DESIGN
of an
AERIAL TRAMWAY.
(CABLEWAY)

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DESCRIPTION

An Aerial Tramway is a device for transporting material in receptacles over wire ropes supported at various elevations above the ground by means of posts or standards. Wire rope has been known for many centuries, and there is a drawing of a ropeway appearing in a book dated 1411.

This method of transportation is particularly well adapted to the handling of sand, stone, coal, ores, and other raw material. The advantages are obvious; they may briefly be enumerated as follows:

1. Unlike a railway, it is almost entirely independent of the topography of the country, and gradients impossible with the former can be operated with ease.
2. The need of expensive cuts, fills, bridges, and circuitous routes is obviated.
3. The material is carriedin a bee-line, and operation over hilly country is only slightly greater than that over level ground.
4. The structures are only slightly affected by rain, snow, floods and storms.
5. The line occupies very little ground space; the posts or towers require some space, but the intervening land is available for any other use and is not at all obstructed.
6. The quantity of material to be handled is no limitation, as lines can be built with capacities as high as 200 tons per hour.
7. The cost of a line is in strict accordance with

its capacity.

8. A line can even be moved from one location to another with comparative ease.

Aerial wire-rope conveyors may be classified under two general headings, namely, Cableways and Ropeways or Tramways. The former may be subdivided into Transporting Cableways, and Hoisting-Transporting Cableways; and the the latter into Single-Rope Tramways and Double-Rope Tramways.

The distinctive feature of a cableway is that a carrier suspended on a main cable by means of rolling sheaves is operated in both directions, motion being imparted to the carriers by means of a comparatively light endless wire rope, of the ordinary or Lang lay, known as the Traction rope. The carriers are usually attached to the traction rope by means of an automatic grip. In the Transporting type the load can merely be conveyed back and forth between the terminals; the Hoisting-Transporting type, in addition to this traveling feature, has a device whereby the load can also be raised and lowered.

Fig.I on the next page shows the simplest form of Gravity Transporting Cableway. Here a carrier suspended from the main cable by rolling sheaves is allowed to coast down, and this develops sufficient power to carry back the empty carriers from the lower terminal. The traction rope is controlled by a braking device. It is evident that such a system is adapted to short spans only, as the weight of the traction would tend to hold back the carrier.

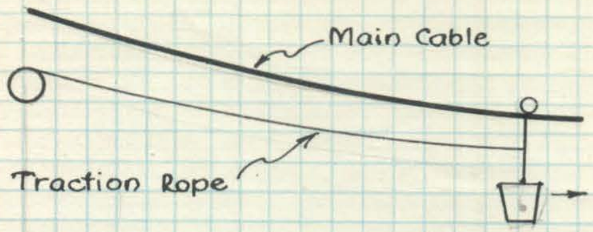


Fig. I.

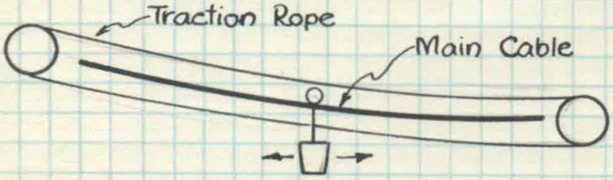


Fig. II.

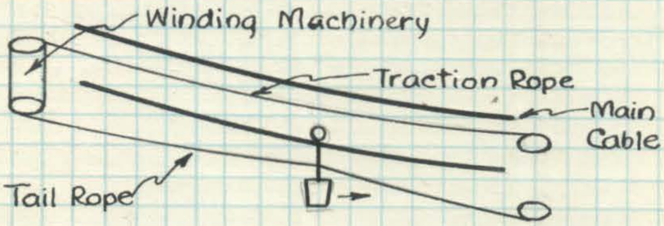


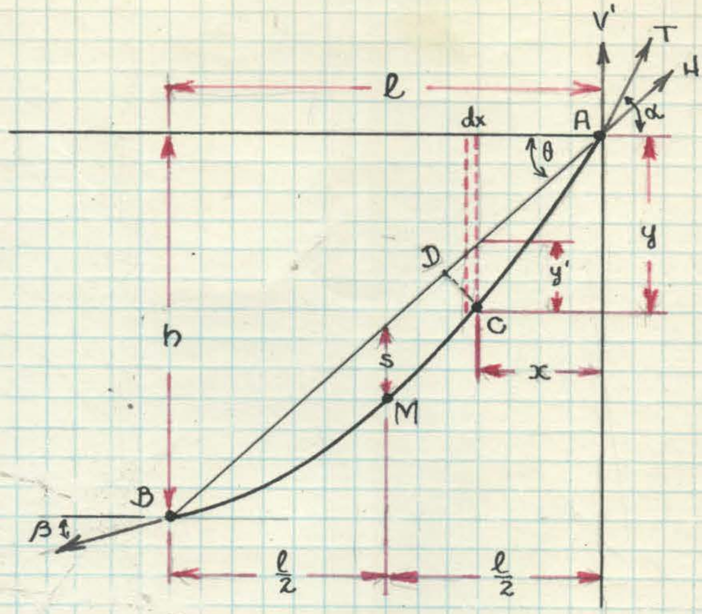
Fig. III

Fig. II shows a similar transporting cableway, which allows more perfect control of the bucket. It consists of a single fixed carrying rope upon which a single carrier is mounted, and drawn back and forth by means of an endless traction rope operated by suitable reversible driving gear at one end, andhaving tightening gear at the other. This arrangement may be used for capacities up to a maximum of 50 tons per hour.

For larger capacities a Twin-Rope Cableway, as shown in Fig. III, is required. This consists of two track cables; the carriers are connected by means of a traction rope and and tail rope.

The endless running Ropeway consists of an endless wire rope to which the carriers may be either detachably connected by means of saddles, or rigidly fixed in position. The rope passes at one end around a driving gear and at the other is attached to a tightening gear. But one rope is employed, which forms both the carrying and hauling rope for the loads. This system is simpler and decidedly cheaper than the cableways above described, but their successful operation is very often a matter of so much difficulty that it is being superseded to a great extent by the fixed-rope cableways.

DERIVATION



A perfectly flexible cable of uniform cross-section will, when suspended, assume the form of the common Catenary, the general equation of which is $y = \frac{c}{2}(e^{\frac{x}{c}} + e^{-\frac{x}{c}}) = c \cosh \frac{x}{c}$

However, even when the supports are at different elevations, the weight of the cable can, with little error, be assumed to be distributed uniformly along the horizontal instead of along the arc of the curve itself, and in this case the curve becomes a parabola, as will be shown later.

In the figure above, take a cable supported at points A and B, & assume its weight to be distributed over the span "l".

Let h = vertical height of A above B

θ = the angle between the horizontal and the chord AB.

T = the tension at A

V' & H' = the components of T along Ay & BA

x & y = the coordinates of any point C

s = sag, or the distance between the chord and the arc at the midspan.

ω = weight of cable per unit length.

Taking any portion of the curve, as AC, there are three forces acting on it, viz, the dead load ωx, the tension at A, T, & the tension at C.

Being in equilibrium, the sum of the moments of these forces about C must equal 0.

$$\therefore \Sigma M_C = -\omega x\left(\frac{x}{2}\right) + V'x - H'(CD) = 0$$

$$\therefore V'x - \frac{\omega x^2}{2} = H'(y\cos\theta - x\sin\theta) \qquad (1)$$

which is the equation of a parabola with its axis parallel to the Y-axis of the figure.

But for our purposes, it is necessary to express this equation in terms of l, h, and s, which are the known quantities in the problem. To do this, it is necessary to consider the whole cable; the forces acting on it are

ωl, its weight

& the tensions at A & B.

Taking moments about B,

$$\Sigma M_B = \omega l\left(\frac{l}{2}\right) - V'l = 0$$

$$\therefore V' = \frac{\omega l}{2} \qquad (2)$$

The forces acting on the upper half AM are the weight, the tension at A, & the tension at M.

Taking moments about M,

$$\Sigma M_M = \frac{\omega l}{2}\left(\frac{l}{4}\right) - V'\frac{l}{2} + H's\cos\theta = 0$$

$$\therefore H' = \left(\frac{V'l}{2} - \frac{\omega l^2}{8}\right) \div s\cos\theta$$

Substituting the value of V' obtained in (2)

$$H' = \frac{\frac{\omega l^2}{4} - \frac{\omega l^2}{8}}{s\cos\theta} = \frac{\omega l^2}{8 s\cos\theta} \qquad (3)$$

Substituting these values of V' & H' into equation (1),

$$V'x - \frac{\omega x^2}{2} = H'(y\cos\theta - x\sin\theta)$$

$$\frac{\omega l x}{2} - \frac{\omega x^2}{2} = \frac{\omega l^2}{8s\cos\theta}(y\cos\theta - x\sin\theta)$$

$$\therefore \frac{8sx}{l^2}\left(\frac{l}{2} - \frac{x}{2}\right) = y\frac{\cos\theta}{\cos\theta} - x\frac{\sin\theta}{\cos\theta}$$

$$\therefore \frac{4sx}{l^2}(l-x) = y - x\tan\theta$$

$$\therefore y = \frac{4sx}{l^2}(l-x) + x\tan\theta$$

$$= \frac{4sx}{l} - \frac{4sx^2}{l^2} + x\frac{h}{l}$$

$$= \frac{x}{l}(4s+h) - 4s\frac{x^2}{l^2} \qquad (4)$$

If y' = the vertical distance of any point such as C below the chord AB,

$$y' = y - x\tan\theta.$$

Substituting the value of y from (4),

$$y' = y - x\frac{h}{l}$$

$$= (4s+h)\frac{x}{l} - 4s\left(\frac{x^2}{l^2}\right) - x\frac{h}{l}$$

$$= 4s\frac{x}{l} + \cancel{\frac{hx}{l}} - 4s\frac{x^2}{l^2} - \cancel{\frac{xh}{l}}$$

$$= \frac{4sx}{l^2}(l-x)$$

The slope at any point on the curve is obtained by taking the first derivative of y with respect to x.

$$y = \frac{x}{l}(4s+h) - 4s\frac{x^2}{l^2} \qquad \text{(from (4))}$$

$$\therefore \frac{dy}{dx} = 4\frac{s}{l} + \frac{h}{l} - \frac{8sx}{l^2}$$

Let α & β = the slope angles at the points of support, A & B.

At A, $x = 0$

" B, $x = l$

$$\therefore \quad \tan\alpha = 4\frac{s}{l} + \frac{h}{l} = \frac{(h+4s)}{l}$$ (6)

$$\& \quad \tan\beta = 4\frac{s}{l} + \frac{h}{l} - \frac{8sl}{l^2} = \frac{(h-4s)}{l}$$

At the vertex of the parabola, the slope is zero, i.e., $\frac{dy}{dx} = 0$.

Let x_0 & y_0 = the coordinates of the vertex

Then $\frac{dy}{dx} = 4\frac{s}{l} + \frac{h}{l} - \frac{8sx_0}{l^2} = 0$

$$\therefore x_0 = \frac{4\frac{s}{l} + \frac{h}{l}}{\frac{8s}{l^2}} = \frac{l(4s+h)}{8s}$$ (7)

And (substituting in (4)), for x

$$y_0 = \frac{x_0}{l}(4s+h) - 4s\frac{x_0^2}{l^2}$$

$$= \frac{l(4s+h)(4s+h)}{8sl} - 4s\frac{l^2(4s+h)^2}{64s^2l^2}$$

$$= \frac{(4s+h)^2}{8s} - \frac{(4s+h)^2}{16s} = \frac{(4s+h)^2}{16s}$$ (7)

Let V and H = the vertical and horizontal components, respectively, of the tension T.

Then, from the figure, it is evident that

$$V = V' + H'\sin\theta =$$

$$= \frac{wl}{2} + \frac{wl^2}{8s\cos\theta}.\sin\theta = \frac{wl}{2}\left(1 + \frac{l\tan\theta}{4s}\right)$$

$$\& \quad H = H'\cos\theta$$

$$= \frac{wl^2}{8s\cos\theta}\cdot\cos\theta = \frac{wl^2}{8s}$$

It is seen that H is dependent only on the weight, the span, and the sag, and is therefore constant, i.e., it is the same at all points on the cable.

From the right-triangular relation,

$$T^2 = V^2 + H^2$$
$$= \left(\frac{wl^2}{8s}\right)^2 + \left[\frac{wl}{2}\left(1 + \frac{l\tan\theta}{4s}\right)\right]^2$$
$$= \frac{w^2l^4}{64s^2} + \frac{w^2l^2}{4}\left(1 + \frac{l\tan\theta}{2s} + \frac{l^2\tan^2\theta}{16s^2}\right)$$
$$= \frac{w^2l^4}{64s^2} + \frac{w^2l^2}{4} + \frac{w^2l^3\tan\theta}{8s} + \frac{w^2l^4\tan^2\theta}{64s^2}$$
$$= \frac{w^2l^2}{4}\left(\frac{l^2}{16s^2} + 1 + \frac{l\tan\theta}{2s} + \frac{l^2\tan^2\theta}{16s^2}\right)$$

Let n = the sag ratio, i.e., $\frac{s}{\text{chord } AB}$,

$$= \frac{s}{l\sec\theta}$$

$$\therefore T^2 = \frac{w^2l^2}{4}\left(\frac{1}{16n^2\sec^2\theta} + 1 + \frac{\tan\theta}{2n\sec\theta} + \frac{\tan^2\theta}{16n^2\sec^2\theta}\right)$$
$$= \frac{w^2l^2}{4}\left(\frac{1}{16n^2\sec^2\theta} + 1 + \frac{\sin\theta}{2n} + \frac{\sin^2\theta}{16n^2}\right)$$
$$= \frac{w^2l^2}{4}\left[1 + \frac{\sin\theta}{2n} + \frac{1}{16n^2}\left(\sin^2\theta + \frac{1}{\sec^2\theta}\right)\right]$$
$$= \frac{w^2l^2}{4}\left(\frac{1}{16n^2} + \frac{\sin\theta}{2n} + 1\right)$$
$$\therefore T = \frac{wl}{2}\left(\frac{1}{16n^2} + \frac{\sin\theta}{2n} + 1\right)^{\frac{1}{2}} \qquad (8)$$

It is seen that the tension is directly proportional to both the weight and length of span; that it increases slightly with θ, i.e., with the difference in elevation between supports; and that it decreases as the sag ratio increases.

To find the length (l') of the arc AB:

$$ds = [(dx)^2 + (dy)^2]^{\frac{1}{2}} = \left[1 + \left(\frac{dy}{dx}\right)^2\right]^{\frac{1}{2}} dx$$

From (4), $\frac{dy}{dx} = \frac{4s}{l}\left(1 - \frac{2x}{l}\right) + \tan\theta$

$$= \left[4n\left(1 - \frac{2x}{l}\right) + \sin\theta\right]\sec\theta$$

Substituting,

$$ds = \left[1 + 8n\left(1 - 2\frac{x}{l}\right)\left\{2n\left(1 - \frac{2x}{l}\right) + \sin\theta\right\}\right]^{\frac{1}{2}} \sec\theta\, dx$$

which may be written $[1+K]^{\frac{1}{2}} \sec\theta\, dx$

where $K = 8n\left(1 - \frac{2x}{l}\right)\left\{2n\left(1 - \frac{2x}{l}\right) + \sin\theta\right\}$

For small sags, as are used in this case, the chord is very nearly equal to the arc, i.e.,

ds nearly $= \sec\theta\, dx$

and therefore $(1+K)$ nearly $= 1$

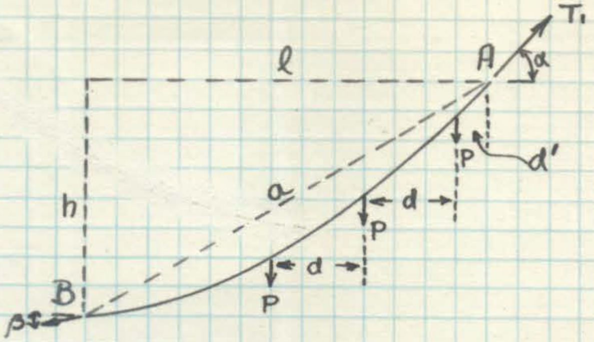
K being small, the expression may be expanded by the binomial theorem as follows

$$\left(1 + \frac{K}{2} - \frac{K^2}{8} + \text{----}\right)$$

$$\therefore\ l' = \int_0^l ds = \int_0^l \left(1 + \frac{K}{2} - \frac{K^2}{8} \cdots\right) \sec\theta\, dx$$

$=$ let a = chord AB

$$l' = a\left(1 + \frac{8}{3}\cos^2\theta\, n^2 - \frac{32}{5} n^4\right)$$



If there are n concentrated loads (P) suspended at a distance d apart, the first being d' from A, the tension T_1 may be obtained by taking moments about B

$$\Sigma M_B = 0 = T_1 a \sin(\alpha - \beta) - \frac{wal}{2} - P(l-d') - P(l-d-d') - P(l-2d-d') - P \text{----}$$

$$\therefore T_1 = \frac{\frac{wcl}{2} + P(l-d') + P(l-d-d') \cdots}{a \sin(\alpha-\beta)}$$

WIND.

The Wind Load on a suspended cable is a factor not to be neglected.

The wind pressure per unit area on a surface is expressed as a function of its kinetic energy in the following formula,

$$P = K\frac{W V^2}{2g}$$

where P = the pressure

W = weight of air per unit volume

V = velocity of wind in feet per second

g = acceleration due to gravity.

K = a coefficient depending on the shape of the surface.

The last part of the equation, $\frac{WV^2}{2g}$, is known as the velocity head.

The value of K for an indefinitely long rectangle of measurable width has been determined to be about 1.83. But, as in the case of wire, the pressure per unit area of projected area of a cylindrical surface is less than that for flat surfaces. The coefficient by which the flat surface pressure must be multipliedin order to give the actual pressure on the cylindrical surface is variously stated as being between .45 and .80, the most widely accepted being about .50.

Then, substituting these values for the coefficient, and moreover, taking the weight of air as .08071 lbs. per cubic foot, and reducing the velocity from feet per second to miles per hour, we have

$$P = .5 \times 1.83 \times \left(\frac{5280}{3600}\right)^2 \times \frac{.08071}{64.4} \times V^2$$

$$= .00246675\ V^2$$

or practically $.0025\ V^2$

where P is in lbs. per square foot, and

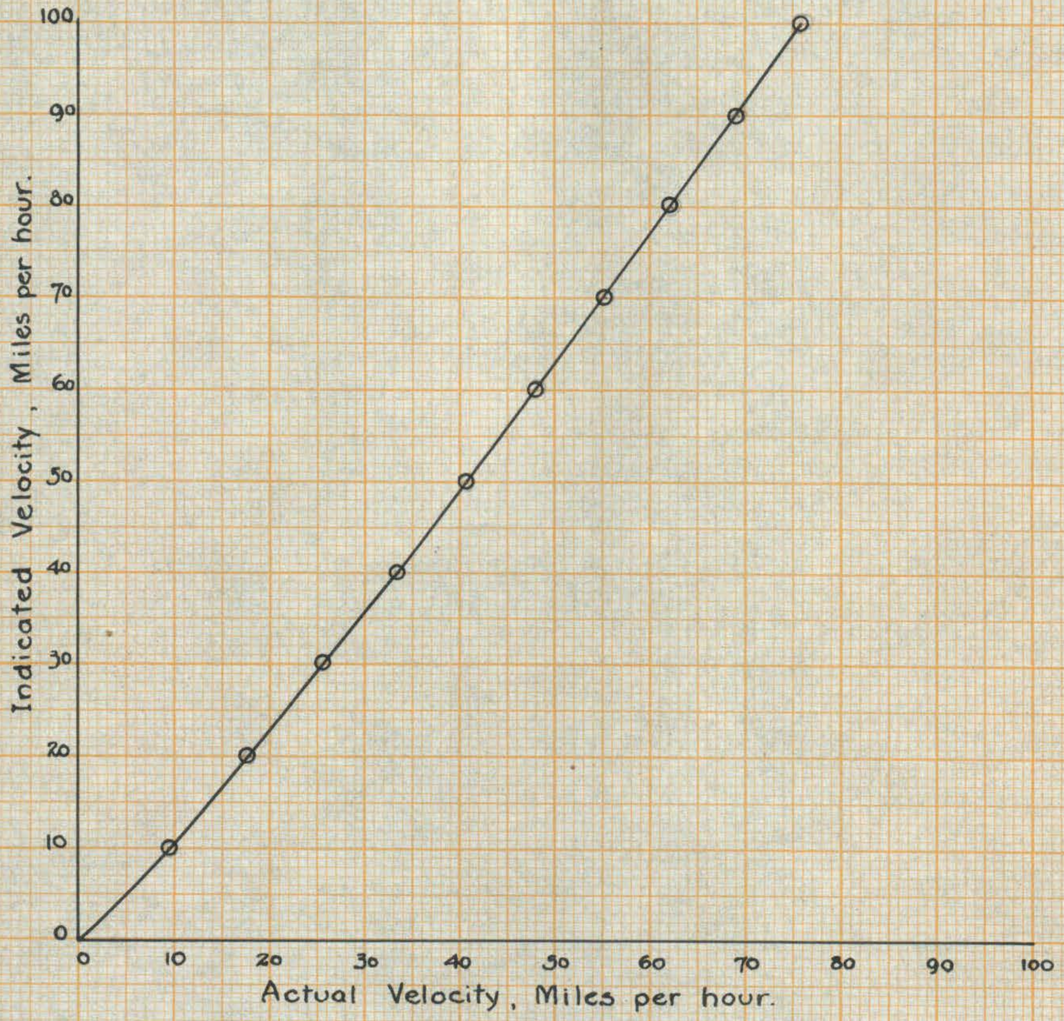
V is in miles per hour.

This velocity V is, of course, the actual velocity of the wind. This true velocity, however, has been found to differ somewhat from the indicated velocity as determined by the U. S. Weather Bureau. Their measurements are made with a cup anemometer over five-minute intervals, and the velocities calculated on the assumption that the cup velocity is one-third (1/3) the wind velocity for all conditions. This is known to be incorrect, especially for high velocities, as shown by the following table, and the accompanying curves.

Indicated Velocity.	Actual Velocity.
10 m.p.h.	9.6 m.p.h.
20	17.8
30	25.7
40	33.3
50	40.8
60	48.0
70	55.2
80	62.2
90	69.2
100	76.2

Curve I shows this relation between the indicated and actual velocities, while Curve II shows the difference in wind pressure when calculated for indicated and actual velocities according to the formula derived above, namely, $P = .0025\ V^2$, as worked out in the following table.

V = Act. Vel. m.p.h.	$P_{ind.}$ #/ft²	$P_{act.}$ #/ft²
10	$.0025\,(10)^2 = .25$	$.25\left(\frac{9.6}{10}\right)^2 = .2305$
20	$.0025\,(20)^2 = 1.00$	$1.00\left(\frac{17.8}{20}\right)^2 = .792$
30	$.0025\,(30)^2 = 2.25$	$2.25\left(\frac{25.7}{30}\right)^2 = 1.650$
40	$.0025\,(40)^2 = 4.00$	$4.00\left(\frac{33.3}{40}\right)^2 = 2.775$
50	$.0025\,(50)^2 = 6.25$	$6.25\left(\frac{40.8}{50}\right)^2 = 4.160$
60	$.0025\,(60)^2 = 9.00$	$9.00\left(\frac{48}{60}\right)^2 = 5.760$
70	$.0025\,(70)^2 = 12.25$	$12.25\left(\frac{55.2}{70}\right)^2 = 7.610$
80	$.0025\,(80)^2 = 16.00$	$16.00\left(\frac{62.2}{80}\right)^2 = 9.680$
90	$.0025\,(90)^2 = 20.25$	$20.25\left(\frac{69.2}{90}\right)^2 = 11.950$
100	$.0025\,(100)^2 = 25.00$	$25.00\left(\frac{76.2}{100}\right)^2 = 14.500$



CURVE SHOWING RELATION BETWEEN
INDICATED & ACTUAL
WIND VELOCITIES.

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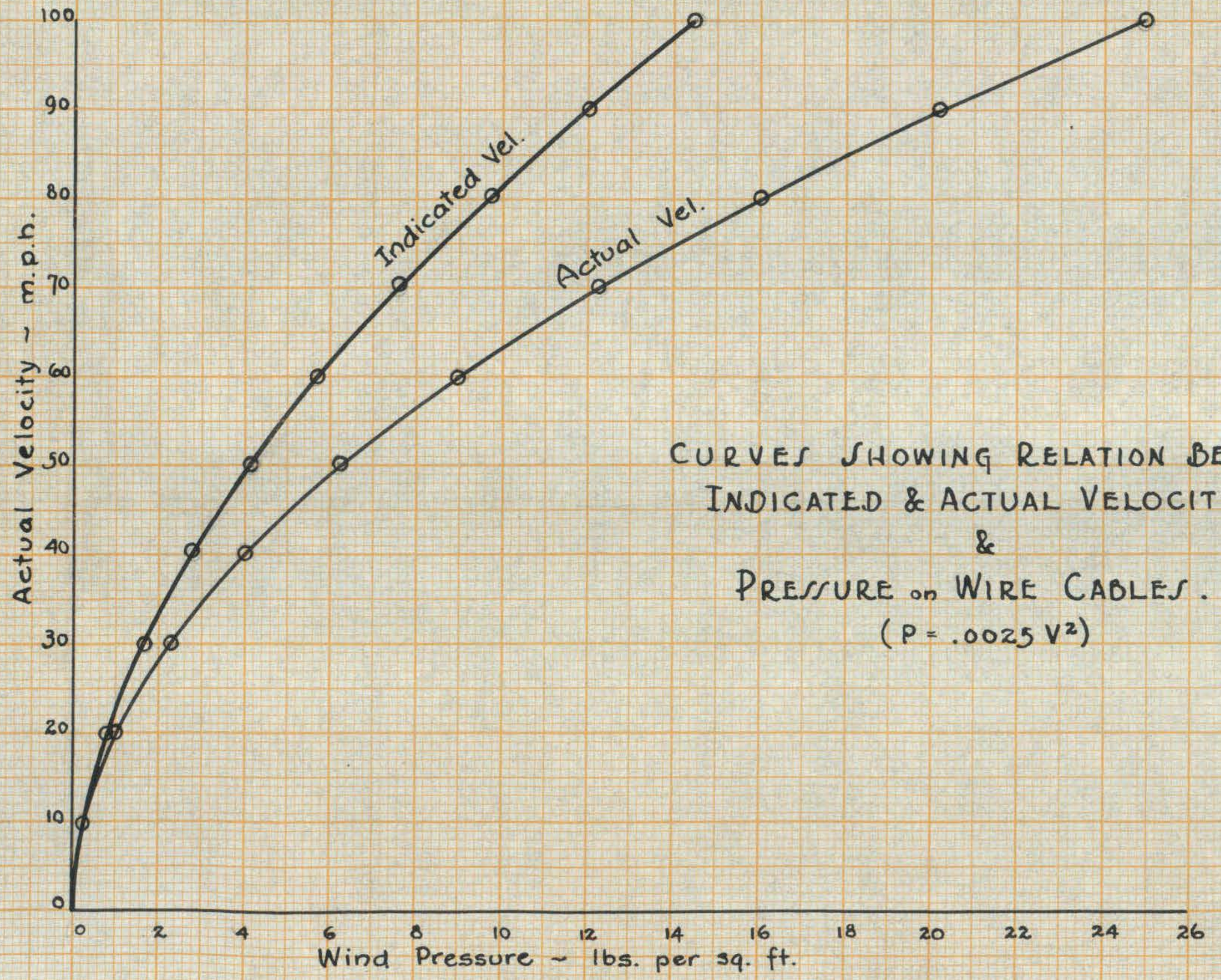
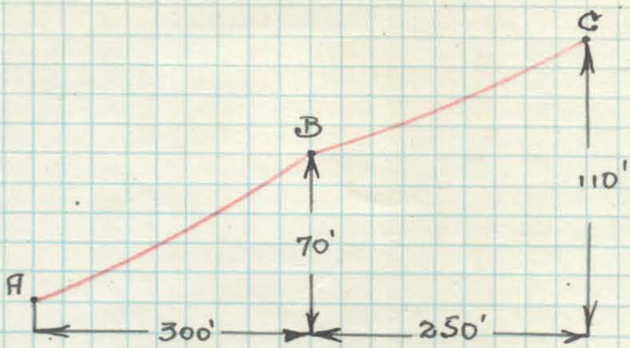


CURVES SHOWING RELATION BETWEEN
INDICATED & ACTUAL VELOCITY
&
PRESSURE on WIRE CABLES.
($P = .0025 V^2$)

Sheet No. 16 of 44 Sheets

TRACK CABLE



Taking the line as shown above:

Span AB:

Chord $= \sqrt{(300)^2 + (70)^2} = 308.06'$

The deflection of the cable is the vertical distance between the cable and the imaginary chord that connects the two supports, measured from the center of the chord. The deflection is limited by the size and stiffness of the wire. Although a large deflection reduces the tension in the cable and thus permits the use of a smaller size, it is also true that the larger the deflection the greater the power required in pulling the load up the steeper slope, and the more pronounced the breaking up of the wire due to bending under the carriage sheaves and over supports.

For Locked Coil Cable, the material generally used for the track rope, the maximum allowable sag consistent with economy is 5% (of the length of the cable).

Maximum deflection will be limited to 4% in this case.

$\therefore$ Sag $s = (.04)(308.06) = 12.3224$ ft.

$$\tan\alpha \text{ (upper support)} = \frac{h+4s}{l} = \frac{70+49.2896}{300} = .39763$$

$$\tan\beta \text{ (lower \;\; " \;\;)} = \frac{h-4s}{l} = \frac{70-49.2896}{300} = .06903$$

$$\left.\begin{aligned} \therefore \alpha &= 21^\circ - 41' \\ \beta &= 3^\circ - 57' \end{aligned}\right\}$$

$$\theta = \tan^{-1}\frac{70}{300} = \tan^{-1} .2333 = \underline{13^\circ - 10'}$$

Assume maximum bucket load to be 4000# and the worst condition of loading to occur when one carrier is just at the support and another at midspan.

$$T_{live} = \frac{4000(300+150)}{308.06 \sin 17^\circ\text{-}44'} = \frac{1,800,000}{(308.06)(.30459)}$$

$$= \underline{19,200\#}$$

Try 1¼" Locked Coil Track-Cable (Crucible Cast Steel), with a breaking strength of 62 tons, or, using a factor of safety of 5, an allowable working stress of 24800#.

The weight of this cable is 3.70 #/ft.

Assuming a maximum actual wind velocity, with no sleet or ice, of 60 miles per hour,

$$P = .0025\,V^2 = (.0025)(60)^2 = 9.0\ \#/ft^2.$$

$\therefore$ Wind Load $= 9.0 \times \frac{1.25}{12} = 0.92$ #/lin. ft. of cable

Combining this uniform load with the dead load,
The Resultant Load $= \sqrt{(3.7)^2 + (.92)^2} = \sqrt{14.54}$
$= 3.81$ #/ft.

$$\therefore T_{dead} = \frac{wl}{2}\left(\frac{1}{16n^2} + \frac{\sin\theta}{2n} + 1\right)^{\frac{1}{2}}$$

$$= \frac{(3.81)(300)}{2}\left(\frac{1}{(16)(.04)^2} + \frac{.2270}{2(.04)} + 1\right)^{\frac{1}{2}}$$

$$= 571.5\sqrt{39.1 + 2.84 + 1} = \sqrt{42.94}\,(571.5)$$

$$= 571.5(6.553) = \underline{3745}^{\#}$$

$\therefore$ Max. $T = 3745 + 19200 = 22,945^{\#}$

$\therefore$ $1\frac{1}{4}$" Locked Coil will be used.

For Span BC,

Chord $= \sqrt{(250)^2 + (40)^2} = \sqrt{64100} = 253.18'$

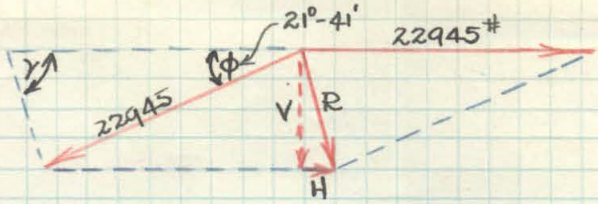
$n = .04$ as in AB

$\therefore s = (.04)(253.18) = 10.127'$

$$\tan\alpha = \frac{40 + 4(10.127)}{250} = .32203$$

$$\tan\beta = \frac{40 - 4(10.127)}{250} = -.00197$$

$$\left.\begin{aligned}\therefore \alpha &= 17^\circ - 51' \\ \beta &= -\,0^\circ - 07' \text{ or practically level.}\end{aligned}\right\}$$



The stresses at the support are as shown in the figure above.

$$R = \sqrt{2(22945)^2(1-\cos 21^\circ\text{-}41')}$$
$$= \sqrt{2(525473025)(1-.92924)}$$
$$= \sqrt{(1050946050)(.07076)}$$
$$= \sqrt{74364942.5} \quad = \quad \underline{8623}^{\#}$$

$$\measuredangle\gamma = \tfrac{1}{2}(180^\circ - 21^\circ\text{-}41') = 79^\circ\text{-}09\tfrac{1}{2}'$$

$$\therefore\; V = 8623 \sin 79^\circ\text{-}09\tfrac{1}{2}' = (8623)(.98218)$$
$$= \underline{\underline{8469}}^{\#}$$

$$H = 8623 \cos 79^\circ\text{-}09\tfrac{1}{2}' = (8623)(.18795)$$
$$= \underline{\underline{1621}}^{\#}$$

TRACTION ROPE

When velocity is imparted to a rope, i.e., when it is accelerated, the stress may be considerably greater than that for a quiescent load. This difference in stress is entirely dependent on the rate of change of velocity.

Taking the simple case of a load being lifted vertically, let

W = weight to be lifted
w = " of rope per foot
a = acceleration in feet per second per sec.
t = time of acceleration
V = velocity in ft/sec.
S = space in which acceleration is made.
l = length of rope
g = force of gravity
K = kinetic energy of the load
k = " " " rope
K_t = total kinetic energy
C = a constant by which the load is increased due to kinetic energy.

Then $K_t = K + k = C(W + wl)$

$$= \frac{WV^2 + wlV^2}{2g} = \frac{V^2}{2g}(W + wl)$$

But $V^2 = 2aS$

$$\therefore C = \frac{2aS}{2g} = \frac{aS}{g} \quad \text{or} \quad a = \frac{Cg}{S}$$

$$V = at \quad \& \quad aS = gC$$

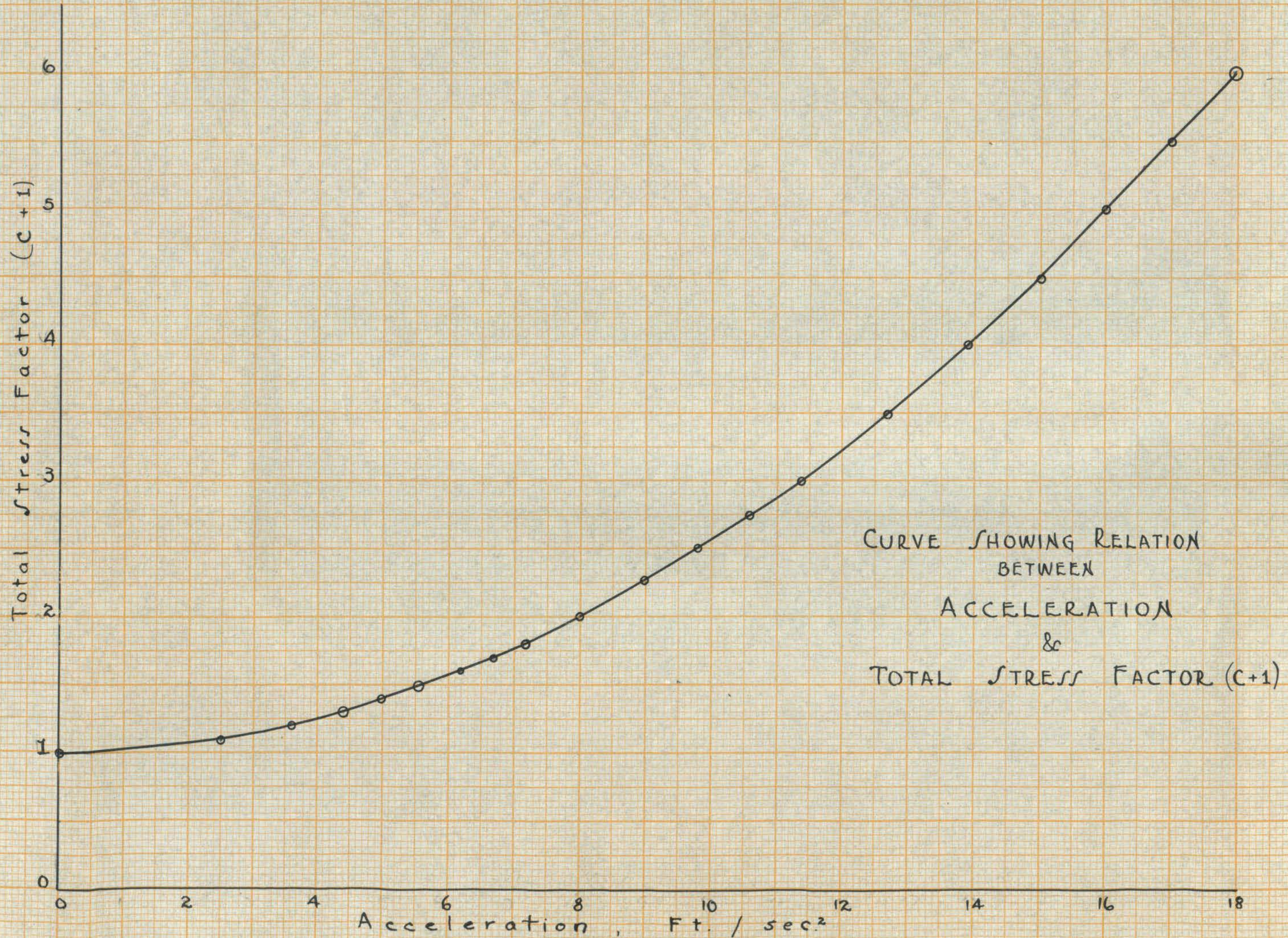
$$\therefore a^2t^2 = 2gC \quad \& \text{ if } t = 1, \quad a = \sqrt{2gC}$$

$$= 8.02\sqrt{C}.$$

The following table is calculated from these formulae, and the curve shows the increase of stress for various accelerations.

C	a ft/sec²	S ft/sec.	C+1 Total Stress Factor	Factor of Safety 10 for a Quiet Load.
0	0	0	1.00	10.00
.10	2.54	1.27	1.10	9.09
.20	3.59	1.79	1.20	8.34
.25	4.01	2.01	1.25	8.00
.30	4.39	2.20	1.30	7.70
.40	5.07	2.54	1.40	7.15
.50	5.67	2.84	1.50	6.67
.60	6.21	3.11	1.60	6.25
.70	6.71	3.36	1.70	5.88
.75	6.94	3.47	1.75	5.72
.80	7.17	3.58	1.80	5.66
.90	7.61	3.81	1.90	5.27
1.00	8.02	4.01	2.00	5.00
1.25	8.97	4.48	2.25	4.44
1.50	9.82	4.91	2.50	4.00
1.75	10.61	5.31	2.75	3.64
2.00	11.34	5.67	3.00	3.33
2.50	12.68	6.34	3.50	2.86
3.00	13.89	6.94	4.00	2.50
3.50	15.00	7.50	4.50	2.22
4.00	16.04	8.02	5.00	2.00
4.50	17.01	8.50	5.50	1.82
5.00	17.93	8.96	6.00	1.67

It is to be noted that there is, especially for long ropes, another factor entering which increases the factor of safety. This is the elasticity and extension of the cable with application of load, the amount of stretch being proportional to the length of the rope. This, however, is not always considered, it being taken as additional safety.



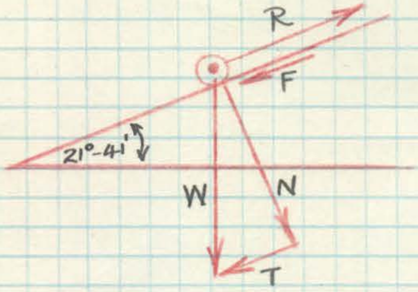
...SSER, CO., NEW YORK, NO. 334 C. R

TRACTION ROPE

Take $V = 600'/\text{min. or } 10'/\text{sec.}$

$S = 5'$

$C = 1$; i.e. $a = 8.02'$ ft/sec²



$W = 4000^{\#}$

$\therefore T = 4000 \sin 21°\text{-}41'$
$= (4000)(.36948) = 1478^{\#}$

$N = 4000 \cos 21°\text{-}41'$
$= 4000(.92924) = 3717^{\#}$

Assuming the friction coefficient to be .03, the frictional resistance $F = (.03)(4000) = 120^{\#}$

$\therefore R - 1478 - 120 = \frac{4000}{32.2} \times 8.02$

$\therefore R = 1598 + (124.2)(8.02)$

$= \underline{2593^{\#}}$

Another load at midspan:

Slope at midspan (150' from support) =

$$\frac{dy}{dx} = 4\frac{s}{l} + \frac{h}{l} - \frac{8sx}{l^2} = \frac{4(12.32)}{300} + \frac{70}{300} - \frac{8(12.32)(150)}{90000}$$

$= .1644 + .233 - .1644 = .233$

$\therefore \tan^{-1} .233 = 13°10'$

$\therefore T_1 = 4000 \sin 13°10' = 4000(.22778) = 911.1^{\#}$

$N_1 = 4000 \cos 13°\text{-}10' = 4000(.97371) = 3895^{\#}$

$F_1 = 120^{\#}$

$a = 8.02\ '/\text{sec}^2$

$\therefore R_1 = \frac{4000}{32.2}(8.02) + 911 + 120 = 2026^{\#}$

Traction Rope (cont.)

Try 5/8" 6-7 Extra Strong Crucible Cast Steel Haulage Rope.

Wt. - .62#/ft.

Allow 10% sag ($n = .1$)

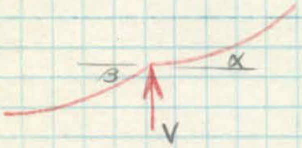
$$T' = \frac{.62 \times 300}{2}\left[\frac{1}{16(.1)^2} + \frac{.2270}{2(.1)} + 1\right]^{\frac{1}{2}}$$

$$= 93\left(\frac{1}{.16} + 1.135 + 1\right)^{\frac{1}{2}} = 93\sqrt{8.385} = \underline{\underline{268.8\#}}$$

$\therefore$ Total Stress = 2593 + 2026 + 269 = $\underline{\underline{4888\#}}$

Ultimate Str. of 5/8" rope = 14.5 tons

$\therefore$ Working Str. = $\frac{14.5 \times 2000}{5}$ = $\underline{\underline{5800\#}}$ $\therefore$ O.K.



$$\tan\alpha = \frac{70 + 4(30.81)}{300} = \frac{70 + 123.24}{300} = .6441$$

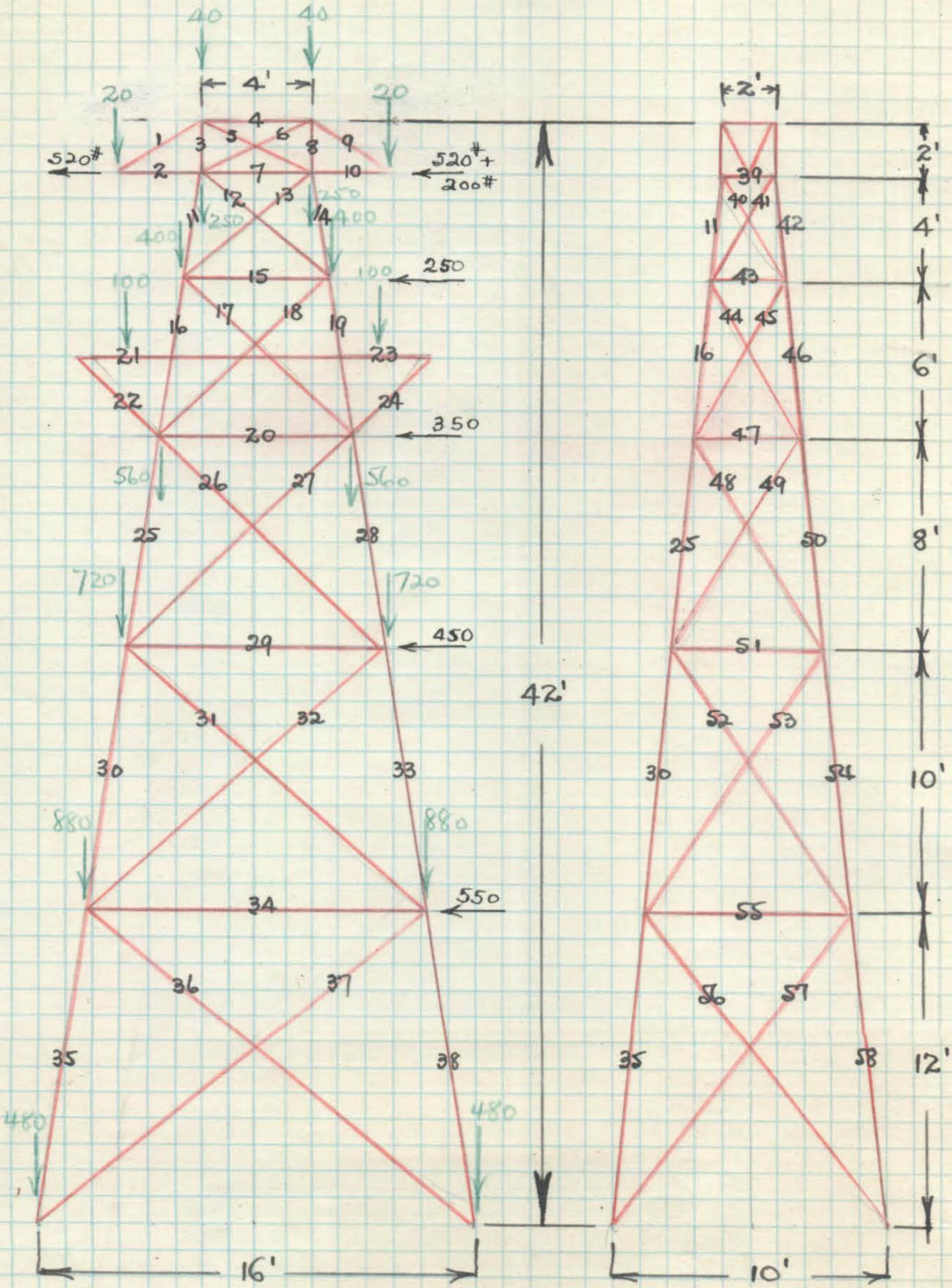
$$\tan\beta = \frac{70 - 4(25.3)}{250} = -\frac{61.2}{250} = -.2448$$

$$\left.\begin{aligned} \therefore \alpha &= 32° - 48' \\ \beta &= -13° - 45' \end{aligned}\right\}$$

$$V = 268.8(\sin 32\text{-}48 + \sin 13\text{-}45)$$

$$= 268.8(.54100 + .23750) = \underline{\underline{209.25\#}}$$

TOWER


40
40
4'
2'
20
20
520#
520# + 200#
250
350
450
550
400
400
250
250
100
100
560
560
720
720
880
880
480
480
42'
2'
4'
6'
8'
10'
12'
16'
10'

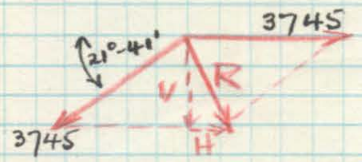
Assume Dead Load = 80# per foot in height for each column.

Wind Load = 50#/ft. in height for each column.

" = .92 #/ft. of cable.

$= (.92) \times 2 \times (564) = 1037$ or say $\underline{\underline{1040^{\#}}}$

Dead Load from Track Cable:



$$R = \sqrt{2(3745)^2(1-\cos 21°\text{-}41')}$$

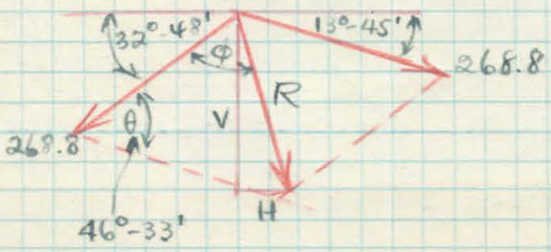
$$= \sqrt{14025025(2-2\times.92924)}$$

$$= \sqrt{14025025(.14152)}$$

$$= \sqrt{1984821} = \underline{\underline{1408^{\#}}}$$

$$\left.\begin{aligned} V &= 1408 \cos 21°\text{-}41' = 1408(.92924) = 1308^{\#} \\ H &= 1408 \sin 21°\text{-}41' = 1408(.36942) = 520^{\#} \end{aligned}\right\}$$

For Traction Cable:



$$R = \sqrt{(268.8)^2(2)(1-\cos 46°\text{-}33')}$$

$$= \sqrt{144516(1-.68775)}$$

$$= \sqrt{144516(.31225)}$$

$$= \sqrt{45122} = \underline{\underline{212^{\#}}}$$

$$\measuredangle \phi = \tfrac{1}{2}(180° - 43°\text{-}33') = 68°\text{-}10'$$

$$\therefore \quad V = 212 \cos[32°\text{-}48' + 68°\text{-}10' - 90°]$$

$$\left.\begin{aligned} &= 212 \cos 10°\text{-}58' = 212(.98153) = 208.1^{\#} \\ H &= 212 \sin 10°\text{-}58' = 212(.19140) = 40.5^{\#} \end{aligned}\right\}$$

Member	Length (Feet)	
1	$\sqrt{2^2+3^2} = \sqrt{13}$	$= 3.61'$
2		$3'$
3		$2'$
4		$4'$
5	$\sqrt{4^2+2^2} = \sqrt{20}$	$= 4.47'$
11	$\sqrt{4^2+\left(\frac{4\times 6}{40}\right)^2} = \sqrt{16+\left(\frac{3}{5}\right)^2} = \sqrt{16.36}$	$= 4.04'$
12	$\sqrt{4^2+(4+3/5)^2} = \sqrt{16+21.16} = \sqrt{37.16}$	$= 6.10'$
15	$4 + 2\left(3/5\right)$	$= 5.20'$
16	$\sqrt{6^2+\left(\frac{6\times 6}{40}\right)^2} = \sqrt{36+.81}$	$= 6.07'$
17	$\sqrt{6^2+(5.20+.9)^2} = \sqrt{36+37.21} = \sqrt{73.21}$	$= 8.56'$
20	$5.20 + 2(.9)$	$= 7.00'$
22	$\sqrt{3^2+3^2} = \sqrt{18}$	$= 4.24'$
25	$\sqrt{8^2+\left(8\times\frac{6}{40}\right)^2} = \sqrt{8^2+1.2^2} = \sqrt{65.44}$	$= 8.09'$
26	$\sqrt{8^2+(7+1.2)^2} = \sqrt{64+67.24}$	$= 11.45'$
29	$7.00 + 2(1.2)$	$= 9.40'$
30	$\sqrt{10^2+\left(10\times\frac{6}{40}\right)^2} = \sqrt{100+1.5^2} = \sqrt{102.25}$	$= 10.11'$
31	$\sqrt{10^2+(9.40+1.5)^2} = \sqrt{100+10.9^2} = \sqrt{218.81}$	$= 14.79'$
34	$9.40 + 2(1.5)$	$= 12.40'$
35	$\sqrt{12^2+\left(12\times\frac{6}{40}\right)^2} = \sqrt{144+(1.8)^2} = \sqrt{147.24}$	$= 12.13'$
36	$\sqrt{12^2+(12.4+1.8)^2} = \sqrt{12^2+14.2^2} = \sqrt{345.64}$	$= 18.59'$
	Bottom $= 12.4 + 2(1.8) = \underline{16'}$ Check.	

Dead Stress

Weight of Structure only

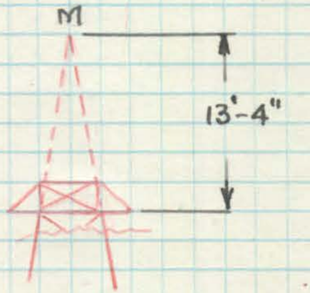
Member		
1	$20\left(\frac{3.6}{2}\right)$	$= +36^{\#}$
2	$36\left(\frac{3}{3.6}\right)$	$= -30^{\#}$
3	$20 + 40$	$= -60^{\#}$
4		$= +30^{\#}$
7	$30 + 303\left(\frac{.6}{4.04}\right)$	$= -303^{\#}$
11	$(60+240)\frac{4.04}{4}$	$= -75^{\#}$
16	$\frac{6.07}{6}(300+400)$	$= -708^{\#}$
22	$\frac{4.24}{3}(100)$	$= -141^{\#}$
25	$\frac{8.09}{8}(700+100+560) = 1.011(1360)$	$= -1376^{\#}$
20	$H.C. 22 + H.C. 25 = 100 + \frac{1.2}{8}(1360)$	$= -304^{\#}$
30	$\frac{10.11}{10}(1360+720) = 1.011(2080)$	$= -2103^{\#}$
35	$\frac{12.13}{12}(2080+880) = 1.011(2960)$	$= -2993^{\#}$

For symmetrical loading like this there will be no stress in the cross-bracing.

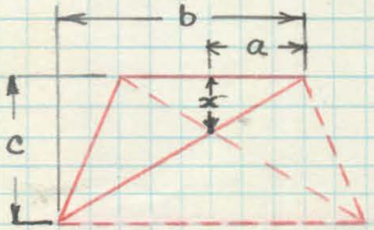
WIND STRESSES

All the wind will be assumed to be acting on the one side. Such a loading, instead of the wind acting on both columns, makes a slight difference in the struts only, & this difference is on the safe side.

The wind bracing is assumed to be incapable of taking compression, and therefore in each panel only one member, that taking tension, is considered.



Distance from top of inclined columns to their point of intersection $= \frac{40}{6} \times 2 = \underline{\underline{13\frac{1}{3}'}}$



Distance from intersection of bracing to strut above:

$$\frac{x}{a} = \frac{c}{b}$$

$$\therefore x = \frac{ac}{b}$$

Panel No.	x
1	$\frac{2 \times 4}{4.6} = 1.74'$
2	$\frac{2.6 \times 6}{6.10} = 2.56'$
3	$\frac{3.5 \times 8}{7.12} = 3.93'$
4	$\frac{4.70 \times 10}{9.55} = 4.93'$
5	$\frac{6.2 \times 12}{12.58} = 5.92'$

WIND STRESSES (BRACING)

Take Moments about M, the intersection of the inclined columns.

Member	Horizontal Component	Stress.
7	$1040 + 200 = 1240$	-1240
12	$[H.C.12](13.33 + 1.74) = 1240(13.33)$	
	$H.C. = \frac{16520}{15.07} = 1100$	$1100\left(\frac{6.10}{4.6}\right) = +1460^{\#}$
15	$[H.C.15](13.33 + 4) = (1240)(13.33) + (250)(17.33)$	
	$\therefore H.C. = \frac{16520 + 4340}{17.33} = \frac{20860}{17.33} = 1200$	$= -1200$
17	$[H.C.17](17.33 + 2.56) = 1240(13.33) + 250(17.33)$	
	$H.C. = \frac{20860}{19.89} = 1050$	$1050\left(\frac{8.56}{6.10}\right) = +1475$
20	$[20](13.33 + 10) = 1240(13.33) + 250(17.33) + 350(23.33)$	
	$[20] = \frac{20860 + 8160}{23.33} = \frac{29020}{23.33} = 1245$	-1245
26	$[H.C.26](23.33 + 3.93) = 29020$	
	$H.C. = \frac{29020}{27.26} = 1065$	$1065\left(\frac{11.45}{8.20}\right) = +1488$
29	$[29](13.33 + 18) = 1240(13.33) + 250(17.33) + 350(23.33) + 450(31.33)$	
	$[29] = \frac{29020 + 14090}{31.33} = \frac{43110}{31.33} =$	-1380
31	$[H.C.31](31.33 + 4.93) = 43110$	
	$\therefore H.C. = \frac{43110}{36.26} = 1190$	$1190\left(\frac{14.79}{10.90}\right) = +1615$
34	$[34](13.33 + 28) = 43110 + 550(41.33)$	
	$[34] = \frac{65810}{41.33} =$	$-1592.$
36	$[H.C.36](41.33 + 5.92) = 65810$	
	$H.C. = \frac{65810}{47.25} = 1391$	$1391\left(\frac{18.59}{14.20}\right) = +1822$

WIND STRESS (COLUMNS)

Take moments about lower right panel point

Member	Vertical Component	Stress.
11	$[V.C._{11}](5.20) = 1240(4)$ $V.C. = \frac{4960}{5.2} = 954$	$954(1.011) = \pm 964^{\#}$
16	$[V.C](7.00) = 1240(10) + 250(6) = 12400 + 1500$ $V.C. = \frac{13900}{7} = 1986$	$1986(1.011) = \pm 2010$
25	$[V.C](9.40) = 1240(18) + 250(14) + 350(8)$ $V.C. = \frac{22300 + 3500 + 2800}{9.4} = \frac{28600}{9.4} = 3040$	$3040(1.011) = \pm 3070$
30	$[V.C](12.40) = 1240(28) + 250(24) + 350(18) + 450(10)$ $V.C. = \frac{34750 + 6000 + 6300 + 4500}{12.40} = \frac{51550}{12.40} = 4160$	$4160(1.011) = \pm 4200$
35	$[V.C](16.0) = 1240(40) + 250(36) + 350(30) + 450(22) + 550(12)$ $V.C. = \frac{49600 + 9000 + 10500 + 9900 + 6600}{16}$ $= \frac{85600}{16} = 5350$	$5350(1.011) = \pm 5420$

Dead Stress due to Cable:

Weight from track cable = 654#

" " traction " = 104#

① = $654\left(\frac{3.61}{2}\right) = +1178$#

② = $654\left(\frac{3}{2}\right) = -1986$

③ = -654

④ = $+986$

⑦ = $986 + \frac{.6}{4}(654) = -1080$

⑪ = $654(1.011) = -661$

⑯ & other columns = $(654 + 104)(1.011) = -766$

Max. symmetrical ~~live~~ loading, with bucket at the tower.

Then W (for each side) = $4234 - 654 + 4000$
(Total - dead + bucket)

= 5580#

① = $5580\left(\frac{3.61}{2}\right) = +10500$#

② = $5580\left(\frac{3}{2}\right) = -8360$

③ = -5580

④ = $+8360$

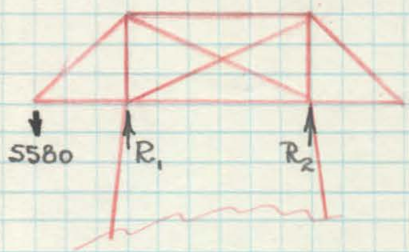
⑦ = $8360 + \frac{.6}{4}(5580) = -9196$

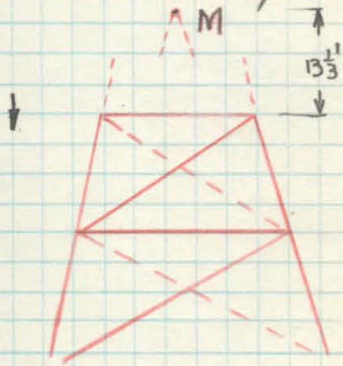
⑪ & all other columns = $5580(1.011) = -5650$

LIVE STRESS

Only one side loaded:

For asymmetrical loading, the cross-bracing will be stressed; and as in the case of wind stresses, the bracing is assumed to take tension only.





$R_1 = \frac{5580 \times 7}{4} = +9750^{\#}$

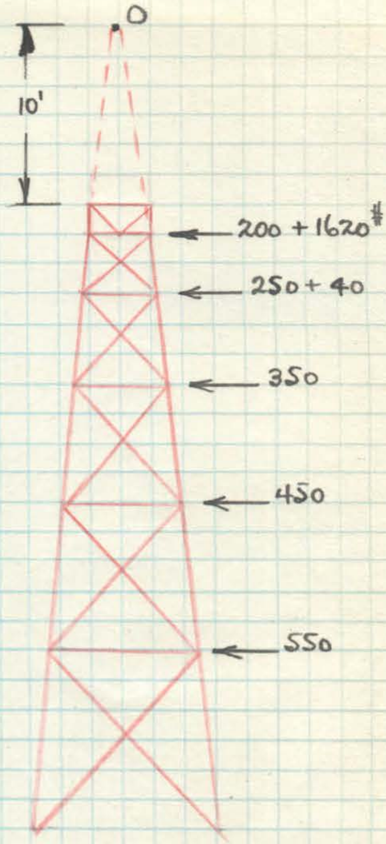
$R_2 = -9750 + 5580 = -4170^{\#}$

11 & other columns	$9750\,(1.011) = -9860^{\#}$
14 & other columns	$4170\,(1.011) = +4220^{\#}$
13.	$\Sigma M_M = 0: \ (5580)(5) = H.C.\,(13.33 + 1.74)$ $\therefore H.C. = \frac{27900}{15.07} = 1850 \qquad \therefore (13) = 1850\left(\frac{6.10}{4.6}\right) = +2455^{\#}$
7	$\Sigma H = 0:$ $5580\left(\frac{3}{2}\right) + 9750\left(\frac{.6}{4}\right) = 8370 + 1462 \qquad = -9832^{\#}$
15	$\Sigma H = 0 \qquad \therefore (15) = H.C\,(13) = -1850^{\#}$
18	$\Sigma M_M = 0: \ 5580(5) = H.C.\,(17.33 + 2.56)$ $\therefore H.C. = \frac{27900}{19.89} = 1402; \quad \therefore (18) = 1402\left(\frac{8.56}{6.1}\right) = +1970^{\#}$
20	$= H.C.\,(18) \qquad = -1402^{\#}$

27	$\Sigma M_M = 0 \quad \therefore H.C. = \frac{27900}{23.33 + 3.93} = \frac{27900}{27.26} = 1022$
	$\therefore (27) = 1022\left(\frac{11.45}{8.2}\right) = +1430^{\#}$
29	$(29) = H.C.\ 27 = -1022^{\#}$
32	$\Sigma M_M = 0 \therefore H.C. = \frac{27900}{31.33 + 4.93} = \frac{27900}{36.26} = 769$
	$\therefore (32) = 769\left(\frac{14.79}{10.9}\right) = +1042^{\#}$
34	$= H.C.\ 32 = -769^{\#}$
37	$\Sigma M_M = 0 \quad \therefore H.C. = \frac{27900}{41.33 + 5.92} = \frac{27900}{47.25} = 591$
	$\therefore (37) = 591\left(\frac{18.59}{14.20}\right) = +775^{\#}$
5 & 6	$(9750 - 5580)\frac{4.47}{2} = 4170(2.235) = +9320^{\#}$

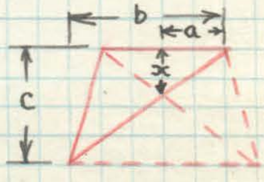
Longitudinal System.

Loading to produce stress in the longitudinal system is as shown in the figure. The loads are the wind loads and the horizontal components from both the track and traction cables.



	Length
39	2'
43	$2 + 2\left(\frac{4}{40} \times 4\right) = 2.80'$
47	$2.8 + 2\left(\frac{4}{40} \times 6\right) = 4.00'$
51	$4.0 + 2\left(\frac{4}{40} \times 8\right) = 5.60'$
55	$5.6 + 2\left(\frac{4}{40} \times 10\right) = 7.60'$
Bottom	$7.6 + 2\left(\frac{4}{40} \times 12\right) = 10'$ check.
40	$\sqrt{2^2 + 4.4^2} = \sqrt{4 + 19.36} = 4.83'$
44	$\sqrt{2.8^2 + 3.4^2} = \sqrt{7.84 + 11.56} = \sqrt{29.40} = 5.42'$
48	$\sqrt{4^2 + 4.8^2} = \sqrt{16 + 23.04} = \sqrt{39.04} = 6.25'$
52	$\sqrt{5.6^2 + 6.6^2} = \sqrt{31.36 + 43.56} = \sqrt{74.92} = 8.65'$
56	$\sqrt{7.6^2 + 8.8^2} = \sqrt{57.76 + 77.44} = \sqrt{135.20} = 11.62'$

Panel No.	Dist. to intersection of diagonals
1	$\frac{1 \times 4}{2.4} = 1.67'$
2	$\frac{1.4 \times 6}{3.4} = 2.47'$
3	$\frac{2.0 \times 8}{4.8} = 3.33'$
4	$\frac{2.8 \times 10}{6.6} = 4.25'$
5	$\frac{3.8 \times 12}{8.8} = 5.18'$



$x = \frac{ac}{b}$

STRESSES (BRACING)

Take Moments about 0, the point of intersection of the inclined columns.

Member	Stress.
39	-1820
41	$(H.C.)(10+1.67) = 1820(10)$ $\therefore H.C. = \frac{18200}{11.67} = 1560 \qquad \therefore (41) = 1560\left(\frac{4.83}{2.4}\right) = +3140^{\#}$
43	$[(43)](10+4) = 1820(10) \qquad \therefore (43) = \frac{18200}{14} = -1300^{\#}$
45	$[H.C.](14+2.47) = 1820(10) + 290(14)$ $\therefore H.C. = \frac{18200+4060}{16.47} = \frac{22260}{16.47} = 1350 \qquad (45) = 1350\left(\frac{5.42}{3.4}\right) = +2150^{\#}$
47	$[(47)](20) = 22260 \qquad \therefore (47) = -1113$
49	$(H.C.)(20+3.33) = 1820(10) + 290(14) + 350(20)$ $\therefore H.C. = \frac{22260+7000}{23.33} = \frac{29260}{23.33} = 1255 \quad \therefore (49) = 1255\left(\frac{6.25}{4.8}\right) = +1632^{\#}$
51	$[(51)](28) = 29260 \qquad \therefore (51) = \frac{29260}{28} = -1044^{\#}$
53	$(H.C.)(28+4.25) = 29260 + 450(28)$ $\therefore H.C. = \frac{29260+12600}{32.25} = \frac{41860}{32.25} = 1298 \qquad \therefore (53) = 1298\left(\frac{8.65}{6.6}\right) = +1700^{\#}$
55	$[(55)](38) = 41860 \qquad \therefore (55) = -1100^{\#}$
57	$(H.C.)(38+5.18) = 41860 + 550(38)$ $\therefore H.C. = \frac{41860+20900}{43.18} = \frac{62760}{43.18} = 1455 \quad \therefore (57) = 1455\left(\frac{11.62}{8.8}\right) = +1925^{\#}$

WIND STRESS in Columns (inc. horiz. pull of cables)

Take Moments about right hand panel point in each panel.

Member	Vertical Component	Stress
11 42	$(V.C.)(2.8) = 1820(4)$ $\therefore V.C. = \frac{7280}{2.8} = 2600$	⑪ $= 2600(1.005) = -2650^{\#}$ ㊷ $= +2650^{\#}$
16 46	$(V.C.)(4.0) = 1820(10) + 290(6)$ $V.C. = \frac{18200 + 1740}{4} = \frac{19940}{4} = 4985$	⑯ $= 4985(1.005) = -5010^{\#}$ ㊻ $= +5010$
25 50	$(V.C.)(5.6) = 1820(18) + 290(14) + 350(8)$ $\therefore V.C. = \frac{32800 + 4070 + 2800}{5.6}$ $= \frac{39670}{5.6} = 7090$	㉕ $= 7090(1.005) = -7130^{\#}$ ㊿ $= +7130$
30 54	$(V.C.)(7.6) = 1820(28) + 290(24) + 350(18) + 450(10)$ $V.C. = \frac{51000 + 6960 + 6300 + 4500}{7.6}$ $= \frac{68760}{7.6} = 9060$	㉚ $= 9060(1.005) = -9110^{\#}$ (54) $= +9110$
35 58	$(V.C.)(10) = 1820(40) + 290(36) + 350(30) + 450(22) + 550(12)$ $V.C. = \frac{72800 + 10430 + 10500 + 9900 + 6600}{10}$ $= \frac{110230}{10} = 11023$	㉟ $= 11023(1.005) = -11080^{\#}$ (58) $= +11080$

MAXIMUM STRESSES

Member	Maximum Possible Stress	Size Angle *
1	+36+1178+10500 = +11714	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{7}{16}$
2	−30−982−8360 = −9372	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{7}{16}$
3	−60−654−5580 = −6294	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
4	+30+982+8360 = +9372	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
5	+9320	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
7	−75−1030−1240−9320 = −11665 or +9350−75−1030 = +8245	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{7}{16}$
11	−303−661−2650−9860 = −13474 +4220+2650−303−661 = +5906	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$
12	+1460+2455 = +3915	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
15	−1200−1850 = −3050	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
16	−708−766−5010−9860 = −16344 +5010+4220−708−766 = +7756	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$
17	+1475+1970 = +3445	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
20	−1245−1402 = −2647	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
22	−141 −294 = −435	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
25	−1376−766−9860−7130 = −19132 +4220+7130−1376−766 = +9208	$3 \times 3 \times \frac{5}{16}$
26	+1488+1430 = +2918	$2 \times 2 \times \frac{5}{16}$
29	−1380−1022 = −2402	$2 \times 2 \times \frac{5}{16}$
30	−2103−766−9110−9860 = −21839 −2103−766+9110+4220 = +10461	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$
31	+1615+1042 = +2657	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
34	−1592−769 = −2271	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$
35	−2993−766−11080−9860 = −24699 −2993−766+11080+4220 = +11541	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$
36	+1822+775 = +2597	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$

*Minimum thickness $= \frac{5}{16}''$

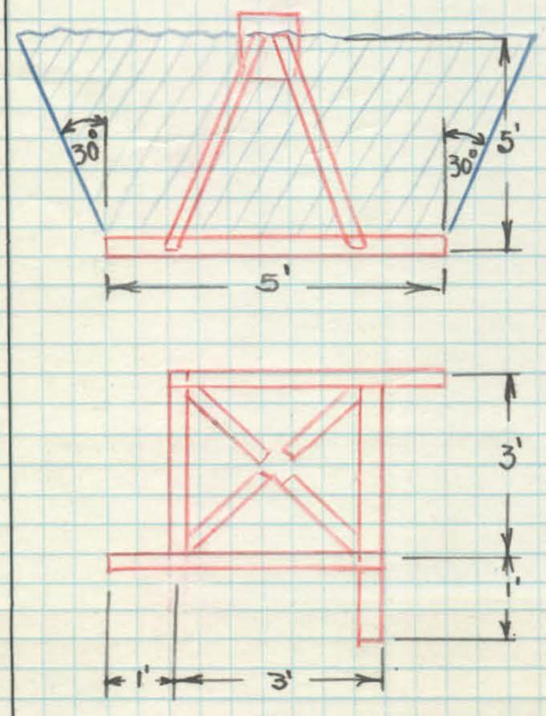
FOOTING.

Maximum vertical Reaction $= (24699)\frac{1}{1.011} - 2597\left(\frac{12}{18.59}\right)$

$= 24430 - 1675 = 22755^{\#}$

" " uplift $= 11541\left(\frac{1}{1.011}\right) = 11415^{\#}$

It is evident that the uplift is the controlling factor. The footing will be designed on the basis that the superimposed earth on the footing must be of sufficient weight to counteract the uplift, the volume of earth to be taken as the frustrum of a pyramid of proportions as shown in the figure (which is conservative)



Try $5'' \times 3'' \times \frac{5}{16}''$ ∠s
(5" side down)
and $2 \times 2 \times \frac{5}{16}''$ struts

Volume of frustrum of pyramid $= \frac{1}{3}(A_1 + A_2 + \sqrt{A_1 + A_2})h$
where A_1 & A_2 = end areas
& h = height.

$A_1 = 4 \times 4 = 16 \text{ ft}^2$

$A_2 = (4 + 2 \times 5 \times \tan 30°)^2$

$= (4 + 10 \times .57735)^2$

$= 9.77^2 = 95.45 \text{ ft}^2$

$\therefore V = \frac{1}{3}(111.45 + 10.55)5$

$= 40.67 \times 5 = 203.35 \text{ ft}^3$

@ $100^{\#}/\text{ft}^3 = \underline{20335}^{\#}$

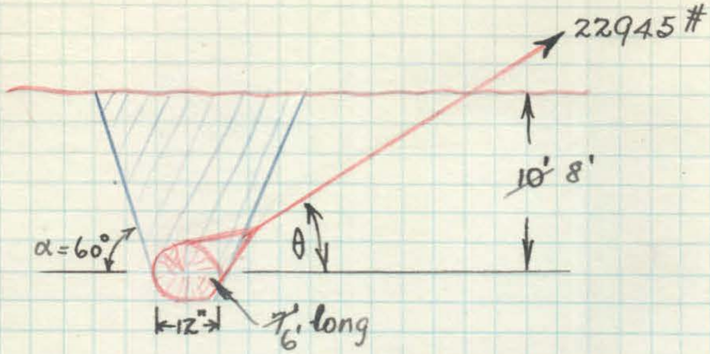
which gives a factor of safety of nearly 2 O.K.

Bearing area $= 4 \times 5 \times 48 = 960 \text{ in}^2 = 6.67 \text{ ft}^2$

$\therefore$ " stress $= \frac{22755}{6.67} = 3410^{\#}/\text{ft}^2$ on soil O.K.

ANCHOR

(Dead Man)



Although there is no particular method of designing an anchorage of this sort, it may be figured (like the the footing for the tower), on the basis that the weight of the superimposed earth must be heavy enough to counteract the uplift.

Take $\alpha = 60°$ & $\theta = 25°$; log 12" x 7' long.

$V.C. = 22945(.42262) = 9697\#$

$\therefore$ Volume of frustum of pyramid $= \frac{1}{3}(A_1 + A_2 + \sqrt{A_1 + A_2})h$

$A_1 = 1 \times 7 = 7\ ft^2$

$A_2 = (1 + 2 \times 10 \times .57735)7 = 12.546 \times 7 = 87.822\ ft^2$

$V = \frac{1}{3}(94.82 + 9.72)10 = 348.5\ ft^3$

@ $100\#/ft^3 = 34850\#$ unnecessarily high.

Try 12" x 6' log imbedded 8'

$A_1 = 6\ ft^2$

$A_2 = (1 + 2 \times 8 \times .57735)6 = 10.24 \times 6 = 61.44$

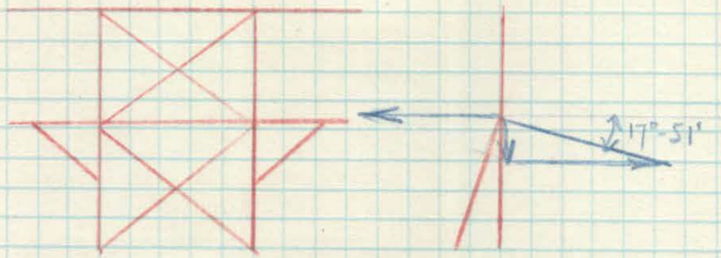
$V = \frac{1}{3}(67.44 + 8.20)8 = 25.21 \times 8 = 201.68\ ft^3$

@ $100\#/ft^3 = 20168\#$ which gives a factor of safety slightly in excess of 2

O.K.

TERMINAL.

For plan of terminal, see blue-print.



V.C. = 22945 sin 17-51 = 22945(.30655)
= 7034#

H.C. = 22945(1-cos 17-51) = 22945(1-.95185)
= 1105#

Assuming thrust to be taken up by strut, & max. load 6000,#

load = 7034 + 6000 = 13034

Using 5×5 Douglas Fir,

$S = \frac{13034}{25} = 522\ \#/in^2$ O.K.

Allowable Str. = $1200\left(1 - \frac{l}{60d}\right) = 1200\left(1 - \frac{108}{60 \times 5}\right) = 768\ \#/in^2$

For concrete foundation, allowing 400#/in² in comp.

$\frac{13034}{400} = 32.6\ in^2$ are req.

For soil at 4000#/ft² $\frac{13034}{4000} = 3.25\ ft^2$

Beam to carry rail.

Assume uniform dead load = 8#/ft
Wt. of rail = 3 "

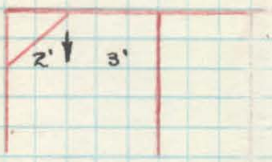
Max. W = 4000# assumed to be distributed over entire span through rail & hanger.

Span = 10' ∴ 400#/ft.

$$M = \frac{411 \times 100 \times 12}{8} = 60{,}800 \text{ in-lbs}$$

Try 5"×8" (Douglas Fir)

$$S = \frac{60800 \times 4 \times 12}{5 \times 8^3} = \underline{\underline{1140 \text{ \#/in}^2}} \ (< 1200 \text{ \#/in}^2) \text{ O.K.}$$

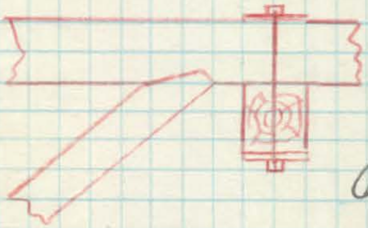


Top beam:

M (without strut) $= \frac{2}{5} \times 4000 \times 36 = 57600$ in-lbs

Assume beam takes 30000 in#

Try 5×6 $S = \frac{30000 \times 3 \times 12}{5 \times 6^3} = 1000\#$ (< 1200) O.K.



W on bolt = 4000+2000+110 = 6110#

$\underline{\tfrac{3}{4}\text{" bolt}} = \frac{6110}{.443} = 13800$ #/in² O.K.

Allowing 310#/in² bearing on Douglas Fir,

area of plate $= \frac{6110}{310} = 19.7$ □" ∴ Use $\underline{\underline{4" \times 5"}}$

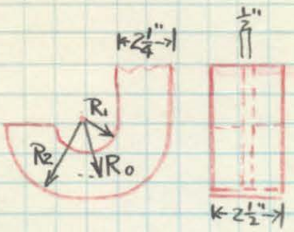
Thickness $t = p\sqrt{\frac{3R}{Af}}$ where p = overhang, R = load

$\underline{\text{Using 2" washer}}$, $t = 1.5\sqrt{\frac{3 \times 6110}{20 \times 16000}} = 1.5\sqrt{.0574} = 1.5 \times .24 = .36$ in

or $\underline{\underline{3/8"}}$

Rail Hanger.

Design as hook or curved beam.



Figured as rectangular section & flanges added for extra safety.

$R_1 = 2"$; $R_2 = 4"$; $d = 2\frac{1}{4}"$

R_o (radius of neutral axis) $= \dfrac{R_1}{\log_\varepsilon \frac{R_2}{R_1}} = \dfrac{2}{\log_\varepsilon 2} = \dfrac{2}{.69315} = 2.886"$

v_o (dist. neutral axis to c.g.) $= R_1 + \frac{d}{2} - R_o = 3\frac{1}{8} - 2.886 = .114"$

$S = \dfrac{P}{A} + S_1$ where $S_1 = \dfrac{M(1 - \frac{R_o}{r})}{b v_o d}$; $M = 4000 \times 2" = 8000"\#$

$S_1 = \dfrac{8000(1 - \frac{2.886}{2})}{.5(.239)2.25} = \dfrac{8000(.443)}{.5(.239)(2.25)} = {}^{+}13180$

$\dfrac{P}{A} = \dfrac{4000}{1.125} = 3550$

$\therefore S_t = 3550 + 13180 = 16730 \ \#/in^2$

Flange: $2\frac{1}{2}" \times \frac{3}{8}"$

Shear on bolt = 4000#

$2 - \frac{1}{2}"$ bolts $= \dfrac{4000}{2 \times .196} = 10200\#$ too high

$2 - \frac{5}{8}"$ bolts $= \dfrac{4000}{2 \times .307} = 6520\#$ O.K.