DESIGN OF A TIMBER BRIDGE AT DELTA MILLS, MICHIGAN

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

Eugene E. Dexter

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Design of a Timber Bridge at Delta Mills, Michigan

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by

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Introduction

This thesis covers the complete drawings and design for a timber bridge on C-533 at Delta Mills, Michigan.

At the present time there is at the site a steel through truss bridge which was built by the R. D. Wheaton and Company of Chicago, Illinois in 1891.

Present Structure

The bridge was one of the best in this section of the state when it was built, but at the present time it has a load limit which inconviences the use of the road as a class A county road which it is. Also the bridge is to narrow for the present and future volume of traffic as it carries only one lane of traffic. It is my object in this thesis to design a bridge wide enough to carry the present and future volume of traffic.

View Looking North

Several types of bridges were investigated and at the time this was designed there was a war priority on steel and therefore I decided to design one of timber. Concrete was also considered but due to the amount of steel reinforcing and the cost of concrete as compared to timber it was decided to be constructed of timber. Timber bridges at the present time are being recognized as one of the foremost types in use in the west and have been experimented with a great extent. The Forest Products Treating Company at Portland, Oregon has a method of treating the timbers as they will last as long as steel if painted and cared for properly.

Specifications which were followed are General Specifications for Timber Bridges and Trestles by Milo S. Ketchum unless otherwise noted.

I wish to take this opportunity to thank Mr. C. J. Hogue, in charge of Technical Service of the West Coast Lumberman's Association for his valuable aid and advice in the preparation of this thesis.

Computations for 20'x240' 4 span timber Bridge

Data. Live load

H-15 load (Michigan State Highway Specification)

Impact $= \frac{50}{L+125}$ used throughout design.

Dead load:

Floor- $1\frac{1}{2}$" Sheet Asphalt, granite chip rolled in @ 14 lbs/sq.ft.

Douglas Fir @ 12% moisture air dried - 15lbs/sq.ft.

Specifications:

General Specifications for Timber Bridges by Milo S. Ketchum, except as noted.

Stresses:

f_s = 16,000 lbs/sq.in.

f_c = 2,000 lbs/sq.in.

C⊥ = 325

C// = 1,406

H = 120

Et = 1,600,000

Horizontal shear 120 lbs/sq.in.

Temperature:

No account has been taken to the temperature which has very little effect on timber. Concrete as noted.

Design of laminated wood floor covered with $1\frac{1}{2}$" of sheet asphalt with granite chips rolled in.

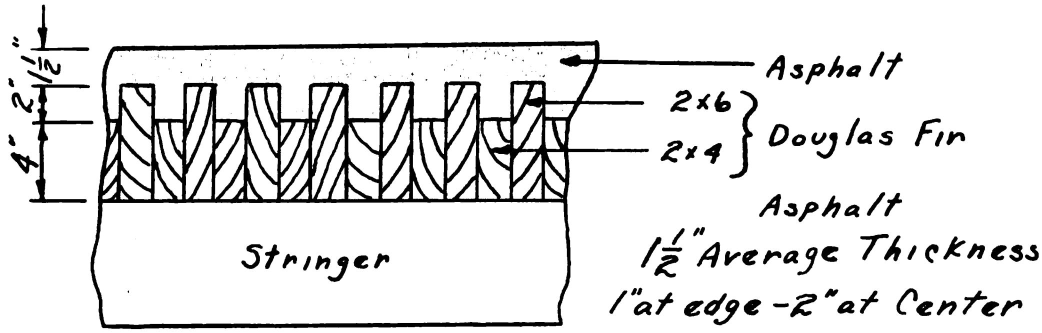


Fig. 1 Laminated Floor

Dead Load	Pound/sq.ft.
Asphalt	14
Laminated wood base	15
Total	29

Then, assume stringers spaced 2' - 6" C-C

$M = 1/14\ WL^2$

$= 1/14\ (29 \times 2.5^2)\ 12 = 155.35$ in.-lbs.

for maximum positive moment due to dead load. This moment occurs in the panel.

Next,

$M = -1/9.5\ WL^2$

$= -1/9.5\ (29 \times 2.5^2)\ 12 = -229$ in.-lbs.

for maximum negative moment due to dead load. This moment occurs at the first intermediate support from the end of the span.

Live Load: H-15

$M = 1/5\ PL$

$= 1/5 \times 12{,}000 \times 2.5 \times 12 = 500$ in.-lbs.

for maximum positive moment due to wheel loads. This moment occurs in the panel.

Next,

$M = -1/7.7\ PL$

$= -1/7.7 \times 12{,}000 \times 2.5 \times 12 = -325$ in.-lbs.

for maximum negative moment due to wheel loads. This moment occurs at the first intermediate support from the end of the span.

For the coefficient of impact

$$C = \frac{50}{L\ 125} = \frac{50}{2.5\ 125} = .392$$

Then, for the maximum positive live-load moment and impact,

L = 500 in.-lbs.

I = <u>196 in.-lbs.</u> (500 x .392)

Total = 696 in.-lbs

which occurs at the center of the end panel, and for the maximum negative moment and impact due to wheel loads.

L = -325 in.-lbs.

I = -<u>128 in.-lbs.</u> (325 x .392)

Total = -453 in-lbs.

which occurs at the first intermediate floor beam from the end of the span.

Distribution of Wheel Loads

A wheel load may be assumed distributed by the asphalt over a retangle with the sides b+2h parallel to the wheel axle and 2h at right angles to the wheel axle where b=width of wheel and h=thickness of asphalt.

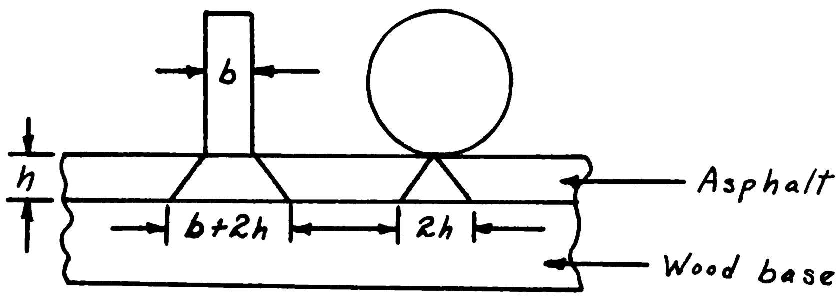


Fig. 2 Wheel Distribution

Since the wheel load is distributed as shown above the load will be carried by,

h = 1" the minimum thickness of asphalt, therefore,

worse case $Z = 2h = 2''$

this load is carried by one plank (2") but maybe assumed that it carried by two planks due the arrangement and fastening of the laminated floor. The flooring is built in the field, the strips being spiked to each other with 20d. spikes spaced about 1ft. centers and then toe-nailed into the stringer with one 20d. spike for each plank crossing a stringer.

For the moment of inertia of two planks

take h = 4+6÷2=5" average h of two planks

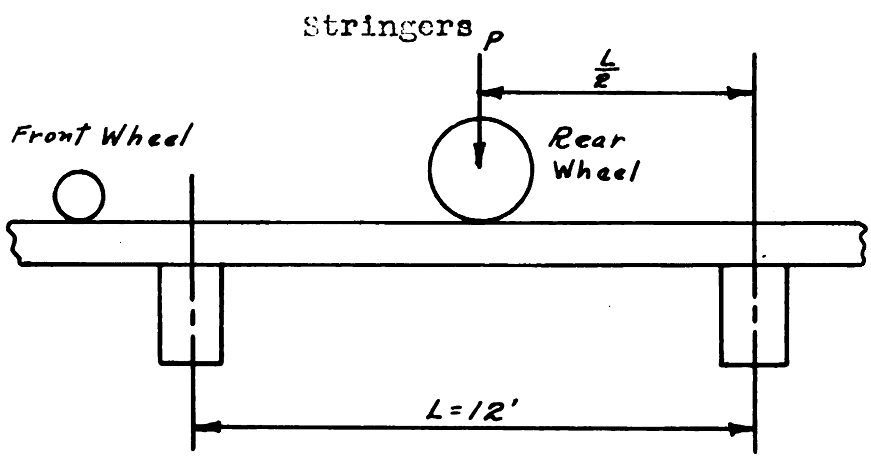
$$I = 1/12\ bh^3$$

$$= 1/12\ (4)\ (5)^3 = \frac{500}{12} = 41.7 \text{ say 42 in. units}$$

Then, for the fiber stress on two planks, when supporting the entire weight of one wheel,

$f = \frac{My}{I} = \frac{(500)(2.5)}{42} = 29.9$ say 30 lbs/sq.in.

Where M = maximum moment in inch-pounds plus impact, y = half the thickness of the plank, and I = moment of inertia of the cross-section of the planks. The allowable value of f = 2000 as given in specifications. Therefore, the wood base is safe.



(a) Rear Wheel at Center of Span

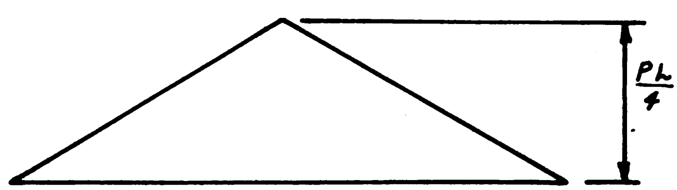


Fig. 3 (b) Moment Diagram

The maximum moment is equal to PL/4 where P denotes the effective load in pounds and L is the panel length in feet. The panel length is 12ft., so that the front wheel load will be in the adjacent panel. This live-load moment must be increased for impact and must then be added to the dead-load moment $WL^2/8$.

Assume stringer 4" x 8" -14'-6"

Dead load

Assumed wt. of stringer	6.5 lbs.per.ft.per 2'-6" wide
wt.of floor	72.5 lbs.per.ft.per 2'-6" wide
	79 lbs.per.ft.per 2'-6" wide

Moment

Live load $= \frac{1200 \times 12 \times 12}{4} =$ 43,200 in.-lbs.

Impact = 43,200 (.392) = 16,920

Dead load $= \frac{79 \times 12^2 \times 12}{8} =$ 17,064

Total 77,184 in.-lbs.

say 77,200 in.-lbs.

Maximum Vertical Shear

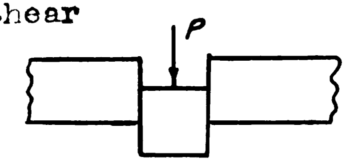


Fig. 4

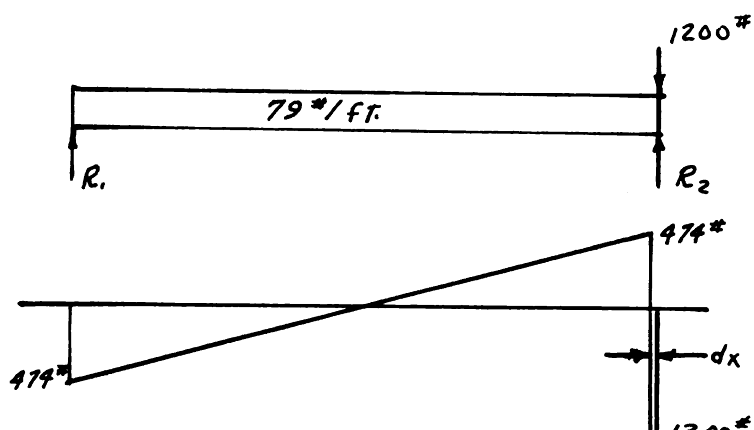


Fig. (5)

Live load			1,200 lbs.
Impact	1,200 (.392) =		470 lbs.
		Total Live	1,670 lbs.
Dead load	79 lbs./ft/		
	79 x 12 =		948 lbs.
	Total load		2,618 lbs.

$\Sigma M_{r2} = 0$

$948 \times 6 = 12R_1$

$R_1 = 474$

$R_2 = 2618 - 474 = 2144$

Maximum shear $S_s = \frac{F}{A} = \frac{2144}{(4)(8)} = 66$ lbs./sq/in.

Allowable 400 lbs./sq.in.

The Outer Stringer

The maximum loading of the stringer placed under the curb ordinarily is less than for the other stringers, but this outside usually is made equally strong because of the high impact stress to which it may be subjected of a truck should strike the curb.

Maximum Horizontal Shear

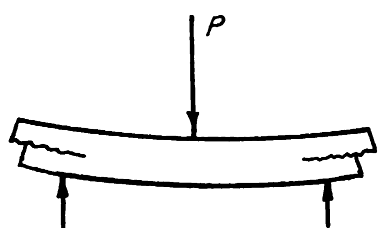


Fig. 6

The maximum horizontal shear stress in a wood beam is calculated by means of the formula,

$H = \frac{3V}{4bh} = \frac{3(2618)}{4(4)(8)} = 63$ lbs./sq.in.

OK allowable 120 lbs./sq.in.

For moment of inertia of the stringer

$I = \frac{1}{12} bh^3$

$= \frac{1}{12} (4)(8)^3 = 171$ in. units say 170 in. units

Then, for the fiber stress of the stringer

$f = \frac{My}{I} = \frac{77,200 (4)}{171} = 1,805$ lbs./sq.in.

O.K. allowable f = 2,000 lbs./sq.in.

Use 4" x 8" stringer

Floor Beam

The floor beams will be spaced 12 c-c. Hence, they must be designed for a live load equal to the full value of the rear wheels of two trucks (Fig.7). The reactions of the wheels are considered as concentrated loads applied to the top of the beam.

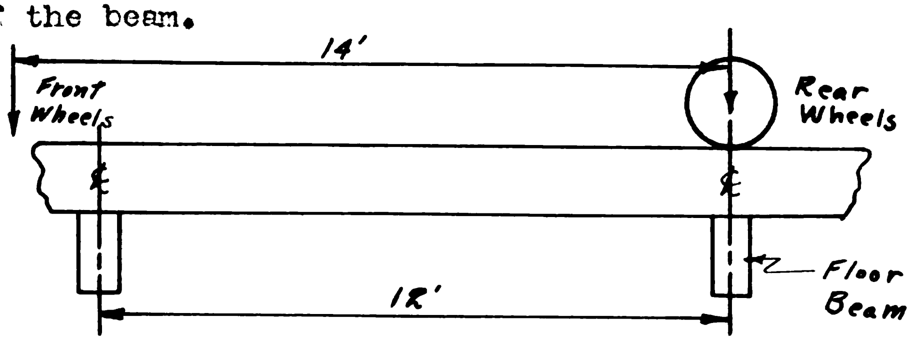


Fig. 7

The dead load is to be considered as a uniformly distributed over the floor beam. This total dead load consists of one panel of the combined beam and approximately ½ of a panel load for an exterior beam, but I prefer to use the same weight of a beam to allow for the possibility of a high impact stress due to the heaving of the roadway approach.

Maximum Moment on Beam

Span of Floor Beam = 22'-8"   Say 23'

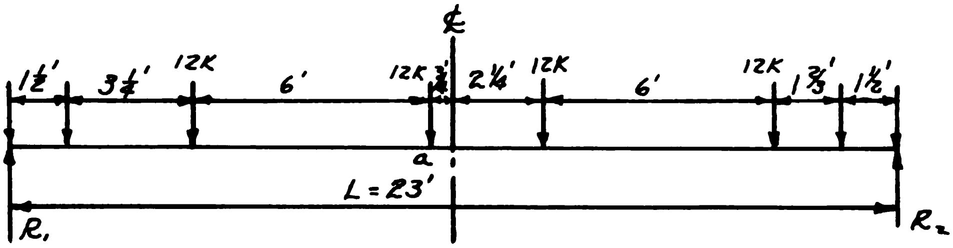


Fig. 8 Position of Loads for Maximum Moment

Dead Load

Wt. of floor in panel = 29 lbs./sq.ft.

29(20)(12) = 6,960 lbs.

Wt. of Stringer in panel = 6.5 lbs./in.ft.

6.5 (14.6)(9) = 854 lbs.

Wt. of floor Beam assume 2-7" x 24" = 74 lbs./ft.

74(23) = 1,702 lbs.

Total dead load = 9,510 lbs.

Uniformly distributed $= \frac{9510}{23} = 414$ lbs./ft.

Live load impact

12,000+12,000 (.392) = 12,470 lbs.

$\Sigma M_{r1} = 0$

4.75 (12,470)+10.75 (12,470)+13.75 (12,470)+19.75 (12,470)

+ 414 (20)(11.5)-23R_2 = 0

59,200+134,000+171,200+246,000+95,200=23R_2

$R_2 = \frac{705,600}{20}$ = 30,640 lbs.

ΣMa = 3(12,470)+9 (12,470)+(414)(10.75)$(\frac{10.75}{2})$-12.25 (30,640)

37,400+112,000+23,900-376,000 = 202,700 ft.-lbs.

Total load = 59,390 lbs. use 14" x 24" for 23 ft.span

Load for Maximum Shear

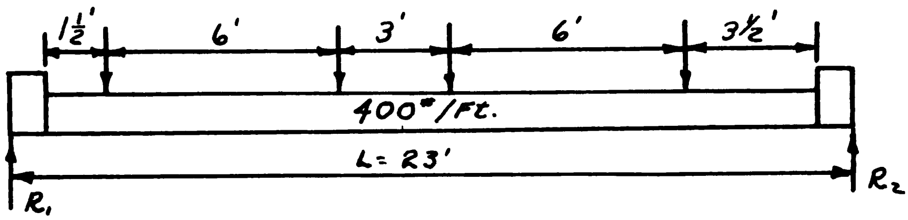


Fig.9 Position of Loads for Maximum Shear

$\Sigma M_{r1} = 0$

$= 3(12,470) + 9(12,470) + 12(12,470) + 18(12,470) - 23(R_2)$

$+ 414(23)(\frac{23}{2}) = 0$

$37,400 + 112,000 + 149,500 + 224,000 + 10,950 = 23R_2$

$R_2 = \frac{533,850}{23} = 23,400$ lbs.

$R_1 = 59,390 - 23,400 = 35,990$ lbs.

Maximum Shear

$S_s = \frac{F}{A} = \frac{35,990}{2(7)(24)} = 107$ lbs./sq. in.

Allowable 400 lbs./sq.in.

Maximum Horizontal Shear

$H = \frac{3W}{4(b)(h)} = \frac{3(57,500)}{4(14)(24)} = 127$ lbs./sq.in.

Slightly high but O.K. because this formula indicates greater stresses in a wood beam than are usually present, particularly with moving or concentrated loads near a support.

For moment of inertia of the beam

$I = 1/12\ bh^3$

$= 1/12\ (14)(24)^3 = 16,128$ inches^4

Then, for the fiber stress of the beam

$f = \frac{My}{I} = \frac{202,700\ (12)(3.5)}{16,128} = 526$ lbs./sq.in.

O.K. Allowable f = 2,000 lbs./sq.in.

Use 2-7" x 24" beam

Design of a 60-ft. Pony-Truss

The roadway is to be 20ft. wide. The truss will be 12' - 0" deep at mid-span and 8' - 0" at the hip, and there will be a 5 panels of 12' - 0" each. The live load will be H-15 loading.

Dead-Load Stresses

Wt. of floor per ft. = 750 lbs.

per foot of truss for the dead load on each truss due to the floor system.

Assume wt. of timber in one truss = 6,000 lbs.

and per foot of truss we have

$\frac{6,000}{60} = 100$ lbs.

An additional 10 lbs./ft. is added for wt. of fastenings

Then, the total wt. per foot of truss is

100+10 = 110 lbs.

then total dead wt. per foot for $\frac{1}{2}$ the bridge

110 + 750 = 860 lbs.

The total dead wt. per panel is

860 x 12 = 10,320 lbs.

Live -load Stresses

The equivalent H-15 loading shown in (Fig.10) will be used.

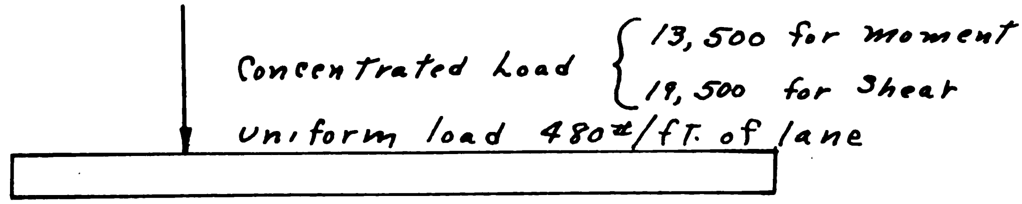


Fig. 10

The maximum load on the truss will occur when the loading is in the position shown in Fig. 11.

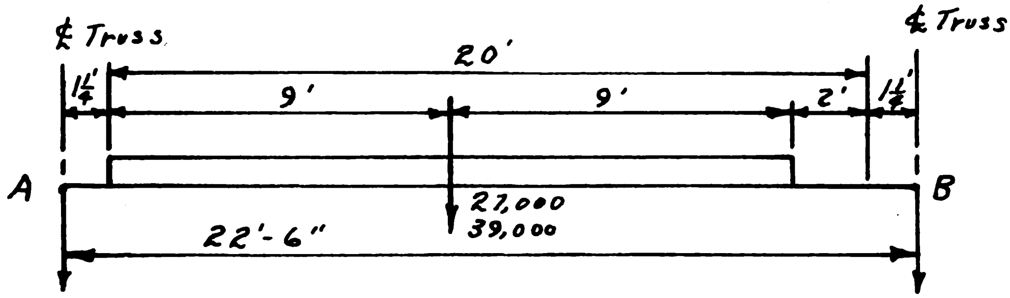


Fig. 11

Taking moments about B (Fig.11)

$$\frac{(480 \times 2)\ 12.25}{22.5} = 520 \text{ lbs.}$$

for the maximum uniform live load per foot of truss. Then, for the panel load,

P = 520 x 12 = 6,240 lbs.

Again, taking moments about B

$$P^{1} = \frac{(2 \times 13,500)\ 12.25}{22.5} = 14,698 \text{ lbs. Say } 14,700 \text{ lbs.}$$

for the panel load due to the concentration to be used in determining moment on truss. In the same manner.

$$P^{11} = \frac{(2 \times 1,950)\ 12.25}{22.5} = 21,231 \text{ lbs. Say } 21,230 \text{ lbs.}$$

for the panel load due to the concentration to be used in determining shear on the truss.

Impact is added to the live and dead load stress and the total panel load is 38,500 lbs. By means of "method of joints" the following stress table was set up.

Stress Table

$U_4L_5 = L_0U_1$	90,390 # comp.
$L_4L_5 = L_0L_1$	75,960 # ten.
$L_4U_4 = U_1L_1$	38,580 # ten.
$L_3L_4 = L_1L_2$	75,960 # ten.
$L_3U_4 = U_1L_2$	19,000 # comp.
	10,910 # ten.
$U_3U_4 = U_1U_2$	77,270 # comp.
$U_3L_3 = U_2L_2$	37,[illegible]40 # ten.
U_2U_3	73,7[illegible]0 # comp.
L_2L_3	64,710 # ten.
$U_2L_3 = L_2U_3$	2,140 # comp.
	14,270 # ten.

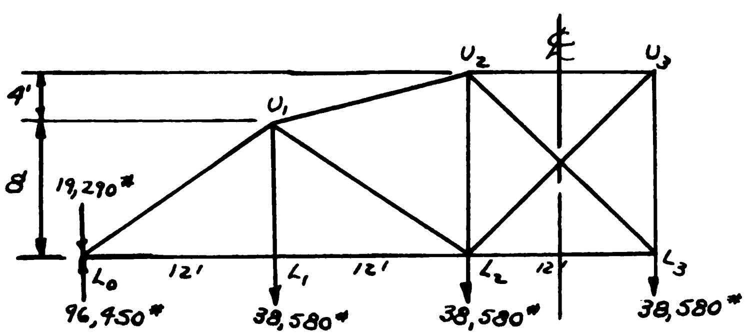


Fig. 12

The truss is small and a short span therefore no wind-load stresses were taken into account.

Camber

The camber is calculated on the basis of 3 inches rise of bottom chord at center-line the camber curve to be parabolic.

Sections and Details

The end post has the largest force acting upon it, therefore design for $L_o U_1$ (Fig.12) and use this design for whole of top chord.

$$K = .641\sqrt{\frac{E}{C}} \qquad C = 1,406$$

$$= .641\sqrt{\frac{1.6 \times 10^6}{1.406}} \qquad E = 1.6 \times 10^6$$

$$= 21.2$$

Assume the top chord to be built up of 3 timers so the resistance to bending is increased which is necessary due to the fact the top chord is in compression. It is also built up for construction purposes. Assume it to be made of a main beam 6" x 6" with a $3\frac{1}{2}$" x 14" timber fastened to the two sides of the main beam (Fig.13).

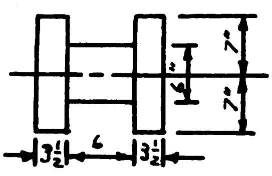


Fig. 13

Ratio l/d where l= Span in inches and d= less dimension in width in inches.

$$d = \frac{6 + 14}{2} = 10''$$

$$1/d = \frac{174}{10} = 17.4$$

For built up columns with a span between eleven times the least dimension and K times the least dimension are classed as intermediate columns. They depend for strength on a combination of crushing strength and resistance to lateral buckling.

Use the following condition in design for the column. When connectors in middle timber are placed at a distance of L/20 from the ends of this timber.

$$K_3 = K \times 1{,}732$$
$$= 21.2 \times 1{,}732$$
$$= 36.8$$

P/A = maximum load per unit of cross-sectional area.

$$P/A = C\left[1-1/3\left(\frac{L}{K_3 d}\right)^4\right]$$

$$P/A = 1{,}466\left[1-1/3\left(\frac{174}{36.8 \times 10}\right)^4\right]$$

$$P/A = 1{,}466\ (.983)$$

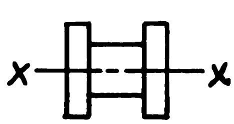
P/A = 1,440 lbs./sq.in. allowable

Cross-sectional Area of end post

$$3.5 \times 14 \times 2 = 98$$
$$6 \times 6 = \underline{36}$$
$$134 \text{ sq.in.}$$

$$P/A = \frac{90{,}390}{134} = 674 \text{ lbs./sq.in.}$$ O.K. allowable 1,440 lbs./sq.in.

The maximum load which the column can carry without buckling.



$$I_1 = \frac{6 \times (6)^3}{12} = 108 \text{ in.}^4$$

$$I_2 = \frac{3.5 \times (14)^3}{12} = 801 \text{ in.}^4$$

$I = 108 + 801 = 909 \text{ in.}^4$

$$Pcr = \frac{\pi^2 EI}{4 L^2} = \frac{9.86 \times 1.6 \times 10^6 \times 909}{4(14.5)2 \times 144}$$

Pcr = 121,000 lbs. O.K. since the load is 90,390 lbs.

Deflection

$$\delta = \frac{Pl^2}{2EI}$$

$$= \frac{90,390 \times (14.5)^2 \times 144}{2(1.6 \times 10^6)(909)}$$

= .939 inch

Diagonals

The diagonals shall be rectangular solid columns all of the same dimensions, therefore design for the largest stresses in the members which would be U_1L_2 (Fig.12) for compression and U_1L_2 (Fig. 12) for tension. The size assumed will be 6" wide for construction purposes and 10" deep for strength.

$$l/d = \frac{198}{6} = 33$$

K = same = 21.2

Long: (l/d radios equal to or greater than "K")

$$P/A = \frac{.274 E}{(l/d)^2} = \frac{.274 \times 1.6 \times 10^6}{1089}$$

P/A = 403 lbs./sq.in. allowable

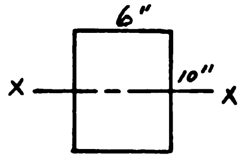
Cross-sectional Area

6 x 10 = 60 sq. in.

P/A 19,000 = 317 lbs./sq.in.

O.K. allowable 403 lbs./sq.in.

The maximum load which the column can carry without buckling.



$$I = \frac{6 \times (10)^3}{12} = 500 \text{ in.}^4$$

$$P_{cr} = \frac{\pi^2 EI}{4l^2} = \frac{9.86 \times 1.6 \times 10^6 \times 500}{4(16.5)^2 (144)}$$

50,000 lbs. O.K. since the load is 19,000 lbs.

Deflection

$$\delta = \frac{Pl^2}{2EI}$$

$$= \frac{19,000 \times (16.5)^2 \times 144}{2 \times 1.6\ 10^6 \times 500}$$

$= .47$ inch

For direct tension the same values as for extreme fiber stress in bending is used.

Cross-sectional area

$6 \times 10 = 60$ sq. inches

$$P/A = \frac{14,270}{60} = 238 \text{ lbs./sq.in.}$$

O.K. allowable 2,000 lbs./sq.in.

Verticals

The verticals shall be a built up column for construction purposes. The design will be for the vertical U_1L_1 which has the largest load of any vertical. The cross-section assumed will be a 4" x 12½" main timber with a 4" x 14" timber fastened to the two 4" sides of the main column. (Fig. 14)

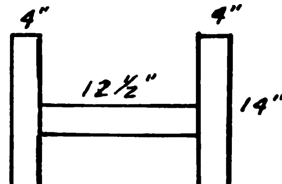


Fig. 14

Since the verticals are always in direct tension the same values as for extreme fiber stress in bending will be used.

Cross-sectional area is

2 x 4 x 14 = 112

4 x 12½ = 50

Total = 162 sq. inches

$P/A = \frac{38,580}{162} = 238$ lbs./sq.in. O.K. allowable 2,000 lbs./sq.in.

Lower Chord

The lower chord shall be assumed as made of 2 - 4" x 10" beams 6" apart to allow for construction at joints. Since the lower chord is always in direct tension it will be designed as the verticals were. The design will be for L_0L_1 which has the largest stress.

Cross sectional area is

2 x 4 x 10 = 80 sq. inches

$P/A = \frac{75,960}{80} = 950$ lbs./sq.in.

O.K. since allowable is 2,000 lbs./sq.in.

A tongue made of 1 - 6" x 12" x 5' timber was used at joint L_0 for construction. Also a 1 - 6" x 10" x 4' timber was used as a splice block in the bottom chord. Four foot was to allow for connectors.

Due to the truss being low and of short span no wind stress will be calculated but a knee brace will be place at joints L_1 L_2 and L_3 running from the joint to the floor beam at a 45° angle.

Design of split rings in joints and bolts.

Bolt hole shall be of a diameter permitting bolts to be driven easily. Minimum spacing is four times the bolt diameter. Spacing between rows should be at least five times the bolt diameter. The end margin should be five times the bolt diameter in tension and four times in compression. The edge margin in tension or compression should be, at least one and one half times the bolt diameter. Margin nearest the edge toward which the load is acting is to be at least four times the bolt diameter.

Joint L2

Chords to diagonals

Angle of load to grain of diagonal is 35°. Allowable load in pounds per connector and bolt at angle of 35° 5512

S. R. load is 37,340 lbs.

8 x 5,512 44,096 value of 8 - 4" SR's.

Results: use 4 - 4" S.R.'s with 3/4" x 14½" bolts with 2 on each bolt. All bolts are spaced according to specifications stated.

Vertical to chords

Angle of load to grain of chords is 90°

S.R. load is 18,670

4 x 4,675 18,700 value of four 4" S.R.'s

Results: use 4 - 4" S.R. with 3/4" x 28" bolt. The 3/4" x 28" bolt carries 5 - 4" S.R.'s; 2 between vertical and chord, 2 between diagonal and chord, and one between the kneebrace and vertical.

The rest of the joints were designed by the same method with the following results:

Joint L_0

End post to bottom chord tongue

12 - 4" S.R.'s with 3/4" x 14½" bolts

2 - 2½" S.R.'s with 5/8" x 14½" bolts

Bottom chord tongue to bottom chord

12 - 4" S.R.'s with 3/4" x 14½" bolts

2 - 2½" S.R.'s with 5/8" x 14½" bolts

Joint L_1

Bottom chord to vertical

4 - 4" S.R.'s with 3/4" x 27" bolts

2 - 4" S.R. on same bolt between kneebrace and vertical.

Joint U_1

Between chord and vertical

8 - 4" S.R.'s with 3/4" x 22" bolts

Between chord and diagonal

4 - 4" S.R.'s on 3/4" x 22" bolts above

2 - 2½" S.R.'s with 5/8" x 22" bolts

Joint U_2

Between chord and vertical

8 - 4" S.R.'s with 3/4" x 22" bolts

Between chord and diagonal

4 - 4" S.R.'s on 3/4" x 22" bolts above

2 - 2½" S.R.'s with 5/8" x 22" bolts

Center line bottom chord splice

24 - 4" S.R.'s with 3/4" x 14½" bolts

on the top chord and the verticals that are made up of three timbers, connectors will be used as a means of fastening. In all verticals and chords five connectors is sufficient, except the vertical between joint L_1 U_1 and L_4U_4 in which three is sufficient.

The floor beams will also be fastened by means of connectors. Using the method above it was determined that 10-5/8" x 29" bolts with 2 - 2½" S.R.'s to each bolt. Due to the fact that the floor beam fastens to the vertical 2½" rings had to be used. The standard washer made by the Timber Engineering Company is used under all bolts throughout the whole structure.

At joints there are end bearings of wood on wood. When wood members are squared and butted end to end, the end tends to bed themselves into each other and the maximum strength will be less than the compressive strength of clear wood. The amount of embedment will vary with the percentage of summer wood and for practical purpose it is not safe to count on more than 75% of the compressive stress for clear wood. Where such end are butted use a piece of 16 gauge galvanized sheet iron. These bearing plates between top chord segments to be placed in the field in a sowout thru the joint made with a finishing hand saw.

Abutments

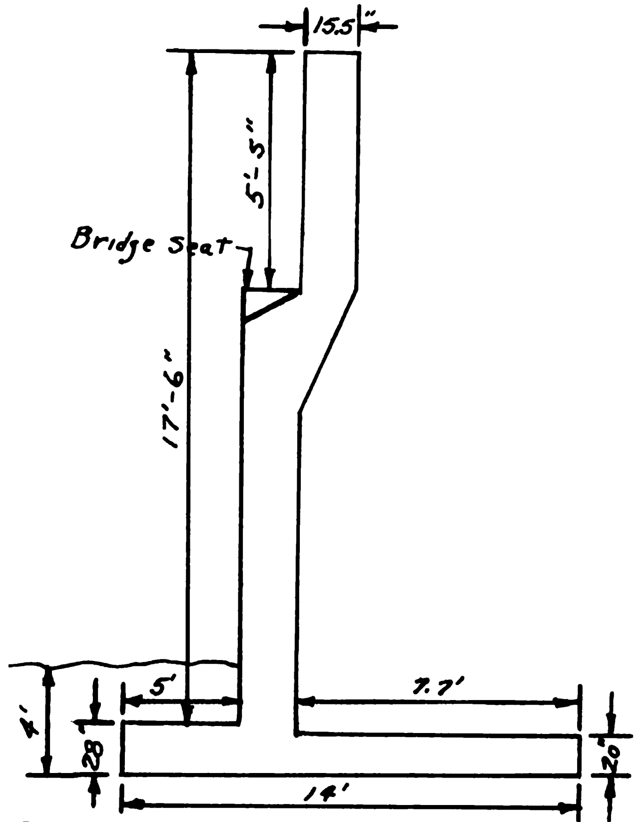
Both abutments will be of the same design due to the fact they are the same height and same loading.

The load on the abutment due to dead and live load and impact per truss is 19?,900#. This load will be considered to be distributed for 10 ft. along the wall. $\frac{192,900}{10} = 19,290$ #/ft.

Design of cantilever wall

assume base = .7 h

toe = .36 b



b = .7 x 20 = 14"

toe dist. = .36 x 14 = 5'

The surcharge due to the live load on the fill will be taken as 6"

Earth Pressure

$$Ce = \frac{wh^2}{2}$$

$$= .27 \frac{100\ (20)^2}{2} = 5,400 \text{ lbs./lin.ft.}$$

act 1/3 up from bottom or 6.5 ft.

wt. of earth

(7.7 x 18) 100 = 13,860 lbs.

wt. of base alab

(2.66 x 5)150 + (1.66 x 7.7) 150 = 3,910 lbs.

wt. of stem

(1.3 x 19.33) 150 = 3,764 lbs.

Moment of forces of base and earth

ΣM toe = o

$(13,?60 + ?,764 + ?,910 + 19,?90)\bar{x} = 13,?60\ (10.15) + ?,910$

$(7) + 19,290\ (5.64) + ?,764\ (5.64) \quad 40,?24\ \bar{x} = 298,176$

$\bar{x} = 7.30$ ft. from toe to 40,?00 lbs. resultant.

Resultant of Earth pressure and weight hit base at

$$\frac{\bar{Y}}{7.30} = \frac{5,400}{40,?24}$$

$\bar{Y} = .97$ ft.

$7.30 - .97 = 6.33$ ft.

eccentricity = $7 - 6.33 = .67$ ft.

Soil Pressure

$$SP = \frac{P}{b}\left(1 \pm \frac{6e}{b}\right)$$

$$\frac{40,?24}{12}\left(1 \pm \frac{6 \times .67}{12}\right) = \begin{matrix} 4,520 \text{ lbs./sq.ft.} \\ 2,2?0 \text{ lbs./sq.ft.} \end{matrix}$$

Sliding $\quad f = .4$

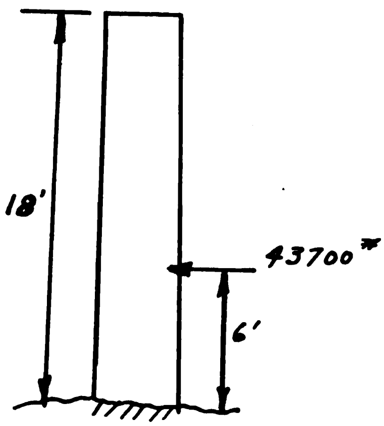
factor of safety = $\frac{40,?24 \times .4}{6,400} = 2.55$ O.K.

Stem

Earth Pressure

$$= .27\ \frac{100 \times (18)^2}{2} = 4,370 \text{ lbs.}$$

act 1/3 up or 6 ft.



Bending Moment

$4,370 \times 6 = 26,2?0$ ft. lbs.

use 3,0?0 lbs concrete

fc = 1,000

fs = 20,000

n = 12

K = 164

U = 100 - 125 lbs./sq.in.

p = .0094

V = 50 - 60 lbs./sq.in.

$$d = \sqrt{\frac{M}{bk}} = \sqrt{\frac{26,200 \times 12}{12 \times 164}}$$

d = 12.6" use 2.9" of protection over bars

therefore D = 15.5" Say 15.5"

As = p bd = .0094 x 12 x 12.5

= 1.51 sq.in.

Use 7/8 in round rod@ 4½ inches

As = 1.60 sq.in.

$$\text{Bond} = \frac{V}{\Sigma o \; jd} = \frac{4,370}{(2.36 \times \frac{12}{3.5})7/8 \times 12.5}$$

= 49.5 lbs./in. O.K.

Unit Shear

$$v = \frac{V}{bjd} = \frac{4,370}{12 \times 7/8 \times 12.5} = 33.2 \text{ lbs./sq.in. O.K.}$$

Temperature changes

Temperature steel to keep the wall from cracking on surface to care for stresses due to temperature changes.

Steel ratio of temperature .003 or .3% of area of concrete. Place 2.3 on front face and 1/3 on back.

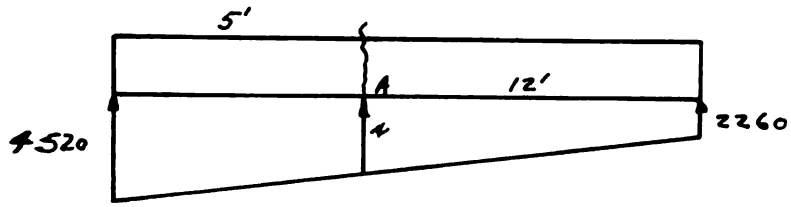
12 x 12.5 = 150

150 x .003 = .45 sq.in. of temperature steel/ft.

front = 1/2 square rod @ 6 inches

back = 1/2 square rod @ 12 inches

Toe Design



$$x = 4,520 - \left(\frac{4,520 - 2,260}{12}\right) 5 = 3,570 \text{ lbs.}$$

$$\Sigma MA = \frac{3,570 \times 5^2}{2} + \frac{950}{2}(5)\left(\frac{5}{3}\right) - 2.5\,(5)(150)2.66$$

Max B.M. = 44,660 ft. lbs.

Necessary to satisfy shear the following

d = 25"

D = 28"

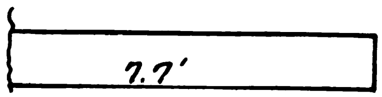
As = .0094 x 25 x 12 = 2.82 sq.inches

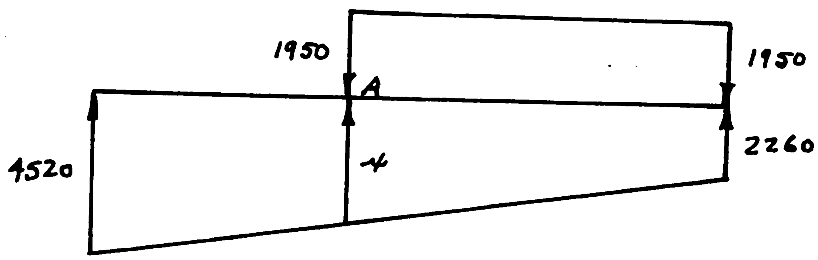
Use 7/8" round bars @ 2.5 inches 2.89 sq.inches

Bond

$$\mu = \frac{44,660}{(2.75)\frac{12}{2.5}(7/8)25} = 120 \quad \text{O.K.}$$

Heel Design





18 x 100 + 1 x 150 = 1,950 lbs.

$$X = 2{,}260 + \left(\frac{4{,}520 - 2.260}{12} \right) 7.7 = 3{,}723$$

$$\Sigma M_A = 2{,}260 \times 7.7 \times \frac{7.7}{2} + \frac{1{,}463}{2} \times 7.7 \times \frac{7.7}{3} - 1{,}950 \times 7.7 \times \frac{7.7}{2} = 23{,}800 \text{ ft. lbs.}$$

d = 17" necessary for shear

D = 17 + 3 = 20 inches

As = .0094 x 12 x 17 = 1.92

Use 7/8" @ 3½" = 2.06 sq.in.

V = 3,0 0 (7.7) - 1,950 (7.7)

V = 8,100

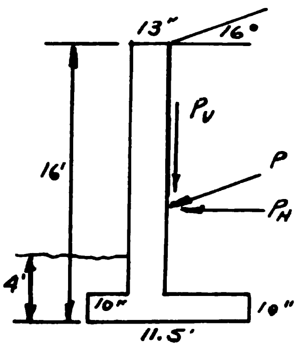
Unit Shear

$$v = \frac{8{,}100}{12 \times 7/8 \times 17} = 50 \text{ OK}$$

Bond

$$\mu = \frac{8{,}100}{2.75 \left(\frac{12}{3\frac{1}{2}}\right) \times 7/8 \times 17} = 58 \text{ OK}$$

Wing Walls



Surcharge angle 16° 40'

Ce = .31

$$p = .31 \frac{100 \times 16^2}{2} = 3{,}968\#$$

Horizontal force of P = 3800 #

Act 1/3 h = 5 1/3 ft.up

base = .7 h

7. x 16 = 11.2' Say 11.5 ft.

$$\text{Toe distance} = \frac{b}{3} = \frac{11.5}{3} = 3.8 \text{ ft.}$$

Pv = 995#

wt. of earth including stem

$(7.7 \times 14.67)\ 100 = 11,300$ lbs.

wt. of base slab

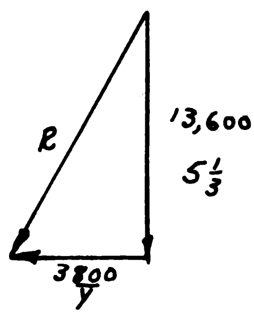
$(1.33 \times 11.5)\ 150 = 2,300$ lbs.

$\Sigma M_{toe} = 0$

$(11,300 + 2,300)\bar{x} = (11,300)\ 7.65 + 2,300\ (5.75)$

$\bar{x} = 7.34$ ft. from tow to 13,600 lbs.

resultant



$$\frac{\bar{Y}}{51/3} = \frac{3,800}{13,600}$$

$\bar{Y} = 1.49$ pt. where R hits base

$7.34 - 1.49 = 5.85'$

Essentricity $= 5.75 - 5.85 = .1$ ft.

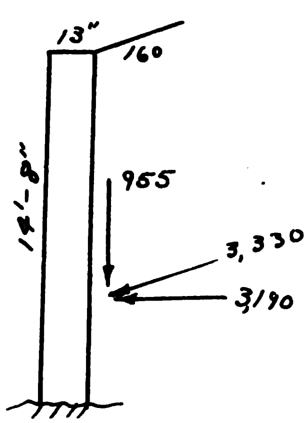
Soil pressure

$$P = \frac{13,600}{11.5}\left[1 \pm \left(\frac{6 \times .1}{11.5}\right)\right] = \begin{matrix} 1,190 \text{ lbs./sq.ft.} \\ 1,120 \quad '' \quad '' \quad '' \end{matrix}$$

Sliding $\quad f = .4$

factor of safety $= \dfrac{13,600 \times .4}{3,800} = 1.43$

Stem



$$E.P. = .31 \quad \frac{100(142/3)^2}{2}$$

$= 3,330$

$Ph = 3,330 \times .958 = 3,190$ lbs.

acts 4.89 ft. up

BM = 3,190 (4.89) = 15,600

$$d = \sqrt{\frac{15,600}{164}} = \sqrt{95} = 9.75 \text{ Say } 10''$$

D = 13"

As = pbd

= .0094 x 12 x 10 = 1.13 sq.in.

use 3/4" ∅ @ 4½" = 1.18 sq.in.

$$\text{Bond} = \frac{3,800}{2.36 \times \frac{12}{4.5} \times 7/8 \times 10}$$

= 69 O.K.

$$\text{Unit Shear} = \frac{3,800}{12 \times 7/8 \times 10}$$

= 36.5 O.K.

Temperature Steel

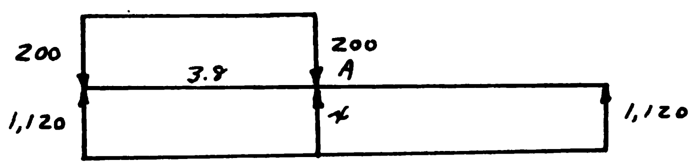
10 x 12 = 120

120 x .003 = .36

front ½" @ 6½"

back ½" @ 13"

Toe design



$$x = 1190 - \frac{(1190 - 1120)\ 3.8}{11.5}$$

= 1167

$$\Sigma Ma = \frac{1167 \times 3.8}{2} + 23 \times \frac{3.8}{2} \times \frac{2(3.8)}{2} - \frac{200(3.8)^2}{2}$$

B.M. = 7,110

$$d = \sqrt{\frac{7,110}{164}} = 6.6'' \qquad D = 10''$$

Say 7"

As = .0094 (7)(12) = .79 sq.in.

use ½" @ 3" = .79

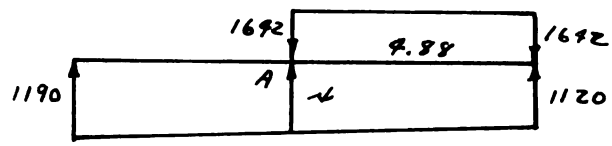
Shear = 1173 x 3.8 - 3.8(200) = 3,710

Unit Shear

$$= \frac{3710}{12x7/8x7} = 51 \text{ O.K.}$$

$$\text{Bond} = \frac{3,710}{1.57x\frac{12}{3}x7/8x7} = 10\_ \text{ O.K.}$$

Heel design



$$x = 1120 + \frac{(1190 - 1120)}{11.5} \times 4.88 = 1150$$

15.17 x 100 + .833 x 150 = 1642

Σ Ma = 0

$$- 1642 (4.88)(2.44) + 1120 (4.88)(2.44) + \frac{30}{2} (4.88)(2.44)$$

= 6,030 tension in top of footing

$$d = \sqrt{\frac{6030}{164}} = 6.05 \text{ Say } 7"$$

D = 10"

As = .0094 x 7 x 12 = .79 sq.in.

Use ½ @ 3" = .79 sq.in.

Shear

= 1135 x 4.88 - 1642 x 4.88 = 2,460

Unit shear

$$v = \frac{2460}{12x7/8x7} = 35 \text{ O.K.}$$

Bond

$$\mu = \frac{2460}{1.57 \times \frac{12}{3} \times 7/8 \times 7} = 6\ .5 \text{ O.K.}$$

Piers

Design the piers to be 21' - 6" high. Assume that the allowable pressure on the footings can be 9,0 0 lbs./sq.ft. The type shown below will be used.

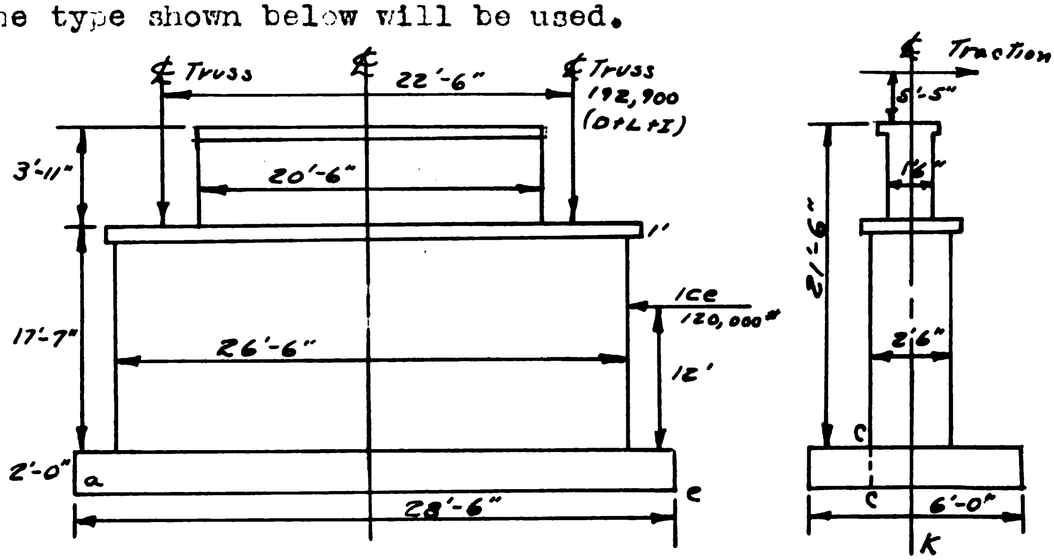


Fig. 15

The cap is assumed to be 3' - 0" wide and 1' - 0" thick. Consider the main shaft to be 2' - 6" thick. The base will be assumed to be 6' - 0" wide, 2' - 0" thick and 28' - 6" long. The portion that supports the stringers will be assumed to be 20' - 6" long and 1' - 6" wide with the a cap 20' - 6" long, 2' - 0" wide and 1' - 0" thick.

Total wt. of pier

Base

28.5 x 6 x 2 x 150 = 51,300 lbs.

Shaft

26.5 x 2.5 x 17.6 x 150 = 175,000 lbs.

Cap

3 x 1 x 27 x 150 = 12,150 lbs.

Upper shaft

20.5 x 1.5 x 2.92 x 150 = 13,450 lbs.

Cap

20.5 x 2 x 1 x 150 = 6,150 lbs.

Total = 258,050 lbs.

For ice pressure (assuming the ice to be 20 inches thick)

200 x 30 x 20 = 120,000 lbs.

For maximum direct load on the bottom of the base

258,050 + 192,900 x 2 = 643,850 lbs.

Dividing this by the area of the base

$$\frac{643,850}{28.5 \times 6} = 3,760 \text{ lbs.}$$

for uniform pressure over the base. Next, take moments about point e on base of ice pressure.

120,000 x 12 = 1,440,000 ft.lbs.

for the moment due to this load tending to overturn the pier.

The moment of inertia of the bottom surface of the base.

$1/12 \times 6 \times (28.6)^3 = 11,700$ ft. units

Then,

$$f = \frac{1,440,000 \times 14.25}{11,700} = 1,755 \text{ lbs.}$$

for the positive pressure per square foot at e and the negative pressure per square foot at a on the base due to ice pressure. Adding this to the uniform load.

e = 3760 + 1755 = 5515

a = 3760 − 1755 = 2005

Next, consider traction (Fig. 15). The traction force will be

$$2 \times 40,000 \times 0.1 = 8,000 \text{ lbs.}$$

Taking moments about the bottom of the base

$$8,000 \times 26.92 = 215,360 \text{ ft./lbs.}$$

for the maximum moment due to traction.

For moments of interia of the bottom surface of the footing about axis K

$$= 1/12 \times 28.5 \times 6^3 = 513 \text{ ft. units}$$

Then

$$f = \frac{215,360 \times 3}{513} = 1260 \text{ lbs./sq.ft.}$$

for the maximum pressure on the footing due to traction. Adding this to the 5515 lbs. obtained due to dead wt. and ice pressure

$$5515 + 1260 = 6,775 \text{ lbs.}$$

for the total maximum pressure per square foot on the footings. O.K. since allowable assumed was 9,000 lbs.

For the moment on the footing along section CC (Fig.15)

$$\left[(6775 \times 7/4)\ 7/8 - (300 \times 7/4)\tfrac{7}{8}\right]12 = 119,0\ \ \text{in.-lbs.}$$

Taking $d = 21"$, $j = 0.87$

$$F = \frac{119,000}{21 \times .87} = 6,520 \text{ lbs.}$$

Then, for the steel required in the bottom of the base

$$6520 \div 16,000 = .407$$

Use 5/8" ϕ bar @ 9" = .41

End Bearings

The expansion of each span will be approximately .018 of a foot which will be taken by the timber span itself.

therefore, both ends of each span will be securely fastened by means of a hinge arrangement.

Vertical load on each plate = 96,450 lbs. assume $2\frac{1}{2}$" pin

Moment on each pin

$1\frac{1}{4}$ x 48,225 = 60,280 in.-lbs. O.K. allowable 65,000 in./lbs.

Shear on pin

4.91 x 44,000 = 216,00 lbs. allowable; actual 96,450 O.K.

Bearing area on masonary

96,450 ÷ 600 = 160 sq. inches

Use 10" x 16" = 160 sq. inches

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