THESIS

written by

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INDEX

DESIGN OF A SQUARE, DOUBLE CIRCUIT, TRANSMISSION TOWER. SUSPENSION TYPE

The design of a double circuit suspension tower is only a small part of the design of a high tension transmission line. The following outline will give one an idea about the range of the work encountered in the design of a transmission line.

Standard Suspension Towers	
	1. Tower
	2. 7' - 14' - 21' Extension
	3. 7' - 14' - 21' Side Hill Extension.
	4. 5' - 10' One legged Extension
	5. Combination Extensions. Special Details.
	6. Earth Footing
	7. Rock Anchor Footing
	8. Transposition Tower Frame.
	9. Cross Arm Extension & Details For Large Line Angles.
	10. Assembly Of Hardware For 1 & 2 String Suspension.
	11. Suspension Clamp
	12. Lower Shield Ring.
	13. Tie down Attachment.
	14. Ground Wire Attachment
	15. Bird Guards
	16. Connections for Dead End on Suspension Tower.
	17. Special Details & Designs For Increased Broken Wire Tension Conditions on Long Spans.
	18. Upper Arcing Horn.

Conductor Data	1. General Catenary Chart for Computing sag & Tension in Conductor and Ground Wire. 2. Sag & Tension Tables in the Sleet & Non-Sleet Regions. 3. Instructions to Fieldmen Regarding Use of Tables and Charts. 4. Templates For Field & Office Use. 5. Test Line
Miscellaneous	1. Data for Location of all Transpositions Throughout the line. 2. Data Sheets showing Locations of all Towers giving the Number, Type, Span, Elevation of Footings, Type of each footing, Length of Span, Jumper Loop Stiffeners, Tie Downs etc. 3. Profiles showing Span & Tower Data. 4. Summary Chart showing sags 5. General design Chart showing Loading used on all Towers, Number of broken Wires considered, Method of Applying Loads to Towers., Calculation of Stresses, Stress Diagrams of All Towers, Summary of all stresses in such Towers 6. Specifications for Structural Steel Work 7. Specifications for Hardware. 8. General Specifications for Construction. 9. Inspection & Testing of Towers, Steel, Hardware, Conductors & Insulators.
Erection Tools	1. Pull up Jacks. 2. Dynanometer 3. Stringing Sheaves. 4. Come along Clamp. 5. Cable hoisting & Lowering Rig. 6. Portable Reel Stands. 7. Tower Raising Hoist 8. Drilling & Excavating Tools 9. Tools for Sagging Line 10. Conductor Splice

Anchor Angle Tower	1. Tower 2. 7'-14'-21' Extensions. 3. Footing For Anchor Tower { Light Loading / Heavy Loading 4. Assembly of Dead End Hardware Arrangement 5. Dead End Cable Clamp. 6. Adjustable Link for Dead End Cable Clamp. 7. Yoke with Attachment For Shield Ring 8. Clevise Attaching Yoke to Insulator on the Dead End of the String. 9. Clevise For Attaching Insulator to the Yoke on the Tower End of the String. 10. Arcing Horn 11. Shield Ring 12. Links For Attaching Yoke to Tower Giving Vertical and also Horizontal Movement. 13. Jumper Loop Details & Connectors. 14. Assembly of Ground Wire Attachment 15. Socket For Dead Ending Ground Line. 16. Bird Guards 17. Connections For Suspension String on Anchor Towers. 18. Clevise For Giving Horizontal & Vertical Movement in Attaching Ground Wire to Tower.
Transposition Towers	1. General Arrangement for Transpositions 2. Special Details.
Disconnecting Towers	1. General Arrangement and Details For Towers. 2. Detail of Platform.
Dead End Racks	1. Location and General Arrangement of Dead End Racks. 2. Design & Detail of Racks.
Terminals	1. Plans of tie end.

GENERAL ASSUMPTIONS & REQUIREMENTS FOR LOADING

The Economical span for wide base towers ranges from 600 to 800 ft. A 700' span is assumed. For this span (from curves) the max vertical sag for stranded copper cables is about 15' at 32° Fahrenheit

Minimum Clearance: 30 ft. between ground and lowest conductor at center of span, giving a required height of 45 ft above the ground to lowest crossarm.

Wind ~~pressure~~ velocity: about 70 miles per hour

Factor of safety for cables = 2.

Factor of safety for steel structure = 2.5

Wind pressure at steel structure = 20#/□'

Factor of safety for cables is made up as follows:

a) Increased loading = 0.30	= 0.30
b) Uncertain strength of material	= 0.20
c) Injuries during erection	= 0.10
d) Errors in erection	= 0.30
e) Deterioration of material	= 0.10
Breaking strength	= 1.00
Total factor of safety	= 2.00

Weight of ice of cable = 57#/cub. ft.

Continued

There will be assumed no ice on cables but an increased wind velocity of approx. 77 miles per hour which will correspond to a pressure of 15# per square foot on the wire cables according the (curves of indicated and actual wind velocity).

~~A~~ Bare stranded copper cables are assumed

Gage = 1,000,000 circular mils.

Diameter = 1.152" Area = .785 □"

Ultimate strength = 55000#/□"

Elastic limit = 33000#/□"

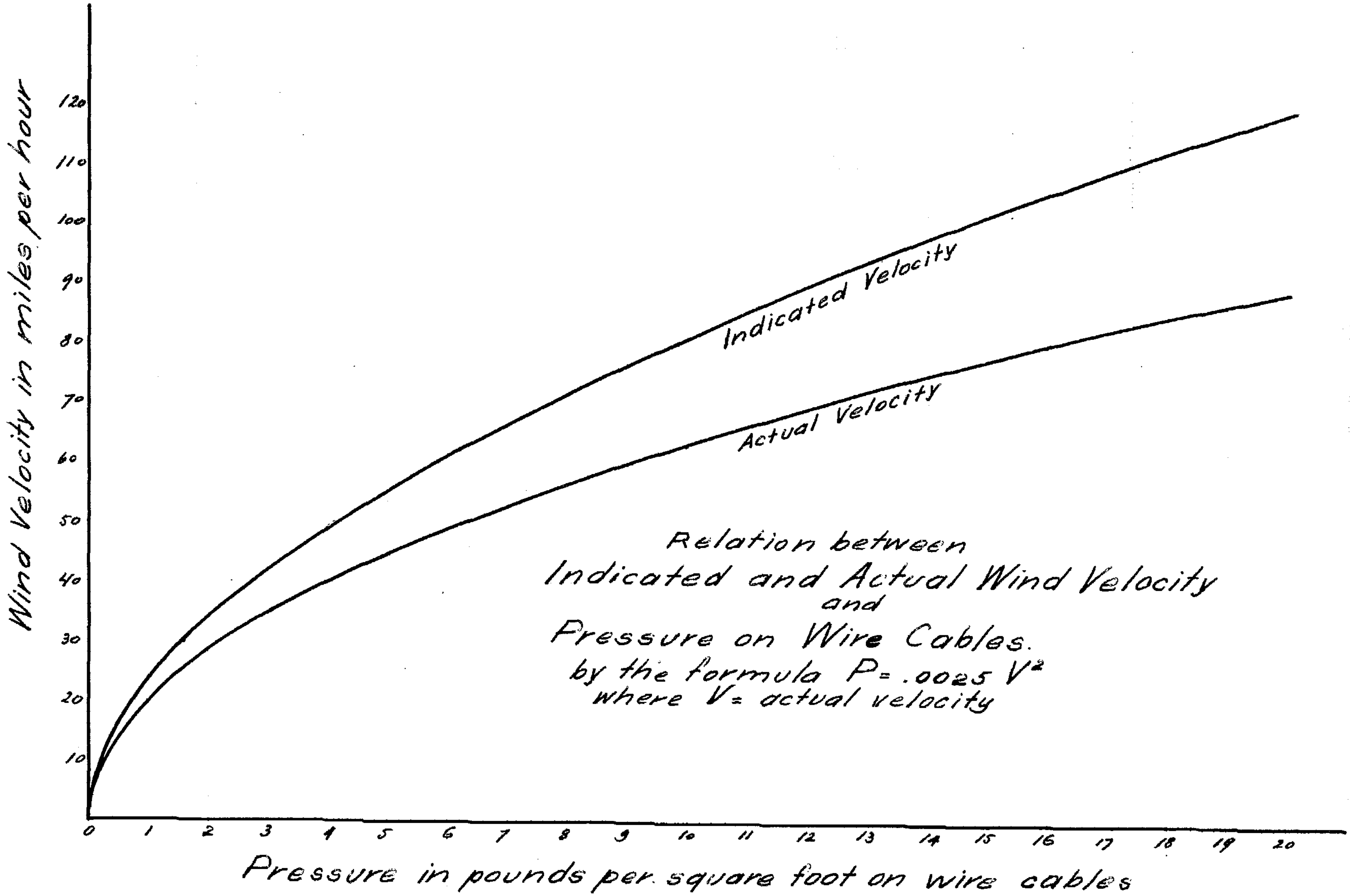
The following loadings are derived from these assumptions:

Wind on wires = 1000# at each wire support normal to the direction of line

Wire pull = 5000# at each of any two conductor supports at the same side of the tower.

Vertical loads = 1000# at each wire support

Usually, in practice, a broken-wire loading of two broken conductors is assumed.

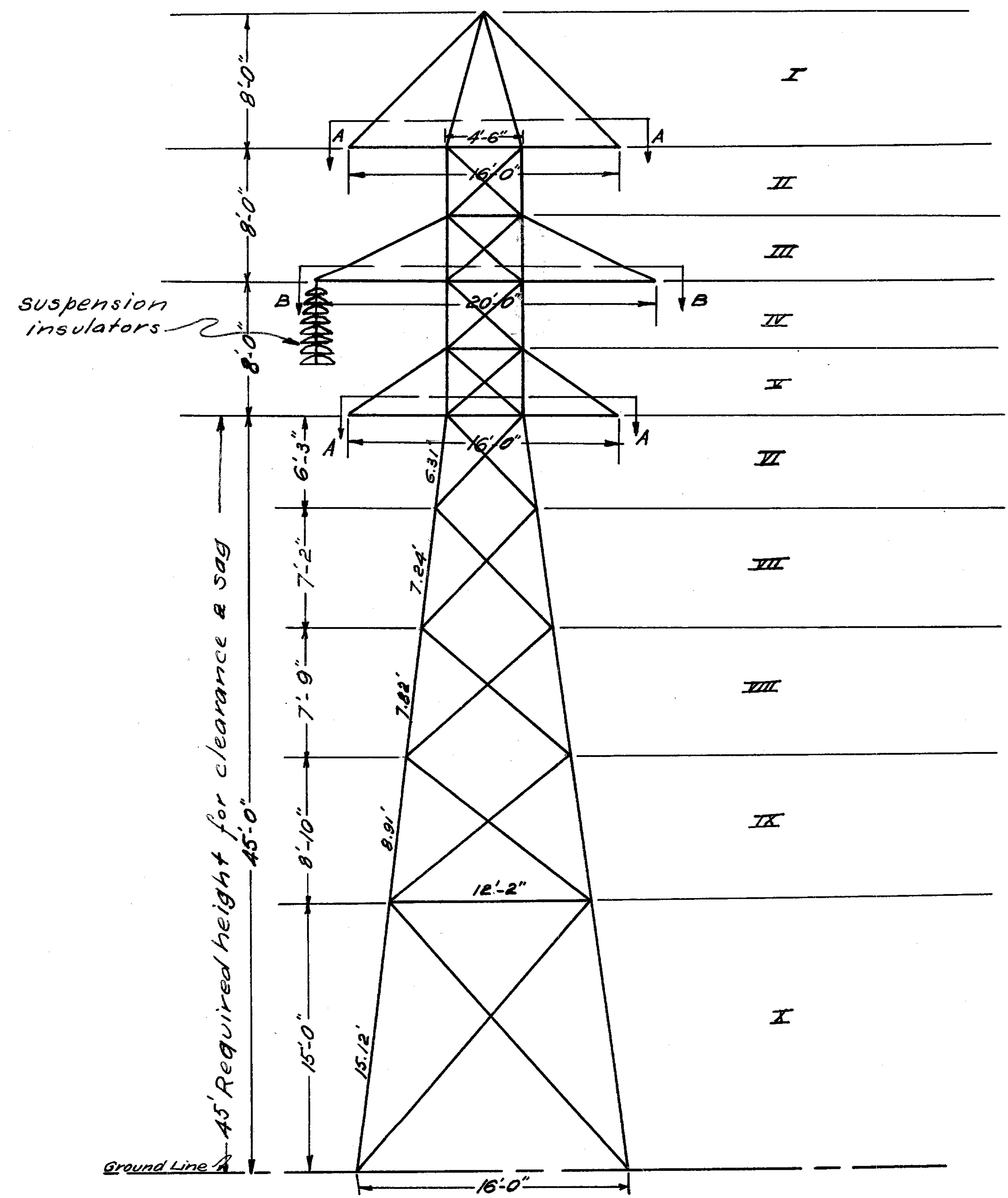

Wind Velocity in miles per hour
120
110
100
90
80
70
60
50
40
30
20
10
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
Indicated Velocity
Actual Velocity
Relation between
Indicated and Actual Wind Velocity
and
Pressure on Wire Cables.
by the formula $P = .0025 V^2$
where V = actual velocity
Pressure in pounds per square foot on wire cables

Square, double circuit Transmission Tower

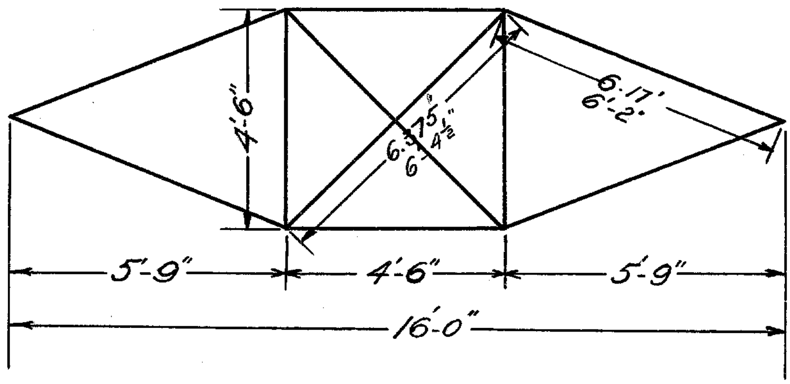
Suspension Type

One ground wire

Scale $\frac{1}{8}$" = 1'-0"

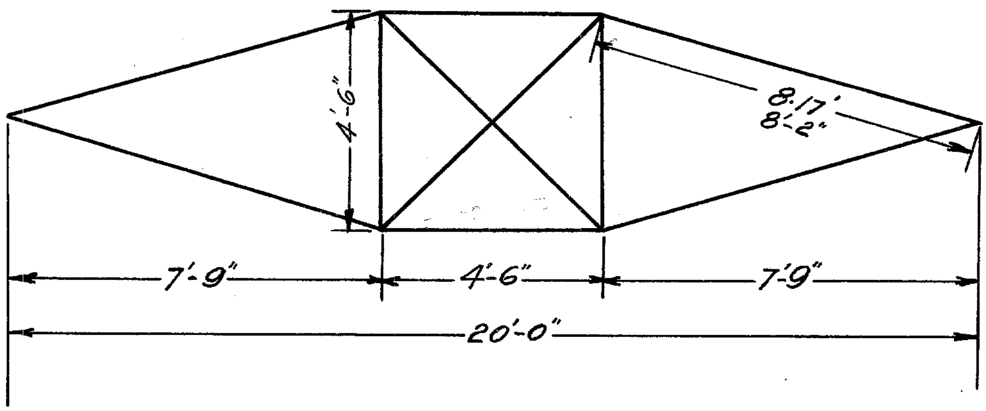


"A- A"



Scale $\frac{1}{4}$" = 1'-0"

"B - B"



Conditions of loading

Wire pull = 5000# at each of any two conductor supports at the same side of the tower

Two broken conductors

Wind on Wires = 1000# at each wire support normal to direction of line

Wind on Tower = 20# per sq. ft. of exposed area of each face normal to direction of line.

Vertical loads = 1000# at each wire support

Dead Load of Tower = 7000#

Redundant Members

We will assume that the double diagonal compressive system of framing is used throughout the tower except in the bottom panel. Investigating the static condition of the structure, we find 135 members, 41 joints and 4 rigid supports.

If m = total number of joints
 n = number of members
 r = number of rigid supports

We then have $n + 3r$ unknown quantities.

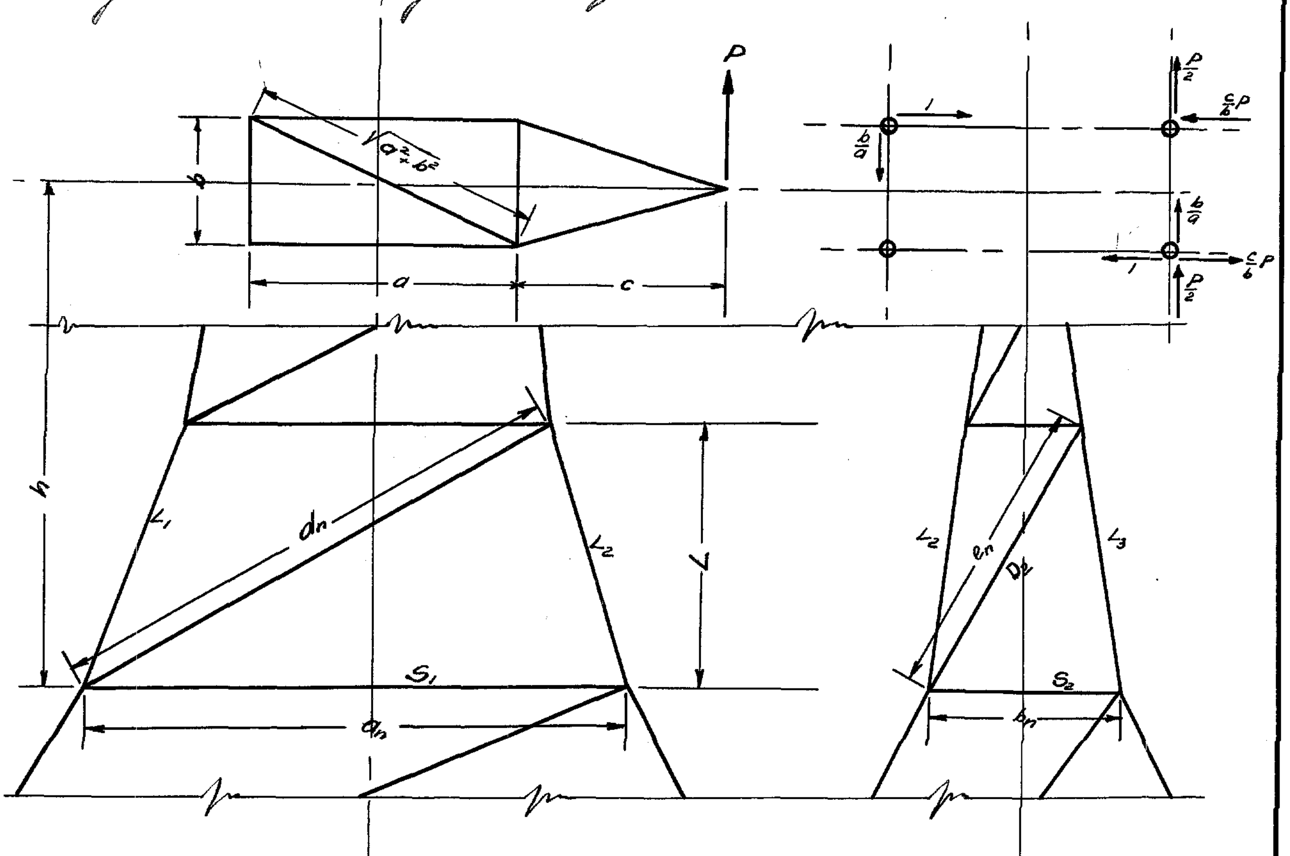
To form a statically determinate structure we must have $n = 3(m-r)$ necessary members.

Number of necessary members $= 3(41-4) = 111$

hence we have 24 redundant members, ~~these~~ 20 horizontal struts, 1 ground wire support member and 3 cross arm diagonals.

Horizontal Bracing

In practice one usually provides for the horizontal bracing of the cross arms. These horizontal diagonals will receive practically no stress when the loading is symmetrical, but, when the tower is subjected to torsion they materially change the stress distribution



L = Corner posts
S = Struts
D = Diagonals
K_1 = Coefficient
K_2 = Coefficient

Assume that:

S = Stress in any of the necessary members

S_1 = Stress in the crossarm diagonal.

S' = Stress in any of the necessary members with the cross arm diagonal removed.

u = Stress in any of the necessary members due to a unit tension in the cross arm diagonal

E = modulus of elasticity A = area of cross section

l = length of any member.

Then from the theory of statically indeterminate structures $S = S' + u'S_1$ (1)

and $S_1 = - \dfrac{\Sigma \frac{S'ul}{AE}}{\Sigma \frac{u^2 l}{AE}}$ (2)

The figure on the former page represents any rectangular tower with a diagonal brace of the cross arm. The external loading consists of an unbalanced pull P, which can be replaced by a couple $\frac{c}{b} P \times b$, and a shear force P, acting at the right side of the tower. The unit tension in the diagonal we assume to be $\sqrt{(\frac{b}{a})^2+1}$, which can be replaced by its components, $\frac{b}{a}$ & 1. Taking any intermediate panel of the tower we get

Member	S'	u	$\frac{l}{A}$	$\frac{S'ul}{A}$	$\frac{u^2 l}{A}$
L_1	$+\frac{hc}{ba_n}P$	$-\frac{h}{a_n} - \frac{bh}{ab_n}$	$\frac{L}{A_1}$	$-\frac{LP}{A_1}\left(\frac{h^2c}{a_n^2 b} + \frac{h^2c}{aa_nb_n}\right)$	$\frac{h^2L}{A_1}\left(\frac{1}{a_n^2} + \frac{b^2}{a^2b_n^2} + \frac{2b}{aa_nb_n}\right)$
L_2	$-\frac{hc}{ba_n}P + \frac{h}{b_n}P$	$+\frac{h}{a_n} + \frac{bh}{ab_n}$	$\frac{L}{A_1}$	$-\frac{LP}{A_1}\left(\frac{h^2c}{a_n^2 b} + \frac{h^2c}{aa_nb_n} - \frac{h^2}{a_nb_n} - \frac{h^2b}{ab_n^2}\right)$	ditto
L_3	$+\frac{hc}{ba_n}P - \frac{h}{b_n}P$	$-\frac{h}{a_n} - \frac{bh}{ab_n}$	$\frac{L}{A_1}$	$-\frac{LP}{A_1}($ " $)$	ditto
L_4	$-\frac{hc}{ba_n}P$	+ " + "	$\frac{L}{A_1}$	$-\frac{LP}{A_1}\left(\frac{h^2c}{a_n^2 b} + \frac{h^2c}{aa_nb_n}\right)$	ditto
S_1	$-K_1\frac{c}{b}P$	$+K_1$	$\frac{a_n}{A_2}$	$-\frac{K_1^2P}{A_2} \times \frac{a_nc}{b}$	$K_1^2\frac{a_n}{A_2}$
S_2	$-K_2P$	$-K_2\frac{b}{a}$	$\frac{b_n}{A_2}$	$+\frac{K_2^2P}{A_2} \times \frac{bb_n}{a}$	$K_2^2\frac{b^2b_n}{a^2A_2}$
S_3	$-K_1\frac{c}{b}P$	$+K_1$	$\frac{a_n}{A_2}$	$-\frac{K_1^2P}{A_2} \times \frac{a_nc}{b}$	$K_1^2\frac{a_n}{A_2}$
S_4	0	$-K_2\frac{b}{a}$	$\frac{b_n}{A_2}$	0	$K_2^2\frac{b^2b_n}{a^2A_2}$
D_1	$+K_1\frac{cd_n}{ba_n}P$	$-K_1\frac{d_n}{a_n}$	$\frac{d_n}{A_3}$	$-\frac{K_1^2P}{A_3} \times \frac{cd_n^3}{ba_n^2}$	$K_1^2\frac{d_n^3}{a_n^2A_3}$
D_2	$+K_2\frac{e_n}{b_n}P$	$+K_2\frac{be_n}{ab_n}$	$\frac{e_n}{A_3}$	$+\frac{K_2^2P}{A_3} \times \frac{be_n^3}{ab_n^2}$	$K_2^2\frac{b^2e_n^3}{a^2b_n^2A_3}$
D_3	$+K_1\frac{cd_n}{ba_n}P$	$-K_1\frac{d_n}{a_n}$	$\frac{d_n}{A_3}$	$-\frac{K_1^2P}{A_3} \times \frac{cd_n^3}{ba_n^2}$	$K_1^2\frac{d_n^3}{a_n^2A_3}$
D_4	0	$+K_2\frac{be_n}{ab_n}$	$\frac{e_n}{A_3}$	0	$K_2^2\frac{b^2e_n^3}{a^2b_n^2A_3}$

Adding up the values of $\frac{S'ul}{A}$ and $\frac{u^2l}{A}$, assuming the summation to extend over the entire structure we get:

see next page

equation (3):

$$\frac{S_1}{\sqrt{1+\left(\frac{b}{a}\right)^2}} = P\,\frac{\Sigma \frac{Lh^2}{A_1}\left(\frac{4c}{a_n^2 b} + \frac{4c}{a a_n b_n} - \frac{2}{a_n b_n} - \frac{2b}{a b_n^2}\right) + \Sigma \frac{1}{A_2}\left(2K_1^2 \frac{a_n c}{b} - K_2^2 \frac{b b_n}{a}\right) + \Sigma \frac{1}{A_3}\left(2K_1^2 \frac{c d_n^3}{a_n^2 b} - K_2^2 \frac{b e_n^3}{a b_n^2}\right)}{4\Sigma \frac{Lh^2}{A_1}\left(\frac{1}{a_n^2} + \frac{b^2}{a^2 b_n^2} + \frac{2b}{a a_n b_n}\right) + 2\Sigma \frac{1}{A_2}\left(K_1^2 a_n + K_2^2 \frac{b^2 b_n}{a^2}\right) + 2\Sigma \frac{1}{A_3}\left(K_1^2 \frac{d_n^3}{a_n^2} + K_2^2 \frac{b^2 e_n^3}{a^2 b_n^2}\right)}$$

This is the general expression for S_1, which will be simplified in our case of the square tower with all four sides of equal rigidity

$a = b \quad a_n = b_n \quad K_1 = K_2$ and

$$S_1 = P\,\frac{2c - a}{4a}\sqrt{2} \quad (4)$$

Ground Wire Support

The common square type of transmission tower generally has its ground wire support formed by four members coming together to a point at the extreme top (see figure). Only three members are necessary to form a rigid support, giving one redundant member.

Wire Pull.

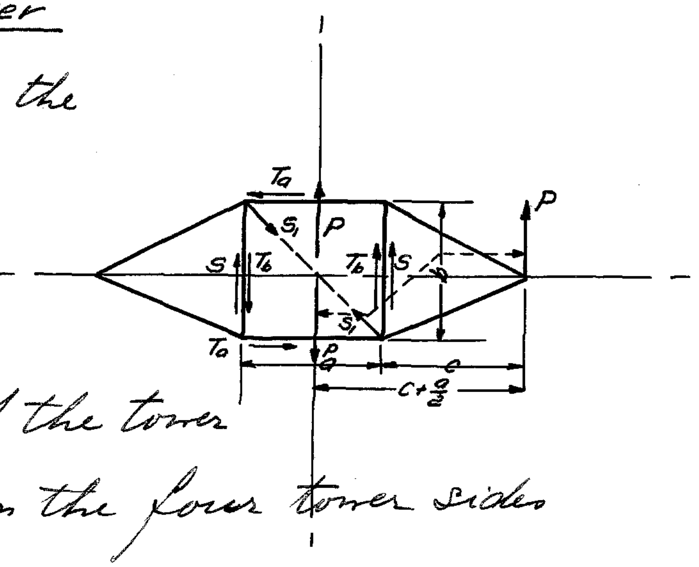
The maximum stresses due to wire pull will, in this case, be produced. when the two upper or two lower conductors are broken. In this case we assume the two upper conductors to be broken.

Double Diagonal Systems

It is found economical to use a double diagonal compressive system of framing for the sides of a tower when its width is small compared to its height. All horizontal struts will then become redundant, and theoretical investigations show that this form of redundancy can be taken care of, with a small percentage of error, by assuming ~~&~~ that each diagonal system carries one-half of the load, and that the struts carry the necessary amount of horizontal shear to give an equal load distribution.

Distribution of Torsion in Tower

The force P, acting at the end of a cross arm, can be replaced by a torsion moment, $P(c+\frac{a}{2})$, and a shear force. P. acting at the center of the tower



Assume the influence on the four tower sides to be expressed by the forces S, T_a, T_b & S_1, located as shown in the figure.

Then $T_a b + T_b a = P\left(c + \frac{a}{2}\right)$

$$S = \frac{P}{2}$$

If $a = b$ as in our case and the tower has diagonal bracing we get applying equation (4)

$$T_b = P\frac{2c-a}{4a} + \frac{P}{2} \quad (5) \qquad T_a = \frac{c}{a}P - P\frac{2c-a}{4a} \quad (6)$$

and $\frac{T_a}{T_b} = 1$ which shows that with diagonal cross arm bracing and equal rigidity of the four sides, the torsion is equally divided between the four sides of a square tower.

To arrive at the stresses due to wire pull and with the two upper conductors broken we can draw separate shear and torsion stress diagrams.

As the tower is square and symmetrical design, the torsion will be divided equally between the four sides. Using equation (4) we get for the upper & lower cross arms:

$$S_1 = 5000\,\frac{11.5 - 4.5}{4 \times 4.5}\sqrt{2} = 1940\sqrt{2}$$

for <u>middle cross arm</u> $S_1 = 5000\,\frac{15.5 - 4.5}{4 \times 4.5}\sqrt{2} = 3060\sqrt{2}$

<u>Upper & Lower Cross Arm</u>

$$S = \tfrac{1}{2} \times 5000 = 2500\#$$

$$T = \frac{5000 \times 8}{2 \times 4.5} = 4450\#$$

<u>Middle cross arm</u>

$$S = \tfrac{1}{2} \times 5000 = 2500\#$$

$$T = \frac{5000 \times 10}{2 \times 4.5} = 5560\#$$

The stresses due to wire pull with 2 broken conductors can also be obtained by drawing a diagram with shear and torsion combined, and algebraically we get using equation (5) & (6)

Upper & Lower Cross Arm

(6) Face A $P_a = \left(\frac{5.75}{4.50}\right) 5000 - 1940 = 4450$ (= Torsion)

(5) Face B $P_b = 5000 + 1940 = 6940$ (= Shear + T)

Face C $P_c = \frac{5.75}{4.5} \times 5000 - 1940 = 4450$ (= T)

Face D $P_d = 1940$ (= T - S)

Middle Cross Arm

Face A $P_a = \frac{7.75}{4.5}(5000) - 3060 = 5560$ (= T)

Face B $P_b = 5000 + 3060 = 8060$ (= S + T)

Face C $P_c = 5560 = 5560$ (= T)

Face D $P_d = \quad = 3060$ (= T - S)

Notes on Stress Diagrams

Separate shear and torsion diagrams are drawn. The stress in the strut at right angles to the face in which the forces are acting, is found in the diagram by drawing the line ac, where bc indicates the stress in question.

We find out of these diagrams, that when the double compressive diagonal system of framing is used, there are no stresses in the corner posts due to torsion.

Then ~~the~~ further the wind diagrams W.W. and W.T. are constructed.

Only the lower half of the diagram for vertical load of wires is constructed. The dash lines indicate the actual stresses in the cross arm members and also the stresses in the struts at right angles to the plane of the diagram.

Dash lines have been used in all diagrams to indicate projected ~~or~~ repeated stresses shown elsewhere in the diagram previously or any stress which ~~did~~ is not part of the diagram proper.

Cross Arm Stresses

There will be some stresses in the horizontal struts which do not appear in the stress diagrams, if we assume that the forces due to wire pull are all acting on the left side of the tower face.

Face A — $\frac{c}{a}P - \frac{S_1}{\sqrt{2}}$
Face B — $+\frac{P}{2}$
Face C — $-\frac{S_1}{\sqrt{2}}$

For Upper & Lower Cross Arms:

Face A — $\frac{5.75}{4.5} \times 5000 - 1940 = +4450$
Face B — $\frac{5000}{2} = +2500$
Face C — $-\frac{1940\sqrt{2}}{\sqrt{2}} = -1940$

Middle Cross Arm

Face A — $\frac{7.75}{4.5} \times 5000 - 3060 = +5560$
Face B — $\frac{5000}{2} = +2500$
Face C — $-\frac{3060\sqrt{2}}{\sqrt{2}} = -3060$

Other cross arm stresses need no special explanation.

Stress Tables

The stresses found by the stress diagrams are put in tables and are in thousands of pounds. One has to remember that a horizontal strut which is located at a point where the corner posts change direction, will be stressed from forces occurring in the tower face at right angles to the one in which the strut is located.

In our case struts I-II and V-VI will receive such stresses due to shear, wind load and vertical load.

Also the cross arm will produce stresses in the struts at the points where they join the tower, so, a vertical load at the end of a cross arm will cause tension in the strut between the upper ends of the hangers and compression in the strut below which joins the ends of the horizontal cross arm members.

To get the maximum load in the strut ~~VI~~ V-VI and the lower cross arm, we have to consider the two lower conductors to be broken.

Corner Posts

Panel	I	II	III	IV	V	VI	VII	VIII	IX	X
S	—	-1.05	-3.30	-6.65	-11.00	-14.05	-15.40	-16.35	-16.95	-17.75
T	—	—	—	—	—	—	—	—	—	-3.50
W.W.	-1.00	-1.55	-2.85	-4.55	-6.75	-8.65	-9.85	-10.70	-11.40	-12.00
W.T.	-0.16	-0.25	-0.47	-0.78	-1.18	-1.56	-1.84	-2.17	-2.53	-3.35
V.L.W.	-0.80	-0.75	-1.25	-1.25	-1.75	-1.75	-1.75	-1.75	-1.75	-1.75
V.L.T	-0.10	-0.25	-0.40	-0.60	-0.75	-0.90	-1.00	-1.25	-1.50	-1.75
	-2.06	-3.85	-8.27	-13.83	-21.43	-26.91	-29.84	-32.22	-34.13	-40.10

	Cross Arms		Hangers		
Panel	A-A	B-B	I	III	V
W.W.	±0.53	±0.54	—	—	—
V.L.W	-0.74	-1.00	+1.38	+1.11	+0.89
C.A.	±6.50	±8.80	—	—	—
	-7.77	-10.54	+1.38	+1.11	+0.89

Diagonals – Face A

Panel	II	III	IV	V	VI	VII	VIII	IX	X
T	± 3.00	± 3.00	± 6.70	± 6.70	± 6.70	± 4.70	± 3.50	± 2.60	± 4.70
W.W.	± 0.95	± 0.95	± 1.65	± 1.65	± 1.00	± 0.70	± 0.50	± 0.40	± 0.73
W.T.	± 0.14	± 0.20	± 0.26	± 0.34	± 0.18	± 0.22	± 0.26	± 0.30	± 0.85
	± 4.09	± 4.15	± 8.61	± 8.69	± 7.88	± 5.62	± 4.26	± 3.30	± 6.28

Diagonals – Face B

Panel	II	III	IV	V	VI	VII	VIII	IX	X
S	± 1.70	± 1.70	± 3.35	± 3.35	± 1.05	± 0.75	± 0.50	± 0.40	± 0.75
T	± 3.00	± 3.00	± 6.70	± 6.70	± 6.70	± 4.70	± 3.50	± 2.60	± 4.70
	± 4.70	± 4.70	± 10.05	± 10.05	± 7.75	± 5.45	± 4.00	± 3.00	± 5.45

Struts - Face A & C

Panel	I-II	II-III	III-IV	IV-V	V-VI	IX-X
S	—	—	—	—	±1.70	—
T	-2.23	—	-2.70	—	—	-1.80
C.A.	-1.94	—	-3.06	—	—	—
W.W	—	—	—	—	—	-0.30
W.T.	—	—	—	—	—	-0.23
V.L.W.	-0.27	+0.95	-0.95	+0.69	-0.69	—
V.L.T.	—	—	—	—	-0.10	—
	-4.44	+0.95	-6.71	+0.69	-2.49	-2.33

Struts - Face B & D

Panel	I-II	II-III	III-IV	IV-V	V-VI	IX-X
S	±1.25	—	±1.20	—	—	±0.30
T	-2.23	—	-2.70	—	—	-1.80
C.A.	+2.50	—	+2.50	—	—	—
W.W.	±0.20	—	±0.15	—	±0.20	—
W.W.	∓0.20	—	—	—	±1.00	—
W.T.	±0.04	—	—	—	±0.18	—
V.L.W.	+0.38	-0.28	+0.28	-0.28	+0.28	—
V.L.T.	—	—	—	—	-0.10	—
-	-0.64 —	-0.28 —	-1.27 —	-0.28 —	-1.20 +1.56	-2.10 —

Strut V-VI

Face	A	B	C	D
S	+0.55	-1.20	-0.55	+1.20
T	-2.23	-2.23	-2.23	-2.23
C.A.	+4.45	+2.50	-1.94	—
W.W.	—	±0.20	—	∓0.20
W.W.	—	±1.00	—	∓1.00
W.T.	—	±0.18	-	∓0.18
V.L.W.	-0.69	+0.28	-0.69	+0.28
V.L.T.	-0.10	-0.10	-0.10	-0.10
	+1.98	-2.13	<u>-5.51</u>	-2.23

2 lower conductors broken.

As for the torsion distribution, all four sides of the tower are of equal rigidity, and for designing purposes we use only the maximum stresses, which are underlined in the stress tables.

Forces on Foundations

Two cases of loading on the foundations can be considered, due to a reversal of the wind, which will reverse the wind load.
A (+) sign in the tables and diagrams indicates a shear force acting inwards and a (-) sign a force acting outwards on the foundation.
The following tables give the forces indicated in the stress diagrams, acting on the foundation

Vertical Forces on Foundations

	Case I				Case II			
	1	2	3	4	1	2	3	4
S	+17,600	+17,600	-17,600	-17,600	+17,600	+17,600	-17,600	-17,600
W.W	-12000	+12000	+12000	-12000	+12000	-12000	-12000	+12000
W.T.	-3,300	+3,300	+3,300	-3,300	+3300	-3,300	-3,300	+3,300
V.L.	-3,500	-3,500	-3500	-3,500	-3,500	-3,500	-3,500	-3,500
	+1,200	+29,400	-5,800	-36,400	+29,400	+1,200	-36,400	-5,800

(-) = Compression
(+) = Tension

Shear Forces on Foundations

Case I

	Direction of Line				Across Line			
	1	2	3	4	1	2	3	4
S	+2300	+2700	-2300	-2700	+2300	+2300	-2300	-2300
T	—	+2800	—	+2800	+2800	—	+2800	—
W.W.	-1500	+1500	+1500	-1500	-2000	+1500	+2000	-1500
W.T.	-430	+430	+430	-430	-930	+430	+930	-430
V.L.	-430	-430	-430	-430	-430	-430	-430	-430
Σ	-60	+7,000	-800	-2,260	+1,740	+3,800	+3,000	-4,660

Case II

	Direction of line				Across line			
Stresses	1	2	3	4	1	2	3	4
S	+2,300	+2,700	-2,300	-2,700	+2,300	+2,300	-2,300	-2,300
T	—	+2,800	—	+2,800	+2,800	—	+2,800	—
W.W.	+1,500	-1500	-1500	+1,500	+2,000	-1,500	-2,000	+1,500
W.T.	+430	-430	-430	+430	+930	-430	-930	+430
V.L.	-430	-430	-430	-430	-430	-430	-430	-430
Σ	+3,800	+3,140	-4,660	+1,600	+7,600	-60	-2,860	-800

Total Maximum Forces on Foundations

Maximum Compression (post 3) = -36,400#

Max. Shear in direction of line = - 4,860#

Maximum Shear across line = - 2860#

Maximum Uplift (post 1. case II) = + 29,400#

Max. Shear in direction of line = + 3,800#

Max. shear across line = + 7,600#

WORKING STRESSES

Ultimate
Maximum Column Stress : $45000 - 165 \frac{l}{r}$

Max. Slenderness Ratio :
- Main Members = 125
- Secondary Members = 175
- Redundant Members = 250

Bearing :

Shop & Field Rivets = 30000#/□" & 24000#/□"

Tension :

Rolled & Cast Steel = 16000#/□"

Bolts (Net area) = 12000#/□"

Shear :

Field & Shop Rivets = 15000#/□" & 12000#/□"

Cast Steel = 9000#/□"

7/8" Bolts = 7000# - 8500#/□"

Allowable Pressure on Soil = 4000#/□'

Assuming a factor of safety of $2\frac{1}{2}$

Safe Working $S_{comp.} = 18000 - 66 \frac{l}{r}$

Common use for calculating safe loads

$S_{comp.} = 16000 - 70 \frac{l}{r}$

The shearing strength of rivets and bolts per square inch multiplied by the area of the cross section and assuming that 48000 - 58000 - lb material is used we have the following shearing values:

Diameter of Rivet or bolt in inches	Ultimate shear 36000#/□"	Working Values (lbs) Shop Rivets 15000#/□"	Working Values (lbs) Field Rivets or bolts 12000#/□"
1/2	7000	2900	2400
5/8	11000	4600	3700
3/4	16000	6600	5300
7/8	21,500	9000	7200

The bearing value of a rivet or bolt is equal to the product of the diameter, the thickness of the thinner riveted piece and the unit bearing value of the material

Bearing Values of Rivets and Bolts

Diameter Rivet or Bolt	Working Values											
	Shop Rivets (30000#/□")						Field Rivets or bolts 24000#/□"					
	Thickness of thinner connected piece (inch).											
	1/8	3/16	1/4	5/16	3/8	7/16	1/8	3/16	1/4	5/16	3/8	7/16
1/2	1800	2800	3700	4700	5600	6500	1500	2200	3000	3700	4500	5300
5/8	2300	3500	4700	5900	7000	8200	1800	2800	3700	4700	5600	6500
3/4	2800	4200	5600	7000	8400	9800	2200	3300	4500	5600	6700	7900
7/8	3200	4900	6500	8200	9800	11500	2600	3900	5300	6600	7900	9200

Minimum Angle Section

Diameter rivet or bolt	Minimum angle
1/2	1 1/2" L
5/8	1 3/4" L
3/4	2 1/4" L
7/8	2 1/2" L

FOUNDATION DESIGN

Specifications & Soil Conditions

Loose Soil, with maximum bearing pressure of 4000 #/sq. ft.
Angle of repose 30°.
Weight of soil = 100 #/cub. ft.
Factor of safety of 2 used.

Calculation of depth of foundation:

Let h = depth of footing below ground level in feet.

r = equivalent radius at top of foundation in ft.

R = radius of footing area in ft.

θ = angle of natural slope of earth.

If a Conical section of **earth*** to be lifted, instead of conical section of **concrete**, the computation in this case, will increase the factor of safety by a ratio of $\frac{150}{100}$ or 1.5. ∴ Computations are then based upon a lift of earthen frustrum, but actually a concrete cone will be put in. Therefore the real factor of safety will become 1.5 × 2 = 3 which is required.

Mamimum Uplift = 29,400 #

Maximum Compression = 36,400 #

~~2x~~ Weight of frustrum of earth ~~F~~ must be equal to 2 × Uplift = 2 × 29400 # = 58800 #

1 cub. ft = 100 # so Volume must be 588 cub. ft approximately.

The Volume of frustrum of cone to be lifted

$V = \frac{\pi}{3} h (r^2 + R^2 + rR)$

$R = r + h \tan. \theta$

$V = \frac{\pi}{3} h (3r^2 + h^2 \tan^2 \theta + 3rh \tan \theta)$

If $\theta = 30°$ $\tan \theta = 0.5774$ and

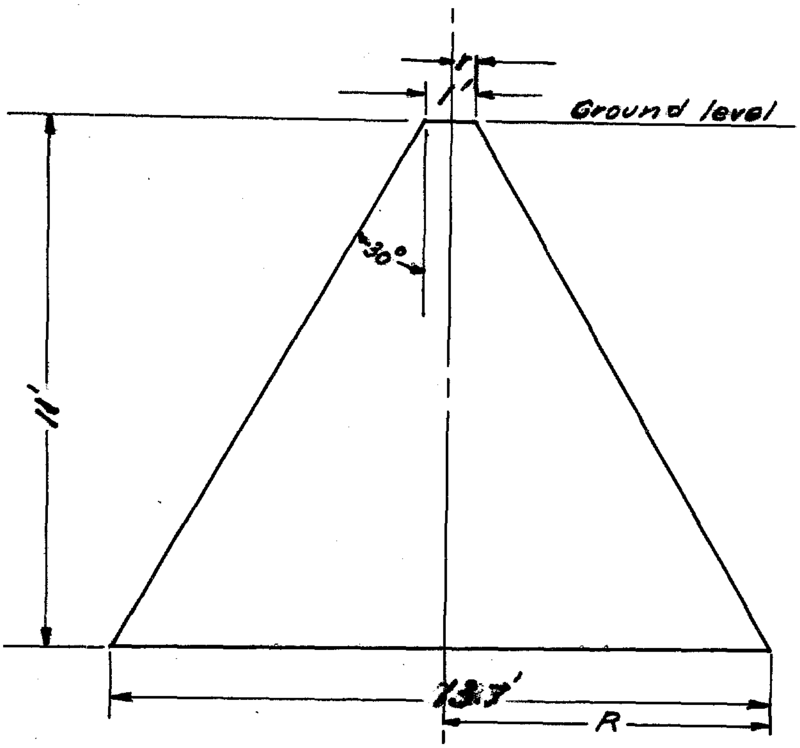
$V = \pi h (r^2 + 0.11 h^2 + 0.58 rh)$

If $r = 0.5$ ft and $h = 11$ ft. the volume of earth to be lifted is then:

$V = 3.14 \times 11 (.25 + 0.11 \times 121 + 0.58 \times 0.5 \times 11) = 580$ cub ft.

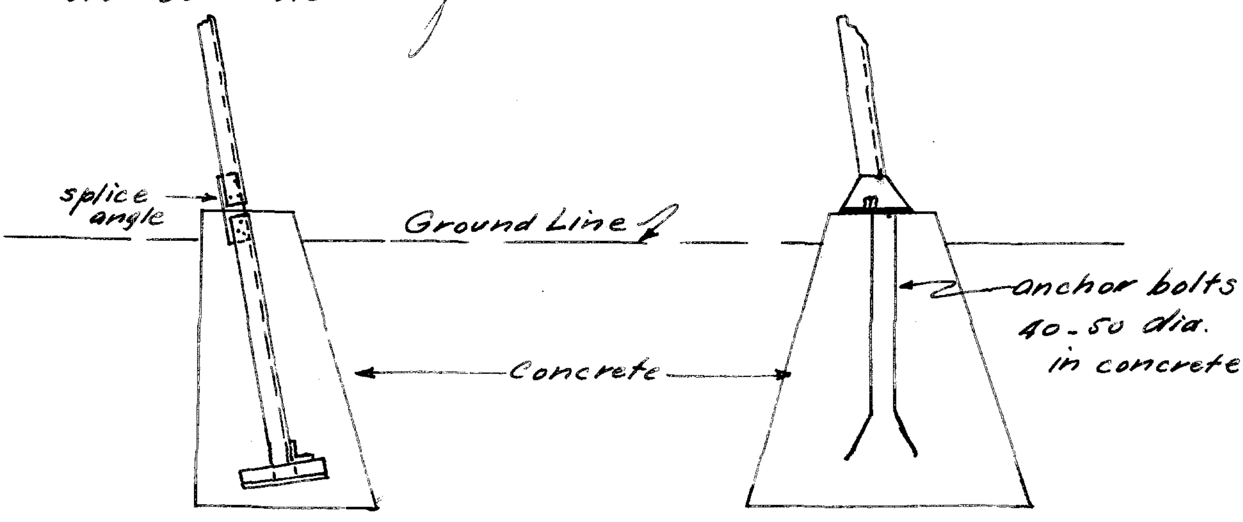
$W = 58000$ # O.K.

$R = 0.5 + 11 \tan. 30° = 6.85$ ft.



Necessary bearing area $= \frac{36400}{4000} = 9.1$ sq. ft.
O.K.

The most commonly used proportions are
1:2:4 Mix for fine work
1:3:6 Mix for mass foundations.
The present practice is to use wet concrete so that it may flow around the form easily, and be easily tamped or puddled.
The cornerposts may be connected ~~in two ways~~ to the footings in two ways.
1°. Anchor stubs built in the footings with sufficient length into the footing, so that the adhesion of the concrete to them will develop their full strength. ∴ sufficient Bond.
2° A base at the lower end of the post section with sufficient bearing area on the mass of concrete in the footing and connected directly to this concrete by means of long bolts or rods, extending to a depth of 40 or 50 times the diameter of the rods.



Thickness of Material & Weight of Structure

Many specifications allow redundant members to be made of material only 1/8" thick, but require a minimum thickness of 3/16" or possibly 1/4" for the main posts.

5/8" and 7/8" bolts are most commonly used.

The actual dead load of the structure is 7160# which is 2.3 % off the assumed weight, which is O.K.

A max. allowable difference of 10 % is used in practice.

Cost of Structure & its Erection.

The following sets of curves show us the relations between the width of base, height of tower and cost of structure. Also the relations of length of span and cost per 1000 ft of line. The cost of each structure has been figured on a basis of \$4.50 per 100 lbs delivered in the field.

It will be seen by reference to the first curve, that the cost of the structure alone is least, when the ratio of width of base to height is about 1 : 4.

The width of base of the structure has an important bearing on the cost of the line, since it affects the cost of foundations, the cost of right of way and the cost of assembling and raising the structure in the field.

According the second set of curves, a span of 425 ft, would be most economical.

The determining factors are the tower cost foundation and insulator cost.

The insulators have been figured at \$5.00 each, erected on the tower.

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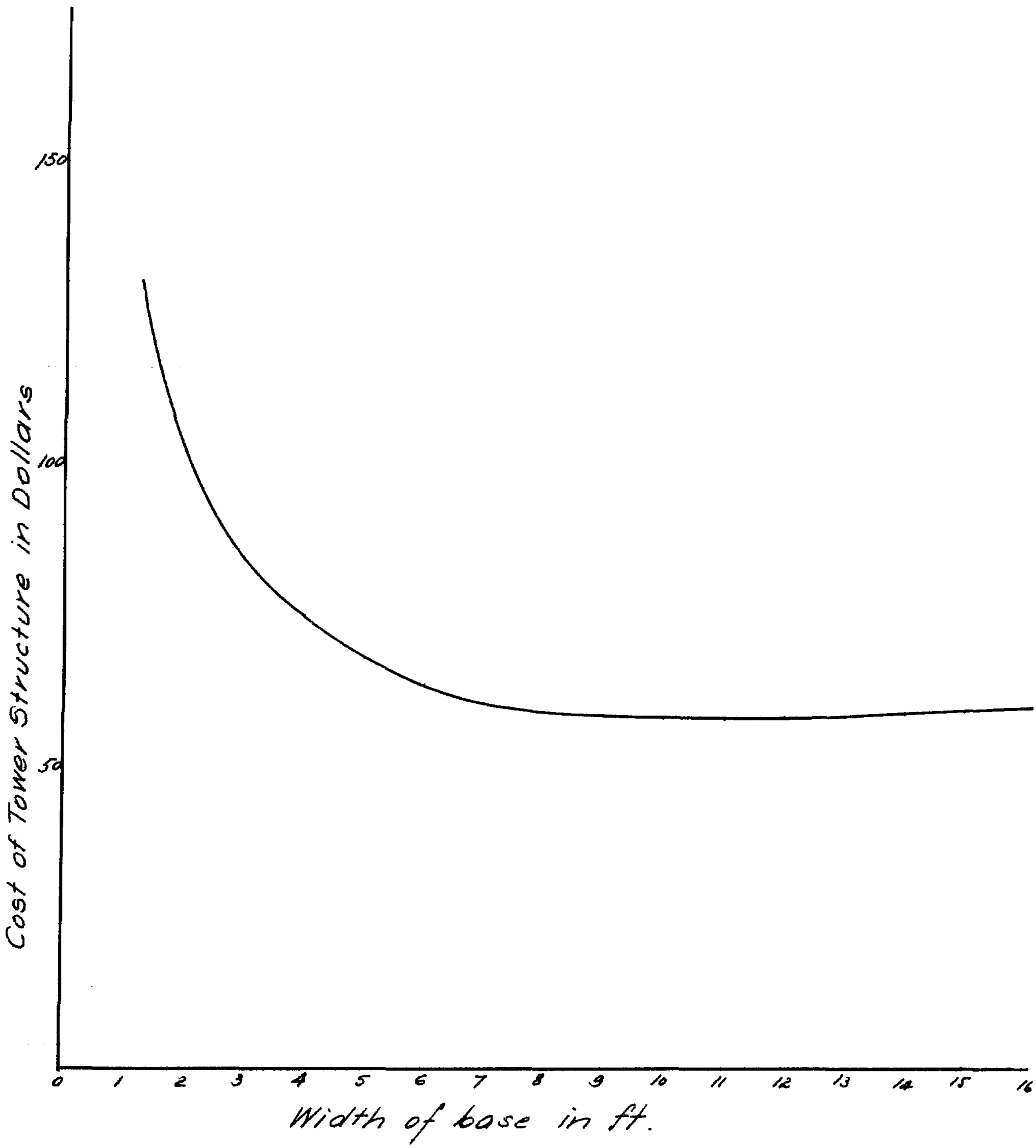
150
100
50
Cost of Tower Structure in Dollars
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Width of base in ft.

DESIGN
OF A
STANDARD SUSPENSION TOWER

WINCKEL, E. E. 1925

CIVIL ENGINEERING DEPT.