

ANALYSIS OF THE LINE LAYOUT,
CABLE AND SHEAVE REQUIREMENTS,
AND FOUNDATIONS OF A 2000 FT.
T-BAR SKI-LIFT

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE
Emery Carlson
1948

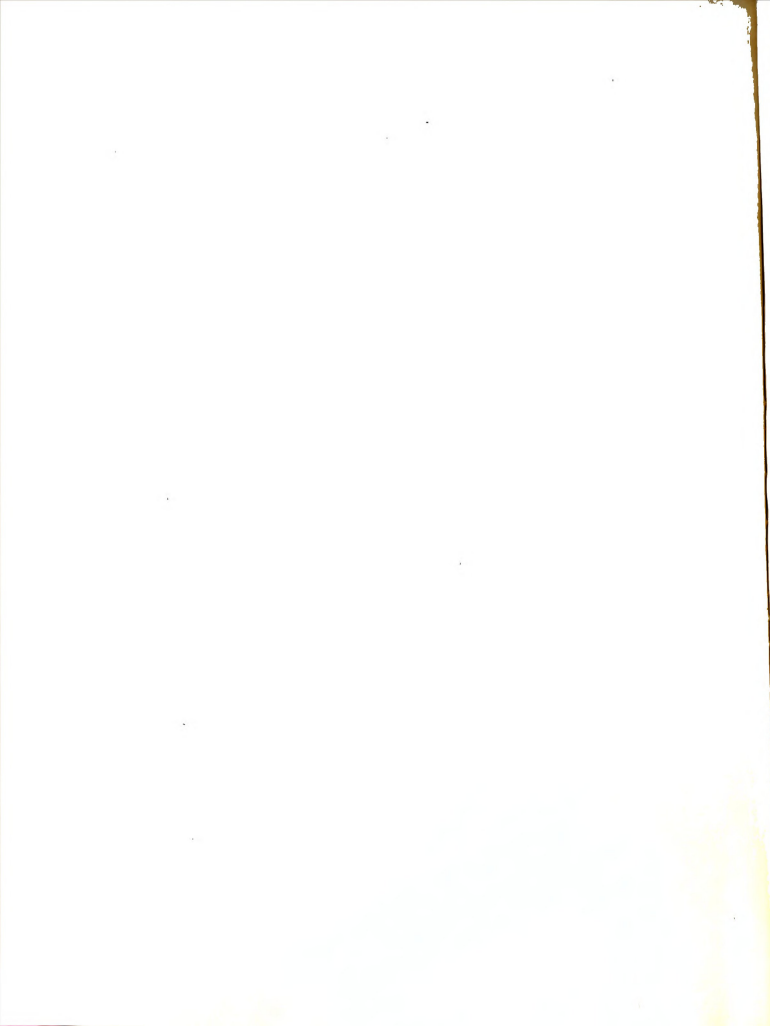
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ANALYSIS OF THE LINE LAYOUT, CABLE
AND SHEAVE REQUIREMENTS, AND FOUN-
DATIONS OF A 2000 FT. T-BAR SKI-LIFT.

A thesis submitted to
The faculty of
MICHIGAN STATE COLLEGE
of
AGRICULTURE AND APPLIED SCIENCE

by
Emery Carlson

Candidate for the degree of
Bachelor of Science

June, 1946

THESIS

6/11/48
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ACKNOWLEDGMENT

The writer wishes to take this opportunity to express his sincere appreciation to Mr. Blair Birdsell, Ass't Chief Engineer, Bridge Division of the John A Roebling's Sons Company; and to Mr. John R. Herr, Ass't Superintendent of American Steel and Wire Company for their kind consideration and advice. It was through the information they furnished that this paper was made possible. In addition, I would like to thank Professor Cade of the Michigan State College faculty for his advice throughout the project.

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PREFACE

I have chosen to analyze the design of a ski-lift for several reasons: first, being an ardent ski enthusiast, I have an intense interest in the subject and have had the opportunity to use and observe several different types of ski lifts while in operation. Secondly, since the sport of skiing has become popular in the United States only in the last ten years, it is a relatively new subject. The need for more and better skiing facilities is apparent to anyone who becomes well acquainted with the sport. I have chosen to analyze a T-bar lift because they seem to be the most practical in both the economic and engineering sense for the majority of slopes that are available in this section of the country. Since there are probably less than half a dozen companies in the whole United States that specialize in this type of work, the available material concerning the design of lifts is very limited. If the material presented here in any way helps further the efforts of those interested in the subject, the work I have put forth will have been well worth while.

INTRODUCTION

The engineering background for this type of structure is not new at all. For many years numerous companies have been designing and building this type structure for transporting freight of various types. The common name applied to these structures is "aerial ropeway" or "aerial tramway." These tramways have been long used by mining companies to carry ore or coal over mountainous terrain over distances up to 72 miles. They have been built and successfully used to transport such articles as sand, gravel, cement, rock, bananas, fish, and even optical instruments. Their use as a mode of transportation has been made economically feasible in instances where the ordinary methods proved too expensive or unhandy. The mining industry has by and large made the greatest use of the aerial tramway, though in recent years it has been used in the construction of several of the larger dams. Boulder Dam is a good example of where an aerial tramway was used to overcome especially difficult transportation conditions.

Development of aerial tramways for passenger conveyances was first developed to high degree by the Europeans. It was not until 1938 that the first aerial tramway was built in the United States. This was built on Cannon Mountain for the State of New Hampshire by the American Steel and Wire Company. Since that time, the sport of skiing and the facilities offered mushroomed until today there are an

estimated one-thousand lifts of every variety, including rope tows.

By applying the known and proven engineering knowledge used in building freight aerial tramways, conversion to passenger type structures was relatively easy. The principle problem lay in the field of safety. An economical, yet absolutely safe, structure was needed; one that required no physical alertness or agility on the part of the rider to safely use the lift.

Commercial tramways consist of four basically different types of structure: bi-cable tramway, twin-cable tramway, reversible tramway, and the mono-cable tramway. Since the fastest, most economical method was desired with no excessive weight requirements, a mono-cable or bi-cable type was chosen as most practical for a ski-lift. Of these two types, the mono-cable type has been most extensively used. Breaking the mono-cable types down into further classes, we find a chair lift, a T-bar lift, and a J-bar lift. The least expensive lifts, most simply constructed, are the T-bar and J-bar lifts. These types are of mono-cable variety and constructed so that the skier remains on the ground and is pulled up the hill. Since this type is better suited to general use than the most expensive chair types used for long steep hauls, I have chosen to analyze one of this type. A T-bar lift needs to be of heavier construction than the J-bar, but is used because of the increased capacity obtained.

The system consists of a row of towers supporting an endless wire rope which is constantly moving while the skiers get on and off the lift. Each towing outfit consists of a

rope grip attaching the outfit to the hauling rope, a hanger extending down from this grip, a so-called spring box on hydraulic take-up attached to the bottom of this hanger and a wooden T-stick which dangles from this spring box. The upper end of the T-stick is attached to a piece of small wire rope which is wound on a drum inside of the spring box. In the case of the hydraulic type take-up, the wire is shortened by the hydraulic cylinder device. Both are being successfully used at the present time.

As a skier mounts the lift, this rope is pulled out and is extended to its full length throughout the uphill travel of the skier. The passenger's skis remain on the ground and he is literally pushed uphill by means of the crossbar of the T-stick which rests against his hips. In normal operation two passengers are carried by each T-stick, although one may comfortably ride using a small degree of balance. As the skier lets loose the T-stick it is pulled up and out of reach of the skiers. At the lower end of the structure an anchorage tower is situated and the power is applied at this point.

At the upper end a tension terminal is located where the counter-weight is suspended, acting against the frame which contains the tension sheave. By controlling the weight of this counter-weight and having the tension frame free to ride on a truss of the tension structure, a known and uniform tension is imparted to the traction rope at all times, under all conditions of loading--no load to full load. As the wire rope wears and stretches, the counterweight lowers and the tension frame moves back.

The power is applied to the rope by a large horizontal

rope with a length of 100 feet and a diameter of 1/2 inch.

Extending from the top of the rope to the bottom of the

hydraulic line is a line of 1/2 inch diameter.

A wooden block is placed at the bottom of the rope

and the rope is pulled down to the bottom of the

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sheave located in the anchorage terminal. The power is transferred to this sheave by means of a set of bevel gears, a right angle speed reducer, and a W-belt drive, or some combination of them. The power unit may be either a gasoline engine, diesel engine, or electric motor. The gasoline or diesel engine are used where electricity is not available, but the electric motor is better suited for most units.

The ground under the rope must be relatively smooth and level perpendicular to the line to keep the skiers from sliding off the tow trail. When in operation, three or four operators are required. One should be at the controls of the power unit at all times. Another operator is needed to pull down the T-stick and aid the skiers in getting started. Also, one operator should be stationed at the upper end of the tow as a safety precaution; if the skier doesn't leave the stick at the upper end he would have a free pathway to continue around the tension sheave. This point of the system presents the only point of danger and this is very remote. If a skier falls off the T-stick or becomes fouled in his skis on the tow path he merely lets go the T-stick and maneuvers himself off the path. The great advantage of this type lift is the simplicity by which it operates plus the large capacity it has. It can be used for any reasonable slope with relative comfort to the skier.

In this paper, I will analyze a lift being installed at Jimmy Peak, Hancock, Massachusetts, by the John A. Roebling Sons Company. Since a complete analysis of all the fixtures making up the lift would require more time than is available and would entail many problems outside the field of the Civil Engineer, I will confine my analysis to the following items:

1. Finding the tension of the Cable throughout the entire line on both the uphill (loaded) side and the downhill (empty) side.
2. Finding the maximum tension and size of rope required.
3. Finding the resultant forces on the towers.
4. Finding the number of sheaves required at each tower.
5. Checking the distance between the rope and the ground at all points for all conditions of loading.
6. Checking the design of the foundations against sliding, overturning, and excessive soil pressure.

In the usual procedure of designing a lift, the engineer would have to first make several assumptions based on his past experience and then check them and make the necessary corrections to finally arrive at workable solution. Taking the profile of the proposed lift and the capacity that is expected the designer proceeds along this line.

First, he establishes the position of the towers. The tension and anchorage terminals are first put in at each end of the line. Next, a tower is put at every sharp break in the profile, sometimes spacing several relatively close if the break is very pronounced. Intermediate towers are then spaced between those needed at critical points. The length

IN THIS ORDER, I AM ADVISED

Timothy Beck, Director, Research, National Bureau of Economic Research

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1. The first step is to identify the problem or question that needs to be answered. This involves understanding the context and the specific requirements of the task.

Journal of Management Education 30(6)p.789-804

1. The first group of people who are interested in the study of the history of the United States are the people who are interested in the history of the United States.

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• *Journal of the American Medical Association*

of span must be adjusted such that the sag of the cable will not be so great as to require unreasonably high towers. A good rough rule to use when picking the length of span is to assume the sag to be five percent of the span. The height of tower used in ski lifts may vary from 20 to 30 feet, but should keep within these limits. Knowing the approximate tower height to be used and clearances required of the cable, the distance between towers can be estimated. Once the tower sites have been selected the designer chooses a likely rope size and, using the sag previously allowed for, and the maximum expected load, will calculate the cable tension along the entire line. If the cable strength is sufficient to resist the maximum tension times the given safety factor the rope selected is used. If not, another trial is made.

After the tension at each tower has been calculated, the towers may be designed and the number of sheaves determined. When the maximum tension has been determined a counterweight is selected to keep a constant tension in the cable at the upper end. The anchorage terminal must be designed to accommodate the drive sheave and power plant and suitable frame structure and foundation to resist the tension of the cable. The tension terminal must be designed to contain a tower for suspending the counterweight and for the tension frame to ride freely upon. Speed of the rope must be such that the skier will have sufficient time to get mounted and this time usually determines the capacity of the lift.

My work will consist of showing the method the designer would use to check his first assumptions. I will use the plans and data given in the completed lift and show how they fall within the specifications. The whole design will be conducted with the following end results in mind:

1. That the tow have a capacity of 560 skiers per hour.
2. That the maximum vertical distance between the rope and ground at any point on the ascending side of the line be 26 feet.
3. That the minimum vertical distance between the rope and ground at any point be 16 feet.

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... thanks to the Patented Constam Ski Lift
—built by **ROEBLING**

It's FASTER: The twin seating on this lift means more skiers are on the way up at one time, therefore less waiting between runs. It's more congenial, too... gives you a chance to compare notes, have more fun.

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Consideration No. 1

Determining whether the distance between the rope and snow is within allowable limits at all points.

Data -- (Distances)

1. Average snow depth -- 2 ft.
2. Distance between snow and contact point between T-stick and skier -- 2 ft.
3. Contracted or minimum length of towing outfit -- 8'-8".
4. Extended or maximum length of towing outfit -- 28'-8".

Note: In both 3 and 4 the lengths stated represent the distance from the center line of rope at the rope grip to the cross bar of the T-stick measured along the towing outfit. The tow outfit is connected to the grip by means of a swiveling joint so that this distance can always be considered to be a straight line.

5. Minimum distance between rope and ground at any point-- 16 ft. (full load conditions)
6. Maximum distance between rope and ground at any point -- 26 ft. (single skier)
7. Towers are 16-21-and 26 ft. high.
8. Coefficient of friction between skis and snow--10%.

(Weights)

1. Total weight of towing outfit -- 68 lbs.
2. Average weight of skier -- 165 lbs.
3. Weight of rope -- .9 lbs/ ft.
4. Two skiers ride on each T-stick.

Consideration No. 1

determining whether the distance between the two points is less than the distance between the two points.

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5. Rope data -- $\frac{3}{4}$ " diameter cable, 47,400 lbs. (max. strength)
6. Safety factor on maximum tension -- 5
7. Spacing of tow outfits along rope -- 96.4 feet.

The data given above is required to make the initial computations.

Immediately, the following condition presents a problem where the first assumption must be made. It concerns the the angle that the tow outfit makes with the vertical as the skier proceeds up the slope between the towers. I have shown this as angle (a) in the sketch labeled Figure 1. Since the tow outfit is extended to its full length all the time the skier is being pulled, the angle between the tow outfit and the vertical would vary as the distance between the ground and the main cable changed. This would cause the components of force exerted on the main cable to vary accordingly. This problem was solved by using the following assumptions:

1. Use an average slope for the entire lift.
2. Use an average rope height above ground for the entire length of the lift. Figure 2 shows angle (a) with assumptions made.

With these two assumptions the following set vertical and horizontal forces were found to act upon the hauling rope. Figure 3 gives a sketch showing the forces and how they act. Figure 4 shows the angles used and distances used in computing the forces. The reaction at the main hauling rope is broken down into a component of tension along the

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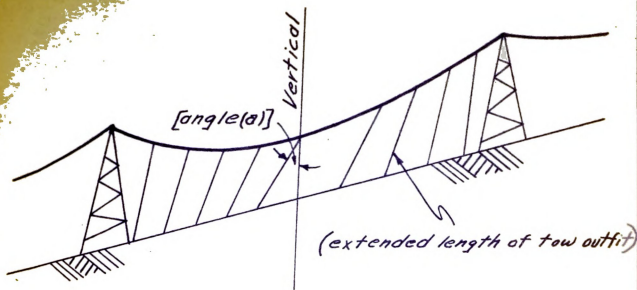
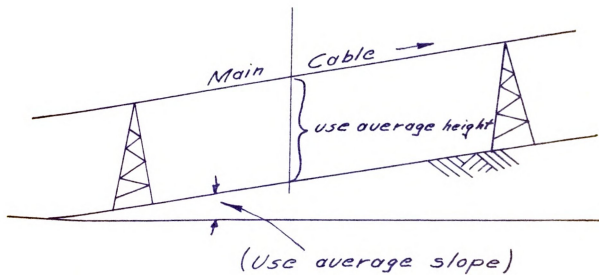
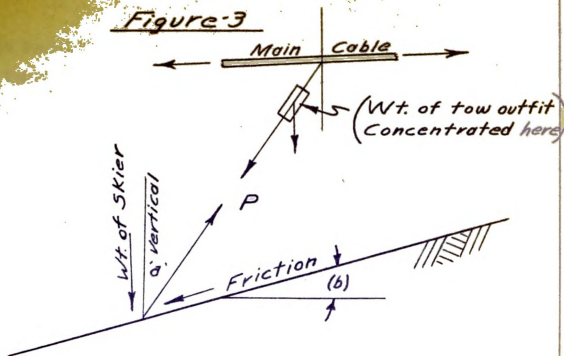
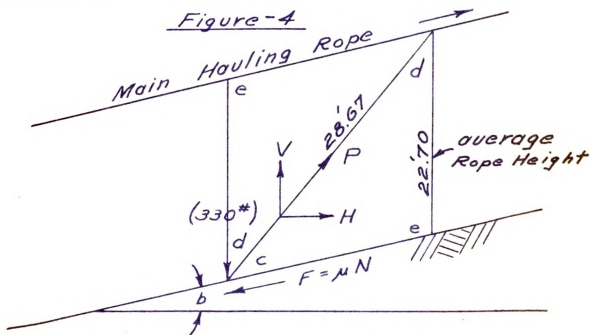
Figure - 1Figure - 2

Figure-3Figure-4

$$\begin{aligned}
 b &= 17^{\circ}07' \\
 c &= 48^{\circ}55' \\
 d &= 23^{\circ}58' \\
 e &= 107^{\circ}07'
 \end{aligned}$$

main hauling rope and a vertical component. The component of tension divided by the spacing of the towing outfits results in an average loss of tension per foot of rope length. Similarly, the vertical reaction plus the weight of the towing outfit divided by the spacing of the towing outfit result in an average vertical load per foot of rope length. This added to the unit weight of the rope gives the total load per foot of rope length for the uphill (loaded) side. The downhill (empty) side loading is simply the weight of the towing outfits and the rope.

Since the loss of tension per foot of cable along the span is known the tension at either end of a span can be calculated knowing one end. From this an average horizontal tension is computed and the following formulas for a uniformly loaded span applied.

**COMPUTATIONS TO FIND THE AVERAGE SLOPE AND AVERAGE HEIGHT
OF ROPE**

Tower Number	Rope Elevation	Ground Elevation	Difference
Anchorage Terminal	130.0	116.0	14.0
1	153.5	141.3 (6 out)	12.2
2	200.6	179.7	20.9
3	292.7	266.5	26.2
4	397.4	371.5	25.9
5	466.7	439.5	27.2
6	534.4	613.5	20.9
7	690.8	664.9	25.9
Tension Terminal	738.5	714.4	24.1
		Total	204.3

Average height of rope -- 22.7'

Total elevation Difference of rope	738.5
	<u>130.0</u>
	608.5

Total elevation Difference of slope	714.4
	<u>116.0</u>
	598.4

Average elevation Difference -- 603.5

Horizontal length of tow -- 1960'

Average slope -- .3079

tan of angle (b)
in Figure 4 - .3079

THE FOLLOWING TABLES SHOW THE RESULTS OF THE ANALYSES OF THE SAMPLES OF THE
 1940

No.	Name	Date	Analysis	
			1940	1941
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Using the figures and dimensions shown in Figure 4, angles b, c, and d were found by the following method:

b -- previously found equal to $17^{\circ}07'$

c -- $90^{\circ} / 17^{\circ}07' = 107^{\circ}07'$

$$\sin c = \frac{(\sin 107^{\circ}07')(22.70)}{28.67} = .7537$$

e -- $48^{\circ}55'$

d -- $90 - b - c = 23^{\circ}58'$

From Analytical Mechanics

$$P = \frac{W (\sin b / u \cos b)}{(\cos c / u \sin c)} = \frac{(330) / (.2943) / .1 (.9557)}{(.6572) / .1 (.7538)} = \underline{\underline{128.67}}$$

Using the Sine Law of Triangles:

Vertical Component

$$V = \frac{128.67 (\sin 48^{\circ}55')}{(\sin 107^{\circ}27')} = \frac{(.7537)(128.67)}{(.9517)} = \underline{\underline{101.91}}$$

Horizontal Component

$$H = \frac{(128.67)(\sin 23^{\circ}58')}{\sin (107^{\circ}27')} = \frac{(128.67)(.4062)}{(.9517)} = \underline{\underline{54.92}}$$

FORCES ACTING ON THE MAIN HAULING ROPE

1. Ascending Side--

A. Vertical Components 101.91 lbs. due to weight of skier
68.00 " due to weight of tow outfit
 169.91 lbs.

Note:

Horizontal spacing of tow outfits equals spacing along rope times cosine of angle (b) of Figure 4--
 $(96.4)(.9557) = 92.13$ feet.

B. Weight per Horizontal Foot $\frac{169.91}{92.13} = 1.85$ lbs.

Weight per Horizontal Foot
 due to Cable = $(.9)(.9557)86"$

Total -- $\frac{2.71 \text{ lbs.}}{\text{per horizontal foot}}$

C. Component along the Rope

$= \frac{54.92}{92.13} = .596$ lbs./Horizontal Foot

$= \frac{54.92}{96.40} = .570$ lbs./foot of rope

2. Descending Side--

A. Vertical Components $\frac{68}{92.13} = .74$ lbs. due to weight of tow outfit

.86 lbs. due to weight of cable
 Total weight per horizontal foot ---- 1.60 lbs.

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B. Weight per Horizontal Foot --- $(1.60)(\sin b)$
 $(1.60)(.2943) = .470 \text{ lbs./foot}$

C. Component along the Rope -- $(.470)(\cos b)$
 $(.470)(.9557) = .449 \text{ lbs./foot of rope}$

3. Total Pull on Cable (assuming no sag)

Ascending - $(2055)(.5697) = 1170.7 \text{ lbs.}$

Descending - $(2055)(.449) = 922.7 \text{ lbs.}$

The problem of figuring the deflection, tension, angles at supports etc. presented a special case not covered by the ordinary formulas for suspended cables. The case of the T-bar lift consisted of a counterweighted span and loads having both vertical and horizontal components. There are numerous equations for vertically suspended loads upon a counterweighted span based on either the catenary or parabolic curves. Upon studying the derivation of formulas presented by F. C. Constanphen in an article published in the A.S.C.E. Proceedings, I attempted to derive a formula for this special case using the same general methods. The formula is based upon a counterweighted span having any number of evenly spaced loads. It applies to any position the loads may assume along the span and is set up for the case where the supports are at different elevations. It is presented as follows:

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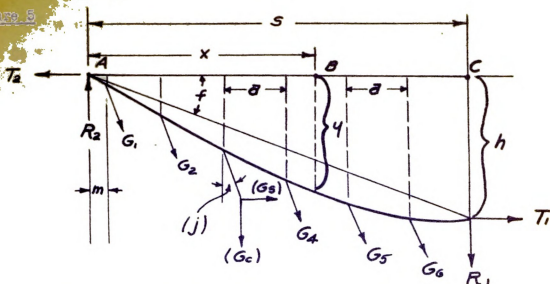
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DERIVATION OF FORMULA

Figure 5

Nomenclature of Quantities

y - Vertical Deflection from Upper Support to Point xy

h - Vertical Difference in elevation of Supports

T - Horizontal Tension of Cable at Lower Support

T - Horizontal Tension of Cable at Upper Support

R - Vertical Tension of Cable at Lower Support

R - Vertical Tension of Cable at Upper Support

a - Horizontal Spacing at loads

m - Horizontal distance from left support to first load

n - Number of concentrated loads

G_s - Horizontal Component of the Concentrated load

G_c - Vertical Component of the Concentrated load

f - Angle between the Horizontal and a chord between supports

s - Horizontal distance between supports

x - Horizontal distance to the left of the point under consideration

w - Weight of cable per foot of horizontal span

t - Horizontal component of Cable tension

P = Number of loads to the left of the Point

$$G_s = G \sin j \quad b = \frac{n(n-1)}{2} \quad c = \frac{p(p-1)}{2}$$

$$G_e = G \sin j$$

In the derivation the cable is treated as a free body.

$$\underline{F_x = 0}$$

$$T - (G_1 + G_2 + G_3 + G_4 + G_5)(\sin j) - T_1 = 0$$

$$\underline{y = 0}$$

$$(R_1 + R_2) - (n G \cos j + ws) = 0$$

$$\underline{M_e = 0}$$

$$R_2 s = (s-m)G + (s-m-a)G_e + (s-m-2a)G_e + (s-m-na)G_e + \frac{ws^2}{2} + T_1 h \\ + G_s(m \tan f) + G_s(m/a) \tan f + G_s((m/20) \tan f) + G_s \\ ((m/na) \tan f) \dots \dots \dots$$

Let $\tan f = x$

$$R = n G_e - \frac{nm G_e}{s} - \left[\frac{n(n-1) G_e a}{2s} \right] + \frac{ws}{2} + \frac{T_1 h}{s} + \left[\frac{nm G_s \tan f}{s} \right] \\ + \left[\frac{n(n-1)a G_s \tan f}{2s} \right]$$

$$\underline{M_b = 0}$$

$$y t = x R - \frac{wx^2}{2} - P x G_s - p m G_s + \left[\frac{p(p-1)a G_s}{2} \right] - \left[p m G_s \tan f \right] \\ - \left[\frac{p(p-1)}{2} a G_s \tan f \right]$$

$$y = \frac{x R}{t} - \frac{wx^2}{2t} - \frac{P x G_s}{t} - \frac{p m G_s}{t} + \left[\frac{p(p-1)}{2t} a G_s \right] - \left[\frac{p m G_s \tan f}{t} \right] \\ - \left[\frac{p(p-1)a G_s \tan f}{2} \right]$$

Substituting R in the equation for y.

$$y = \frac{x}{t} \left[n G_e - \frac{nm G_e}{s} \right] - \left[\frac{x}{t} \frac{b G_e a}{s} \right] + \frac{wsx}{2t} + \frac{T_1 x h}{st} + \frac{x}{t} \left[\frac{nm G_s \tan f}{s} \right] \\ + \frac{x}{t} \left[\frac{b G_s a \tan f}{2s} \right] - \frac{wx^2}{2t} - \frac{p x G_s}{t} - \frac{p m G_s}{t} + \frac{c a G_s}{t} \\ - \left[\frac{p m G_s \tan f}{t} \right] - \left[\frac{c a G_s \tan f}{t} \right]$$

Since $\lim_{n \rightarrow \infty} \frac{1}{n} = 0$ we have $\lim_{n \rightarrow \infty} \frac{1}{n} = 0$

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Reducing terms

$$y = \frac{x G_c}{t} \left[\frac{n-nm}{s} - \frac{ba}{s} \right] + \frac{x}{t} \left[\frac{ws}{2} + \frac{T h}{s} \right] + \left[\frac{x G_s \tan f}{ts} \right] \left[\frac{nm+ba}{2} \right] \\ - \frac{x}{t} \left[\frac{wx}{2} + p G_s \right] + \left[\frac{G_s}{t} \right] \left[\frac{pm+ca}{s} \right] - \left[\frac{G_s \tan f}{t} \right] (pm+ca)$$

Example: Application to Span 3

$$s = 300 \quad G_s = (128.67)(.9138) \quad 117.7$$

$$x = 150 \quad G_c = (128.67)(.4062) \quad 52.2$$

$$G = 128.67 \quad n = 5 \quad p = 1$$

$$j = 23 \text{ } 58' \quad f = 17 \text{ } 04'$$

$$b = \frac{3(2)}{2} \quad 3 \quad \tan f = .3070$$

$$c = \frac{1(1-1)}{2} \quad 0 \quad m = 58.42$$

$$w = .86 \text{ lbs./foot} \quad a = 91.58$$

$$h = 92.1 \quad T = t = 6583$$

$$t = \tan = 6672$$

$$y = \frac{(150)(52.2)}{6672} \left[\frac{3 - \frac{(3)(58.42)}{300} - \frac{3(91.58)}{300}}{2} \right] + \frac{150}{6672} \left[\frac{(986)(300)}{2} \right] \\ + \frac{(6583)(92.1)}{300} + \left[\frac{(150)(117.7)(.307)}{(300)(6672)} \right] \left[\frac{3(58.42 + \frac{(3)(91.58)}{2})}{2} \right] \\ - \frac{150}{6672} \left[\frac{.86(150)}{2} + 117.7 \right] + \left[\frac{117.7}{6672} \right] (58.42) \\ - \left[\frac{(117.7)(.307)(58.42)}{6672} \right] = \underline{\underline{47.54}}$$

Obviously, the above formula is much too cumbersome and far too precise to be practical for the design of a ski lift of this type where considerable leeway is permissible. No attempt has been made to derive a simplified formula covering the special cases of one or two loads centered on the span, although they could be derived by the same methods.

Checking the deflection in Span 3 by this formula, I find the deflection at the center to be 47.54 feet. Using the approximate method used by the J. H. Roebling Company, I find the deflection at the center to be 50.61. Although there appears to be considerable discrepancy in the results the approximate method gives a larger result and would be more toward the safe side in this case than were the reverse true. It is my belief that the derived formula gives the more accurate results since no assumptions are used in its derivation, while the approximate method requires using an average of values and assumes a uniform load which is not the case. Since my derived formula is too unwieldy in its present state and since time is too limited to further investigate, I will use the approximate method in my calculations.

Using the estimated value of 8000 lbs. as the tension at the lower end of the ascending side of the Cable, I worked out the tension along the line toward the top. I found the tension at the top would exceed the strength of the cable I had chosen after applying the safety factor. A more practical method is to start from the upper end

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ALL INFORMATION CONTAINED HEREIN IS UNCLASSIFIED DATE 08-29-2007 BY 60322 UCBAW

—, *et al.*, 1997, *Journal of the American Academy of Child and Adolescent Psychiatry*, 36, 1111-1120.

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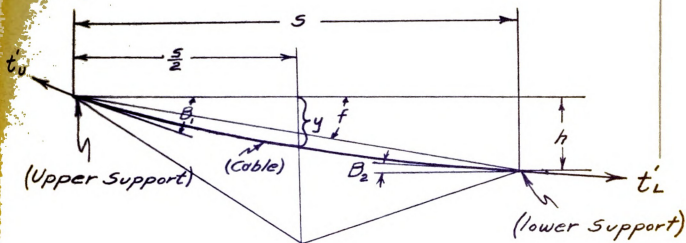
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where the tension is known and kept constant by the counter-weight. Then as one works down along the span the only thing limiting the design is the amount of sag. If the tension became so low at the lower end of the cable as to allow excessive sag, a longer cable and higher initial tension at the upper end would have to be used. On my second trial I worked from the upper end and found the sag to be within limits of the specifications so only two trials were necessary. I calculated the descending line in the same manner from the upper end down.

Figure - 7



THE FOLLOWING NOMENCLATURE AND FORMULAS WERE USED IN THE DESIGN OF THE CABLE.

- y -- Vertical deflection from upper support to point xy
 s -- Horizontal distance between supports
 h -- Vertical Difference in elevations of supports
 w -- Weight per foot of horizontal length of span
 f -- angle between the horizontal and a chord between supports
 B_1 -- Angle between the horizontal and a tangent to a cable curve at the upper support
 B_2 -- Angle between the horizontal and a tangent to cable curve at the lower support
 t_L -- Horizontal component of cable tension at lower support
 t'_L -- Maximum cable tension at lower support
 t_v -- Vertical component of cable tension at lower support
 t_u -- Horizontal component of cable tension at upper support
 t_{uv} -- Vertical component of cable tension at upper support
 t'_u -- Maximum cable tension at upper support
 τ -- Average horizontal component of cable tension in the span

Cable tension approaching and leaving each tower is equal making t'_u of one span equal to t'_L of the next higher span.

Since the horizontal component of tension in the case of this ski lift is not the same throughout the span an average value of the horizontal component at either end will be used instead.

$$y = \frac{ws^2}{8t} + \frac{h}{2}$$

$$B_1 = \frac{ws}{2t} + \tan f$$

$$B_1 = \frac{ws}{2t} - \tan f$$

$$t_u = (t'_L \text{ of next higher span}) (\cosine B_1)$$

$$\tan = t_u - \frac{(\text{length of span})}{s} (\text{Tension Difference per Horizontal Foot})$$

$$t_L = t_u - (\text{length of span})(\text{tension difference per horizontal foot})$$

$$t'_U = t_u \sec B_1$$

$$t'_L = t_L \sec B_2$$

$$t_{UV} = t'_U \sin B_1$$

$$t_{LV} = t'_L \sin B_2$$

Since the maximum deflection is all that is required it is necessary to calculate the deflection only at the center of the span.

Because of the large slope angle B_2 will be found to be an angle above the horizontal exerting an upward pull at the tower which in the case of towers 1 and 5 exceeds the downward pull of the cable in the adjacent span. This condition requires the use of depressor sheaves.

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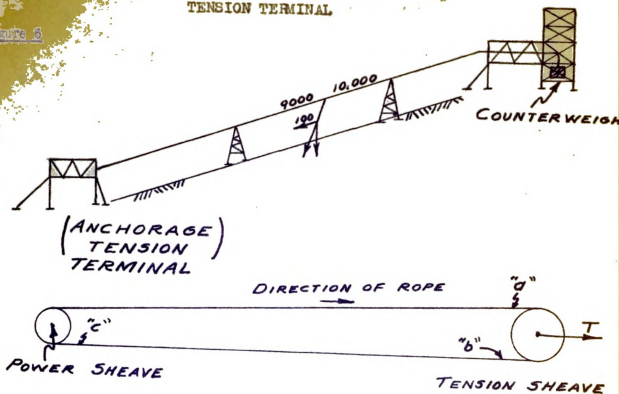
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TENSION TERMINAL

Figure 6



Tension at Point T is equal to the weight of the counter weight.

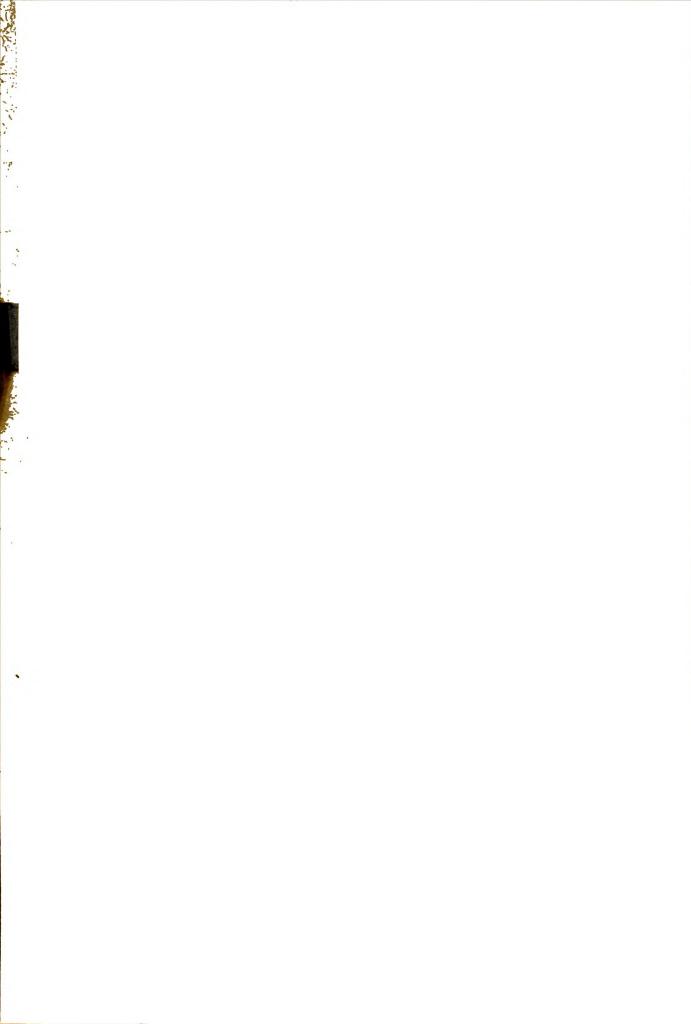
Increased tension in cable due to bending over sheaves applied at either side of tension sheave (a & b) and is .33% of maximum

$$\text{rope tension} = (.0033)(9480) = 31\#$$

Tension at C may be roughly estimated as follows:

$$\text{Assume tension at b} = 9200$$

$$c = 9200 - 31 - 1700 = 8000\#$$



SPAN 1 -- 1st TRIAL

$$w = 2.71 \text{ lbs./foot}$$

$$\text{Assume: } t'_L = 8000\# \quad B_2 = 8^\circ 54' \text{ from angle } f$$

Trial:

$$t_L = (8000)(\cos 8^\circ 54') = (8000)(.988) = 7904$$

$$t_u = (7904) \times (150)(.596) = 7993$$

$$\tau = 7949$$

$$\tan B_2 = \frac{(2.71)(150)}{(2)(7949)} - (.1566) = 7^\circ 28'$$

$$t_L = (8000)(.9915) = 7932$$

$$t_u = (7932) \times (89) = 8021$$

$$\tau = 7977$$

$$\tan B_2 = \frac{(2.71)(.0247)(150)}{(2)(7977)} - (.1566) = .1310 \quad B_2 = 7^\circ 31'$$

$$\tan B_1 = (.0247) \times (.1566) = .1813 \quad B_1 = 10^\circ 17'$$

$$t'_L = (t)(\sec B) = (7904)(1.0087) = 7972$$

$$t'_u = (t_u)(\sec E) = (7993)(1.0165) = 8120$$

$$t_{uv} = (t_u)(\sin B) = (8120)(.1794) = 1457\# \times$$

$$y = \frac{(2.71)(150)^2}{(8)(7949)} \times 11.75 = 12.71$$

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DATA FROM 1ST TRIAL OF ASCENDING SIDE

Span No.	Trial No.	t _L	t _u	t _{au}	B ₂	B ₁	t' _u	t'	tuv	t'v
1	1	7904	7993	7949	7°28'					
	2	7932	8021	7977	7°31'	10°17'	8120	7972	1457	
2	1	7993	8082	8038	12°59'			8196		
	2	7912	8001	7957	12°59'	15°43'	8132	8119	2203	1824
3	1	8082	8261	8171	14°25'			8345		
	2	7877	8056	7967	14°22'	19°43'	8461	8131	2855	2015
4	1	8182	8361	8272	14°40'			8458		
	2	8199	8396	8298	14°41'	20°19'	8915	8476	3095	2145
5	1	8701	8862	8783	12°07'			8899		
	2	8719	8880	8800	12°08'	16°35'	9265	8918	1875	2644
6	1	8700	8924	8812	21°17'			9337		
	2	8637	8861	8749	21°16'	26°46'	9927	9268	4472	3362
7	1	9540	9644	9592	16°34'			9953		
	2	9515	9619	9567	16°34'	19°09'	10185	9927	3340	2830
8	1	9959	10084	10022	11°11'					
	2	9990	10115	10053	11°12'	14°16'	10184	10437	2572	1978

Span No.	γ	Length of Tot. Span	Diff. in Hor. Tens.	$\frac{1}{2}$ Total Diff. Tens.	ϵ	$\tan \epsilon$	$\frac{1}{2}$	$\tan B_1$	$\tan B_2$
1	12.71	150	89	45	$8^{\circ}54'$	.1560	11.75	.1613	.1310
2	24.50	150	89	45	$14^{\circ}22'$	.2560	23.55	.2307	.2307
3	49.87	300	179	89	$17^{\circ}04'$	.3070	46.05	.3580	.2560
4	56.81	331.3	197	98	$17^{\circ}32'$	.3160	52.35	.3702	.2618
5	37.42	269.7	161	81	$14^{\circ}23'$	.2565	34.65	.2979	.2149
6	89.28	375	224	112	$24^{\circ}06'$	.4472	83.85	.5049	.3895
7	29.27	175	104	52	$17^{\circ}52'$	.3223	28.20	.3471	.2975
8	25.22	210	125	63	$12^{\circ}45'$	.2262	23.75	.2543	.1961



SPAN 8 -- 2nd TRIAL

Assume: $B_1 = 14^{\circ}30'$ from 1st Trial

$$t_u = 9200 \#$$

$$t_u = (9200)(\cos B_1) = (9200)(.9680) = (8906)$$

$$t_L = 8906 - 125 = 8781$$

$$\tau_u = 8906 - 63 = 8843$$

$$\tan B_1 = \frac{(2.71)(210)}{(2)(8843)} \div (.2262) = .2584 \quad B_1 = 14^{\circ}59'$$

Assume: $B_1 = 14^{\circ}59'$

$$t_u = (9200)(.966) = 8887$$

$$t_L = (8887 - 125) = 8762$$

$$\tau_u = (8887 - 63) = 8824$$

$$\tan B_1 = \frac{(2.71)(210)}{(2)(8824)} \div (.2262) = .2584 \quad B_1 = 14^{\circ}59'$$

$$\tan B_2 = (.2262) - (.0322) = .1940 \quad B_2 = 10^{\circ}59'$$

$$t_u' = (t_u)(\sec B) = (8906)(1.0352) = 9219 \# \text{ Maximum}$$

$$t_L' = (t)(\sin B) = (8781)(1.0187) = 8945 \#$$

$$t_{uv} = (t_u)(\sin B) = (9219)(.2585) = 2383$$

$$t_{Lv} = (t)(\sin B) = (8945)(.1905) = 1704$$

$$y = \frac{2.71(210)^2}{(8)(8843)} \div 25.75 = 25.44$$

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DATA FROM SECOND TRIAL OF ASCENDING SIB

Spantrial No. No.	tu	t	ten	B ₁	B ₂	t' u	t'	tuv	t v
1	1	6477	6432	10°40'		6587			
	2	6473	6384	10°40'	7°08'	6587	6434	1219	738
2	1	6517	6472	16°02'		6780			
	2	6516	6427	16°02'	12°40'	6780	6587	1873	1445
3	1	6771	6681	20°12'		7205			
	2	6762	6672	20°12'	13°50'	7205	6780	2488	1583
4	1	7196	7097	20°46'		7682			
	2	7183	7084	20°46'	14°11'	7682	7205	2724	1765
5	1	7668	7587	16°56'		8030			
	2	7661	7520	16°56'	11°46'	8028	7682	2339	1566
6	1	7743	7631	27°12'		8690			
	2	7729	7505	27°12'	20°50'	8690	8030	3972	2856
7	1	8305	8253	19°20'		8945			
	2	8441	8357	19°19'	16°23'	8945	8690	2959	2451
8	1	8906	8781	14°59'		9200			
	2	8887	8762	14°59'	10°59'	9210	8945	2383	1704

70	37E
20	
72	20C
12	
30	28C
10	
10	28E
10	
15	30E
15	
2	80
0	
0	8A
0	
0	87

Span No.	γ	Length of Span	Diff. in Hor. Tens.	Assumed B_1 for 1st Trials	r	$\tan r$	$\frac{h}{2}$	$\tan B_1$	$\tan B_2$
1	12.94	150	69	$10^{\circ}30'$	$8^{\circ}54'$	.1560	11.75	.1882	.1260
2	24.75	150	89	$16^{\circ}00'$	$14^{\circ}28'$	.2560	25.55	.2874	.2246
3	50.61	300	179	$20^{\circ}00'$	$17^{\circ}04'$	.3070	46.05	.3678	.2462
4	57.59	351.5	197	$20^{\circ}30'$	$17^{\circ}32'$	.3160	52.35	.3793	.2527
5	37.69	269.7	161	$17^{\circ}30'$	$14^{\circ}23'$	.2565	34.65	.3045	.2083
6	90.11	375	224	$27^{\circ}00'$	$24^{\circ}06'$	.4472	83.85	.5140	.3804
7	29.44	175	104	$20^{\circ}00'$	$17^{\circ}52'$	.3223	28.20	.3506	.2940
8	25.44	210	125	$14^{\circ}30'$	$12^{\circ}45'$	.2262	25.75	.2564	.1940

DATA FROM FIRST TRIAL OF DESCENDING SLIDE

Spectral No. No.	t _n	t	tan	B ₁	B ₂	t _u	t'	t _{uv}	t''
1	1	7293	7256	7°59'	7430				
	2	7358	7297	9°46'	7°59'	7431	7555*	1273	1022
2	1	7305	7270	15°15'		7563			
	2	7297	7226	15°15'	13°28'	7563	7430	2014	1730
3	1	7424	7353	16°46'		7652			
	2	7434	7293	16°46'	15°21'	7651	7563	2326	2002
4	1	7729	7651	19°19'		8175			
	2	7715	7559	19°19'	15°43'	8175	7852	2504	2127
5	1	8088	8025	15°49'		8414			
	2	8095	7968	15°49'	12°56'	8414	8175	2317	1830
6	1	7959	7870	25°53'		8854			
	2	7964	7798	25°53'	22°15'	8852	8414	3364	3186
7	1	8567	8526	18°43'		9025			
	2	8548	8466	18°43'	17°01'	9025	8854	2896	2592
8	1	9010	8911	13°46'		9209			
	2	8936	8837	13°46'	11°43'	9209*	9025	2190	1833

HIGH-9200
LOW-7355

*Assumed Maximum Tension

DATA

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Span No.	Y	Span Length	Diff. in Hor. Tens.	Assumed B ₁ for lat trial	δ	$\tan \delta$	μ 2	$\tan B_1$	$\tan B_2$
12.37		150	71	11°00'	8°54'	.1560	11.75	.1790	.1402
23.91		150	71	15°00'	14°22'	.2560	23.55	.2725	.2395
48.50		300	141	19°00'	17°04'	.3070	46.05	.3396	.2744
55.23		351.3	156	19°00'	17°32'	.3160	52.35	.3503	.2814
56.47		269.7	127	16°00'	14°23'	.2565	34.65	.2933	.2295
87.42		375	170	26°00'	24°06'	.4472	85.85	.4553	.4091
28.91		175	82	18°20'	17°52'	.3223	28.20	.2367	.2059
24.74		210	99	12°00'	12°45'	.2262	25.75	.2451	.2073



SELECTION OF ROPE

Since the lift has been designed for a maximum tension of 9200# and a safety factor of 5 is required, the following rope was selected:

6 x 19 Standard Hoisting Rope
3/4 inch diameter of
Monitor Steel and a
breaking strength of
47,600 lbs., and weight
of .90 lbs./foot.

This conforms with the weight used in the design and agrees with the rope selected by the Roebling Company. A standard hauling rope was selected instead of the locked coil type usually used on tramways because a ski lift would not wear the rope nearly as severely and a great saving in rope cost could be had.

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DISTANCE BETWEEN ROPE AND SNOW

Span No.	(Y)		Clearance on		Change Req'd
	Ascending	Descending	Ascending	on Descending	
1	12.94	12.37	11.86	12.43	cut 6.14
2	24.73	23.91	16.2	17.02	1.8
3	50.61	48.50	24.1	26.2	fill 1.0
4	57.59	55.23	20.6	23.0	None
5	37.89	36.47	23.3	24.7	None
6	90.11	87.42	17.8	20.5	None
7	29.44	22.91	22.2	22.7	None
8	25.44	24.74	13.2	13.9	cut 6

Note: Maximum allowable clearance = 28'

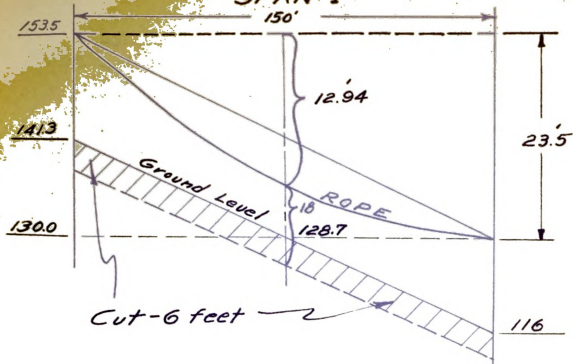
Minimum allowable clearance = 18'

Date		Time		Location		Weather		Remarks	
1900	1901	1902	1903	1904	1905	1906	1907	1908	1909
1910	1911	1912	1913	1914	1915	1916	1917	1918	1919
1920	1921	1922	1923	1924	1925	1926	1927	1928	1929
1930	1931	1932	1933	1934	1935	1936	1937	1938	1939
1940	1941	1942	1943	1944	1945	1946	1947	1948	1949
1950	1951	1952	1953	1954	1955	1956	1957	1958	1959
1960	1961	1962	1963	1964	1965	1966	1967	1968	1969
1970	1971	1972	1973	1974	1975	1976	1977	1978	1979
1980	1981	1982	1983	1984	1985	1986	1987	1988	1989
1990	1991	1992	1993	1994	1995	1996	1997	1998	1999
2000	2001	2002	2003	2004	2005	2006	2007	2008	2009
2010	2011	2012	2013	2014	2015	2016	2017	2018	2019
2020	2021	2022	2023	2024	2025	2026	2027	2028	2029
2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
2040	2041	2042	2043	2044	2045	2046	2047	2048	2049
2050	2051	2052	2053	2054	2055	2056	2057	2058	2059
2060	2061	2062	2063	2064	2065	2066	2067	2068	2069
2070	2071	2072	2073	2074	2075	2076	2077	2078	2079
2080	2081	2082	2083	2084	2085	2086	2087	2088	2089
2090	2091	2092	2093	2094	2095	2096	2097	2098	2099

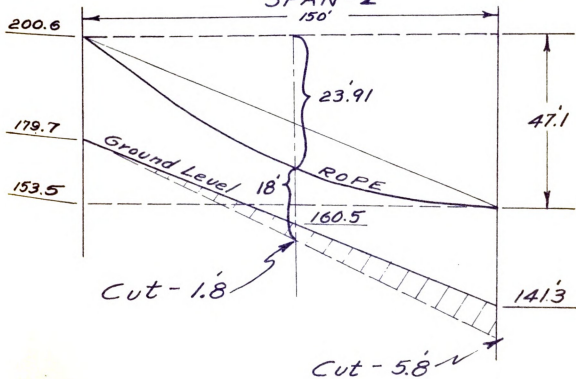
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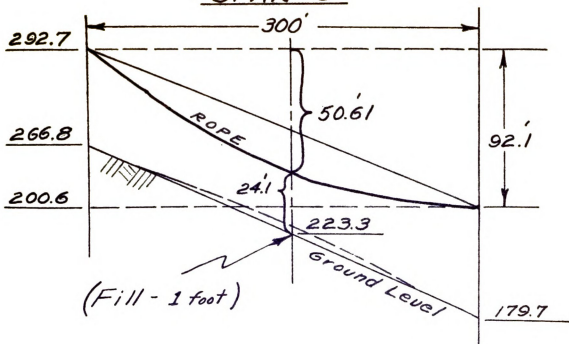
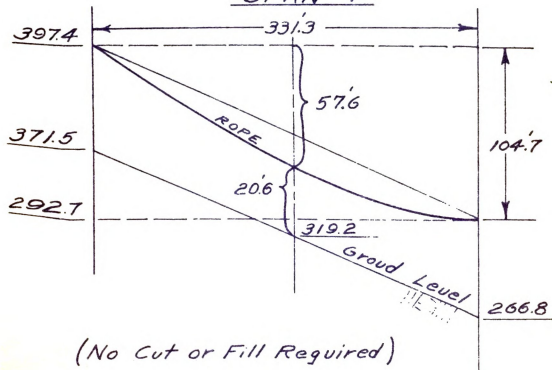
SPAN-1



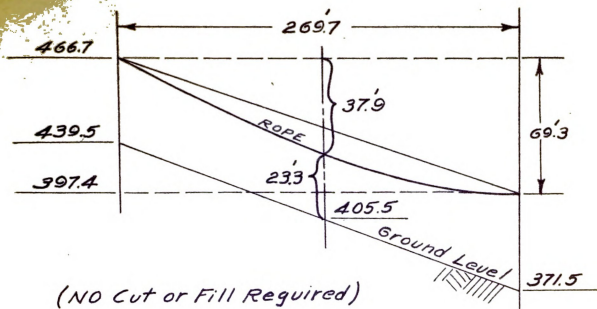
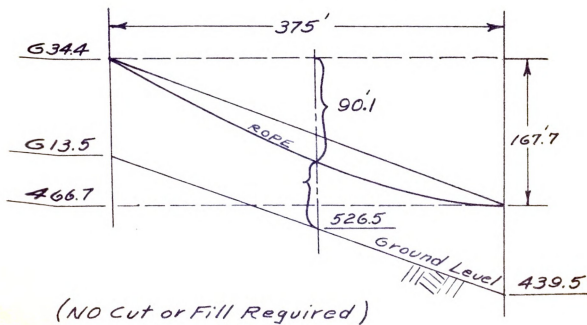
SPAN-2



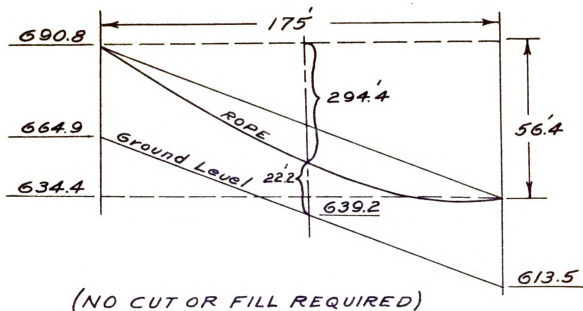
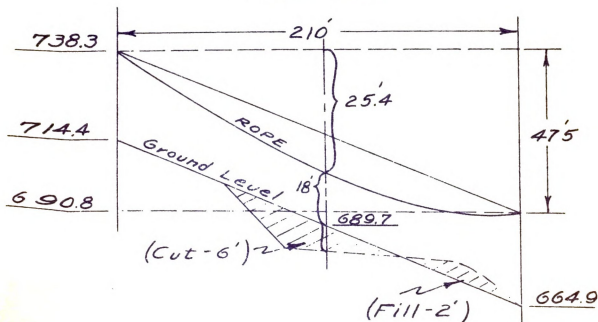


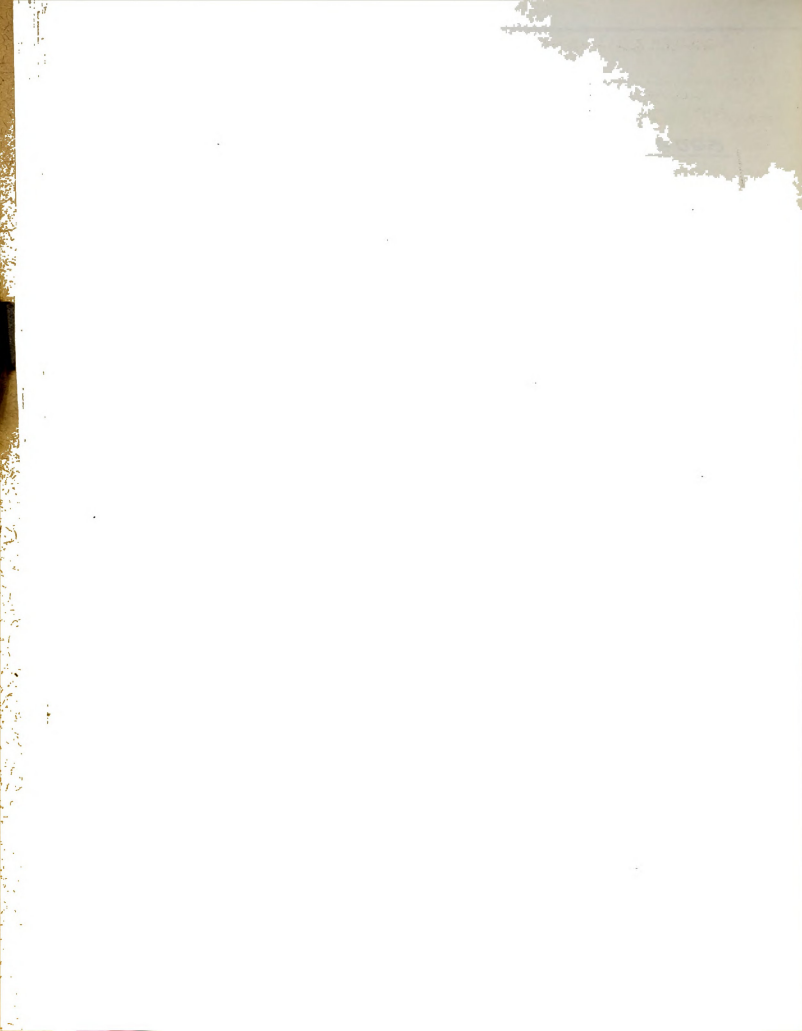
SPAN-3SPAN-4



SPAN-5SPAN-6



SPAN-7SPAN-8



SHEAVE LOADS ON THE SHEAVE UNITS AT EACH TOWER

41

	Ascending Side			Descending Side		
	Horizontal	Vertical	Resultant	Horizontal	Vertical	Resultant
1	46	186	194	132	457	476
2	67	290	298	4	12	13
3	224	723	757	125	399	418
4	337	1158	1210	253	774	814
5	176	517	546	307	869	922
6	608	1521	1630	502	1272	1370
7	321	1255	1300	259	1063	1100

Each sheave takes one-half the resultant load which in no case exceeds the following specifications:

Maximum allowable pressure on each support sheave along the line -- 700 lbs.

Maximum allowable pressure on each hold down or depress sheave along the line -- 525 lbs.

Minimum allowable pressure on any sheave -- 200 lbs.

DATA CONCERNING THE TOWERS

The total weights of the various towers can be assumed to be as follows:

1. 16 ft. tower -- 4,250 lbs. applied along a vertical line 1'6" uphill from the center line of downhill anchor belt group.
2. 21 ft. tower -- 4,500 lbs. applied along a vertical line 2'0" uphill from the center line of downhill anchor belt group.
3. 26 ft. tower -- 5,100 lbs. applied along a vertical line 2'4" uphill from the center line of downhill anchor belt.

The forces acting through the sheaves can be assumed to act through the center line of the tower and at the rope elevation.

The weight of the concrete foundations were figured using 140 lbs. per cubic foot and are as follows:

1. 16 ft. tower -- 8,660 lbs.
2. 21 ft. tower -- 10,000 lbs.
3. 26 ft. tower -- 11,300 lbs.

Resisting force of the soil against sliding is taking 1000 lbs. per square foot as recommended by Peele's Handbook of Mining Engineering.

Soil pressure figured by Formula $f_s = \frac{P}{A} \left(1 \pm \frac{G_0}{B} \right)$

DATA SHOWING SAFETY FACTOR AGAINST OVERTURN,
AND RESULTING SOIL PRESSURES

	Tower Number	Weight of Foundation in Lbs.	Weight of Tower in Lbs.	Vertical Sheave Pressure	Horizontal Sheave Pressure	Overturning Moment about (a)	Resisting Moment about (d)	Overturning Safety Fact.	Distance (e) in ft.	Force-(F) lb.	Area (A) in Sq. inches	Soil Pressure fs in p.s.f.	fs in p.s.f.
1	8660	4250	345	103	2634	39700	15.1	.72	12910	17.7	1170	292	
2	10000	4500	196	43	990	71030	72	.75	14700	20.4	1110	330	
4	11300	5100	1041	308	8750	91700	10.5	.08	17440	23.1	817	685	
5	11300	5100	785	263	15610	71400	4.6	.59	15610	23.1	1070	277	
6	10000	4500	1471	572	13400	79800	6.0	.01	15970	20.4	784	760	
7	11300	5100	1202	310	8800	92000	10.6	.09	17600	23.1	808	717	

Overturning Safety Factor never dropped lower than 4.6 for any Tower giving a wide margin of safety. The Soil Pressure shows safe pressures for any ordinary soil conditions.

1. The first part of the paper is devoted to a general discussion of the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β . It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

2. In the second part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

3. In the third part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

4. In the fourth part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

5. In the fifth part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

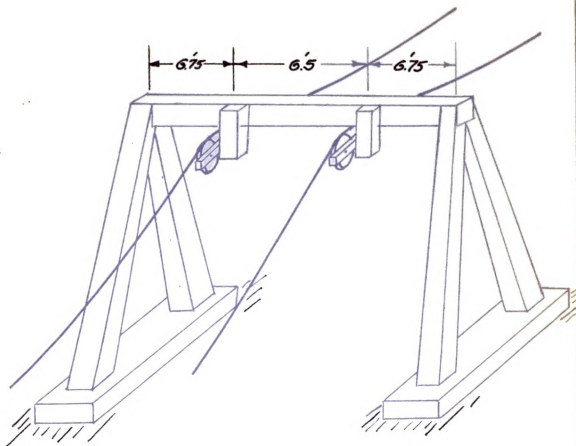
6. In the sixth part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

7. In the seventh part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

8. In the eighth part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

9. In the ninth part of the paper the problem of the existence of solutions of the system of equations (1) and (2) for arbitrary values of the parameters α and β is solved. It is shown that the system has solutions for arbitrary values of the parameters α and β if and only if the condition $\alpha + \beta = 1$ is satisfied.

Stresses Transferred To The Tower and Foundations



Forces on Right Foundation :

$$\frac{13.25}{20} \text{ or } (.663) \times (\text{Resultant of ascending side})$$

- plus -

$$\frac{6.75}{20} \text{ or } (.338) \times (\text{Resultant of descending side})$$

Forces on Left Foundation :

$$(.663) (\text{Resultant of descending side})$$

$$(.338) (\text{Resultant of ascending side})$$



FORCES TRANSFERRED TO FOUNDATIONS

	Ascending Side				Descending Side			
	Horizontal		Vertical		Horizontal		Vertical	
	.66	.34	.66	.34	.66	.34	.66	.34
1	30	16	83	43	87	45	302	155
2	44	23	192	98	3	1	9	4
3	148	76	477	246	83	42	263	136
4	222	115	764	394	167	86	537	277
5	116	60	341	176	203	104	609	313
6	402	206	1005	517	332	170	904	466
7	212	109	828	427	191	98	726	374

Foundations which will be checked are as follows:

1. Descending side of Tower 1.
2. Ascending side of Tower 2.
3. Ascending side of Tower 4.
4. Descending side of Tower 5.
5. Ascending side of Tower 6.
6. Ascending side of Tower 7.

and the other is the same as the first.

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	Ascending Side		Descending Side	
	Horizontal	Vertical	Horizontal	Vertical
1	75	238	103	345
2	43	1.96	20	107
3	190	613	159	509
4	308	1041	282	931
5	220	654	263	785
6	572	1471	538	1421
7	310	1202	300	1153

Note:

Only foundations under the sides marked "x" will be checked.

Each different type of loading will be checked. Only the worst loading conditions for each type loading need be checked. If they check all right, the others with less load may be assumed to also be correct.

Checking over the forces acting on the foundations as given in the above table, six different possible types of loading exist. Different height towers, varying length of span, and varying slope causes the differences in cable tensions and angles. Thus, different load conditions are created.

Foundations will be checked for three possible types of failure--sliding, overturning, and soil bearing failure in the direction of the lift. Since the lift is assumed to be nearly perfectly aligned from tower to tower in a

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straight line, there will be no transverse forces acting on the towers excepting wind. Although the wind forces may be considerable and include the wind forces on $1/2$ the span of cable on each side of the tower, they are not enough to cause failure of the foundation because of the wide transverse spread.

1. The first part of the paper is devoted to a general discussion of the problem.

2. The second part is devoted to a detailed analysis of the case of a single particle.

3. The third part is devoted to a detailed analysis of the case of a system of particles.

4. The fourth part is devoted to a detailed analysis of the case of a system of particles.

5. The fifth part is devoted to a detailed analysis of the case of a system of particles.

6. The sixth part is devoted to a detailed analysis of the case of a system of particles.

Checking all Foundations Against Sliding

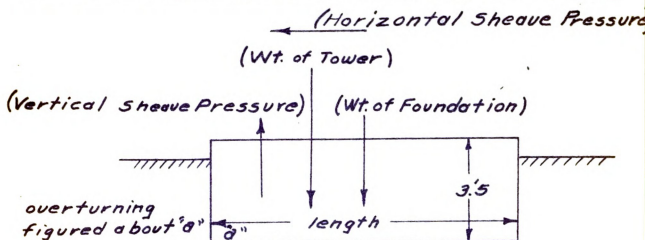
$$\text{Face Area} = (3 \times 2.46) = 7.38 \text{ sq.ft.}$$

$$\text{Resisting Force} = (7.38)(1000) = 7380 \text{ Lbs.}$$

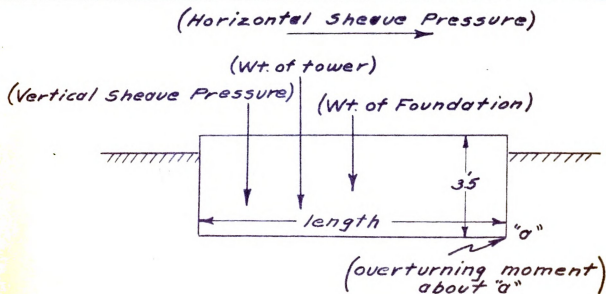
$$\text{Greatest Sliding Force on Foundation} = 572 \text{ lbs}$$

$$\text{Safety Factor} = \frac{7380}{572} = 12.9 \text{ OK}$$

Two General Cases of Foundation Loading



This case applies to foundations at towers 1 & 5

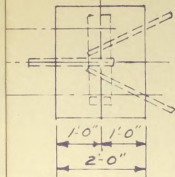


This case applies to foundations at towers 2, 3, 4, 6,



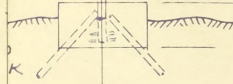
ER

x 2'-1 1/2" - 5" DIA. HOLES FOR ANCHOR
BOLTS



NT TO FIT)

TOP OF CONCRETE



REINFORCING Rod - DRILL & GROUT 1'-0" IN
ROCK (BEND TOP)

Top of Bolt

Top of CONCRETE

NOTING - ROCK TYPE

d/s	
EYE BAR	(F.B.O.)
HEX SEMI-FINISHED NUT (F.B.O.)	
ANCHOR BOLT	(F.B.O.)

3'-0" MINIMUM
FINISHED

TH

FOOTING SHALL PENETRATE



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