ft
4a
4
2000
1000
5a
6
16°
5
810 in = 67.5 ft
Rails
890 in = 74.2 ft
1000 in = 83.3 ft
1990 in = 166 ft
192 in = 16 ft
2a
1
2
1800 in = 150 ft
2078 in = 173.2 ft

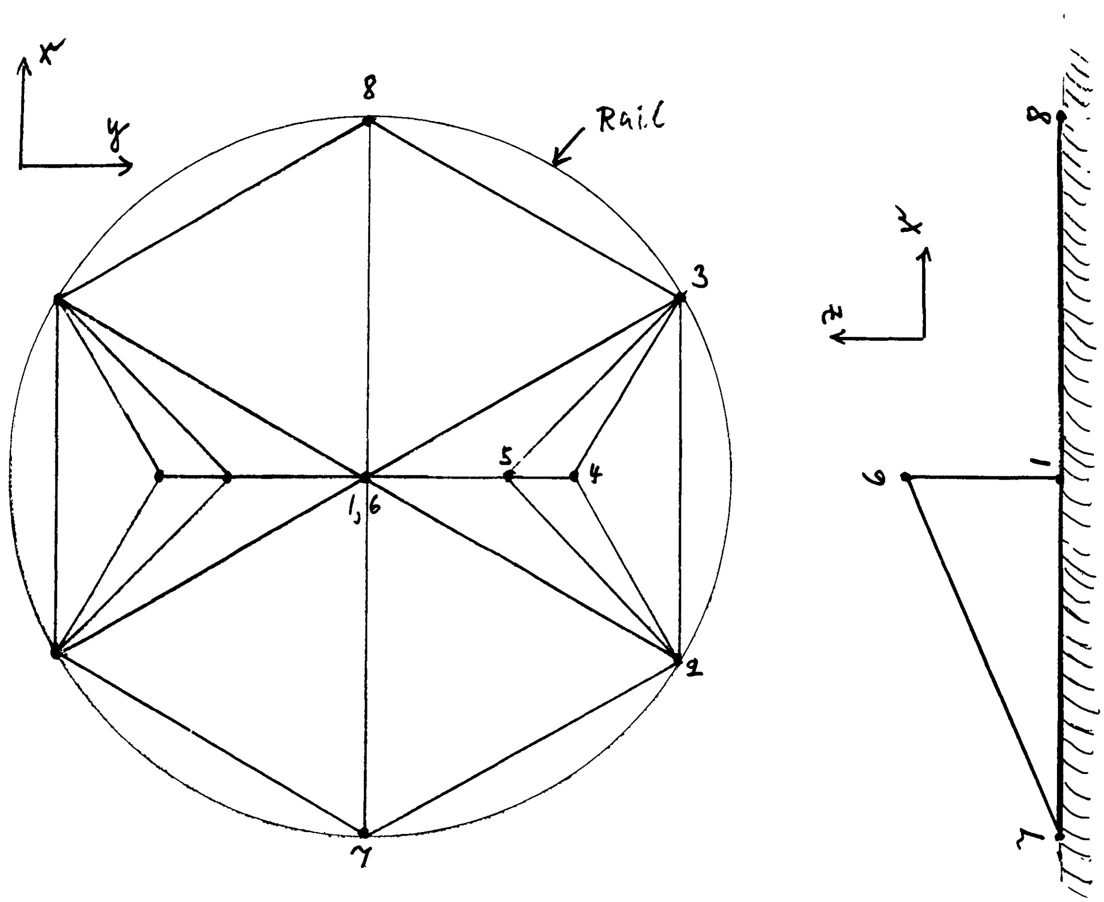
Fig. 1 : Sizes

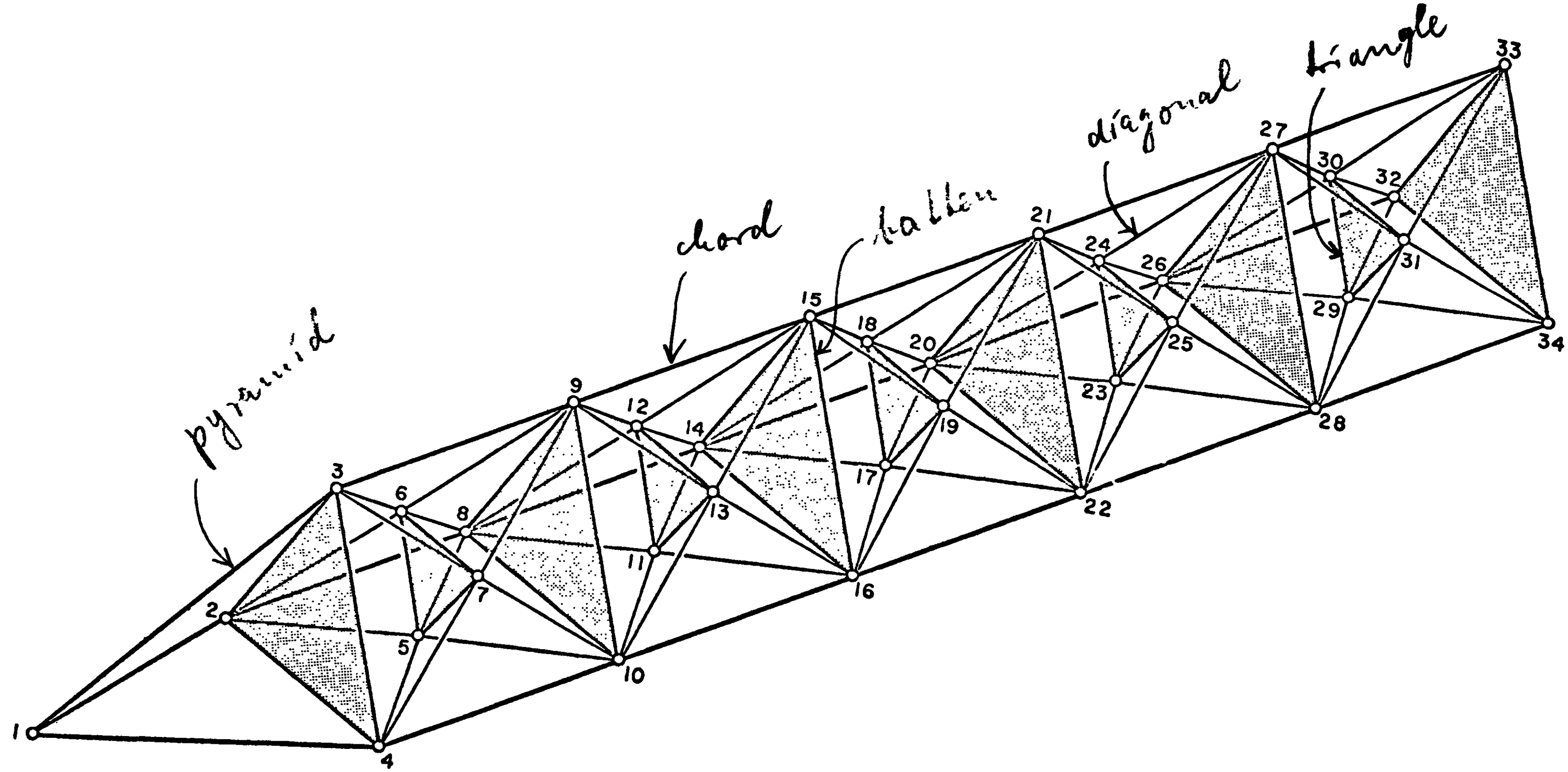
Fig. 2 : Towers



Restraints :

point	x y z
1	
2	
3	
4	
5	
6	
7	
8	





Fig, 3

Points and connections of ½ member.

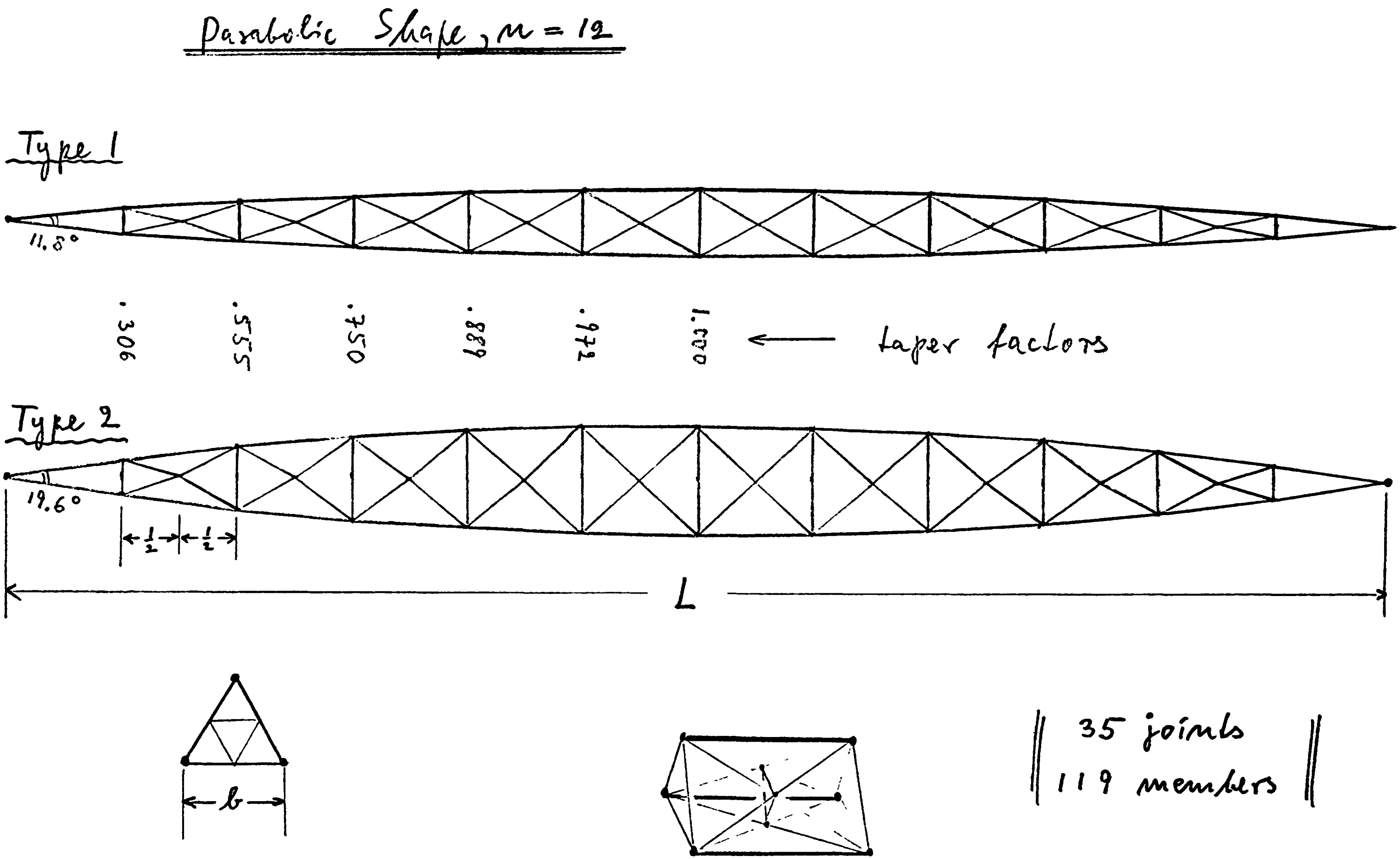


Fig. 4: Geometry of built-up members

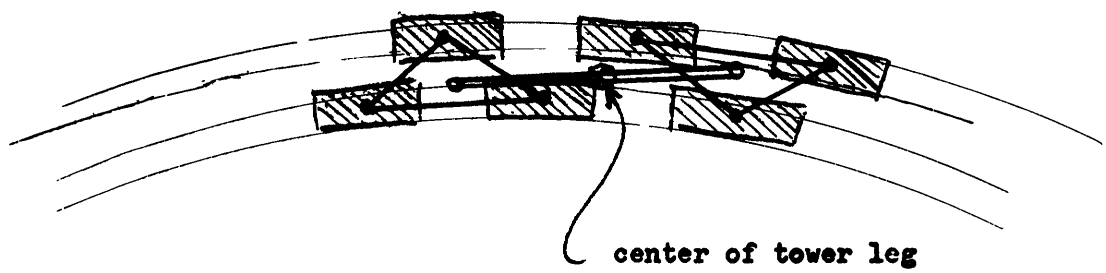


Fig. 5: Support from 6 gondolas for one tower leg.