An Analysis of the Design

of Supply House for Burro Mountain Copper Co.

A Thesis Submitted to

The Faculty of

MICHIGAN AGRICULTURAL COLLEGE

By

V. R. Cooledge

Candidate for the Degree of

Bachelor of Science

June, 1917.

THESIS

cop 2

The Author takes great pleasure in thanking C. A. Melick, Associate Professor of Civil Engineering at the Michigan Agricultural College for his valuable suggestions and encouragement during the progress of this work.

The Author also wishes to thank Mr. C. U. Cooledge of the Burro Mountain Copper Company who very kindly furnished the blueprints and data regarding the design of the Supply House.

Introduction

The building of which this is the Analysis is the Supply House of the Burro Mountain Copper Company, located near Tyrone, New Mexico.

It is used as a general warehouse. It is 60' by 125', has a concrete basement, steel columns and 18" I beam girders supporting 12" I floor beams on which is a 3" plank floor designed to carry 400 pounds per square foot. The walls are tile and the roof is supported by wooden trusses. There is an overhead 4 ton trolley and a 1½ ton elevator.

The method used in the design is as follows:

1. Estimate Weight and Dead Load Panel Concentrations for Combination Roof Truss.
2. Draw Stress Diagrams and Tabulate Stresses in Roof Truss for Dead Load - Max. Snow Load - Min. Snow Load, and Wind Load.
3. Analyze Roof Purlins and Rafters.
4. Tabulate Max. Stresses and Analyze Truss Members.
5. Analyze Joint details for Each Typical Joint.
6. Analyze Wall Footings for Max. Wall Height and TrussLoading.
7. Analyze Reinforced Concrete Lintel - 10 ft. Span.
8. Analyze 3 in. Flooring.
9. Analyze Steel Floor Beams B. L - B 14 in.
10. Analyze Pipe Columns.

11. Analyse Built-Plate and Angle Columns.

12. Analyse All Members of Elevator Framing and Supporting Beam C 1 and C 2.

13. Analyse Column Footings.

14. Analyse Reinforced Concrete Stairs.

15. Analyse Walls and Buttresses for Oil Sump.

16. Analyse I Beam Trolley.

17. Analyse Cross Girders of Hand Trolley Support.

18. Analyse Beams for Hand Trolley Support.

19. Analyse Columns D 1 - D 4 in. Hand Trolley Support.

20. Analyse Bracing and Tie Rods, Hand Trolley Support.

21. Summary.

I N D E X .

The different articles are taken up in the order just named as follows:

Index to Pocket Drawings.

Index of References.

A. H. B. - American Civil Engineer's Hand Book.

S. H. B. - Ketchum's Structural Engineers Hand Book.

A. R. E. A. - American Railway Engineering and Maintenance of Way Association.

Index of Symbols used.

M	Bending moment.
s	Fibre stress in pounds per sq. in.
I	Moment of Inertia.
c	Distance to extreme fibre.
S_1	Safe working stress for the column.
S_2	Safe end bearing stress, compression with the grain.
l	Length of column.
d	Least side of column or net depth of beam.
p	Steel ratio.
k	Ratio of depth of neutral axis to depth d.
j	Ratio of lever arm of resisting couple to depth d.
S_s	Stress in pounds per sq. in. in the steel.
S_c	Stress in pounds per sq. in. in the concrete.
b	Breadth of beam.
U	Bond stress per unit length of beam.
V	Total vertical shear in any section.
M_R	Resisting Moment.
P	Concentrated load.
l	Length of beam.
A	Area.
r	Least radius of gyration.
ϕ	A constant taken from A.H.B.
I_{yy}	Moment of inertia around axis yy
I_{xx}	Moment of inertia around axis xx
d_1	Inside diameter of pipe column.
d_2	Outside diameter of pipe column.

w	Weight of earth per cu. ft.
ϕ_1	The angle of repose.
P	Horizontal pressure of the earth on the wall.
W	Vertical pressure on the soil.
b	Breadth of footing.

Article 1.

Estimate of Weight and Dead Load Panel Concentrations for Combination Roof Truss. See Fig. 1a - Exhibit A.

Span 60'-0" Rise 15'-0".

Spacing of Trusses = 12' - 6" C. to C.

A - Estimate of weight of Truss.

-1- Wooden members. Wt. per cu. ft. Oregon pine (Douglas Fir) = 32.14# (P368 A.H.B.)

Size	Length	Wt. / ft.	Weight
2 x 10	204 ft	4.46	910
2 x 8	108	3.58	386
4 x 6	22	5.36	118
6 x 8	36	10.72	385
6 x 6	92	8.04	740
4 x 8	36	7.16	257
		Total =	2796.

-2- Estimate of weight of bolts. (Table 103 S.H.B.)

No	Size	Wt./ 100	Weight
54	1"x7½"	252#	136#
28	¾ x 7½	131	37
2	1 x 21	552	11
2	1-3/8x33	1400	28
2	1-3/8x28	1200	24
		Total =	236#

-3- Estimate of wt. of hanger bolts. (T6 and T103 S.H.B.)

No	Diam.	Length	Wt./ft.ea.	Wt.
2				
2	¾	5.25	1.502	15.8#
2	1-1/8	9.16	3.379	61.9
2	7/8	8.75	2.044	35.8
1	7/8	9.16	2.044	18.6
	Wt. of nuts and heads			10.0
			Total =	142.1#v

-4- Wt. of special parts = 30#

Total wt. of Truss.
(1) Wooden members - - 2796#
(2) Bolts - - - - - - - 236
(3) Hanger bolts - - 142
(4) Special parts - - 30

Total = 3204#

Wt. of Truss per hor. sq. ft. $= \dfrac{3204}{60 \times 12.5} = 4.26\#$

B Estimate of wt. of Roof covering.

-1- Corrugated steel roofing. (P25 S.H.B.) Corrugated steel to cover 100 sq. ft. using 2 corrugation side lap and 6 inch end lap = 128 sq. ft. Area to be covered = 2 x 33.6 x 12.5 = 840 sq. ft. Wt. of 100 sq. ft. #20 galvanized corrugated sheets with 3" corrugations = 178#

$1.28 \times 8.40 \times 178 = 1970\#$

$\dfrac{1970}{12.5 \times 60} = 2.53\#$ per hor. sq. ft.

-2- Nailing strips.

2 x 4 nailing strips spaced 3'-1"

$\dfrac{2 \times 4 \times 12 \times 11 \times 12.5 \times 32.14}{1728 \times 30 \times 12.5} = .655\#$/ hor. sq. ft.

-3- Rafters 2 x 6 spaced 24" C. to C.

$\dfrac{2 \times 6 \times 12}{1728} \times \dfrac{12.5 \times 33.6 \times 32.14}{2 \times 30 \times 12.5} = 1.51\#$ / hor. sq. ft.

-4- Purlins 4 x 8

$\dfrac{4 \times 8 \times 12}{1728} \times \dfrac{5 \times 12.5 \times 32.14}{30 \times 12.5} = 1.19\#$ / hor. sq. ft.

Wt. of Roof covering.

(1)	Corrugated steel	2.53# / hor. sq. ft.
(2)	Nailing strips	.66# / " " "
(3)	Rafters	1.51# / " " "
(4)	Purlins	1.19# / " " "
	Total	5.89# / " " "

C Panel loads.

-1- Dead panel load.

Wt. of Truss	4.26# / hor. sq. ft.
Wt. of Roof Covering	5.89# / hor. sq. ft.
Total	10.15# / " " "

Horizontal projection of one panel = 7.5'

Dead panel load = 10.15 x 7.5 x 12.5 = 950#.

-2- Snow panel load (fig. 1 p. 4 S.H.B.)

Latitude 34° - 1/4 pitch. Snow load = 10# / hor. sq. ft.

(Note - For Pacific Coast and Arid regions use 1/2 the tabular values)

Min. snow load = 5# / hor. sq. ft. Min. snow panel load = $\frac{5}{10.15}$ x 950 = 469# Max. snow load.

From the fig. on p 4 S.H.B. the max. snow load is smaller than the minimum, therefore it is neglected.

-3- Wind panel load (SHB p. 6 Fig. 3). P = 30# / sq. ft. A = 26° Use Duchemin formula. From the curve normal pressure = 22# Panel load = 22 x 8.4 x 12.5 = 2300#.

-4- By Art 27 p. 56 S.H.B. Vertical panel loading at 30# / hor. sq. ft. = $\frac{30}{10.15}$ x 950 = 2800# / hor. sq. ft.

Article 2.

Tabulation of stresses in roof truss for dead load, minimum snow load, and wind load.

See Fig. 1b, Exhibit A; Fig. 2, Exhibit B and table I

TABLE 1

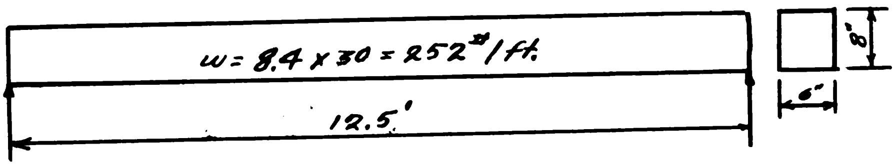
Member	Vertical Load P=1000#	Wind on Left P=1000#	Wind on Right P=1000#	Dead Load P=950# ①	Min. Snow Load P=469# ②	Wind on Left P=2300# ④	Wind on Right P=2300# ⑤	Vertical Loading P=2800# ⑥
Aa	-7800	-4500	-2500	-7400	-3660	-10350	-5750	-21800
Bc	-6700	-3750	-2500	-6360	-3140	-8620	-5750	-18780
Ch	-2200	-1130	-630	-2090	-1030	-2600	-1450	-6180
Di	-1120	-375	-630	-1062	-524	-862	-1450	-3140
aI	+6950	+525	+1680	+6600	+3260	+1210	+3860	+19400
bI	+6950	+525	+1680	+6600	+3260	+1210	+3860	+19400
eI	+6450	+366	+2530	+6120	+3025	+842	+5820	+18100
fI	—	+366	—	—	—	+842	—	—
gI	+6950	—	+3380	+6600	+3260	—	+7780	+19400
ab	0	0	0	0	0	0	0	0
bc	-1100	-1250	0	-1042	-515	-2870	0	-3080
cd	+1000	+300	+840	+950	+469	+690	+1930	+2800
de	-700	+375	-1200	-664	-328	+862	-2750	-1960
hi	-1100	-1250	0	-1042	-515	-2875	0	-3080
ef	-500	0	+825	+475	+235	0	+1900	+1400
fg,	—	-390	—	—	—	-900	—	—
fg	-700	—	-1200	-664	-328	—	-2750	-1960
gh	—	-2275	—	—	—	-5280	—	—
fh	-4475	—	-2525	-4250	-2100	—	-5800	-12550
gg'	0	+260	+260	0	0	+600	+600	0
dh	-3975	-2800	-1680	-3375	-1860	-6440	-3860	-11,150

Article 3.

Analysis of Roof Purlins and Rafters.

(Art. 26 P 56 SHB) Roof covering should be designed for a normal load of not less than 30# / sq. ft.

(A) Analysis of Roof Purlins.



(Art 29 P 651 and P 368 A. H. B.) Allowable unit extreme fibre stress for Oregon Pine (Douglas Fir) = 130% x 800# = 1040# / sq. in.

$$\text{Max. } M = \frac{252 \times 12.5^2}{8} \times 12 = 59{,}100''\#$$

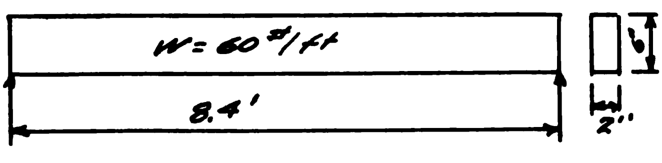
$$I = \frac{6 \times 8^3}{12} = 256 \qquad c = 4''$$

$$\text{Resisting moment} = \frac{sI}{c}$$

$$59100 = \frac{s \times 256}{4}$$

s = 924#/ sq. in which is well within the allowable working stress.

(B) Analysis of Rafters.



Analyze for a normal load of 30# / sq. ft.

$$\text{Max } M = \frac{60 \times \overline{8.4}^2}{8} \times 12 = 6{,}360''\#$$

$$M = \frac{sI}{c}$$

$$I = \frac{2 \times 6^3}{12} = 36$$

$$c = 3''$$

$$6360 = \frac{s \times 36}{3}$$

$s = 530\#$ which is well within allowable working stress.

Article 4.

Tabulation of Max stresses and analysis of Truss Members.

Refer to table I

To find maximum load combine 1, 2, 4, - 1, 2, 5, 1, 4 - 1, 5 or 6.

Member	Max. Comp	Max. Load	Max. Tension	Max. Load	Size of Mem.	Min Sect.
Aa	21800	6			6 x 8	48 sq. in
Bc	18780	6			6 x 8	48
Ch	6180	6			4 x 8	32
Di	3140	6			4 x 8	32
aI			19400	6	6 x 10	40
bI			19400	6	6 x 10	40
eI			18100	6	6 x 10	40
fI)			842	1,4	6 x 10	40
gI)			19400	6	6 x 10	40
ab	-	-	-	-	dia = 3/4"	
bc	4427	1,2,4			6 x 6	36
cd			2349	1,2,5	dia 1-1/8	
de	3742	1,2,5			6 x 6	36
hi	4432	1,2,4			4 x 6	24
ef			2610	1,2,5	7/8 dia	
fg_1	900	1,4			6 x 6	36
fg	3742	1,2, 5			6 x 6	36
gh)	5230	1,4			6 x 8	32
fh)	12550	6			6 x 8	32
dh)	11150	6			6 x 8	32
gg'			600	1,5	7/8 dia	

Top Chord members aA and cB

Max. Stress = 21800# compression

Length of chord per panel = 8.4'

Min sec. = 48 sq. in.

Safe end bearing stress Oregon Pine (Douglas Fir) = 1200# x 1.30 = 1560# (A. M. B. p. 657).

Column formula adopted by A. R. E. A.

$$S_1 = S_2 \left(1 - \frac{l}{60\ d} \right)$$

$$S_1 = 1560 \left(1 - \frac{8.4' \times 12}{60 \times 6} \right) = 1560 \times (1 - .28)$$

= 1120# safe working stress for the column.

The allowed stress in aA and eB = 48 x 1120 = 53,750#

Top chord members hc and iD

Max. stress = 6180# compression.

Length = 8.4'

Min. section = 32 sq. in.

The allowable stress = 32 x 1120 = 35800#

Bottom chord members aI, bI, eI and fI

Max. Stress = 19400# tension

Allowed stress in tension = 1.30 x 800 = 1040#

Min. net section = 40 sq. in.

Allowed stress in the member = 40 x 1040 = 41600#

Tie rod ab

Max. stress = 0

Member cd

Max stress = 3349# tension

Allowed tensile stress in steel = 16000#/ sq. in.

Net area of steel in a 1 1/8" rod = .994 sq. in.

Allowed stress in rod = .994 x 16000 = 15,900#.

Member ef - 7/8" rod.

Max. stress = 2610# tension

Net area = 60 sq. in.

Allowed stress = .60 x 16000 = 9600#

Member gg^1 - 7/8" rod

Max stress = 600# tension

Allowed stress = 9600#

Member be

Max. stress = 4427# comp.

L = 8.4'

Min. sect. = 36 sq. in.

Allowed stress = 36 x 1120 = 40,400#

Members dh, fh, gh.

Max. stress = 12550# compression

L = 7.5'

Min. Section = sq. in.

Formula for columns A.R.E.A. (A.E.B. p. 657)

$S_1 = 1560 \left(1 - \frac{7.5 \times 12}{60 \times 4} \right) = 1560 \times .625 = 975\#$

Allowed stress in the column = 32 x 9.75 = 31200#

Member de

Max. stress = 3742# compression

$L = \sqrt{7.5^2 + 7.5^2} = 10.6'$

Min. section = 36 sq. in.

$S_1 = 1560 \left(1 - \frac{10.6 \times 12}{60 \times 6} \right) = 1015\#$

Allowed stress = 36 x 1015 = 36,500#

Member hi

Max. Stress = 4432# comp.

L = 8.4'

Min. section = 24 sq. in.

Allowed stress = 24 x 1120 = 26,800#

Member fg_1

Max. Stress = 900# compression

L = 10.6'

Min. section = 36 sq. in.

Allowed stress = 36 x 1015 = 36,500#

Member fg

Max. stress = 3742# compression.

L = 10.6

Min. section = 36 sq. in.

Allowed stress = 36,500#

Summary of Analysis of Truss members.

Members	Allowed Stress	Actual Stress
Aa, Bc	-53,750	-21,800
Ch, Di	-35,800	- 6,180
aI, bI, cI, fI, gI,	+41,600	+19,400
ab		0
bc	-40,400	- 4,427
cd	+15,900	+ 3,349
de	-36,500	-3742
hi	-26,800	- 4,432
ef	+ 9,600	+ 2,610
fg_1	-36,500	- 900
fg	-36,500	-3,742
gh, dh	-31,200	-12,550
gg'	+ 9,600	+ 600
fh *	-31,200	-12,550

* fh is the same member as gh.

Article 5.

Analysis of Joint Details for each Typical Joint. See Fig. 1, Exhibit A.

Analysis of Joint U.

The area of the bearing surface of be on the en the upper chord = 5 sq. in. at an angle of 83° with the fibres and 36 sq. in. at an angle of 7° with the fibres. Allowed bearing stress of be then equals (by A.H.B. p 651 and 700) 1.3 x 959 x 5 + 1.3 x 221 x 36 = 16,600# which is sufficient.

There is no stress in the rod ab.

The min. section of the upper chord at the joint = 38 sq. in. The allowable compression then, at the joint, of Aa or Bc = 38 x 1120 = 42,500# which is sufficient.

Analysis of Joint U_2.

Area of the plate on the end of ed = $7\frac{1}{2}$ x 3 - 1.23 = 21.27 sq. in.

Allowed stress in ed = 21.27 x 260 = 5530# which is sufficient.

Bearing surface of Be on dh = $2\frac{1}{2}$ x 6 = 15 sq. in. at an angle of 61° with the fibres and $16\frac{1}{2}$ x 6 = 104 sq. in. at 8°.

Allowable comp. at 61° = 963 x 1.3 = 1250#.

Allowable comp. at 8° = 224 x 1.3 = 290#.

Allowable comp. in Be = 15 x 1250 + 16.5 x 6 x 290 = 18,750 plus 29000 = 47,750 which is sufficient.

The bearing surface of dh on Be is $2\frac{1}{2}$ x 6 = 15 sq. in. at an angle of 60° with the fibres.

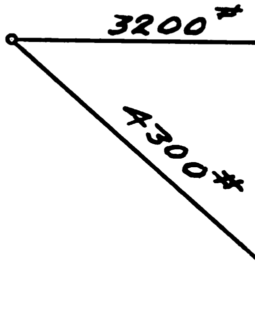
Allowable stress # / sq. in = 950 x 1.3 = 1235#.

Allowable stress in dh then equals 15 x 1235 = 18,500#

The horizontal component of the stress in Ch $= \frac{30}{33.6} \times 6180 =$ 5,550# which must be transmitted through the block to the bolt and washer. The area of the washer = 18 - 1 = 17 sq. in. The surface of the washer makes an angle of 27° with the fibres. Allowed stress (A.H.B. p 700) = 1.30 x 341 = 443# Then the force which can be transmitted thru the block = 17 x 443 = 7530# which is sufficient.

Analysis of Joint U_0 .

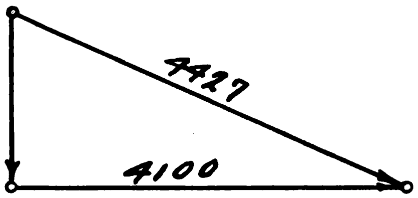
The outward thrust of the compression member which is equal to the pull of the tension member or 19,400# is resisted by the shear of the lower chord along the plane be and by the horizontal component of the tension of the bolts. The allowable shear with the grain = 1.30 x 130 x 6 x 16 = 16,200# leaving a remaining stress of 3,200# to be taken by the bolts. The horizontal compentent of the stress in the bolt = 3200#.



The stress resited by the bolts = 4300#. There are 2 1-3/8 bolt which are sufficient. The 3,200# horizontal stress must be carried by the projection of the plate whose area = 6 x ¾ = 4.5 sq. in. Allowed comp. perpendicular to the fibre = (1200 x 1.3) x 4.5 = 7000# which is sufficient. The horizontal shear that can be taken by the plate = 1.30 x 130 x 6 x 48 = 48,600 which is sufficient. The plate must take a force of 4300#. The area of the plate = 99 sq. in. Allowable compression = 99 x 260 = 25,740 which is sufficient.

Analysis of Joint L_2 .

With wind on the left the maximum stress in be = 4427# and maximum stress in de equals 130#. The horizontal component of these foces must be carried by the projection into the tension member.



The horizontal component = 4100# - 100 = 4000#.

Allowed bearing vaule = 1.3 x 1200 = 1560#

Area of bearing = 6 x .5 = 3 sq. in.

Allowed bearing = 3.0 x 1560 = 4,680# which is sufficient.

The area of the plate = 21.27 sq. inches.

Allowed stress in cd = 260 x 21.27 = 5530 which is sufficient.

Article 6.

Analysis of wall footings for Max. Wall Height and Truss Loading.

(A) Wt. of Pilaster (Fig. 3).

Wt. of concrete = 150# / cu. ft.

Vol. of footing = 2.72 x 2.66 x 2 = 14.50 cu. ft.

Vol. of Concrete wall = 9.65 x 2.72 x 2.66 = 69.8 cu. ft.

Vol. of lintel = $\frac{20}{12}$ x $\frac{10}{12}$ x 2.72 = 3.77 cu. ft.

Total volume = 88.07 cu. ft.

Weight = 88.07 x 150 = 13,250#

Vol. of solid tile = 13.58 x 1.66 x 2.72 = 61.3 cu. ft.

Wt. of solid tile = 120# / cu. ft. (kent p 177)

Wt. of tile = 61.3 x 120 = 7,350#

Wt. of pilaster = 13,250 plus 7350 = 20,550#

(B) Truss loading (Fig. 3)

Reaction due to dead load = 9800#

Reaction at right with wind on right = 6315#

Vertical component of wind reaction = 6315 x $\frac{30}{33.6}$ = 5650#.

Total truss loading taken by the pilaster = 5650 plus 9800 = 15,450# Total pressure on the footing = 20,550 plus 15,450 = 36,000#.

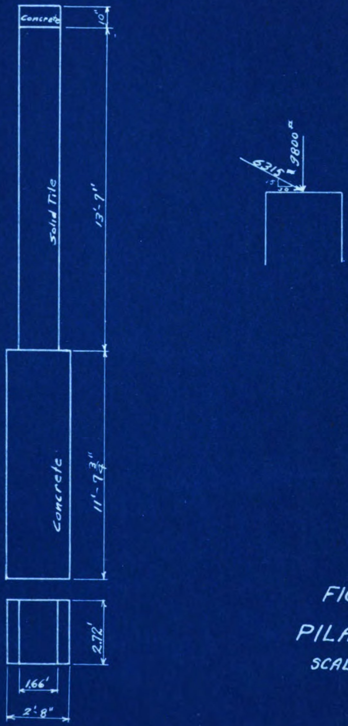


FIG. 3
PILASTER
SCALE ¼" = 1'

(C) Wt. of wall between pilasters (Fig. 4)

Vol. of concrete = 9.65 x 9.79 x 2 plus 2 x 2.66 x 9.79 plus $\frac{16.25}{12}$ x $\frac{10}{12}$ x 9.79 = 189.0 plus 52.0 plus 11.0 = 252 cu. ft.

Wt. of concrete = 252 x 150 = 37,800 #

Vol. of tile = 9.79 x 13.58 x 1 = 132 cu. ft.

Vol. of solid tile = approximately 2/3 x 132 = 88 cu. ft.

Wt. per cu. ft. of solid tile = 120#

Wt. of tile = 10,600#

Total wt. of wall between pilasters = 37,800# plus 10,600# = 48,400#

Total wt. on footing exclusive of floor beams = 36,000plus 48,400 = 84.400#

Wt. per ft. = $\frac{84.400}{12.5}$ = 6740#.

There are two floor beams between each pair of pilasters, (Fig. 4a)

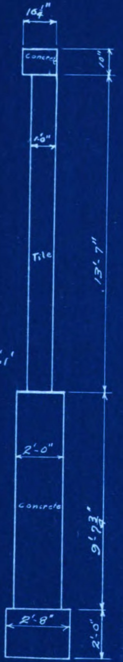
The floor was designed for a uniform loading of #400#/ sq. ft.

Load on the two floor beams = 12x 15 x 400 = 72,000#

Wt. of 2 - 15" Is @ 42# 15' long = 1260#.

Load on wall = $\frac{73260}{2}$ = 36,630#.

Load per ft. = $\frac{36.630}{12.5}$ = 2930#.



SCALE ¼" = 1'

FIG. 4

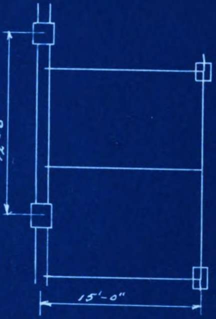
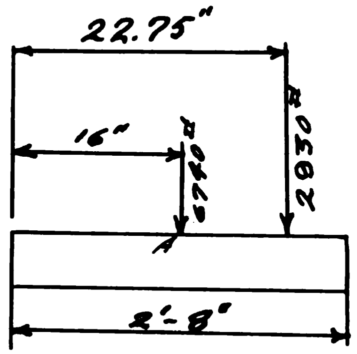


FIG 4a

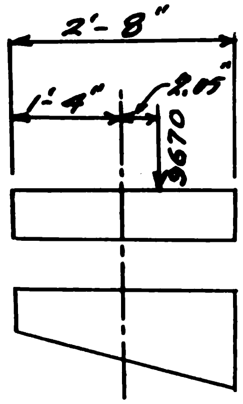
WALL BETWEEN
PILASTERS



Taking moments about A, $2930 \times 6.75 = 9670\ \bar{x}$

$\bar{x} = \frac{2930 \times 6.75}{9670} = 2.05$ inches.

Test of wall footing.



From A.H.B. P 587

$S_1 = \frac{V}{b}\ (1 \text{ plus } 6\ \frac{e}{b})$

$S_1 = \frac{9670}{2.66}\ (1 + 6\ \frac{\frac{2.05}{12}}{2.66}) = \frac{9670}{2.66}\ (1.014)$

$= 3690\#/$ sq '

$S_2 = \frac{V}{b}\ (1 - 6\ \frac{e}{b}\ )$

$S_2 = \frac{9670}{2.66}\ (1 - .014) = 3580\#/$ sq. ft.

Allowed pressure = 4000# / sq. ft.

Wt. of wall at max. height. (Fig. 5)

Volume of concrete = 2.66 x 6 x 1.5 plus 6 x 2 x 9.25 =

23.9 plus 111.0 = 134.9 cu. ft.

Wt. of concrete = 150 x 134.9 = 20,300#

Vol. of tile = 6 x 1 x 305 = 183.0 cu. ft.

Vol. of solid tile = x 183 = 120 cu. ft.

Wt. of tile = 120 x 120 = 14,400#.

Total wt. of wall = 20,300 plus 14,400 = 34,700#

Wt. per ft. = $\frac{34,700}{6}$ = 5780#.

SCALE $\frac{3}{16}'' = 1'$

FIG 5

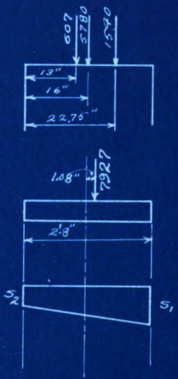


FIG 5a

Wt. transmitted to wall by one floor beam =

$$\frac{3.7 \times 13.0 \times 400}{2} \text{ plus } \frac{13 \times 40}{2} = 9000 \text{ plus } 260 = 9260\#.$$

Wt. per ft. $= \frac{9260}{6} = 1540\#$

Max. Wt. transmitted to wall by D11 = 3640#.

Wt. per ft. $\frac{3640}{6} = 607\#$.

Test of footing.

Take moments about A (fig. 5a).

$1540 \times 6.75 - 607 \times 3 = 7927\ Y$

$Y = 1.08$

$$S_1 = \frac{7927}{2.66}\left(1 + 6\,\frac{\frac{1.08}{12}}{2.66}\right) = \frac{7927}{2.66}(1.076) = 3200\#.$$

$$S_2 = \frac{7927}{2.66}\left(\overset{.924}{1-.076}\right) = 2760\#$$

Allowed pressure = 4000#

Article 7.

Analysis of Reinforced concrete lintel 10' span. (A.H.B. p. 460) See Fig. 6 and 6a.

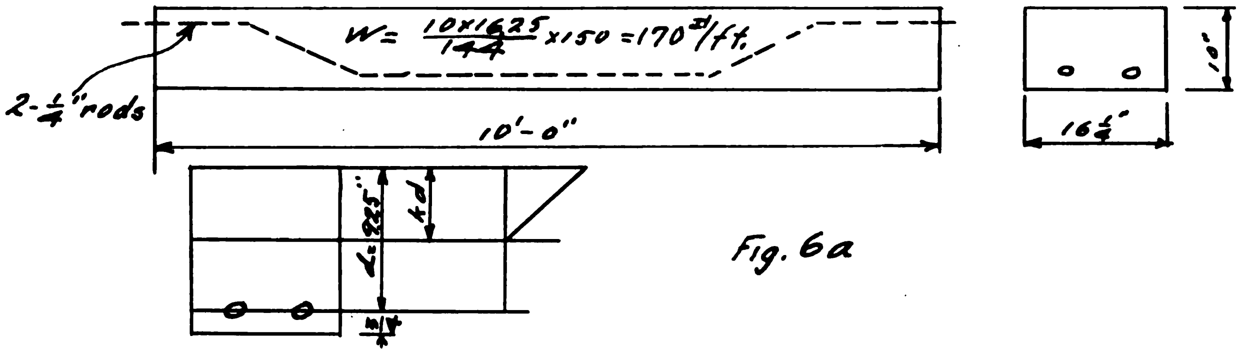


Fig. 6a

Max. M = $\frac{wl^2}{8}$ = $\frac{170 \times 10^2}{8} \times 12$ = 25,500"#

p = $\frac{2 \times .0491}{9.25 \times 16.25}$ = $\frac{.0982}{150}$ = 0.00655

k = $\sqrt{np\,(2 + np)} - np$ = $\sqrt{15 \times .00655\,(2 + 15 \times .00655)}$ -

15 x .00655 = .356

jd = d (1- 1/3 k) or j = (1 - $\frac{.356}{3}$) = .782

Ss = $\frac{M}{pjbd^2}$ = $\frac{25,500}{.782 \times .00655 \times 16.25 \times 9.25^2}$ = $\frac{25500}{7}$ =

3640# / sq. in.

Sc = $\frac{2M}{jkbd^2}$ = $\frac{2 \times 25,500}{.782 \times .356 \times 16.25 \times 9.25^2}$ = $\frac{51000}{388}$ =

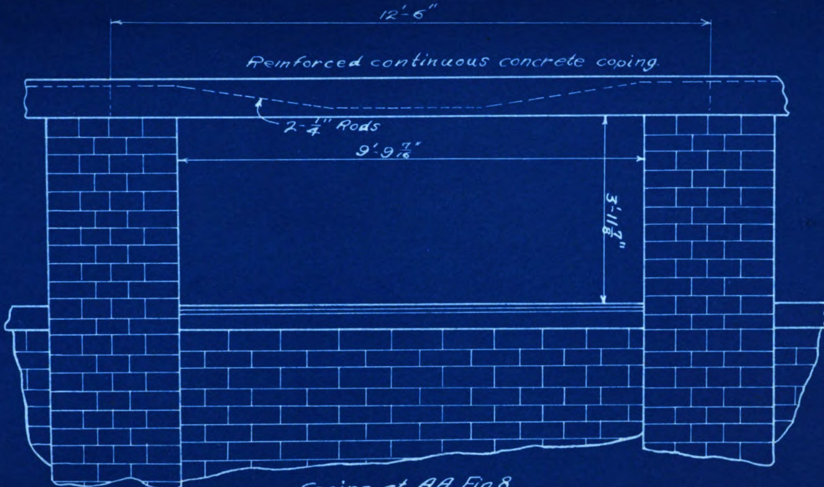
132# / sq. in.

Max. shear = 850#

U = 2 (80 x .785) = 125.6

u = $\frac{V}{jd}$ or V = ujd = 125.6 x .782 x 10 = 985#

Then resisting shear = 985#.

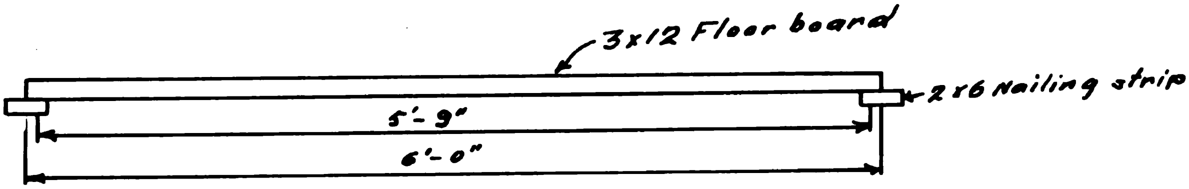


Coping at AA Fig. 8

Fig 6

Article 8.

Analysis of 3" flooring.



$$\text{Max. Moment} = \frac{wl^2}{8} = \frac{400 \times \overline{5.75}^2}{8} \times 12 = 19850\text{"\#}$$

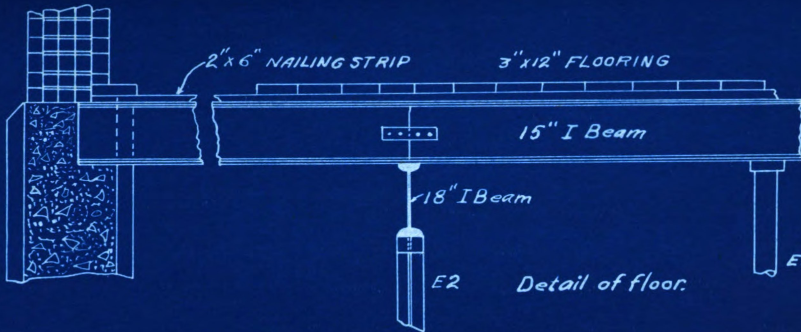
$$M_R = \frac{sI}{e}$$

$$I = \frac{bd^3}{12} = \frac{12 \times \overline{3}^3}{12} = 27$$

$$e = 1.5"$$

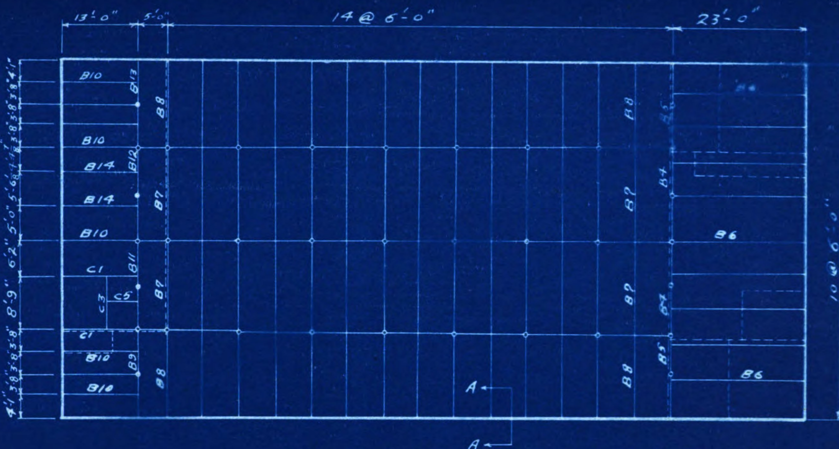
$$19{,}850 = \frac{s \times 27}{1.5}$$

$s = \dfrac{19850 \times 1.5}{27} = 1100\#$ / sq. in. which is slightly above allowable stress.



Detail of floor.

Fig. 7



FLOOR PLAN
Fig. 8
Scale $\frac{1}{16}'' = 1'$

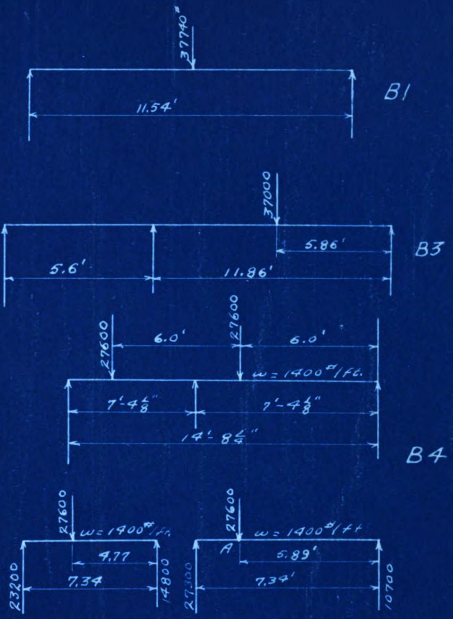


Fig. 8a

FLOOR BEAMS

M_R = 79000'#

Max. Mom. on left half = $22,200 \times 2.79 - \dfrac{1400 \times \overline{2.79}^2}{2}$ = 56550'#

M_R = 79000'#

Analysis of B 6   12" - I @ 40#   See Fig. 8b.

B6 was designed so that if the building were lengthened a stringer could be run along the center of the beams B6 and it would hold up a wt. of 400# / sq. ft.   Then the beam would be designed as though it were half as long.

Max. M = $\dfrac{2400 \times \overline{11.15}^2}{8}$ = 39,600'#

M_R = 60,000'#.

Analysis of the beam B6 which holds up a partition and a live load of 100# / sq. ft.

Max. moment = $\dfrac{900 \times \overline{22.3}^2}{8}$ = 56000'#

M_R = 60,000'#

Analysis of B7.   15" - I @ 42#. See Fig. 8c.

Max. m. = $\dfrac{2400 \times \overline{14.7}^2}{8}$ = 64,800'#

M_R = 79000'#

Analysis of the beam B 7 on which the column D3 of trolley

framing rests.

Wt. / ft. due to floor load. equals 2200#.

Wt. / ft. due to partition load equals 200#.

Max. wt. transmitted by D3 equals 3333# (from Art. 20).

R_1 = 17000 plus $\frac{5.5}{15}$ x 3333 = 19,950#

R_2 = 20850#

Max. M = 19,100 x 5.4 - 12960 x 2.7 = 68,000

M_R = 79,000'#

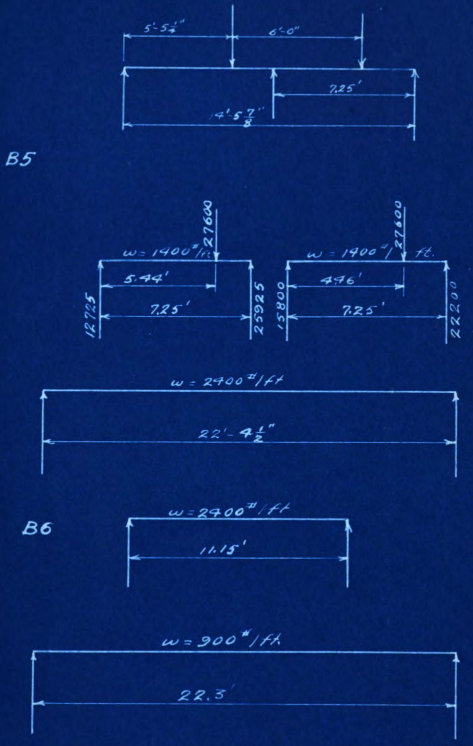


Fig 8b
FLOOR BEAMS

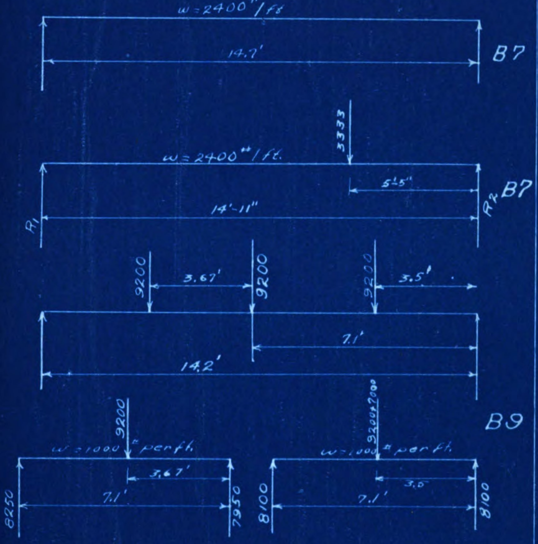


Fig 8c

FLOOR BEAMS

Analysis of B 8. 15" - I @ 42#.

B8 has same load and very nearly the same span as B7 and need not be analyzed.

Analysis of B9. 15" - I @ 42#

Max M on right half of beam $= 8100 \times 3.5 = 28400$'#.

$M_R = 79000$'#.

Max. M on left half of beam $= 8100 \times 3.5 - \frac{1000 \times \overline{3.5}^2}{2} =$ 22,280'#

$M_R = 79000$'#.

Analysis of B10. 12" - I @ 31.5#. See Fig. 8d.

Max. $M = \frac{1468 \times \overline{12.44}^2}{8} = 28,500$ '#.

$M_R = 48,000$'#.

Analysis of B11. 15" - I @ 42#.

From Article 12 the load carried to B11 by c1 = 11,928#.

$R = \frac{7340}{2}$ plus $\frac{5.96}{7.34} \times 11928 = 13350$

$R_2 = 6088$#.

Max. $M = M_A = 11928 \times 1.38 - \frac{1000 \times \overline{1.38}^2}{2} = 15,500$'#

$M_R = 79000$'#.

Analysis of B13. 15" - I @ 42#.

$R_1 = 8200$# $R_2 = 8460$#.

Max. $M = M_A = 8460 \times 2.46 - \frac{1000 \times \overline{2.46}^2}{2} = 22400$'#

$M_R = 79,000$'#

Analysis of B 14. 15" - I @ 42#.

Max. M same as B10 = 28500'#.

$M_R = 79,000$'#.

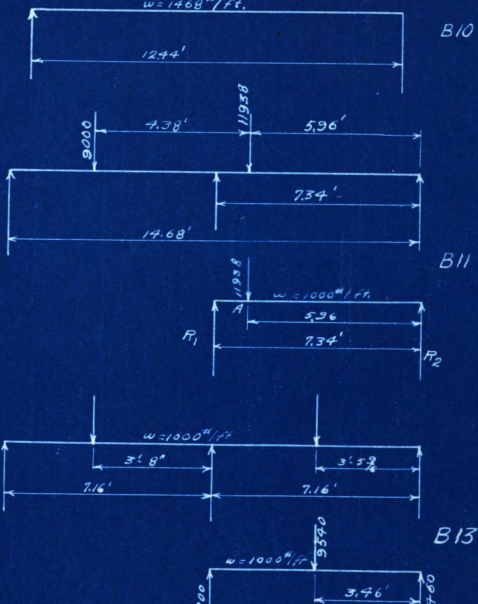


Fig 8d
FLOOR BEAMS

Summary of floor beam Analysis.

Beam	Size	Actual Stress	Allowed Stress
Beam			
B1	18" - 60# - I	106800	125000
B2	Same as B1	Same as B1	Same as B1
B3	18" - 60# - I	109500	125000
B4	15" - 42# - I	38130 55370	79000
B5	15" - 42# - I	67470 56550	79000
B6	12" - 40# - I	56000 39600	60000
B7	15" - 42# - I	68000	79000
B8	Same as B7	Same as B7	Same as B7
B9	15" - 42# - I	28400 32280	79000
B10	12"-31.5# - I	28300	48000
B11	15" - 42# - I	18300	79000'#
B13	15" - 42# - I	33400	79000
B14	15" - 42# - I	285000	79000

Article 10.

Analysis of Pipe Columns (figs. 8, 9 & 20).

Greatest value of P on any of the pipe

column = 40,000#.

Inside dia. (Standard wrt. iron pipe) =

5.055"

Outside dia. = 5.563"

A = 24.31 - 19.99 = 4.32 sq. in.

l = 8.08 x 12 = 96.5"

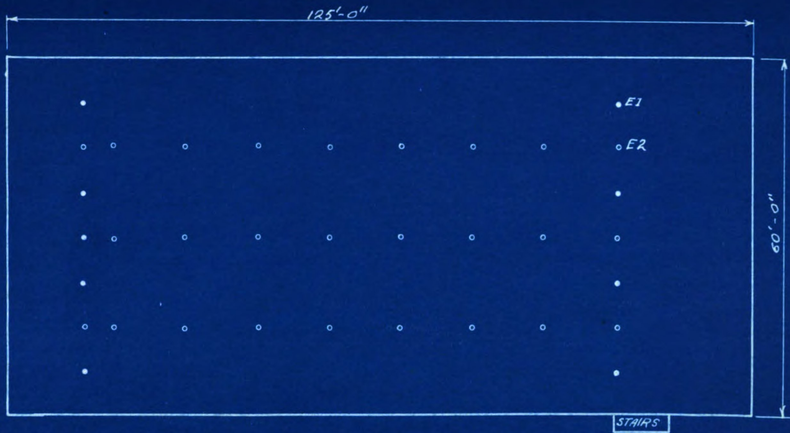
$r = \frac{\sqrt{d^2 + d^2}}{4} = \frac{7.510}{4} = 1.877$

$\frac{l}{r} = 51.3$

Assume $\phi = \frac{1.95}{36000}$ (A.H.B. p 307) $\frac{P}{A} = \frac{S}{1 + \phi(\frac{l}{r})^2}$

$$\frac{40,000}{4.32} = \frac{S}{1 + \frac{1.95}{36,000}(51.3)^2} = \frac{S}{1.1425}.$$

$S = \frac{40,000 \times 1.1425}{4.32} = 10,600\#$ which is well within allowed stress for wrt. iron.



FOUNDATION PLAN

scale $\frac{1}{16}$" = 1'

E1 - Pipe Columns
E2 - Built Columns

Fig. 9

Article 11.

Analysis of Built Plate and Angle Column. (See Figs. 11 and 11a)

Fig 11a

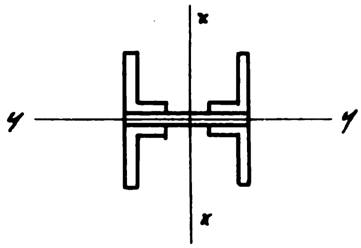


Plate 7" x ¼"

4Ls 3½x2½x¼

$l = 6' - 6" = 78"$

A of 4Ls = 5.76 sq. in.

A of Plate = 1.75 sq. in.

Total A = 7.51 sq. in.

Iyy (Ls) = 16 Ixx (Ls) = 51.

Iyy (Pl) = .01.

I = 16.01

$r^2 = \frac{16.01}{7.51}$ $r = 1.45$

$\frac{l}{r} = \frac{78}{1.45} = 54.0$

P = 74000

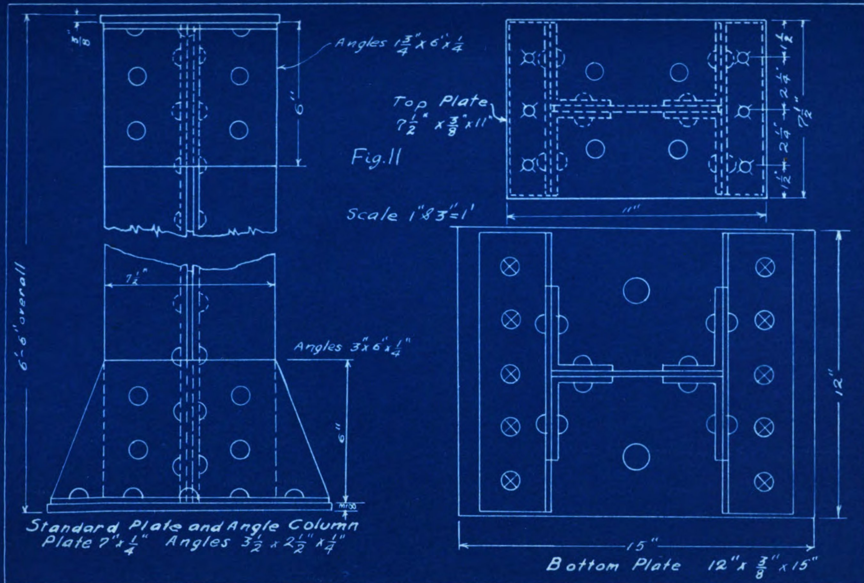
$\frac{l}{r} = 54$

$\frac{P}{A} = S - \frac{Cl}{r}$

C = 220 (A.H. B. p 308)

A = 7.51 sq. inches.

$S = \frac{74000}{7.51}$ plus $\frac{220}{4}$ x 54 = 12820# which is not excessive.


Angles 1¾" x 6" x ¼
6"
5/8"
6'-6" overall
7½"
Angles 3" x 6" x ¼"
6"
Standard Plate and Angle Column
Plate 7" x ¼" Angles 3½ x 2½" x ¼"
Top Plate
7½" x 3/8" x 11"
Fig. 11
Scale 1" 8 3" = 1'
1½ 2¼" 2¼" 1½
7½"
11"
12"
15"
Bottom Plate 12" x 3/8" x 15"

Article 12.

Analysis of all members of elevator framing and supporting beam C1 and C3. (See Fig. 14 and 14a)

$1\frac{1}{2}$ ton elevator. (Use concentrations of elevator on p 331 Vol II Hool).

Find load transmitted to C4.

Load at a $= \frac{2.5}{8.75} \times 4000$ plus $\frac{6.25}{8.75} \times 800 = 1758$#.

Load at b = same as at a = 1758#.

Load at c $= \frac{6.75}{8.75} \times 1000$ plus $\frac{2.5}{8.75} \times 6200 = 2540$#.

Load from C and D is carried to the columns by a beam above C4.

Load caused by D $= \frac{1.75}{10.25} \times 1250$ plus $\frac{3}{10.25} \times 1250 = 578$#.

Load caused by C $= \frac{1}{10.25} \times 1250$ plus $\frac{2.25}{10.25} \times 1250 = 400$#.

Load on C2' from C. and D $= \frac{6.75}{8.5} \times 578$ plus $\frac{8.0}{8.5} \times 400 = 836$.

Load on C2 = 978 - 836 = 142#.

Multiply the stresses by two due to the impact caused by stopping and starting the elevator. Taking moments about left reaction

3516 x 2 plus 3516 x 3.5 plus 5080 x 7.5 plus 284 x 8.5

$= R_2 \times 8.5$

$R_2 = \frac{7032 \text{ plus } 12260 \text{ plus } 36200 \text{ plus } 2400}{8.5} = 6800$#

$R_1 = 7268$#

Max. M occurs at b and equals - 6516 x 5.0 plus 5080 x 4

= 12260'#

M_R of C4 which is a 10" - I @ 25# = 33000'#

Max. load on column C2 = 7268#

$L = 13.5^{1}$

C2 is a 10" - I @ 25#

$r = .97$

$\frac{l}{r} = \frac{13.5}{.97} = 13.9$

$A = 7.37$

(A.H.B. P 307) $\phi = \frac{1.95}{25000}$

$$\frac{P}{A} = \frac{S}{1 + \phi\left(\frac{l}{r}\right)^2}$$

$$\frac{7268}{7.37} = \frac{S}{1 + \frac{1.95}{25,000}(13.9)^2}$$

$S = \frac{7350}{7.37} = 1000$#/ sq. in. which is safe.

Analysis of C1, 15" - I @ 42#. (See Fig. 14b)

$R_1 = \frac{4.32}{12.8} \times 7268$ plus ~~[illegible]~~ $\frac{8.5}{12.8} \times 6120$ plus $\frac{2.2}{12.8} \times 7040 =$ 8490# .

$R_2 = 20428 - 7490 = 11,938$#

Max. $M = 8490 \times 8.5 - 6120 \times 4.25 = 46000$'#

$M_R = 79000$'#

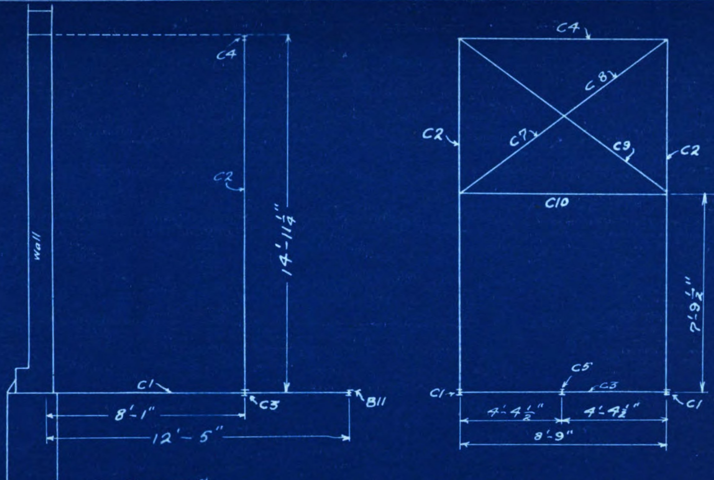
Analysis of C3, 12" - I @ 31.8# (Fig. 14c)

Load on beam = load carried by C2 to the mid point.

$P = 4.375 \times 4.32 \times 400 = 7560$#

Max. $M = \frac{7560 \times 8.75}{4} = 16500$'#

$M_R = 48,000$'#



C1 - 15" - 42# - I
C2 - 10" - 25# - I
C3 - 12" - 31.5# - I
C4 - 10" - 25# - I
C5 - 8" - 20.25# - I
C7 - L $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$
C8 - L $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$
C9 - L $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$
C10 - L $3 \times 3 \times \frac{1}{4}$

Fig. 14

ELEVATOR FRAMING

SCALE $\frac{1}{4}'' = 1'$

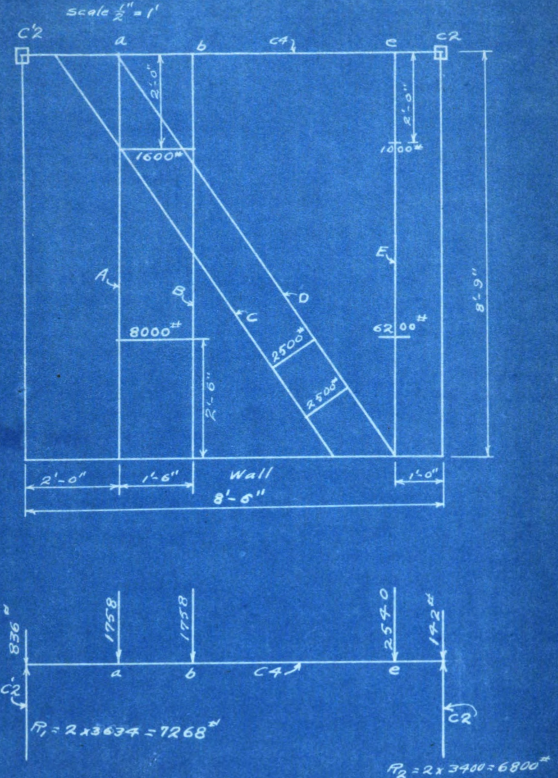
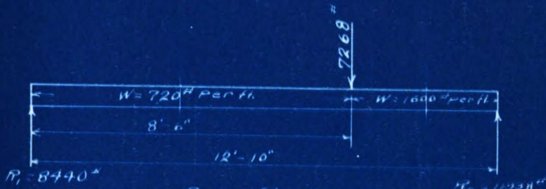


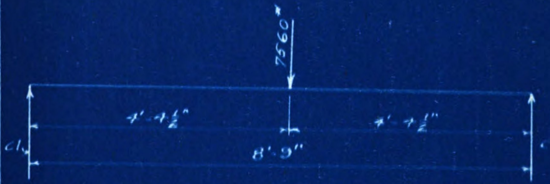
FIG. 14a

$1\frac{1}{2}$ T. ELEVATOR

V.R. Cooledge



Beam C1



Beam C5

Fig 14 b

Article 13.

Analysis of Column Footings.

(Fig. 8 and 9.)

The maximum pressure on the column footings is 74,000#.

The area of footing is 3 x 4 = 12 sq. ft.

The bearing pressure on the soil then equals $\frac{74000}{12}$ =

6,180# = 3.1 ton.

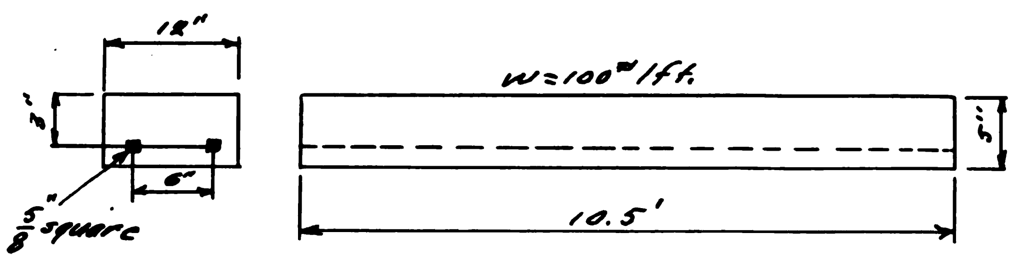
Article 14.

Analysis of Reinforced concrete stairs.

According to Hool Vol. II stairs for commercial and manufacturing plants are designed for 100# per horizontal sq. ft.

Then the normal load in case of a stairs having a rise of 7(-7½" and a run of 11'-5¼" = 83.5#.

The transverse reinforcing is used only for stiffening and to prevent shrinkage cracks.



$$\text{Max. } M = \frac{100 \times 10.5^2}{8} = 1380'\# = 1380 \times 12 = 16{,}550''\#.$$

$$p = \frac{.78}{36} = 0.0216$$

$$k = \sqrt{2 \times 0.0216 \times 15 \text{ plus } (0.0216 \times 15)^2} - 0.0216 \times 15 =$$

$$.542$$

$$j = 1 - .180 = 820.$$

As determined by the steel

$M_s = 14000 \times 0.0216 \times .820 \times 12 \times 3^2 = 26800''\#$ which is safe.

As determined by the concrete Res M

$M_c = \frac{1}{2} \times 650 \times .542 \times .820 \times 12 \times 3^2 = 15600''\#$ which is a little small.

Max. shear = 525#.

For bond $80 = \dfrac{V}{2 \times 4 \times 5/8 \times 820.}$ $V = 328\#$ which is smal l.

Article 15.

Analysis of wall and buttress of Oil Sump. (See Fig. 15).

Span of buttresses 10'-9".

Thickness of buttress 1'-0".

Weight of earth 100# / cu. ft.

Weight of concrete 150# / cu. ft.

By Rankine's formula with horizontal surcharge and vertical

wall $P = \frac{1}{2} wh^2 \left(\frac{1 - \sin \phi_1}{1 + \sin \phi_1} \right)$

$\phi_1 = 33^\circ - 46'$

$P = 50 \times \overline{7.7}^2 \left(\frac{1 - .55581}{1 + .55581} \right) = 843\#$

Wt. of earth on footing $= \frac{1}{2} \times 6.7 \times 100 = 335\#$.

Wt. of masonry $= \left(\frac{1.5 + 2.1}{2} \times 6.7 \text{ plus } 3.1 \times 1 \right) 150$

$= 2260\#$.

(See Fig. 15a)

$$X = \frac{335 \times 2.8 \text{ plus } 1810 \times 1.51 \text{ plus } 450 \times 1.5}{2595}$$

$$= \frac{938 \text{ plus } 2730 \text{ plus } 675}{2595} = 1.67'$$

Total vertical pressure of earth and masonry is 2595 x 10.75

or 27900#.

Total horizontal pressure of earth is 843 x 10.75 or 9080#.

The graphical solution (Fig. 15c) shows that the resultant

pressure lies well within the middle third.

Bearing stress - see Fig. 15b.

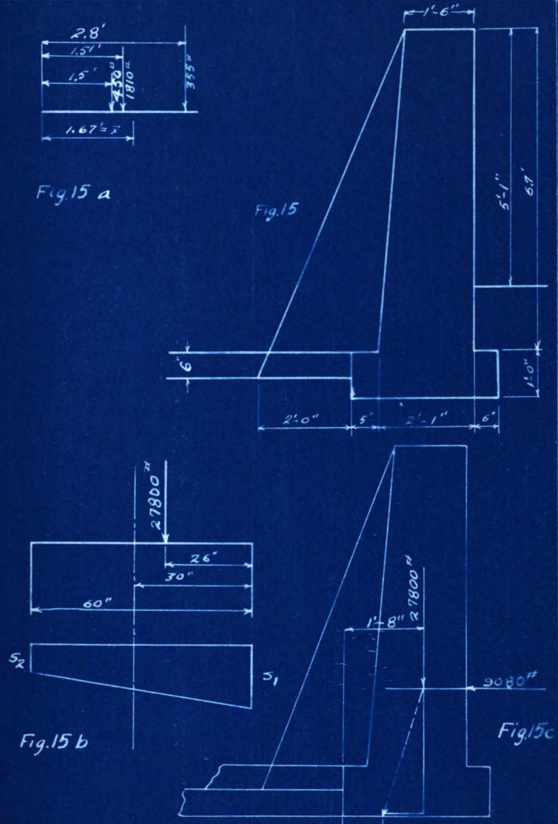
$S_1 = \frac{P}{b} \left(1 \text{ plus } 6 \frac{e}{b} \right)$ (A. H. B. p. 587)

$S_1 = \frac{27900}{60} \left(1 \text{ plus } 6 \frac{4}{60} \right) = 648.0\# / \text{sq. in.}$

$S_2 = \frac{27900}{60} \left(1 - 6 \frac{4}{60} \right) = 279.0\# / \text{sq. in.}$

There is a 4" pipe braced between buttresses which adds to

the factor of safety.



WALL OF OIL SUMP

Article 16.

Analysis of I beam Trolley.

10" I beam (Fig. 12 exhibit C).

Analysis of Rail #1 (Fig. 12b)

The most dangerous place is the 8'-1" span.

Max. moment occurs when load is at the center.

Neglecting the wt. of beam Max. Moment $= \frac{Pl}{4} = \frac{8000 \times 8.1}{4}$

$= 16200$'#.

Max. allowed bending moment for the lightest 10" I beam in

S. H. B. is 33000'# for a 25# I beam.

Max. shear = 8000#.

Analysis of Rail #2 (Fig. 12b)

Max. $M = \frac{8000 \times 14.5}{4} = 29000$# which is safe.

Max. shear = 8000#.

Analysis of Rail #3 (Fig. 12b).

Twisting moment = .5 x 8000 = 4000'# (From A.H.B. p 310)

$S = \frac{Pe\,c}{J}$

Pe = 4000 x 12 = 48000"#

c = distance to extreme fibre = 5".

J = polar inertia = Ix plus Iy.

$S = \frac{48000 \times 5}{128,99} = 1860$ #/ sq. in.

Max. M in beam = 4828 x 1.75 = 8450'#

$M = \frac{SI}{C}$

$8450 \times 12 = \frac{S \times 122.1}{5}$

S = 4150# / sq. in.

Resultant stress = $\sqrt{4150^2 \text{ plus } 1850^2}$ = 4540# / sq. in.

which is safe.

Max. shear is 8000#.

Fig. 12b.

Twisting moment = 8000 x 5/12 = 3340'#.

$S = \frac{Pe}{J}$

$S = \frac{3340 \times 12 \times 5}{128.99}$ = 1550# / sq. in.

Max. M. is 3500 x 2.16 or 7550.

M is $\frac{sI}{c}$

$7550 \times 12 = \frac{s \times 122.1}{5}$ = 3700# / sq. in.

Resultant stress = $\sqrt{1550^2 \text{ plus } 3700^2}$ = 4010# / sq. in.

which is safe.

Max. shear is 8000#.

Analysis of Rail #4 (Fig. 12b)

Max. M is $\frac{8000 \times 15.5}{4}$ = 31,000'# which is safe.

Max. shear is 8000#.

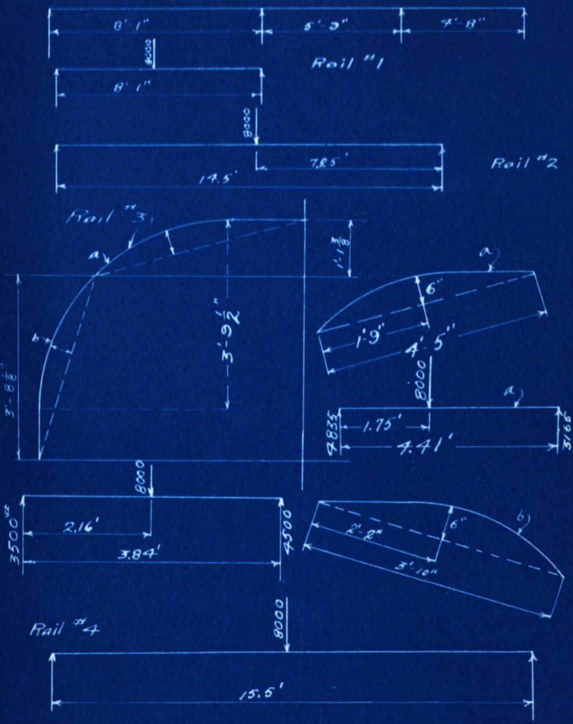


Fig. 12b

TROLLEY RAILS

Article 17.

Analysis of Cross griders of Hand Trolley support (fig. 12 Exhibit e)

Analysis of D16 8" - I @ 20.25# (See Fig. 12e)

Max. load on D16 = 8000#

Max. moment = $\frac{8000 \times 4}{4}$ = 8000#

Max. shear = 4000 plus 2 x 20.25 = 4030.5#.

M_R (by S.H.B.) = 20,000'#

Analysis of D14 8" - I @ 20.25#

Max. load on D14 = 8000#.

Max. moment = 4540 x 3.79 = 17,200'#

M_R = 20,000'#.

Max. shear = 4540#.

Analysis of D13 when trolley is at end of track. 8" I @20.25#.

Taking moments about O.

-2400 x 1.35 plus 177 x 4.38 - 2400 x 8.5 = R_2 x 8.75

R_2 = $\frac{22620}{8.75}$ = 2580#

R_1 = 3043#.

Max. M = M_A = 3043 x 1.35 - $\frac{20.25 \times 1.35^2}{2}$ = 2534'#

M_R = 20,000'#.

Max shear 2580.

Analysis of D 12 8" - I @ 18#.

Max. load on D 12 8000#.

Max. Moment 4410 x 3.79 = 16700'#.

M_R (S.H.B.) = 19,000'#

Max. shear = 4410.

Analysis of D 11 8" I @ 18#. See Fig. 12e.

Max. load on D11 8000#.

Max. moment is 4360 x 3.79 or 16,500'#.

M_R (S.H.B. p. 24) = 19,000'#.

Max. Shear = 4360#.

Analysis of D 5 8# I @ 18#.

Max. Load 8000#.

Max. Moment is 4850 x 4.25 or 20,500.

M_R = 19000'#.

Max. Shear = 4850#.

Analysis of D 3 8"-I @ 18#.

Max. Load 8000#.

Max. moment is 3980 x 5.42 or 21,600'#.

M_R = 19000'#.

Max. shear = 4020.

The beam D3 hasn't a large enough section.

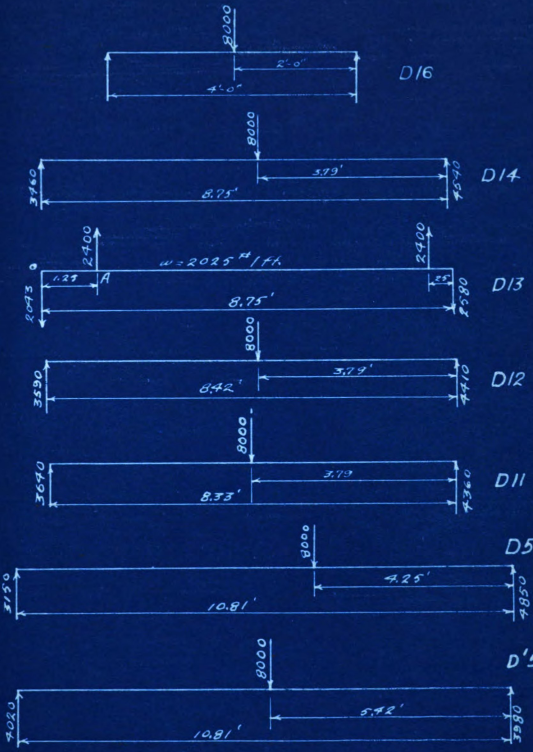


Fig. 12c

TROLLEY CROSS GIRDERS

Article 18.

Analysis of Beams for Hand Trolley Support. (See Fig. 12, Exhibit C and 12a).

Analysis of D'11   8" I @ 18#.

Max. Moment is 1290 x 4.92 or 6350'#.

M_R = 19000'#.

Max. shear is 1860#.

Analysis of D8.   8" I @ 18#.

Max. M neglecting the braces is 2520 x 6.44 or 16,200'#.

M_R is 19,000'#.

Analysis of D18 -   8" I @ 22.75#.

$R_1 = \frac{15}{18} \times 4000 = 3333$#

$R_2 = 3/18 \times 4000 = 667$#.

Max. M = 3333 x 3.0 = 9,999'#

M_R = 21,000'#.

Analysis ofD8 when there is an uplift of 2043# 6'-5" from right end of beam.

Left reaction = $\frac{6.4}{14.5} \times 2043 = 900$#.

Right reaction = 2043 - 900 = 1143#.

Max. moment is 1143 x 6.4 = 7320'#.

M_R = 19000'#.

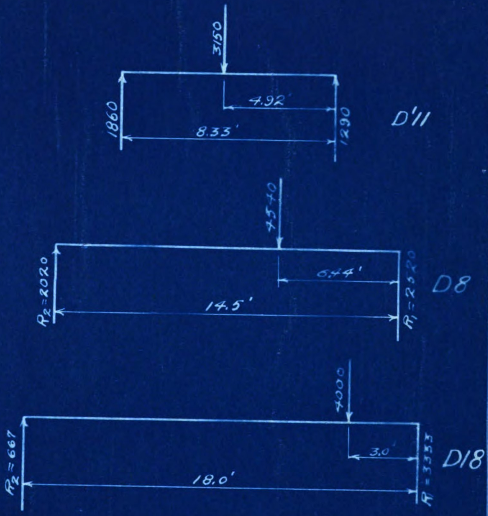

3150
4.92'
1860
8.33'
1290
D'11
4540
6.44'
R2=2020
14.5'
R1=2520
D8
4000
3.0'
R2=667
18.0'
R1=3333
D18

Fig 12a

Article 19.

Analysis of columns D1 and D4 inclusive, Hand Trolley Support. (See Fig. 19, Exhibit C).

Max. load on D2 is 4360# (by Art. 17)

Length equals 12.44'.

D2 is an 8" 23.75# I beam.

Least radius of gyration is .81.

$\frac{l}{r} = \frac{12.44}{.81} = 15.4$

$A = 6.76$ sq. in.

$P = 4360.$

(A. H. B. p. 307) $\phi = \frac{1.25}{25,000}$

$\frac{P}{A} = \frac{S}{1 + \phi \left(\frac{l}{r} \right)^2}$

$\frac{4360}{6.76} = \frac{S}{1.0186}$

$S = \frac{1.0186 \times 4360}{6.76} = 658\#$ / sq. in.

Bearing value of concrete is 400# / sq. in. (by A. H. B. p. 515)

Bearing area is 10" x 12".

Allowed bearing then is 12 x 10 x 400 or 48,000#.

Analysis of D1

Max. load on D1 is 1860#.

This column is the same length and of the same section as D2 and need not be analyzed since D2 was found to be very safe.

Analysis of D3 8" - 23.75# I beam.

Max. load on D3 occurs when the trolley is under D3 and is equal to 3333#.

Length is the same as D2 and the load is less. Therefore it is seen that it is safe.

Analysis of D4   8" I @ 22.75#.

$L = 15.1$

Max. load occurs when the trolley is at the end of the track and is equal to 6400#.

$r = .81.$

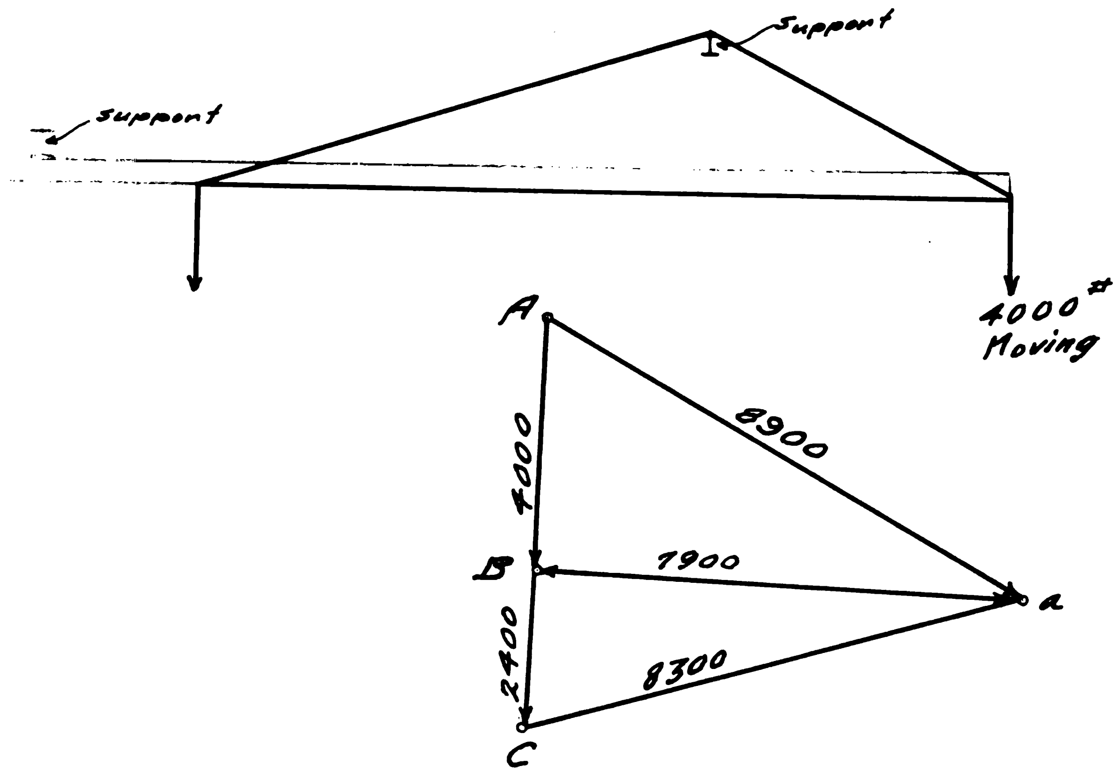
$$\frac{l}{r} = \frac{15.1}{.81} = 18.6$$

$$\frac{6400}{6.76} = \frac{s}{1 + \frac{1.44}{25,000}(18.6)^2}$$

$s = \frac{6400 \times 1.02}{6.76} = 965$ # / sq. in. which is safe.

Article 20.

Analysis of Bracing and tie rods. Fig. 12, Exhibit C.



Max. stress in aA which is a 1-1/8" tie rod is 8900#.

Allowed stress is 0.994 x 14000 or 13,900#.

Max. stress in aC which is a 1" rod is 8300#.

Allowed stress is .785 x 14,000 or 11,000#.

The uplift on each end of D13 is 2400# which is resisted by four ½" rivets which are good for 4 x .196 x 16000 or 12500#.

Analysis of bracing under a load due to trackage equal to 10% of the trolley load. (See fig. 13).

Member	Thrust	Shear	Moment.
Aa	plus 330	190	1188'#
Ac	plus 100	0	1188'#
cb	plus 310	- -	- -
cd	- 400	100	686'#
ee	- 300	340	735'#
gh	0	100	735'#
hi	- 310	- -	- -
hB	- 100	0	1360'#
jB	-335	200	1188'#

Resisting moment of the 8" I beam is 19,000'# which is safe.

The braces D 19 carry a compression of 310# or a tension of 310#.

D 19 - 2 x 2 x 3/16 L

$l = 7.65$

$r = .62$ $\frac{l}{r} = 12.35$

Allowed pressure by A. R. E. A. formula (S.H.B. p. 80) 14000# / sq. in.

Allowed pressure = .71 x 14000 = 9,950#, which is safe.

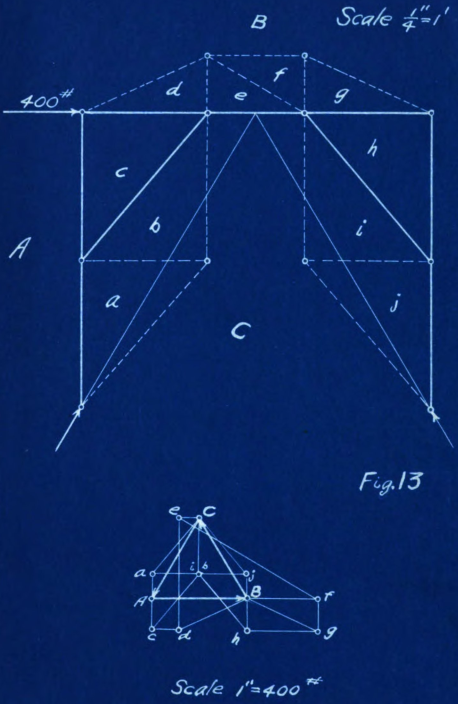

Scale $\frac{1}{4}'' = 1'$
B
400#
d
e
f
g
c
h
b
i
A
a
j
C
e
C
a
i
b
j
A
B
f
c
d
h
g
Scale 1" = 400#

Fig. 13

Article 21. SUMMARY.

The fibre stress in the roof purlins was found to be 924# per square inch while the allowable fibre stress = 1040# per sq. inch.

The fibre stress in the rafters was 330# per sq. in. which is well within allowed stress.

All members of the roof truss were found to be very safe.

The pressure of wall footing on the earth was found to be less than the allowed value of 4000# per sq. foot.

The reinforced concrete lintel is safe both for flexure and shear.

The stress in the fibering is 1100# per sq. inch, which is slightly too large.

The floor beams were all found to be strong enough but some had a factor of safety larger than necessary.

The stress in the pipe columns is well within the allowed stress.

The stress in the built columns is well within the allowed stress.

All members of the elevator framing were within allowed stress.

The bearing pressure of the column footings on the earth is 3.1 tons per square foot which exceeds the allowed value of 1.75 tons. This would indicate that the column footings are too small not knowing the practical conditions or the method of determining the safe bearing load.

The walls and buttresses of the oil sump were found to exert a bearing pressure on the soil which does not exceed the allowed value .

The trolley rails were found to be safe.

The Max. mom. in the cross girder D6 of the Trolley slightly exceeds the allowable resisting moment.

The beams for the Hand Trolley support are all well within the allowed stress.

The stress in the columns of the Hand Trolley Support was found to be well within the allowed unit.

The stress in the bracing and tie rods of the Trolley was found to be within the allowed units.

Pocket has: 3 Exhibits