

THESIS

Tell Bros. Terminal
for Lansing, Michigan

**SUPPLEMENTARY
MATERIAL**
IN BACK OF BOOK

The Design of a Bus Terminal
for Lansing, Michigan.

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

By
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Master of Science.
June 1932.

Acknowledgement.

The author of this thesis wishes to acknowledge the valuable assistance of Professor C. L. Allen in the preparation of the material presented.

W.B.E.

Table of contents.

Introduction

I Description of the building and the materials used.

II Design

Roof beams

3rd floor beams

2nd floor beams

1st floor beams & abutments

Miscellaneous

III Drawings

Conclusion.

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Introduction

The thesis herewith presented was chosen for two reasons. firstly, it presents two features of Civil Engineering construction; namely a building and a bridge, and secondly, it offers a structure which has long been needed in the city of Lansing.

Part One

**A Description of the building
and the materials used.**

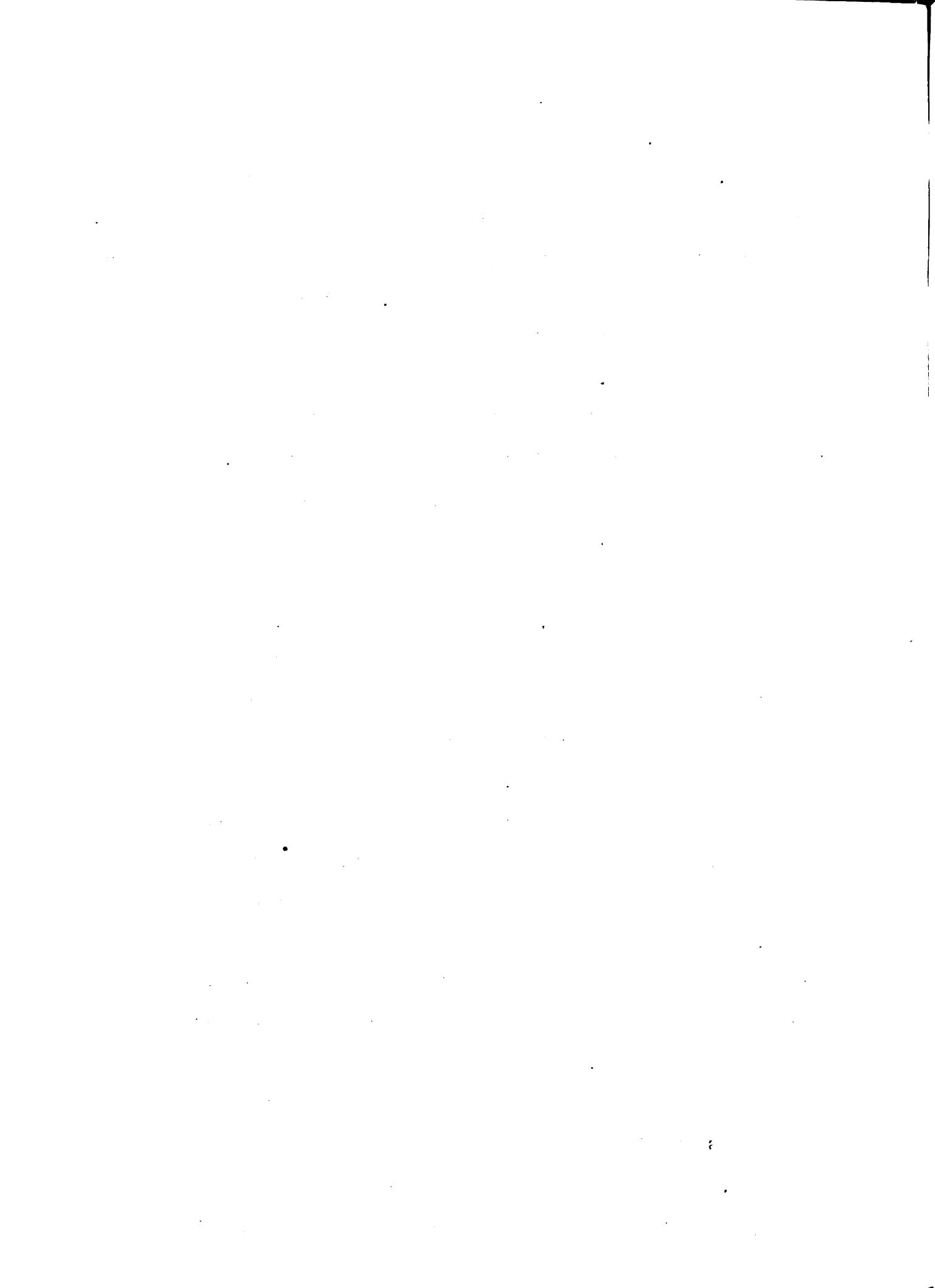
The structure, the design of which is presented in this thesis, is to be used as a bus terminal and office building. The lower floor will serve as a bus garage and loading station and as a waiting room for the bus passengers. The upper two floors may be used as offices for the various bus companies occupying the building, club rooms for the bus drivers or offices for companies other than the bus operating concerns.

The building is situated on a bridge spanning the Grand River at the foot of East Ottawa Street. It is one hundred and eighty feet long, seventy feet wide and three stories high.

The design was undertaken with the intention of securing a light weight, fire-proof structure.

The roof of the building is composed of tar and asbestos built up roofing on corrugated aluminum sheets supported on lightweight steel trusses which are in turn supported on steel I beams. The side panels are fifteen feet by fifteen feet with the trusses spaced at five foot intervals. The center panels are ten feet by fifteen feet with trusses fixed at the middle of the ten foot distance. A plaster ceiling supported on metal lath is hung from the underside of the roof frame to provide a dead air space for insulation and to provide fire protection for the roof itself.

The floors are of the American Institute of Steel Construction Battledock floor construction "T" sections, composed of steel plates welded to small I beams spaced twenty-four inches center to center. These



sections are welded to the floor frame and are protected from fire on the under side by patented fireproof blocks which are applied after the floor is laid. The surfaces of the upper floors and of the waiting room are covered with linoleum, laid directly upon the steel plates. The floor in the garage section of the first floor is covered with two inches of asphalt which serves as a cushion for the heavy loads and also deadens the noise.

The exterior walls of the building are of hollow-tile faced with limestone with steel sash windows and aluminum spandrels.

The interior walls are of hollow gypsum block, plastered on one or both sides as necessary.

The columns, like the rest of the frame, are of rolled steel sections being fastened by means of welded joints.

The piers, footings and abutments are of reinforced concrete, the west abutment having two openings through it to allow for outlets for storm sewers which are laid down Ottawa Street.

The elevator is a one thousand pound steel cab type with an overhead drive using a one to one wrap traction system. The machinery and cab are supported by means of steel columns, and the shaft is of six inch terra cotta tile blocks.

The stairs are of steel with tile covered treads.

While a heating system for the building has not been designed, the use of aluminum spandrels provides convenient space for the new convector type heaters in the

offices and waiting room, while unit heaters would probably be the most satisfactory in the garage section. Since the building has no basement, the most desirable method of heating would be to purchase heat. This could be conveniently done, as the city heating plant which supplies steam to downtown buildings is on the adjoining property.

Part Two

Design

Roof beams

The first step in the actual design of the building is the design of the roof system. The roof itself is composed of tar and asbestos roofing on corrugated aluminum sheets. The panels are 15 feet by 15 feet with light steel trusses 5 feet c. to c.

The load on the roof is as follows:

Weight of roofing	- 2 pounds per sq. ft.
Weight of sheet aluminum	- 1 pound " " "
Live load (Lansing Bldg. Code)	- <u>30</u> pounds " " "
The total load is	33 pounds per sq. ft.

The weight of the suspended ceiling is 10 pounds per square foot making a total load upon the roof system of 43 pounds per square foot.

The allowable load on B & S 16 gage corrugated aluminum supported at 5 foot intervals is 39 pounds per square foot which makes this type of roof safe to use.

The framing plan and the design of the roof beams follows.

Note: The number of inches of 3/8" fillet weld required for any joint equals the reaction divided by 3000.

DESIGN OF BEAM TYPE R.B.N.

Load @ each truss point

$$15 \times 5 \times 33 = 2475\#$$

$$15 \times 5 \times 10 = 750\#$$

$$M_{max} = 3225 \times 5 \times 12 = 193,500 \quad \text{from a to b}$$

Assume wt. of beam = 6# /ft.

$$M_b = \frac{6 \times 15 \times 15 \times 12}{8} = 2025\#$$

$$\text{Total } M = 195,525$$

$$S = \frac{195,525}{18,000} = 10.9$$

use 7" 17.5# std. I

$$s = 11.11$$

DESIGN OF BEAM TYPE R.B.E.

$$\text{Ceiling} = 10 \times 5 = 50$$

$$\text{Load} = 33 \times 5 = 165\# \text{ per lin. ft.}$$

Assume wt. of beam 5# per lin. ft.

$$\text{Total load} = 220\# \text{ per lin. ft.}$$

$$M = \frac{110}{220} \times 15 \times 15 \times 12 = 74,500\#$$

$$S = \frac{74,500}{18,000} = 4.15$$

use 5", 10# std. I

$$s = 4.84$$

DESIGN OF BEAM TYPE R.B.M.

$$15 \times 5 \times 10 = 750$$

$$\text{Con Load} = 15 \times 5 \times 33 = 2975$$

$$M_{max} = 3225 \times \frac{10}{4} \times 12 = 96,750$$

$$M_b = \frac{5 \times 10 \times 10 \times 12}{8} = 750$$

use 6" 12.5# Std. I

$$97,500$$

$$s = 7.27$$

$$s = \frac{97,500}{18,000} = 5.42$$

DESIGN OF BEAM TYPE R.B.H.

assume wt. = 4#

$$\text{Load} = 47 \times 2.5 = 117.5\# / \text{ft.}$$

$$M = \frac{117.5 \times 11 \times 11 \times 12}{8} = 21,300\#$$

$$S = \frac{21,300}{18,000} = 1.19$$

use 3", 5.7# std. I.

$$s = 1.67$$

DESIGN OF BEAM TYPE R.B.I.

ass. wt. = 4"

$$\text{Load} = 47 \times 5 = 235\# / \text{ft}$$

$$M = \frac{235 \times 15 \times 15 \times 12}{8} = 79,300$$

$$S = \frac{79,300}{18,000} = 4.41$$

use 5", 10# std. I.

$$s = 4.84$$

DESIGN OF BEAM TYPE R.B.J.

assume wt. = 5#

$$\text{Load} = 48 \times 5 = 240\# / \text{ft.}$$

$$M = 240 \times 11 \times 11 \times 12 = 43,500\#$$

$$S = \frac{43500}{18000} = 2.42$$

use 4" - 7.7 std. I.

$$s = 3.0$$

DESIGN OF BEAM TYPE R.B.Q.

assume wt. 5" /ft

$$\text{Load} = 48 \times 5\frac{1}{2} = 264\# / \text{ft.}$$

$$M = \frac{264 \times 11 \times 11 \times 12}{8} = 48000\#$$

use 4"x 7.7# std. I.

$$s = 3.0$$

$$S = \frac{48,000}{18,000} = 2.67$$

DESIGN OF BEAM TYPE R.B.V.

Assume wt. = 5# /ft

$$\text{Load} = 48 \times 5 = 240\# / \text{ft.}$$

$$M = \frac{240 \times 11 \times 11 \times 12}{8} = 43,500$$

use 4", 7.7# std. I.

$$s = 3.$$

$$S = \frac{43500}{18000} = 2.42$$

DESIGN OF BEAM TYPE R.B.R.

assume wt. = 6#

$$\text{Conc Load} = (8.5 + 6.5) \quad 5 \times 43 = 3225$$

$$M_{\max} = 195,525$$

$$s = 10.9$$

use 8" - 18.4# std. I.

$$s = 4.22$$

DESIGN OF BEAM TYPE R.B.T.

$$\text{Conc Loads} = 3225$$

wt. of beam = 8#

$$M_{\text{conc}} = 3225 \times 2 \times 12 = 387,000$$

$$M_b = \frac{8 \times 20 \times 20 \times 12}{8} = 4,800$$

$$M_{\max} = 391,800$$

use 10" 25.4# std. I.

$$s = \frac{391800}{18000} = 21.7$$

$$s = 24.42$$

DESIGN OF BEAM TYPE R.B.S.

assume wt. of beam 5# /ft

$$\text{Conc Loads} = (\text{assume } 2' \text{ supp. by beam})$$

$$-43 \times 2 \times 5 = 430\#$$

$$M = 430 \times 5 \times 12 = 25,800$$

$$M_b = \frac{5 \times 16 \times 16 \times 12}{8} = 1,920$$

$$M_{\max} = 27,720$$

use 3" 5.7# std. I.

$$s = \frac{27720}{18000} = 1.54$$

$$s = 1.67$$

DESIGN OF BEAM TYPE R.B.U.

ass. wt. = 5# /ft

$$\text{Conc Load (assume } 6' \text{ supported by beam)}$$

$$= 43 \times 6 \times 5 = 1,290$$

$$M = 1290 \times 5 \times 12 = 77,400$$

$$M_b = \frac{5 \times 16 \times 16 \times 12}{8} = 1,920$$

$$M_{\max} = 79,320$$

use 5", 10# std. I.

$$s = \frac{79320}{18000} = 4.42$$

$$s = 4.84$$

DESIGN OF BEAM TYPE R.B.F. assume wt. of beam = 5# /ft

Uniform Load $33 \times 2.5 + 10 \times 2.5 + 5 = 112.5\# /ft$

$$M = \frac{112.5 \times 15 \times 15 \times 12}{8} = 38,100$$

$$s = \frac{38,100}{18,000} = 2.12$$

use 4", 7.7# std. I

$$s = 3.0$$

DESIGN OF BEAM TYPE R.B.W. assume wt. of beam 7# /ft

Conc. Loads = $(10 \times 7.5 \times 5) + (33 \times 7.5 \times 5) + (10 \times 9 \times 5) + 33(9 \times 5) = 3545$

$$M_{con} = 3545 \times 5 \times 12 = 212,700\#$$

$$M_b = \frac{7 \times 15 \times 15 \times 12}{8} = 2,381\#$$

$$M_{max} = 215,081 \#$$

$$s = \frac{215,081}{18,000} = 12.0$$

use 8", 18.4# std. I

$$s = 14.22$$

DESIGN OF BEAM TYPE R.B.K. assume wt. of beam 5# per ft.

Conc. loads = $(10 \ 33) \ (5) \ (9) = 1935$

$$M_{con} = 1935 \times 5 \times 12 = 116,100$$

$$M_b = \frac{5 \times 15 \times 15 \times 12}{8} = 1,690$$

$$M_{max} = 117,790$$

$$s = \frac{117,790}{18,000} = 6.55$$

use 6", 12.5# std. I

$$s = 7.27$$

DESIGN OF BEAM TYPE R.B.L. assume wt. of beam 5# /ft

Conc. loads $43 \times 6.5 \times 5 = 1400$

$$M_{con} = 1400 \times 5 \times 12 = 84,000$$

$$M_b = \frac{5 \times 15 \times 15 \times 12}{8} = 1,690$$

$$M = 85,690$$

$$s = \frac{85,690}{18,000} = 4.76$$

use 5#, 10# std. I

$$s = 4.84$$

DESIGN OF BEAM TYPE R.B.P. assume wt. of beam = 6# /ft.

$$\text{Conc. loads} = 43 \times 6.5 \times 5 = 1400$$

$$M_{\text{con.}} = (2100 \times 10 - 1400 \times 5)12 = 168,000 \text{ "#}$$

$$M_b = \frac{6 \times 20 \times 20 \times 12}{8} = 3,600$$

$$M_{\text{max}} = 171,600 \text{ "#}$$

$$s = \frac{171600}{18000} = 9.55$$

use 7" 15.3# std. 1.

$$s = 10.34$$

DESIGN OF BEAM TYPE R.B.A. assume wt. of beam = 5# /ft

$$\text{uniform load} = 43 \times 2.5 = 108$$

$$\text{wt. of beam} = 5$$

$$M = \frac{113 \times 18 \times 18 \times 12}{8} = 55,000 \text{ use 4" 8.5# std. 1}$$

$$s = \frac{55000}{18000} = 3.06$$

$$s = 3.15$$

DESIGN OF BEAM TYPE R.B.B.

assume wt. 5# /ft.

$$\text{Uniform load} = 43 \times 5 = 215\#$$

$$\text{wt. of beam} = \frac{5\#}{220}$$

$$M = \frac{220 \times 18 \times 18 \times 12}{8} = 107,000 \text{ use 6", 12.5# std. 1.}$$

$$s = \frac{107,000}{18,000} = 6.5$$

$$s = 7.27$$

DESIGN OF BEAM TYPE R.B.C.

assume wt. 5# /ft

$$\text{uniform load} = 43 \times 25 = 108$$

$$\text{wt. of beam} = 5\#$$

$$M = \frac{113 \times 13 \times 13 \times 12}{8} = 50,700 \text{ use 4", 7.7# std. 1}$$

$$s = \frac{50700}{18000} = 2.82$$

$$s = 3.0$$

DESIGN OF BEAM TYPE R.B.D.

assume wt. = 10#

Uniform load = 220#

$$M = \frac{220 \times 13 \times 13 \times 12}{8} = 89,600$$

use 5", 12.75 std. I

$$S = \frac{89600}{18000} = 4.97$$

$$s = 5.40$$

DESIGN OF BEAM TYPE R.B.Y.

assume wt. = 5# /ft

Assume beam supports 10'

$$\text{Conc. load} = 43 \times 5 \times 10 = 2150$$

$$M_{\text{con}} = 2150 \times 5 \times 12 = 129,000 \text{ " \#}$$

$$M_b = \frac{5 \times 15 \times 15 \times 12}{8} = 1,690$$

$$M_{\text{max}} = 130,690$$

Use 6", 12.5# std. I

$$S = \frac{130,690}{18000} = 7.14$$

$$s = 7.27$$

DESIGN OF BEAM TYPE R.B.Z.

assume wt. = 6# /ft

Assume Beam supports 13'

$$\text{Conc. load} = 43 \times 5 \times 13 = 2800$$

$$M_{\text{con}} = 2800 \times 12 \times 5 = 168,000 \text{ " \#}$$

$$M_b = \frac{6 \times 15 \times 15 \times 12}{8} = 2,025 \text{ " \#}$$

$$M_{\text{max}} = 170,025$$

use 7", 15.3# std. I

$$S = \frac{170025}{18000} = 9.5$$

$$s = 10.34$$

DESIGN OF BEAM TYPE R.B.X.

assume wt. = 5# /ft

Assume beam supports 7'

$$\text{Con. load} = 43 \times 7 \times 5 = 1510$$

$$M_{\text{con.}} = 1510 \times 5 \times 12 = 90600$$

$$M_b = \frac{5 \times 11 \times 11 \times 12}{8} = 910$$

$$M_{\text{max}} = 91510$$

use 5", 12.25# std. I

$$S = 91510 = 5.1$$

$$s = 5.40$$

DESIGN OF BEAM TYPE R.B.G.

assume wt = 5# /ft

$$\text{Load} = 5 (48) = 240 \# /ft$$

$$M = 240 \times 5.5 \times 5.5 \times 12 = 10900\#$$

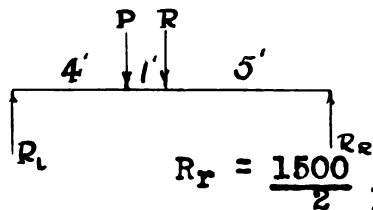
$$S = \frac{10900}{18000} = .61$$

use 3" 5.7# std. I.

$$s = 1.67$$

DESIGN OF BEAM TYPE RB.O.

assume wt. = 5# /ft



$$R = 40 \times 5 \times 7.5 = 1500\#$$

$$P = 43 \times 6.5 \times 5 = 1400\#$$

$$R_R = \frac{1500}{2} + \frac{4}{10} \times 1400 = 1310$$

$$M = 1310 \times 5 \times 12 = 78,600$$

$$M_d = \frac{5 \times 10 \times 10 \times 12}{8} = 750$$

$$M = 79,350$$

$$S = \frac{79350}{18000} = 4.41$$

use 5", 10.0# Std. b

$$s = 4.84$$

Third floor beams

The next step to be considered in the design is that of the third floor system. The type floor used is the A.I.S.C. Battledock floor with linoleum covering and patented fireproof ceiling.

There are two types of floor required, one a 3 inch beam with 3/16 inch plate where the span is fifteen feet and the other a 4 inch beam with 3/16 inch plate where the span is eighteen feet.

The load upon the section having the fifteen foot span is as follows:

Weight of floor	10.5 pounds per square foot
Weight of fireproofing	13.0 " " " "
Live load (Lans. Bldg. Code)	<u>50.</u> " " " "
Total load	73.5 pounds per square foot
Tabulate safe load	85.0 " " " "

The load upon the section having the eighteen foot span is as follows:

Weight of floor	11.5 pounds per square foot
Weight of fireproofing	13.0 " " " "
Live load	<u>50</u> " " " "
Total load	74.5 pounds per square foot
Tabulated safe load	100 " " " "

A check upon the safety of the 3 inch floor follows, the section used being one "T" section 24 inches wide and 3 3/16 inches deep.

$$L.L. = 100\# / \text{lineal foot}$$

$$\text{Max. L.L. moment} = \frac{1}{12} \times 100 \times 15 \times 15 \times 12 = 22,500\#$$

$$\text{Max. D.L. moment} = \frac{1}{12} \times 23.5 \times 15 \times 15 \times 12 = 5,300\#$$

$$\text{Max. Total moment} = 27,800\text{"}\#$$

$$I_{c.g.} \text{ of plate} = \frac{1}{12} \times 24 \times \left(\frac{3}{16}\right)^3 = .00132$$

$$A \text{ of plate} = 24 \times 3/16 = 4.5 \text{ square inches}$$

$$I_{c.g.} \text{ of I beam} = 2.5 \quad A = 1.64 \text{ " "}$$

$$\bar{Y} \text{ of plate} = 3.09375$$

$$\bar{Y} \text{ of I beam} = 1.5$$

$$4.5 \times 3.094 \quad 1.64 \times 1.5 = 6.14 \times C$$

$$C = 2.668$$

$$A = .425$$

$$I = .00132 \quad 4.5 \quad (.425)^2 \quad 2.5 \quad 1.64 \quad (1.168)^2$$

$$I = 5.55$$

$$F_s = \frac{27,800 \times 2.668}{5.55} = 13,480 \text{ } \# / \text{sq. in.}$$

$$\text{Allowable } f_s = 18,000$$

The framing plan and the design of the third floor beams follows.

Note: The number of inches of 3/8" fillet weld required for any joint equals the reaction divided by 3000.

Floor Beam Type F.B.3N.

$$\text{Total load /ft} = 73.5\# \text{ on floor}$$

$$\text{load /ft on frame} = 73.5 \times 15 = 1100\#$$

$$\text{assume wt. of beam} = 20\# / \text{ft}$$

$$\text{wt. of wall (4" hollow Gypswm 12' high with plaster)} = 336\# / \text{ft.}$$

100

$$M = \frac{1}{12} \times 1460 \times 15 \times 15 \times 12 = 328,500''\# \times \frac{12}{8}$$

$$S = \frac{328,500 \times 12}{18,000 \times 8} = 27.4 \quad \text{use 10" 35\# std. I beam}$$

$$s = 29.16$$

Floor Beam Type FB3M

Total load /ft on floor = 73.5# load on frame = 1100#

Assume wt. of beam = 15# /ft

Total load 1115# /ft.

$$s = \frac{1115 \times 10 \times 10 \times 12}{12 \times 18000} = 6.2 \times \frac{12}{8} = 9.3$$

use 7, 15.3# std. I beams = 10.34

Floor Beam Type FB3W

Floor load /ft. on frame = (74.5 x 9) (73.5 x 7.5) = 1230

Assume wt. of beams = 25# /ft

Wt. of wall = 336# /ft

$$M = \frac{1}{12} \times 1590 \times 15 \times 15 \times 12 = 358000''\# \times \frac{12}{8}$$

$$s = \frac{358,000 \times 12}{18,000 \times 8} = 28.4 \quad \text{use 10' 35 std. I beam}$$

$$s = 29.16$$

Floor Beam Type FB3E1

3" beam as reqd. by floor

Floor Beam Type FB3G

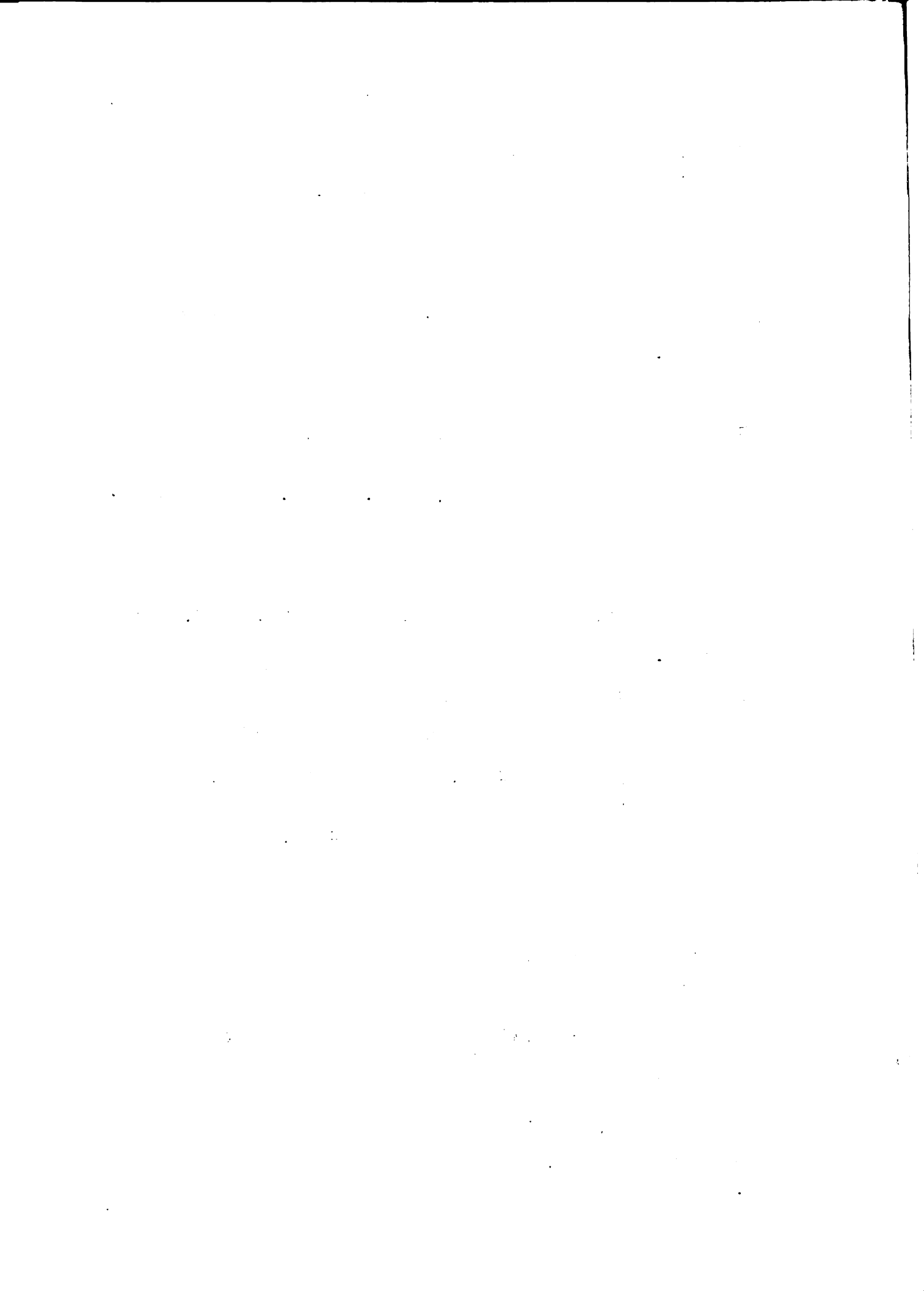
3" beam as reqd. by floor (see also FB3G1)

Floor beam Type FB3D

3" Beam as reqd. by floor

Floor beam Type FB3B

4" Beam as reqd. by floor



Floor Beam Type FB3E

assume wt. 10# /ft.

wall load = 336# /ft

$$M = \frac{1}{12} \times 350 \times 15 \times 15 \times 12 = 74,200" \# \times 12/8$$

use 6" 12.5# std. I beam

$$S = \frac{74200}{18000} = 4.12 \times \frac{12}{8} = 6.17 \quad s = 7.27$$

Floor Beam Type FB3K

Floor load /ft = (73.5 x 6.5) (74.5 x 9) = 1148#/'

Assume wt. of beam = 25#

Wt. of wall = 336 #/'

Total load 1510 #/'

$$S = \frac{1510 \times 15 \times 15 \times 12}{12 \times 18000} \times \frac{12}{8} = 28.4 \quad \text{use 10" 35#/' std. beam}$$

$$s = 29.16$$

Floor Beam Type FB3Q

assume wt. of beam = 10# /'

Wall load = 336#/'

$$S = \frac{350 \times 11 \times 11 \times 12}{8 \times 18000} = 3.52 \quad \text{use 5", 10# std. I beam}$$

$$s = 4.84$$

Floor Beam Type FB3I

same as FB3E 6" 12.5# std. I beam

Floor Beam Type FB3Y

Assume wt. of beam = 15# /ft

Assume 12' of floor carried by beam (max)

$$s = \frac{900 \times 12 \times 12 \times 12}{8 \times 18,000} = 10.8 \quad \text{use 7" 17.5# std. I beam}$$

$$s = 11.11$$

Computations for load on wall supporting beam

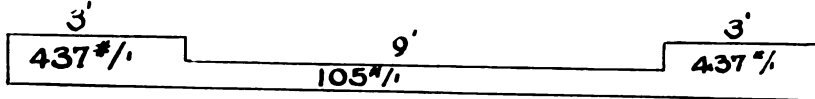
wt. of wall 6 x 10 x 32 = 1920#

wt. of windows = 10 x 9 x 5.5 = 495#

$$\text{Wt. of spandrel } \frac{9 \times 3.5 \times 1}{12} \times 170 = 450\#$$

$$\frac{1}{4} \text{ wt. of stone facing} = \frac{1}{4} \times \frac{6 \times 10 \times 2}{12} \times 160 = 400\#$$

$$\text{Wt. of plaster} = 6 \times 10 \times 5 = 300\#$$

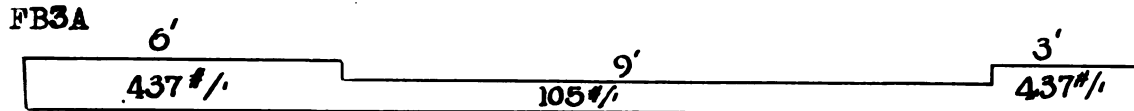


$$R = 1785$$

$$M = (1785 \times 7.5 - 437 \times 3 \times 6 - \frac{105 \times 4.5 \times 4.5}{2}) 12$$

$$\text{F.B.3F. } M = 53,800 \quad S = \frac{53,800}{18,000} = 3 \quad \text{use 4", 8.5\# std. I}$$

$$s = 3.45$$



$$R_L = 2680\#$$

$$R_R = 2200$$

$$M_{\max} = (2680 \times 6.57 - 437) \times 6 \times 3.57) 12 = 99,500$$

$$S = \frac{99,500}{18,000} = 5.53 \quad \text{use 6\#, 12.5\# std. I}$$

$$s = 7.27$$

$$\text{FB3C } \text{load} = 437\# / \text{ft}$$

$$M = \frac{437 \times 13 \times 13 \times 12}{12} = 74,000\# \times \frac{12}{8}$$

$$S = \frac{74,000}{18,000} = 6.15 \quad \text{use 7" 12.5\# std. I}$$

$$s = 6.88$$

Load on FB3L

Assume wt. of beam = 15\# /ft.

$$\text{Same as FB3F} + 73.5 \times 5.5\# / \text{ft}$$

$$M = 53,800 + \frac{1}{8} \times 405 \times 15 \times 15 \times 12 = 188,800\#$$

$$M = 188,000\# \quad \text{use 7" 17.5\# std. I}$$

$$S = 10.4$$

$$s = 11.11$$

FB3J

Use 10\#, 15.3\# std. channel for stairs

$$s = 13.38$$

FB3G1

Use 10" 15.3# std. channel for stairs

$$s = 13.38$$

FB3H wall load = 437# /ft.

assume wt. = 15# /ft

$$M = \frac{452 \times 11 \times 11 \times 12}{12} = 54600" \# \times \frac{12}{8}$$

$$s = \frac{54600 \times 12}{18000} = 4.59 \quad \text{use 5", 10# std. I}$$

$$s = 4.84$$

FB3K

assume wt. = 15# /ft.

Wall load = same as FB3H assume wt. 15# /ft.

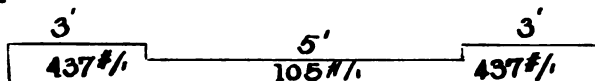
$$M \text{ wall load} = 53,800$$

$$M = \frac{(7.5 \times 89.5)}{12} \times 15 \times 15 \times 12 = 151,500 \times \frac{12}{8} = 227,500" \#$$

$$M_{\max} = 281300" \# \quad \text{use 9" 21.8# std. I}$$

$$s = \frac{281,300}{18,000} = 15.6 \quad s = 18.87$$

FB3X



assume wt. of beam = 15# /ft

$$M_{\text{floor}} = (15 + \frac{7.5 \times 73.5}{12}) \times 11 \times 11 \times 12 = 66,600" \#$$

$$M \text{ wall} = (1574 \times 5.5 - 1311 \times 4.0 - 263 \times 1.25)12 = 37,000" \#$$

$$M = 103,600" \# \quad \text{use 6" 12.5# I}$$

$$s = \frac{103,000}{18,000} = 5.75 \quad s = 7.27$$

FB3B

assume wt = 15# /ft.

Wall load = 437# /ft.

floor load assume 220# /ft

$$M = \frac{682 \times 16 \times 16 \times 12}{12} = 172,000" \# \times \frac{12}{8}$$

use 8" 20.5# std. I

$$S = \frac{172000}{18000} \times \frac{12}{8} = 14.35$$

$$s = 15.05$$

FB3V

assume wt = 15# /ft

Wall load = 437 # /ft

Floor load 588# /ft

$$M = \frac{746 \times 16 \times 16 \times 12}{12} = 191,000" \# \times \frac{12}{8} \text{ use 8" } 23\# \text{ std. I}$$

$$S = \frac{191000}{18000} \times \frac{12}{8} = 15.9$$

$$s = 16.05$$

FB3Z

assume wt. 15# /ft

Floor load = 73.5 x 11.5 = 845 # /ft

$$N = \frac{860 \times 15 \times 15 \times 12}{12} = 194,000" \# \times \frac{12}{8}$$

$$S = \frac{194000}{18000} \times \frac{12}{8} = 16.2$$

use 9" 21.8# std. I

$$s = 18.87$$

XB3A

assume wt. = 10# /ft.

Wt. of wall 280# /ft

$$M = \frac{1}{12} \times 290 \times 13 \times 13 \times 12 = 49000" /ft \times \frac{12}{8}$$

$$S = \frac{49000}{18000} \times \frac{12}{8} = 4.18 \text{ use 5", 10# I}$$

$$s = 4.84$$

XB3D

wt. of wall 280# /ft

assume wt. 10# /ft

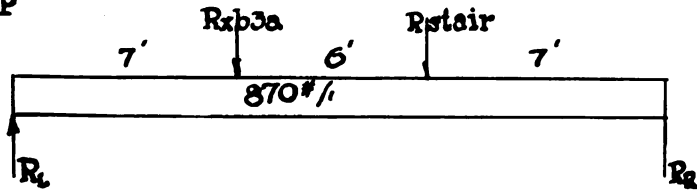
$$M = \frac{1}{12} \times 290 \times 6 \times 6 \times 12 = 10500" \# \times \frac{12}{8}$$

use 3", 5.7# std. I

$$S = \frac{10,500}{18,000} \times \frac{12}{8} = .9$$

$$s = 1.67$$

FB3P



assume 30" /ft

$$R_{xb3a} = 1595$$

$$R_{stair} = 2950$$

$$R_L = 8700 + \frac{13}{20} \times 1595 + \frac{7}{20} \times 2950 = 10770$$

$$R_R = 11,175$$

Max moment occurs 10.54' from R_L

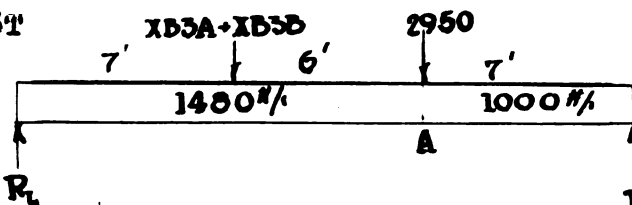
$$M = (10,770 \times 10.54 - 1595 \times 3.54 - 870 \times 10.54 \times 5.27)12 = 718,200 \text{ "#}$$

$$S = \frac{718,200}{18,000} = 40.5$$

use 12" std. I beam 40.8# /ft.

$$s = 44.82$$

FB3T



floor load left of A = 1148# /ft

" " right of A = 670# /ft

Wall load = 280 #/ft

Wt. of beam = 50# /ft

$$R_L = \frac{13.5}{20} \times 13 \times 1480 + \frac{13}{20} \times 5985 + \frac{7}{20} \times 2950 + \frac{3.5}{20} \times 7000 = 18,850$$

$$Xb3A + Xb3B = 5985 \#$$

$$R_R = 16325$$

Max Moment occurs 8.68' from R_L

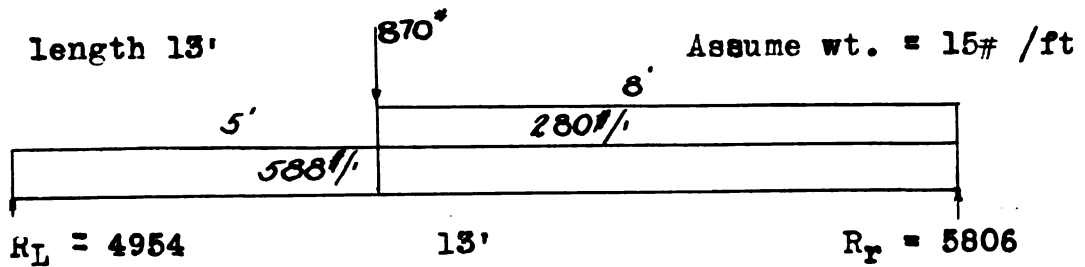
$$M = (18,850 \times 8.68 - 5985 \times 1.68 - 1480 \times 8.68 \times 4.34)12 = 1,282,000 \text{ "#}$$

$$S = \frac{1,282,000}{18000} = 71.4$$

use 14" - 58" C Beam

$$s = 78.25$$

XB3C length 13'



$$R_R = 5/13 \times 870 + \frac{9}{13} (280 \times 8) + \frac{13 \times 603}{2} = 5806$$

$$x = \frac{5806}{868} = 6.7' \text{ from rt. end}$$

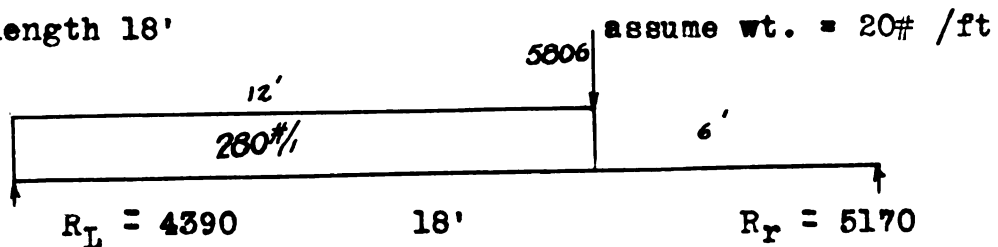
$$M_{max} (5806 \times 6.7 - 868 \times 6.7 \times \frac{6.7}{2}) 12 = 232,500''\#$$

use 8" 18.4# std. I

$$S = \frac{232500}{18000} = .12.9$$

$$s = 14.22$$

XB3B length 18'



$$R_L = 180 + \frac{1}{3} (5806) + \frac{2}{3} (12 \times 280) =$$

$$M_{max} = (5170 \times 6 - 120 \times 3) 12 = 367,920''\#$$

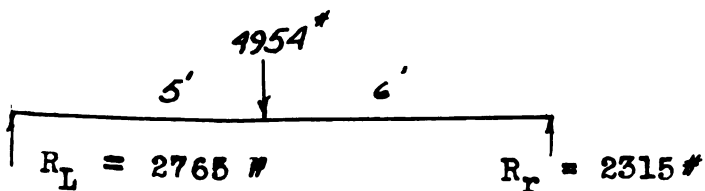
$$S = 20.4$$

use 9" 25# std I

$$s = 20.31$$

FB3V

assume wt. = 10# /ft



$$R_L = 5.5 \times 10 + \frac{6}{11} \times 4954 = 2765$$

$$R_R = 5.5 \times 10 + \frac{5}{11} \times 4954 = 2315$$

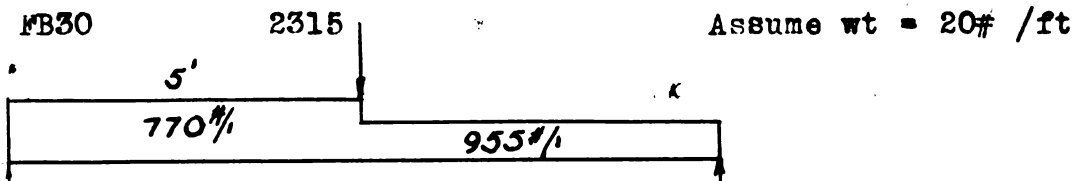
FB3V (cont.)

$$M = (2765 \times 5 - 50 \times 2.5)12 = 165,000$$

$$S = \frac{165000}{18000} = 9.15$$

use 7" 15.3# std I

$$s = 10.34$$



$$R_L = \frac{3}{4} \times 5 \times 770 + \frac{1}{4} \times 955 \times 5 + \frac{1}{2} (2315) = 5250 \quad 100 = 535$$

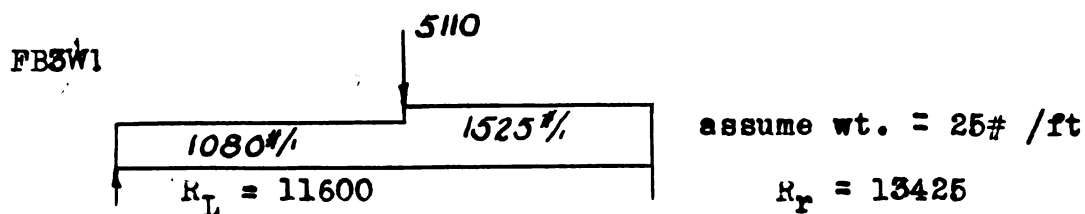
$$R_R = \frac{3}{4} \times 5 \times 955 + \frac{1}{4} \times 5 \times 770 + \frac{1}{2} (2315) = 5710 \quad 100 = 5810$$

$$M_{max} = (5350 \times 5 - 790 \times 5 \times 2.5)12 = 203,000\text{"#}$$

$$S = \frac{203,000\text{"#}}{18,000} = 11.3$$

use 7" -20# std. I

$$s = 11.97$$



$$R_L = \frac{7}{15} \times 5710 + \frac{3.5}{15} \times 7 \times 1525 + \frac{11}{15} \times 8 \times 1080 = 11,600$$

$$M = (11600 \times 8 - 8640 \times 4) 12 = 700,000\text{"#}$$

$$S = \frac{700,000}{18,000} = 38.8$$

use 12" 40.8# std. I

$$s = 44.82$$

Second floor beams

The loading and type of floor used on the second floor is the same as on the third floor with the exception of the concentrated load at the midpoint of the transverse beams caused by the column loads applied at these points. This load is taken as 50,000 pounds, computations for which are shown in the section on columns.

The framing plan and design of the second floor beams follows:

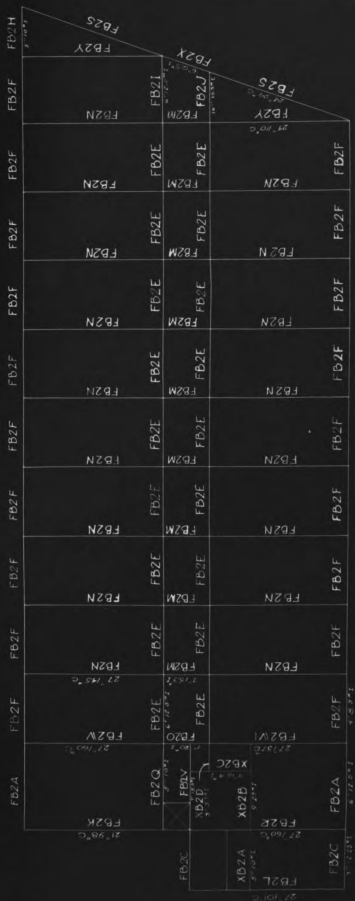
Note: The number of inches of $3/8$ " fillet weld required for any joint equals the reaction divided by 3000.

SECOND FLOOR

FRAMING PLAN

UNION BUS TERMINAL
LANSING, MICHIGAN

SCALE 3/64" = 1'-0"
W.B. EDWARDS



FLOOR BEAMS

FB2N length 30'

2ND FLOOR

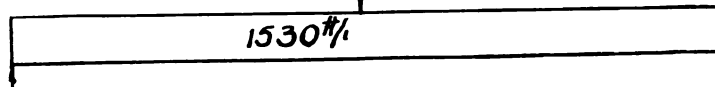
assume wt 150# /ft

Floor load = 15 x 73.5# /ft

Wall load = 280 # /ft

Wt. of beam = 150# /ft

Conc. load @ Mid pt. = 50,000#



$$M_{max} = (25,000 \times 15 + \frac{1}{8} \times 1530 \times 30 \times 30)12 = 6,865,500$$

$$s = \frac{6,865,500}{18000} = 381$$

use a 27 "C" I beam 145# /ft

R = 47,950

s = 408.05

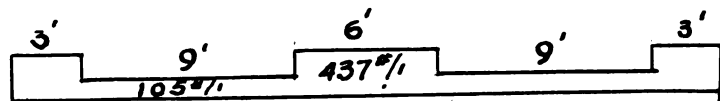
FB2K length 30'

wt. 150# /ft

Floor load = 9 x 74.5 = 670.5# /ft

Wt. of beam 150.0# /ft

Wall load



Conc. load = 2 x 7860 (FB3K) = 15720#

2 x 1990 (RBK) = 3980#

Roof column 138#

1 x 1440 (FB3B) = 1440#

1 x 1980 (RBB) = 1980#

3rd floor column

588
23846#

use 25,000 as design load

FB2K (cont.)

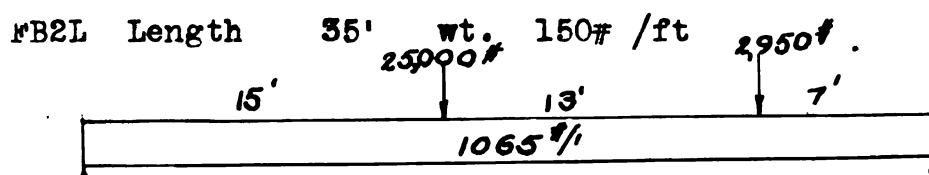
$$R = 6 \times 437 + 9 \times 105 + 15 \times 820 + \frac{1}{2} \times 25,000 = 28,367\#$$

$$M_{max} = (28,367 \times 15 - (437 \times 6 + 105 \times 9 + 820 \times 15) 7.5) \times 12 = 3,680,000\text{"}\#$$

$$S = \frac{3680,000}{18,000} = 204.2$$

use 21" 98# C Beam

$$s = 209.24$$



Location of windows uncertain-whole beam designed to carry full wall load.

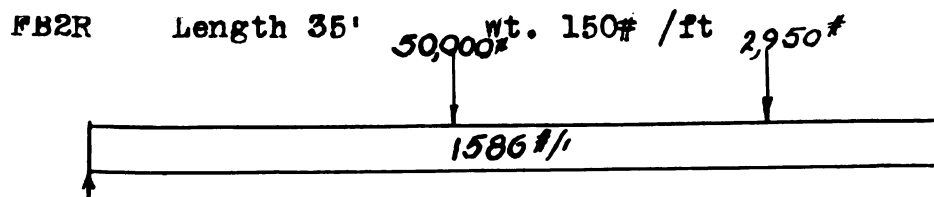
$$R_L = \frac{20}{35} \times 25000 + \frac{1065 \times 35}{2} + \frac{7}{35} \times 2950 = 33,513$$

$$R_R = 31711$$

$$M_{max} = (33513 \times 15 - 1065 \times 15 \times 7.5) 12 = 4,500,000$$

$$S = \frac{4,500,000}{18,000} = 250 \text{ use 27" - 101# C Beam}$$

$$s = 264.72$$



$$R_L = \frac{20}{35} \times 50000 + \frac{7}{35} \times 2950 + \frac{35 \times 1586}{2} = 56,910$$

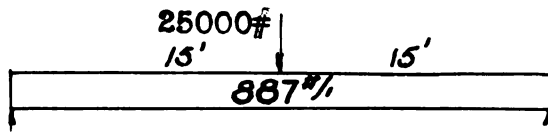
$$R_R = 51540$$

$$S = \frac{(56910 \times 15 - 1586 \times 15 \times 7.5) 12}{18000} = 449.9$$

use 27" - 160# C Beam

$$s = 450.13$$

FB2S



wt. 100# /ft

floor load 350# /ft

$$M = (25000 \times \frac{32}{4} + \frac{1}{8} \times 880 \times 32 \times 32) 12 = 3,752,400 \text{ in}^2$$

$$S = \frac{3752400}{18000} = 209.0$$

use 24" - 94# c Beam

$$s = 225.02$$

$$R = 26,580\#$$

FB2Y use a concentrated load of

37,500# @ mid point

Wall load = 280# /ft.

assume wt = 100#

let floor load = 735# /ft (10 sq. ft. of floor)

$$M = (37500 \times \frac{30}{4} + \frac{1}{8} \times 1115 \times 30 \times 30) 12 = 4,878,000$$

$$S = \frac{4878000}{18000} = 271$$

use 24,110# C Beam

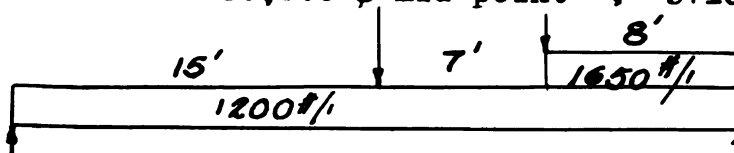
$$s = 276.83$$

FB2W1

wt. 150# /ft

span 30'

Conc. load = 50,000 @ mid point , 5710# 8' from R_L



$$R_L = 25,000 + \frac{4}{30} \times 8 \times 1650 + \frac{19}{30} \times 22 \times 1200 = 43840$$

$$M_{max} = (43840 \times 15 - 18000 \times 7.5) 12 = 6,271,200$$

$$S = \frac{6,271,200}{18,000} = 348$$

use 27" 137# C Beam

$$s = 358.73$$

FB2A

Same as **FB3A**

FB2B

" " **FB3B**

FB2C

" " **FB3C**

FB2D

" " **FB3D**

FB2E

" " **FB3E**

FB2F

" " **FB3F**

FB2G

" " **FB3G**

FB2H

" " **FB3H**

FB2I

" " **FbbI**

FB2J

" " **Fb3J**

FB2M

" " **FB3M**

FB2O

" " **FB3O**

FB2Q

" " **FB3Q**

FB2V

" " **FB3V**

FB2A

Same as FB3X

XB2A

" " XB3A

XB2B

" " XB3B

XB2C

" " XB3C

XB2D

" " XB3D

First floor beams
(including girders)

The design of the first floor system varies somewhat from that of the other floors in that the live loads are greater, and the longitudinal members are girders which transmit the stress to the abutments and the pier.

The loads on the north section are as follows:

15 foot span.

L. L.	150 pounds per square foot.
2" Asphalt	20 " " " "
Fireproofing	15 " " " "
Weight of floor	<u>15</u> " " " "
Total load	200 " " " "

Using 5", 10# I's with 1/4" plate the safe load is 240 pounds per square foot.

18 foot span

Load 200 pounds per square foot.

Using 5" 14.75# I's with 1/4" plate the safe load is 220 pounds per square foot.

The loads on the south and middle sections are as follows:

15 foot span

L. L.	100 pounds per square foot
Fireproofing	15 " " " "
Weight of floor	<u>15</u> " " " "
Total load	130 pounds per square foot

Using 4" 7.7# I's with 3/16 plate the safe load is 150 pounds per square foot.

18 foot span

Load 130 pounds per square foot

Using 4" 10.5# I's with 3/16 plate the safe load is 135 pounds per square foot.

The framing plan with the design of the members follows.

Note: The number of inches of 3/8" fillet weld required for any joint equals the reaction divided by 3000.

Brid

Load

M =

S =

Load

M

S

Loa

M =

Bridge floor beams (North side)

rBlN Span 30' Wt. 100# /ft

$$\text{Load} = (15 \times 200) + 100 = 3100\# /ft$$

$$M = \frac{1}{8} 3100 \times 30 \times 30 \times 12 = 4,185,000$$

$$S = \frac{4,185,000}{18,000} = 232$$

use 21" C Beam Wt 104# /ft s = 235.74

$$\text{reaction} = 3100 \times 15 = 46,500\#$$

rBlW Span 30' Wt. 150# /ft

$$\text{Load} = (16.5 \times 200) + 150 = 3450\# /ft$$

$$M = \frac{1}{8} \times 3450 \times 30 \times 30 \times 12 = 4,655,000\#$$

$$S = \frac{4,655,000}{18,000} = 259$$

use 21" C Beam Wt. 120# /ft s = 2.72"

$$\text{Reaction} = 3450 \times 15 = 51.750\#$$

rBlY Span 30' Wt. 100# /ft

$$\text{Load} = (13 \times 200) + 100 = 2700\# /ft$$

$$M = \frac{1}{8} \times 2700 \times 30 \times 30 \times 12 = 3,650,000\#$$

$$S = \frac{3,650,000}{18000} = 203$$

Use 21" 98# C Beam s = 209.24

$$\text{reaction} = 2700 \times 15 = 40,500\#$$

Bridge floor beams (North side)

Wt. 1000' span 30'

$$\text{Load} = (12 \times 200) + 100 = 2500 \text{ lb}$$

$$M = \frac{1}{8} \times 2500 \times 30 \times 30 \times 12 = 4,185,000$$

$$S = \frac{4,185,000}{18,000} = 232$$

Use 21" x 36" beam at 104' span

$$\text{Reaction} = 2500 \times 12 = 30,000$$

Wt. 1800' span 30'

$$\text{Load} = (12.5 \times 200) + 180 = 2450 \text{ lb}$$

$$M = \frac{1}{8} \times 2450 \times 30 \times 30 \times 12 = 4,685,000$$

$$S = \frac{4,685,000}{18,000} = 260$$

Use 21" x 36" beam at 180' span

$$\text{Reaction} = 2450 \times 12 = 29,400$$

Wt. 1000' span 30'

$$\text{Load} = (12 \times 200) + 100 = 2500 \text{ lb}$$

$$M = \frac{1}{8} \times 2500 \times 30 \times 30 \times 12 = 4,185,000$$

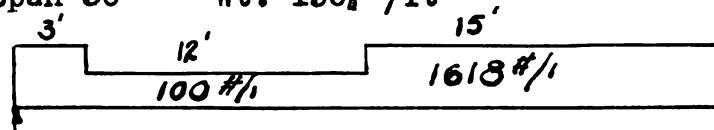
$$S = \frac{4,185,000}{18,000} = 232$$

Use 21" x 36" beam at 104' span

$$\text{Reaction} = 2500 \times 12 = 30,000$$

Bridge floor beams (North side cont.)

RBLK Span 30' wt. 150# /ft



$$\text{Floor load} = 9\frac{1}{2} \times 200 = 1900 \# / \text{ft}$$

$$\text{Dead load} = 150\#$$

$$\text{Total uniform load} = 2050\# / \text{ft}$$

$$R_L = \frac{3 \times 1618 \times 28.5 + 12 \times 100 \times 21 + 15 \times 1618 \times 7.5 + 2050 \times 30 \times 15}{30}$$

$$R_L = \frac{138,500 + 25200 + 182,000 + 920,000}{30}$$

$$R_L = 42,190$$

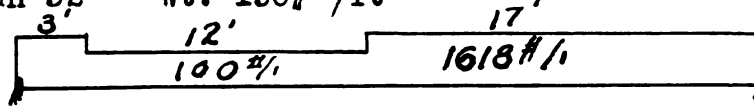
$$R_R = 49,650$$

$$M_{\text{max}} = (49650 \times 15 - 182,000) \times 12 = 6,753,000 \text{"}\#$$

$$s = \frac{6,753,000}{18,000} = 375$$

use 24" 150# "C" beam $s = 384.94$

RBLK Span 32' wt. 150# /ft



$$\text{Max floor load} = 5.5 \times 200\# / \text{ft @ R end (N)}$$

Since this beam rests directly on the abutment the area need only be large enough to withstand the compressive stress.

Max load /ft = 1100 150 1618 = 2868# which load would require only a fraction of an inch of web thickness to provide for joints and floor connections. use 8" - 18.4# std. "I".



Floor Beams (Middle).

FBLM Span 10' Wt. 25# /ft.

L. L. = 100# /sq. ft.

Load = (130 x 15) + 25 = 1975# /ft

Partition load = (15' high) = 420# /ft

total load 2395

$M = \frac{1}{8} \times 2395 \times 10 \times 10 \times 12 = 360,000\text{"#}$

$S = \frac{360000}{18000} = 20$

use 9" 25# std. "I" S = 20.31

reaction = 2395 x 5 = 12,000#

FBLV Span 11' Wt. 10# /ft

Load = 420# /ft

$S = \frac{1}{8} \times \frac{420 \times 11 \times 11 \times 12}{18000} = 4.35$

use 5" 10# std. I S = 4.84

reaction = 420 x 5.5 = 2310#

FBL0 span 10' Wt. 50#

Load = (130 x 16) 50 = 2130 420 = 2550# /ft

$M = 2310 \times 5 \times 12 + \frac{1}{8} \times 2550 \times 10 \times 10 \times 12 = 526,100\text{"#}$

$S = \frac{526,100}{18,000} = 29.3$

use 12" 31.8" std. I S = 35.97

FBLX

See FBL5

Use 8" 18.4# Std. "I"



Bridge floor beams (Middle)

FB1C Span 13' Wt. 50# /ft

wall load = 1918# /ft

wt. 50# /ft

Total load = 1968# /ft

$$S = \frac{1968 \times 13 \times 13 \times 12}{8 \times 18000} = 27.8$$

use 10" 30# C Beam

$$s = 31.91$$

XB1D Span 4' Wt. 20# /ft

Wall load = 1918# /ft

Floor load = 6.5 x 130 = 845

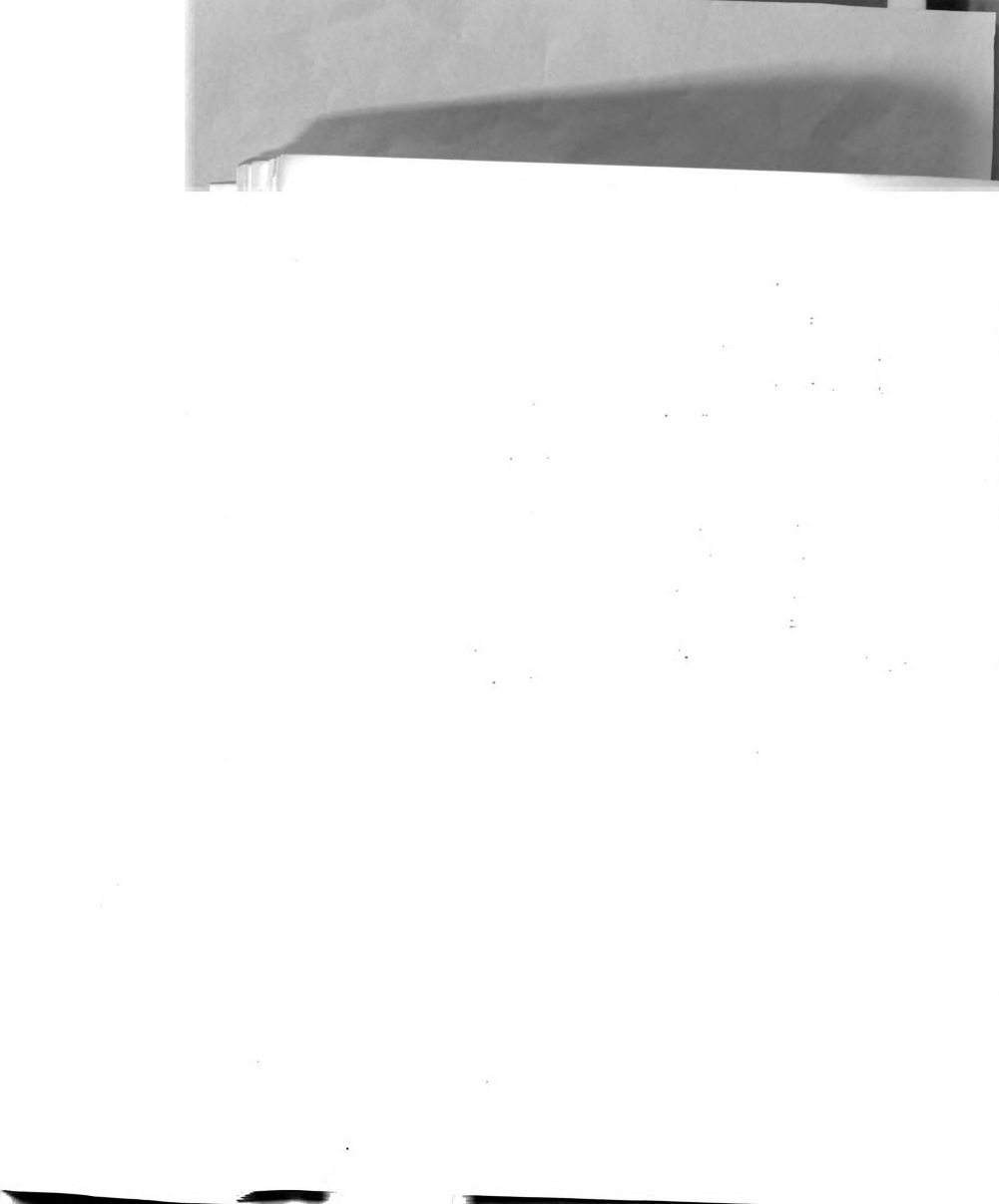
Total load = #/ft

$$s = \frac{2778 \times 4 \times 4 \times 12}{8 \times 18000} = 3.71$$

use 8" 18.4# std. I*

$$s = 14.22$$

* for floor connections



(South Side)

BRIDGE FLOOR BEAMS RB1L

Wt. 100# /ft

Partition load - 450# /ft

Floor load - 130 x 15 = 1950# /ft

Wt. of beam = 100# /ft

Total load 2500# /ft

$$M = \frac{1}{8} \times 2500 \times 30 \times 30 \times 12 = 3,375,000 \text{ "#}$$

$$S = \frac{3,375,000}{18,000} = 187.5$$

use - 27"C" beam

85# /ft

$$s = 216.20$$

$$\text{Reactions} = 2500 \times 15 = 37,500\#$$

FB1Z

Span 30'

Wt. 100# /ft

Floor load = 130 x 130 = 1690# /ft

Partition load = 450# /ft

Wt. = 100# /ft

2240# /ft

$$M = \frac{1}{8} \times 2240 \times 30 \times 30 \times 12 = 3,024,000$$

$$S = \frac{3,024}{18} = 168.5$$

use 21" 80# "C" Beam

$$\text{Reactions} = 15 \times 2240 = 33,600\#$$

$$s = 170.9$$

FB1U

See FB1S

8" - 18.4# Std "I"

FB1R

Span 30'

Wt. 120# /ft

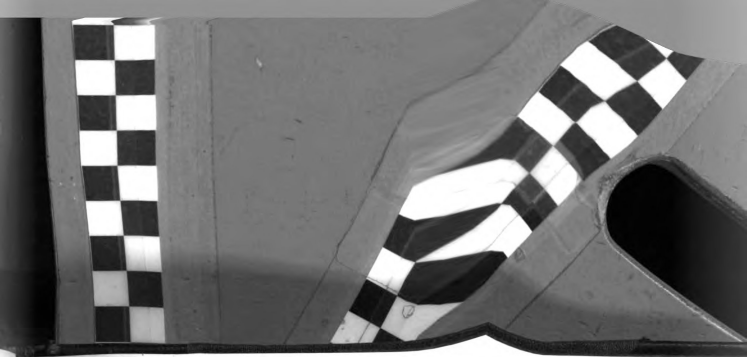
Floor load = 17 x 130 = 2210

Partition load = 450

Weight

120
-2780# /ft





$$S = \frac{27800 \times 30 \times 30 \times 12}{8 \times 18000} = 208.5$$

use 27" - 85# "C" Beam

$$\text{Reactions} = 15 \times 2780 = 41,700\#$$

$$s = 216.20$$

FBLT

Length 30'

Wt. 100# /ft

$$\text{Floor load} = 130 \times 15 = 1950\# / \text{ft}$$

$$\text{Partition load} = 450\# / \text{ft}$$

$$\text{Wt. of beam} = 100\# / \text{ft}$$

$$\text{Total load} = 2500\# / \text{ft}$$

$$M = 3,375,000\#\text{'}$$

$$s = 187.5$$

use 18" - 100# C Beam $s = 19557$

$$\text{Reactions} = 37,500\#$$

FBLP

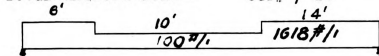
Length 30'

Wt. 150# /ft

$$\text{Floor load} = 130 \times 5.5 = 715\# / \text{ft}$$

$$\text{Wt.} = 150\# / \text{ft}$$

$$\text{Total uniform load} = 665\# / \text{ft}$$



WALL LOADS

$$R_L = \frac{1618 \times 6 \times 27 + 10 \times 100 \times 19 + 14 \times 1618 \times 7 + 665 \times 30 \times 15}{30}$$

$$R_L = 24,620$$

$$R_R = 28,690$$

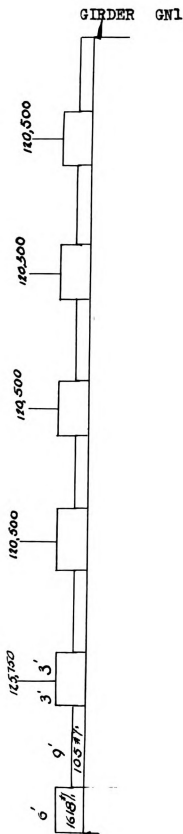
$$M_{14'} \text{ from rt} = 12(28,690 \times 14 - 2283 \times 7) = 462,500\#\text{'}$$

$$S = \frac{462,500}{18000} = 25.7$$

use 10" "C" Beam

$$\text{Wt. } 26\# / \text{ft} \quad s = 27.63$$





$$90 R_L - (239 \times 9)(4.5 + 19.5 + 34.5 + 49.5 + 64.5 + 79.5)$$

$$- (1618 \times 6) (12 + 27 + 42 + 57 + 72 + 87)$$

$$- 120,500 (12 + 27 + 42 + 57) - 125750 (72)$$

$$R_L = \frac{(252 \times 2079) + (9708 \times 297) + (120,500 \times 138) + (125750 \times 72)}{90}$$

$$R_L = 322,891$$

$$R_R = 12,474 + 58,248 + 482,000 + 125750 - 322,891 = 355,581$$

1. The first part of the document is a list of names and their corresponding addresses. The names are listed in a column on the left, and the addresses are listed in a column on the right. The names are: John Doe, Jane Smith, and Bob Johnson. The addresses are: 123 Main St, 456 Elm St, and 789 Oak St.

Name	Address
John Doe	123 Main St
Jane Smith	456 Elm St
Bob Johnson	789 Oak St

GIRDER GN1 (Cont.)

Max shear = 355,581# @ right end

The max moment will occur @ pt. 48' from the left end

M (48' from rt. end)

$$12 (322891 \times 48 - 29,124 \times 30 - 127,750 \times 30 - 120,500 \times 15 - 231 \times 27 \times 22.5 - 1618 \times 3 \times 1.5) = 106,050,816\text{"#}$$

$$D = \frac{1}{12} \text{ span} = \frac{1}{12} \times 90 \times 12 = 90\text{"} \quad \text{use } 96\text{"}$$

$$\text{use } 96 \times \frac{1}{2} \quad \text{Web} \quad A = 48 \text{ sq. in.}$$

$$\text{Using web stiffeners } \frac{V}{A} \text{ may} = 10,000 \quad A = \frac{355,581}{10,000} = 35.56\text{"}$$

Assume wt. = 370# /ft

$$\text{Dead M (48' from L end)} = (16,650 \times 48 - 48 \times 370 \times 24) \frac{1}{2} = 4,470,000\text{"#}$$

$$A = \frac{110,520,800}{18,000 \times 94} - \frac{1}{8} \times 96 \times \frac{1}{2} = 65.3\text{"} - 6.00 \text{ sq. in.} = 59.3 \text{ sq.}"$$

Use	1 Channel 15" - 55#	=	16.11 sq. in.
	1 Pl 16" x 1"	=	16.00 sq. in.
	1 Pl 17 x 1"	=	17.00 sq. in.
	1 Pl 18 x 1/2"	=	9.00 sq. in (comp. flange only)
	1 Pl 19 x 1/2"	=	<u>9.50 sq. in.</u>
			68.11 sq. in.

$$\text{Allowable comp.} = 18,000 - 170 \times \frac{180}{17} = 16,200\text{#} / \text{sq. In.}$$

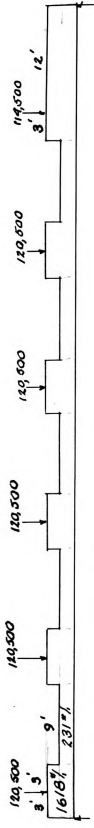
$$H = 96 \quad 2 (1.63 - .82) = 97.62$$

$$A_c = \frac{110,520,800}{97.62 \times 16,200} - 6.00 = 64.0 \text{ sq. in.}$$

$$A_T = \frac{110,520,800}{97.62 \times 18000} - 6.00 = 57 \text{ sq. in.}$$



GIRDER GN 2



$$90 R_L = 120,500 (27 + 42 + 57 + 72 + 87) + (9 \times 231)(19.5 + 34.5 + 49.5 + 64.5 + 79.5)$$

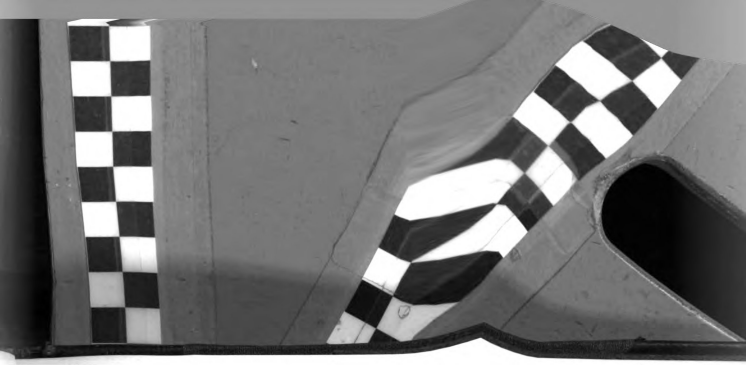
$$1618 \times 6 (27 + 42 + 57 + 72 + 87) (1618 \times 15)(7.5) \quad 114,500 \times 12$$

$$R_L = \frac{34,342,500 + 2,766,780 + 514,553 + 36,405 + 1,374,000}{90}$$

$$R_L = \frac{39,034,233}{90} = 433,714$$

$$R_R = 602,500 + 10,395 + 72,810 + 114,500 - 433,714$$

$$R_R = 366,391$$



GIRDER GN2 (Cont.)

Max shear = 433,714# @ left end

Max moment will occur @ a pt. 48' from the left end.

$$D = \frac{1}{12} \times 90 \times 12 = 90" \quad \text{use 96" web}$$

Using web stiffeners $\frac{V}{A} = 10,000$

$$\text{Req'd Area} = \frac{433714}{10,000} = 43.3714 \text{ sq. in.} \quad \text{use 96"x}\frac{1}{2}\text{" web}$$

Assume wt. = 400# /ft

$$M \text{ 48' from left} = (433,714 \times 48 - 120,500 \times 3 \times 30 - 1618 \times 18 \\ \times 30 - 231 \times 27 \times 22.5 - 3 \times 1618 \times 1.5)12 + \\ (400 \times 45 \times 48 - 400 \times 48 \times 24)12 =$$

$$106,424,864 + 4,502,000 = 110,926,864"$$

$$\text{Approx. A} = \frac{110,926,864}{18,000 \times 94} - \frac{1}{8} \times 96 \times \frac{1}{2} = 59.7 \text{ sq. in.}$$

use 1 Channel 15" - 55" 16.11 sq. in.

1 Pl 16" x 1" 16.00

1 Pl 17" x 1" 17.00

1 Pl 18" x $\frac{1}{2}$ " 9.00 (comp. flange only)

1 Pl 19 x $\frac{1}{2}$ 9.50

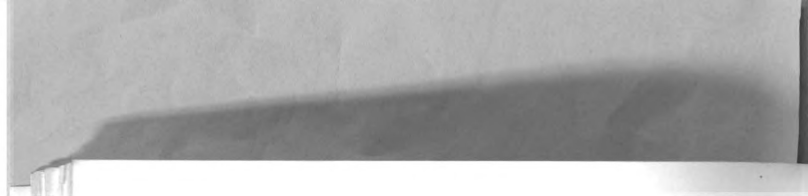
68.11 sq. in.

$$\text{Allowable comp.} = 18,000 - 170 \times \frac{180}{17} = 16200 \# / \text{sq. in.}$$

$$H = 93.36$$

$$A_c = \frac{110,926,864}{97.62 \times 16,200} - 6.00 = 64.1 \text{ sq. in.}$$

$$A_T = \frac{110,926,824}{97.62 \times 18000} - 6.00 = 57.9 \text{ sq. in.}$$



4. The fourth part of the document discusses the implications of the study and the conclusions drawn from the results. It highlights the significance of the findings and their potential applications in the field of research.

5. The fifth part of the document provides a summary of the key points discussed throughout the document. It reiterates the importance of accurate record-keeping and the need for transparency in financial reporting.

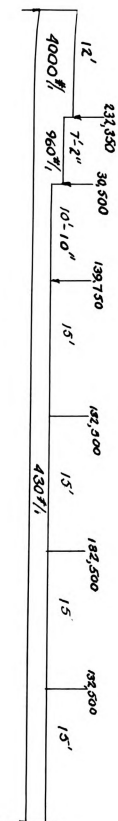
6. The sixth part of the document includes a list of references and a bibliography, citing the various sources used in the study. It also includes a list of figures and tables, providing a clear overview of the data presented in the document.

7. The seventh part of the document includes a list of appendices, providing additional information and data that support the findings of the study. It includes a list of figures and tables, providing a clear overview of the data presented in the document.

8. The eighth part of the document includes a list of footnotes, providing additional information and data that support the findings of the study. It includes a list of figures and tables, providing a clear overview of the data presented in the document.

9. The ninth part of the document includes a list of references and a bibliography, citing the various sources used in the study. It also includes a list of figures and tables, providing a clear overview of the data presented in the document.

10. The tenth part of the document includes a list of appendices, providing additional information and data that support the findings of the study. It includes a list of figures and tables, providing a clear overview of the data presented in the document.



GM1

$$R_L = \frac{84}{90} \times 12 \times 4000 + \frac{78}{90} \times 232,350 + \frac{71}{90} \times 30,500 + \frac{60}{90} \times 139,750 + \frac{45 + 30 + 15}{90} \times 135,000$$

$$+ \frac{74.5}{90} \times 7 \frac{1}{6} \times 960 + \frac{35}{90} \times 430 \times 70 \frac{5}{6}$$

$$R_L = 506,060$$

$$R_R = 390,120$$



GM 1 (Cont.)

Max. shear = 506,060# @ left end

Max. moment is @ middle

$$\text{Web Area} = \frac{506,060}{10,000} = 50.606 \text{ sq. in.}$$

use 96" x $\frac{9}{16}$ " web

assume wt. = 400# /ft.

$$M (\text{max.}) = (408,120 \times 45 - 830 \times 45 \times 22.5 - 265000 \times 22.5)12$$

$$M = 139,750,300\text{"#}$$

use 1 channel 15"-55# - 16.11 sq. in.

1 Pl. 16 x 1 - 16.00

1 Pl. 17 x 1 - 17.00

1 Pl. 18 x 1 - 18.00

1 Pl. 19 x 11/16 13.06*

80.17 sq. in.

$$\text{Allowable compression} = 18,000 - 170 \times \frac{180}{17} = 16,200$$

Required Area

$$A (\text{Comp.}) = \frac{139,750,300}{16,200 \times 100} - \frac{1}{8} \times 54 = 79.45 \text{ sq. in.}$$

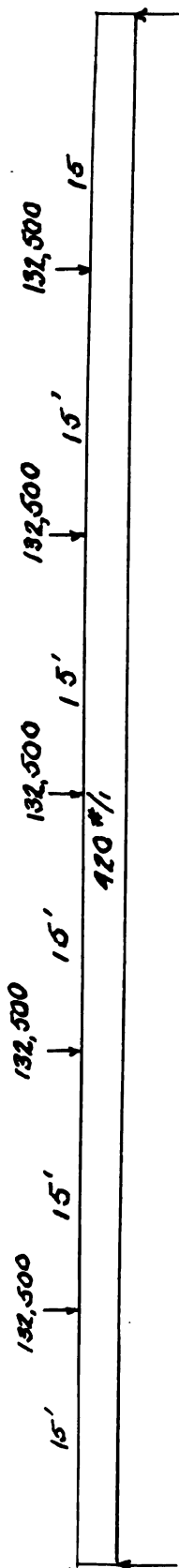
$$A. (\text{Ten}) \frac{139,750,300}{18,000 \times 100} - 6.75 = 70.75 \text{ sq. in.}$$

* use 19"x $\frac{1}{8}$ " on tensile flange

making tensile area = 76.61 sq. in.



GIRDER GM2



$$R_L = R_R = 420 \times 45 + 2.5 (132,500) = 340,150$$

41808-002

GM 2 (Cont.)

Max Shear = 340,150 @ end

Max Moment is @ middle

$$\text{Req'd. area} = \frac{340,150}{10,000} = 34.015 \text{ sq. in.}$$

use 96" x 3/8" web

assume wt. = 400# /ft

$$M \text{ max.} = (198,750 \times 45 + \frac{1}{8} \times 820 \times 90 \times 90) 12 = 117,288,000 \text{"}\#$$

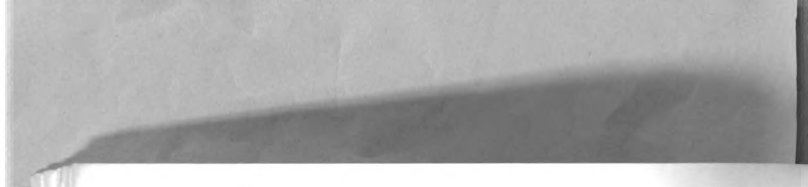
$$\text{Approx. A} = \frac{117,288,000}{96 \times 18,000} - 4.5 = 63.5 \text{ sq. in.}$$

Use 1 channel	15" - 55#	= 16.11 sq. in.
1 Pl	16 x 1"	= 16.00 sq. in.
1 Pl	17 x 1"	= 17.00 sq. in.
1 Pl	18 x $\frac{3}{4}$ "	= 13.50 sq. in.
1 Pl	19 x $\frac{1}{2}$ "	= <u>9.50</u> (comp.)
		72.11 sq. in.

$$\text{Allowable comp.} = 18,000 - 170 \frac{180}{17} = 16,200$$

$$A_c = \frac{117,288,000}{99 \times 16,200} - 4.5 = 69.00 \text{ sq. in.}$$

$$A_T = \frac{117,288,000}{99 \times 18,000} - 4.5 = 61.30 \text{ sq. in.}$$



GIRDER G M 3

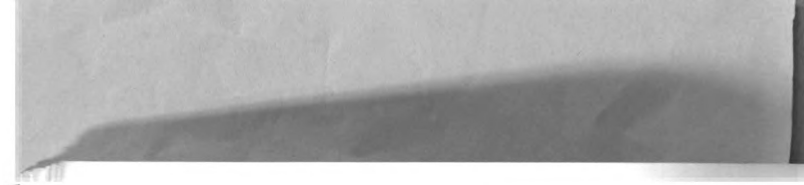


$$R_L = \frac{88.5 \times 3 \times 2500 + 67 \times 34,300 + 74 \times 186,700 + 28 \times 56 \times 430 + 67 \times 20,700 + 56 \times 129,700 + 41 + 26 + 11 \times 123,500}{90}$$

$$R_L = \frac{88.5 \times 3 \times 2500 + 67 \times 34,300 + 74 \times 186,700 + 28 \times 56 \times 430 + 67 \times 20,700 + 56 \times 129,700 + 41 + 26 + 11 \times 123,500}{90}$$

$$K_L = 404,750$$

$$K_R = 368,730$$



100

GM 3 (Cont.)

Max. shear = 404,750# @ left end

Max. moment is @ A point 49' from the left end.

$$\text{web area} = \frac{404,750}{10,000} = 40.475 \text{ square inches}$$

use 96" x $\frac{1}{2}$ " web

Assume weight = 400# /ft

$$M (\text{max}) = (386,730 \times 41 - 123,500 \times 45 - 830 \times 41 \times 20.5)12$$

$$M = 103,217,160\text{"}\#$$

use 1 channel 15" - 55# 16.11 sq. in

1 Pl 16"x 1" 16.00

1 Pl 17" x 1" 17.00

1 Pl 18 x 3/4" 13.50

62.61 sq. in

Allowable compression = 16,100# /sq. in

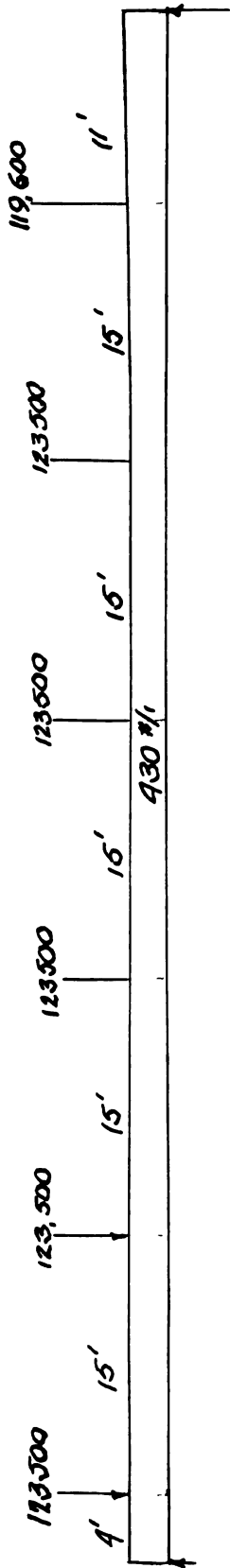
Required flange areas

$$A (\text{comp.}) = \frac{103,217,160}{98.75 \times 16,100} - 6.0 = 58.9 \text{ sq. in}$$

$$A (\text{Tens}) = \frac{103,217,160}{98.75 \times 18,000} - 6.0 = 52.0 \text{ sq. in}$$



GIRDER G M 4



$$R_L = 45 \times 430 + \frac{123,500}{90} (86 + 71 + 56 + 41 + 26) + \frac{119,600}{90} (11)$$

$$R_L = 418,140$$

$$R_R = 357,600$$



G M 4 (cont.)

Max shear = 418,140 @ left end

Max Moment is @ 49' from left end

$$\text{web area} = \frac{418,140}{10,000} = 41.814 \text{ sq. in.}$$

use 96 x 1/2" web

assume wt. = 400 lb/ft

$$\begin{aligned} M (\text{max}) &= (357,600 \times 41 - 123,500 \times 15 - 119,600 \times 30 - 430 \\ &\quad \times 41 \times 20.5) + (400 \times 45 \times 41 - 400 \times 41 \times 20.5) 12 = \\ &\quad 103,368,730 \text{ in}^2 \end{aligned}$$

use 1 channel 15" - 55# 16.11

1 Pl 16" x 1" 16.00

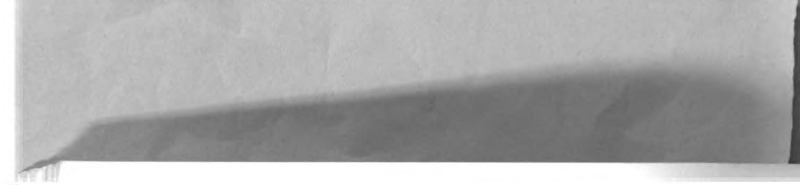
1 Pl 17" x 1" 17.00

1 Pl 18" x 3/4" 13.50

allowable comp. = 16,100 62.61 sq. in.

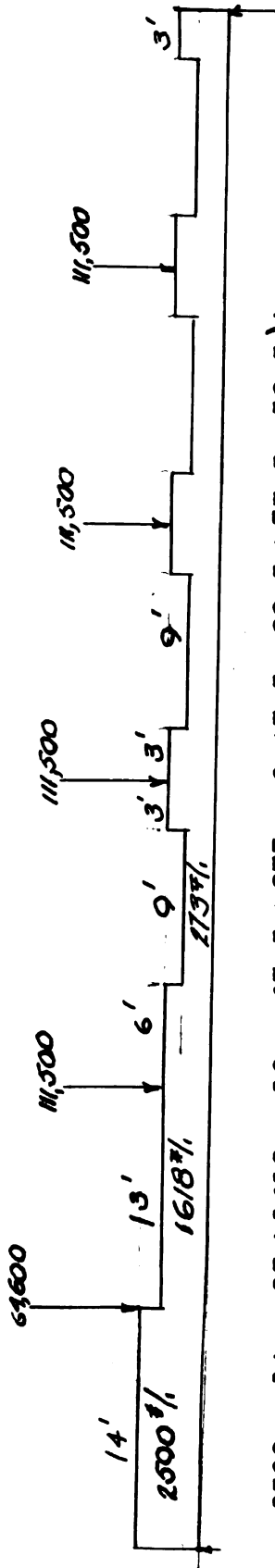
$$A_c = \frac{103,368,780}{98.75 \times 16,100} - 6.0 = 59.2 \text{ sq. in.}$$

$$A_t = \frac{103,368,780}{98.75 \times 18,000} - 6.0 = 52.2 \text{ sq. in.}$$



11/11/11

GIRDER GSI



$$K_L = 2500 \times 14 \times 83 + 1618 \times 19 \times 67.5 + 273 \times 9 \times (7.5 + 22.5 + 37.5 + 52.5) +$$

$$1618 \times 6 \times (15 + 30 + 45) + 1618 \times 3 \times (1.5) + 111500 \times (15 + 30 + 63) + 115700 \times 45$$

$$64,600 \times 76 + 90$$

$$R_L = 319,415$$

$$R_R = 292,725$$

GS 1 (Cont.)

Max shear = 319,415 @ left end

Max. moment is @ the middle

$$\frac{V}{A} = 10,000$$

$$\text{Req'd. area} = \frac{319415}{10,000} = 31.9415 \text{ sq. in.}$$

use 96 x 3/8" web

assume wt. = 400# /ft

$$M = (292,725 \times 45 - 223,000 \times 22.5 - 273 \times 27 \times 22.5 - 1618 \times 18 \times 22.5) 12 + \frac{1}{8} \times 400 \times 90 \times 90 \times 12 = 92,486,500 \text{ "lb}$$

$$\text{Approx. } A = \frac{92,486,500}{18000 \times 96} - \frac{1}{8} \times 96 \times \frac{3}{8} = 49.5 \text{ sq. in.}$$

use 1 channel 15" 55# 16.11 sq. in.

1 Pl 16" x 1" 16.00 sq. in.

1 Pl 17" x 1" 17.00 sq. in.

1 Pl 18 x 1/2" 9.00 sq. in.

58.11 sq. in.

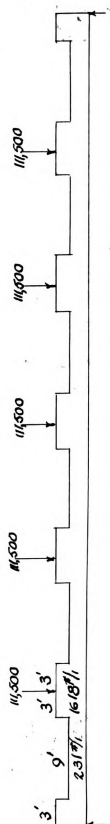
$$\text{Allowable comp.} = 18,000 - 170 \times \frac{180}{16} = 16,100\# / \text{sq. in.}$$

$$A_c = \frac{92486500}{98.5 \times 16100} - 4.5 = 54.5 \text{ sq. in.}$$

$$A_T = \frac{92,486,500}{98.5 \times 18,000} - 4.5 = 57.7 \text{ sq. in.}$$

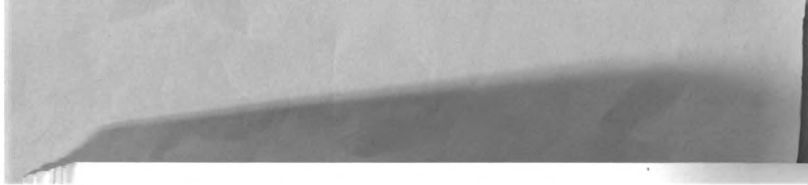


G52



$$R_L = R_R = (18 \times 1618 + 27 \times 231 + 2.5 \times 111,500) = 314,105$$





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G 3 2 (Cont.)

Max. shear = 314,105# @ right end

Max Moment is @ the middle

$$D = \frac{1}{12} \times 90 \times 12 = 90" \quad \text{use 96" web}$$

using web stiffeners $\frac{V}{A} = 10,000$

$$\text{Req'd. Area} = \frac{314,105}{10,000} = 31.4105 \text{ sq. in.}$$

use 96" x 3/8" web

Assume wt. = 400 #/ft

$$M = (369,855 \times 22.5 + \frac{1}{8} \times 400 \times 90 \times 90) 12 = 88,260,000 \text{ "#}$$

$$\text{Approx. A} = \frac{88,260,000}{18000 \times 96} - \frac{1}{8} \times 96 \times \frac{3}{8} = 46.5 \text{ sq. in.}$$

use 1 channel 15" - 55# 16.11 sq. in.

1 P1 16" x 1" 16.00

1 P1 17" x 1" 17.00

1 P1 18" x 1/4" 4.50

53.61 sq. in (comp.)

$$\text{Allowable comp.} = 18,000 - 170 \times \frac{180}{16} = 16,100 \text{ #/sq. in.}$$

$$A_c = \frac{88,260,000}{98.0 \times 16,100} - 4.5 = 52.0 \text{ sq. in.}$$

$$A_T = \frac{88,260,000}{98.0 \times 18000} - 4.5 = 45.5 \text{ sq. in.}$$



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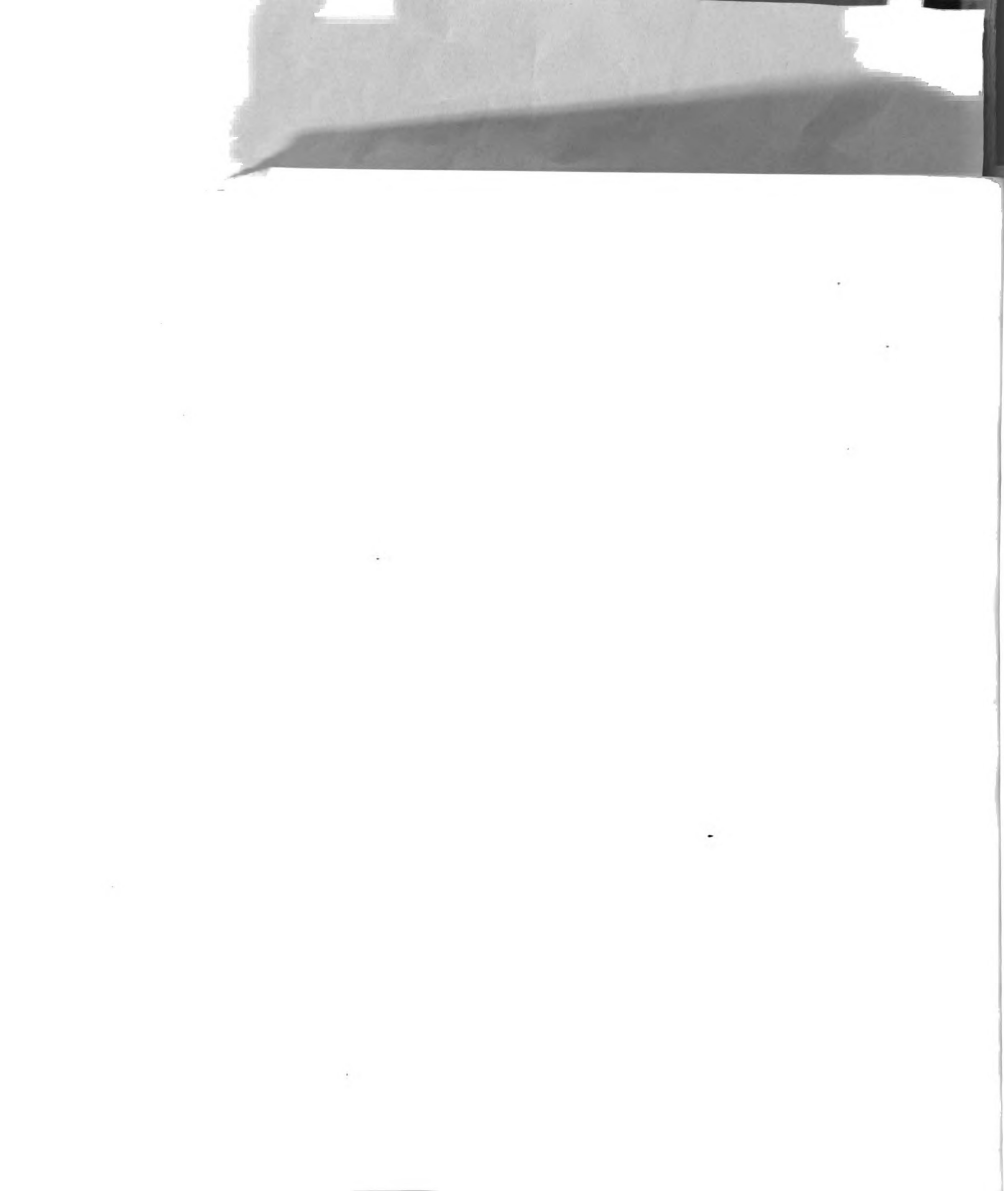
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Columns



To provide equal resistance to wind stresses in all panels, all columns on each floor are designed to support the maximum load occurring on any column on that floor.

These computations with the size of columns chosen follow:



ROOF COLUMNS

Max Load on Roof Column

$$RBE = 7.5 \times 220 = 1650$$

$$RBB = 9 \times 220 = 1980$$

$$2 RBW = 2 (3545) + 15 \times 20 = \underline{7390}$$

11020# which would require an
area 1 sq. in, But for joints
a larger beam must be used.

use 6" 12.5# std. "I"

Total load transmitted to column below =

$$11,020 \quad 11 \times 12.5 = 11,158\#$$

for design load use 11,500

Check stress for partial loading

$$RBE = (75 \quad 2.5 \times 30) 7.5 = 1125$$

$$RBB = (77.5 \quad 2.5 \times 30) 9 = 1373$$

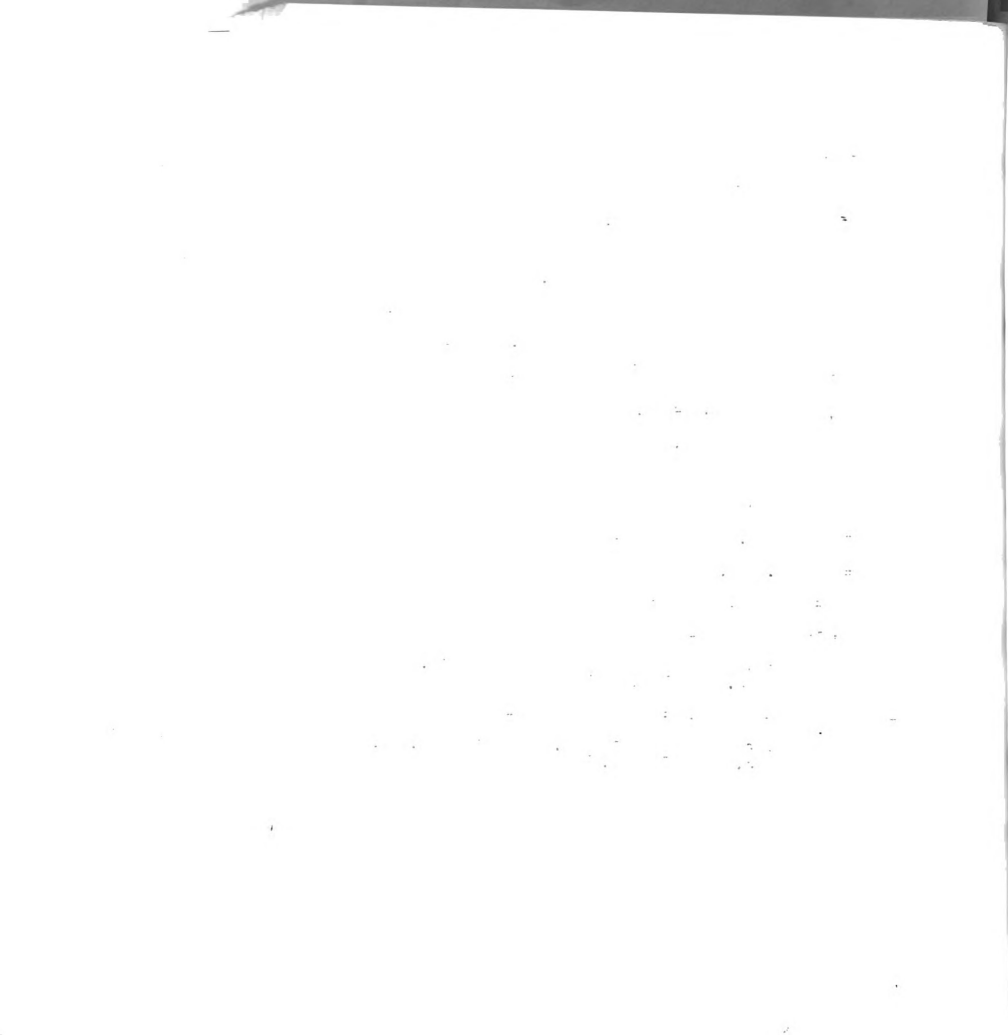
$$RBW_{L+D} = 3545 \quad 7.5 \times 20 = 3695$$

$$RBW_D = 18 \times 75 \quad 225 = 1575$$

$$s = \frac{4468}{3.61} + \frac{2120 \times 3 \times 3}{21.8} = 2115\# / \text{sq. in.}$$

$$RBB = 1980 \quad RBE = 75 \times 7.5 = 640\# \quad 2 RBW_{\frac{1}{2}} = 3695$$

$$s = \frac{6315}{3.61} + \frac{1340 \times .115 \times .115}{1.1} = 1770\# / \text{sq. in.}$$



load on interior column for 3 rd floor.

$$2 \text{ (FB3W)} = 1590 \times 15 = 23,850$$

$$\text{FB3E1} = 73.5 \times 2 \times 7.5 = 1,100$$

$$\text{FB3B} = 74.5 \times 2 \times 9 = 1,330$$

$$\text{load from roof} = \underline{11,500}$$

$$47,780$$

$$\text{Assume } 8" \text{ } 18.4\# \text{ } A = 5.34 \text{ sq. in.}$$

check for eccentric load

$$s = \frac{16075}{5.34} + \frac{10775 \times 4 \times 4}{56.9} = 6080$$

Use conc. load of 50,000#

Load on Interior of Beam (lb)

Loads on bridge.

(column)

$$R_L (FB2N) = 25,000 \quad 22,900 = 47,900\#$$

$$2R (FB2F) = 2(1785) = 3,570$$

$$R_L (FB3N) = 11,000$$

$$2R (FB2F) = 2 (1785) = 3,570$$

$$R_L (RBN) = 3,357$$

$$2R (RBF) = \underline{1,725}$$

$$71,122\#$$

$$\text{Wt. of Roof Column} = 138\#$$

$$\text{" " 3rd Floor Column} = \underline{388\#}$$

$$\text{Load on Column} = 71,648$$

Assume an 8" Beth. H Col. 14' long 27#

$$S = \frac{71,848}{7.89} + \frac{47,900 \times 4 \times 4}{89.7} = 9,120 \quad 8,540 = 17,660\# / \text{sq. in.}$$

$$\text{Total conc. load on bridge) = 71,848} \quad 27 \times 14 = 72,240\#$$

from above)

$$\text{use } 74,000\#$$

Uniform load

$$\text{Wt. of windows} = 10 \times 5.5 = 55\# / \text{ft} \quad \text{Wt. of wall} = 448\# / \text{ft}$$

$$\text{Wt. of 3' of wall} = 32 \times 3 = 96\# / \text{ft} \quad \text{Wt. of 14' of facing} = 370\# / \text{ft.}$$

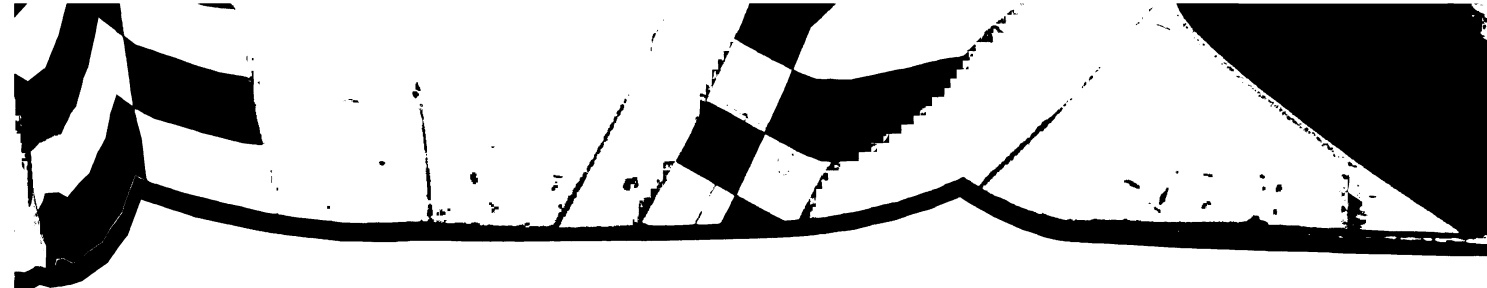
$$\text{Wt. of facing} = \underline{80\# / \text{ft}} \quad \frac{1}{2} \text{wt. of 20' of facing} = \underline{400\# / \text{ft.}}$$

$$231\# / \text{ft}$$

$$1618\# / \text{ft.}$$



Labels on inside



Piers and Abutments



Design of Pier & Abutments.

According to a survey made by H. J. Olver and Max Wygant for a thesis on flood control, the elevation of Ottawa Street is 123.12. This corresponds with 93.0 on the map included in this thesis. High water is at 121.0 or 90.9. This would necessitate the bottoms of the girders being at an elevation of 92.0 giving a clearance of approximately 12 feet over the flow as of December 1, 1931 which is also the present condition (April 1932).

The heights of the girders are 8 feet 8 inches making the elevation of the first floor 100.67 feet or 7.67 feet above the present level of Ottawa Street and 13.32 feet above the road atop the east bank.

The only available data as to bearing conditions as obtained from the city sanitary engineer shows a stratum of blue clay at a depth of twelve feet below the center of the road on the east bank. It is assumed that this soil extends down for several feet so that the base of the pier and the abutments will bear upon it.

West Abutment.

The loads upon the west abutment are as follows, from north to south; 410,540#, 524,060#, 422,750#, and 337,415#.

If six inch rollers are used on this end of each girder, and each roller is 24 inches long the number of rollers required is as follows:

Under northermost girder (GN1)

$$\frac{410,540}{500 \times 6 \times 24} = 4.8 \text{ use 5 rollers}$$



Under (GM1)

$$\frac{524,060}{600 \times 6 \times 24} = 6.06 \text{ use 6 rollers}$$

Under (GM3)

$$\frac{422,750}{600 \times 6 \times 24} = 4.9 \text{ use 5 rollers}$$

Under (GS1)

$$\frac{337,415}{86,400} = 3.9 \text{ use 4 rollers}$$

The allowable bearing stress for steel on concrete is 600 pounds per square inch. The required area under the maximum load would be $\frac{524,060}{600} = 873.4$ square

inches or at 24 inches wide, this would be 36.4 inches long which is the minimum width at the top of the abutment.

The allowable bearing stress on blue clay is four tons per square foot. The total load on the abutment is 1,694,765 pounds. This would require a base area of $\frac{1,694,765}{8000} =$

222 square feet. If the abutment is equal in length to the building (70 feet) the required width is 222 or $\frac{3.17}{70}$ feet,

but since the required top width is 36.4 inches the base area must be larger than required.

Since all the buildings along the west bank have retaining walls along the river's edge, there will be little danger of the water washing behind the abutment; therefore, a straight type abutment will be satisfactory.

The height of the abutment from the base to the seat of the girders is taken as twenty feet.

With such a large load being produced by the bridge, the weight of the abutment itself and the earth pressure against the back of the abutment will be of minor



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importance, therefore if the resultant of the girder loads passes through the center of the base the abutment will probably be safe. A check of this is shown in the accompanying data and drawing.

East Abutment.

The concentrated loads on the east abutment are, from north to south, 414,700#, 419,400#, 435,400# and 387,855#. There is also a wall load which is a distributed load along the top of the abutment. The maximum wall load is 1618 pounds per foot.

Using six inch rollers 24 inches wide under each girder, the number of rollers required is as follows:

Under (GN2)

$$\frac{414,700}{600 \times 6 \times 24} = 4.8 \quad \text{use 5 rollers}$$

Under (GM2)

$$\frac{419,400}{86,400} = 4.9 \quad \text{use 5 rollers}$$

Under (GM4)

$$\frac{435,400}{86,400} = 5.03 \quad \text{use 5 rollers}$$

Under (GS2)

$$\frac{387,855}{86,400} = 4.5 \quad \text{use 5 rollers}$$

The required area under the maximum load is $\frac{435,400}{600}$

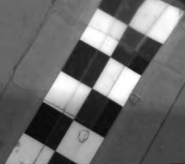
726 square inches or at 24 inches wide, 30.25 inches which is the length of the bearing surface which will be required.

The required area of the base is $\frac{1,770,615}{8000} = 225.3$

square feet. The required width at 70 feet long is 3.22 ft.

To provide against any danger of the dirt being





washed away from behind the abutment, wings will be extended back on both sides of the abutment.

The design of the abutment is shown on the abutment drawings.

Middle Pier.

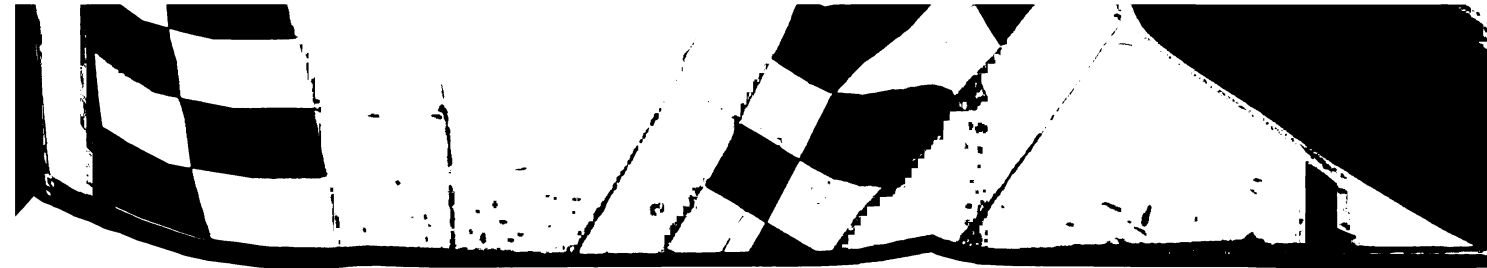
The pier at the center of the bridge supports the fixed ends of all the girders.

The maximum sum of the loads on the pier at any point is 898,770# at the reaction of GM1 and GM2. If a bearing plate 24 inches wide is used, the width of the top of the pier, as determined by this load is $\frac{898,770}{600 \times 24} = 62.5$ inches.

The total load on the pier is 3,304,265 pounds. The base area required is $3304265 \div 41303$ square feet or at 70 feet long this would require a width of 5.9 feet.

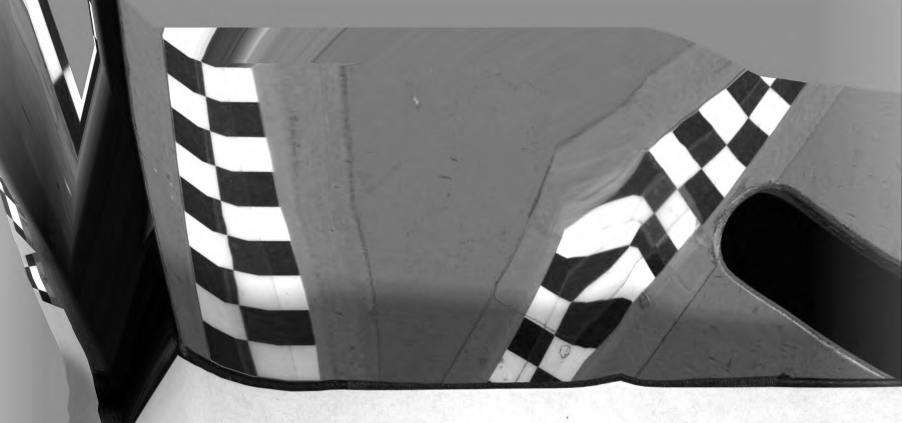
The design of the pier is shown on the drawing of the pier and abutments.





Elevator





DESIGN OF ELEVATOR

Since the building is very low, the smallest elevator will be large enough.

The floor area used will be 25 square feet. For an area of 25 square feet:

Width = 6' - 0" Depth = 4' - 3"

G - gate opening = 3' - 9"

The design load for this size elevator = 75# / square foot.

Live load = $75 \times 25 = 1875$ use 2000#

Assume wt. of frame = 1000#

Wt. of cab = 16# /sq. ft. of floor = 400#

Wt. of platform = 12# / sq. ft. = 300#

Total wt. of car 1700#

Counter weight* = $1700 + .40 \times 2000 = 2500\#$

Assume# wt. of cables = 150#

Max. live load = $1700 + 2000 + 2500 + 150 = 8350\#$

L.L. + 100% impact = 8350 $8350 = 16,700\#$

Max "Rope Pull" (load on cables) = 3,850#

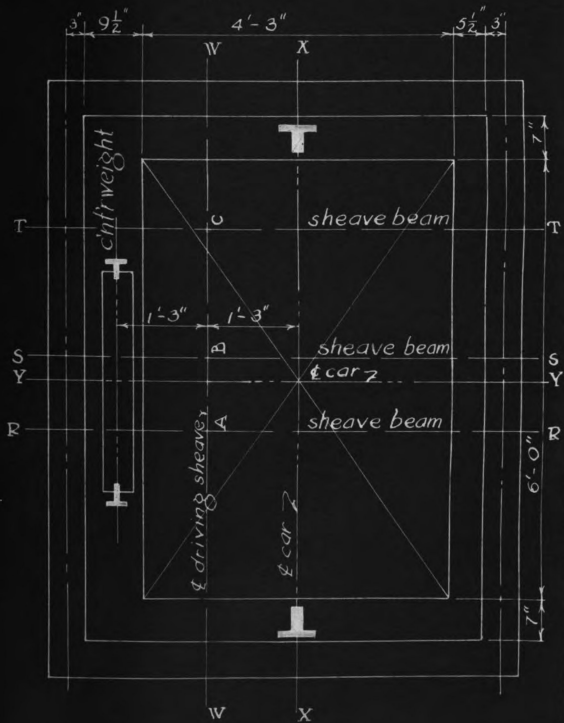
Load on supports = 16,700#

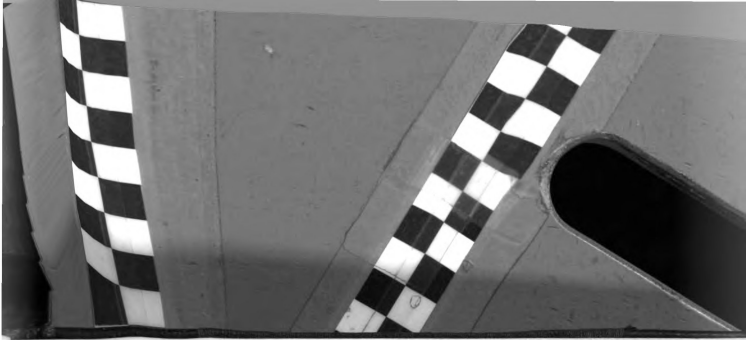
Assume# wt. of machine = 3,000#

Total load on supports = 19,700# (This checks very closely with the load of 19200# specified by westinghouse k & n elevators.)

* Counterweight = wt. of car 40% live load

Assumptions made after following similar design in "Architectural construction" by voss & varney.





Loads @ A, B, & C are proportioned as follows:

$$A = 10,900 \quad B = 6,100 \quad C = 2,700$$

A B C = 19,700 which is the total load.

Assume distance from L of supporting beams to edge of hatchway = 3"

Length of sheave beams = car width + front clearance + rear clearance + 6"

$$\text{Length} = 4' - 3" + 5\frac{1}{2}" + 9\frac{1}{2}" + 6" = 6' - 0"$$

Beam R - R.

$$\frac{M}{S} = 12 \frac{(10900 \times 4.084 \times 1.916)}{6 \times 12,000} = 14.20$$

$$\text{Use } 9" \quad 21.8\# \quad I \quad S = 18.87$$

Beam S - S.

$$\frac{M}{S} = 12 \frac{6100 \times 4.08 \times 1.916}{6 \times 12,000} = 7.95$$

$$\text{Use } 9" \quad 13.4\# \quad S = 10.51$$

Beam T - T.

$$\frac{M}{S} = 12 \frac{2700 \times 4.084 \times 1.916}{6 \times 1200} = 3.52$$

$$\text{Use } 9" \quad 13.4\# \quad S = 10.51$$

roads A, B, & C are proportioned as follows:

A = 10,000 B = 10,000 C = 10,000

A B C = 10,000 which is the total load.

Assume distance from C to A is 100 feet and from C to B is 100 feet.

Pathway = 200

$$R = \frac{10,900 \times 4.084}{6} = 7,430$$

$$R_1 = \frac{10,900 \times 1.916}{6} = 3,470$$

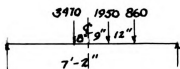
$$S = \frac{6,100 \times 4.084}{6} = 4,150$$

$$S_1 = \frac{6,100 \times 1.916}{6} = 1,950$$

$$T = \frac{2,700 \times 4.084}{6} = 1,840$$

$$T_1 = \frac{2,700 \times 1.916}{6} = 860$$

Front supporting beam



$$R_L = \frac{3470 \times 4.25 + 1950 \times 3.25 + 860 \times 1.5}{7.16}$$

$$R_L = 3125$$

$$R_R = 2165$$

due to elevator and machinery

$$R_L = R_R \quad 273 \times 3 \times \frac{7.16}{2} = 2935\# \text{ due to floor load}$$

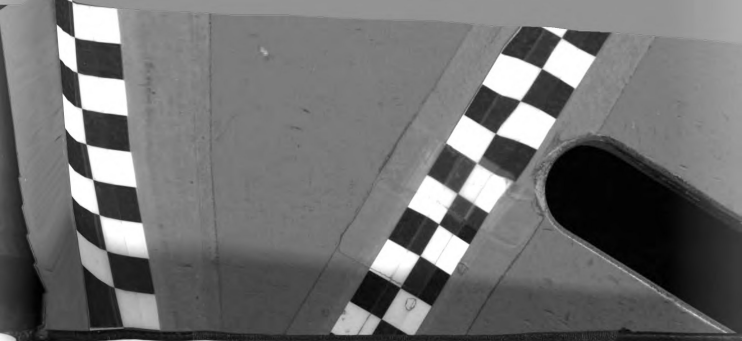
$$\text{Total reaction} = R_L = 6050, R_R = 5100$$

$$S = \frac{(2935 \times 7.16 + 3125 \times 3.583 - 3470 \times .667)12}{12000} = 14.14$$

$$\text{Use } 9" - 21.8\#I$$

$$s = 18.87$$





Rear supporting beams.

Load due to machinery.

$$R_L = 6660$$

$$R_R = 4610$$

$$R_L = R_R = 2935 \text{ due to floor load}$$

Total reactions

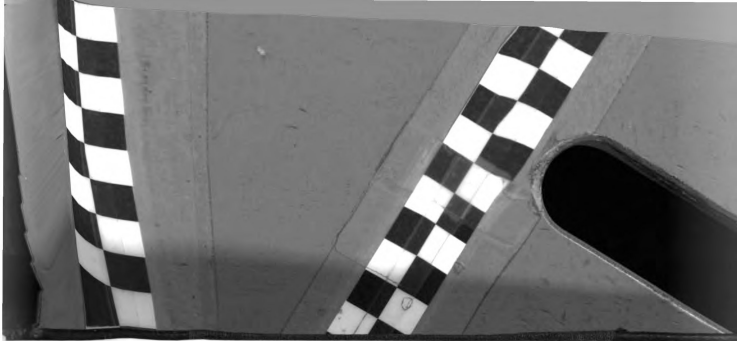
$$R_L = 9595$$

$$R_R = 7545$$

$$s = 5.25 + 18.90 = 24.15$$

Use 10" 25.4# I

$$s = 24.42$$



Penthouse Design.

Floor area

$$7' - 2'' \times 11' - 0''$$

Live load = 250# /sq. ft.

Wt. of floor) 23# /sq. ft
and fireproofing

total 273# /sq. ft.

Span 7' - 2"

Use 3" beams with 3/16" plate.

Supporting beams

Loads

$$980 = 3.58 \times 273 \text{ #/ft from floors}$$

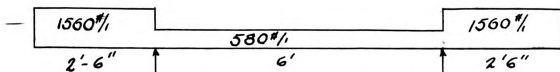
$$224 = 7 \times 32 \text{ #/ft from wall}$$

$$187 = \frac{2}{12} \times 7 \times 160 \text{ #/ft from stone facing}$$

$$125 = 3.58 \times 35 \text{ #/ft from roof}$$

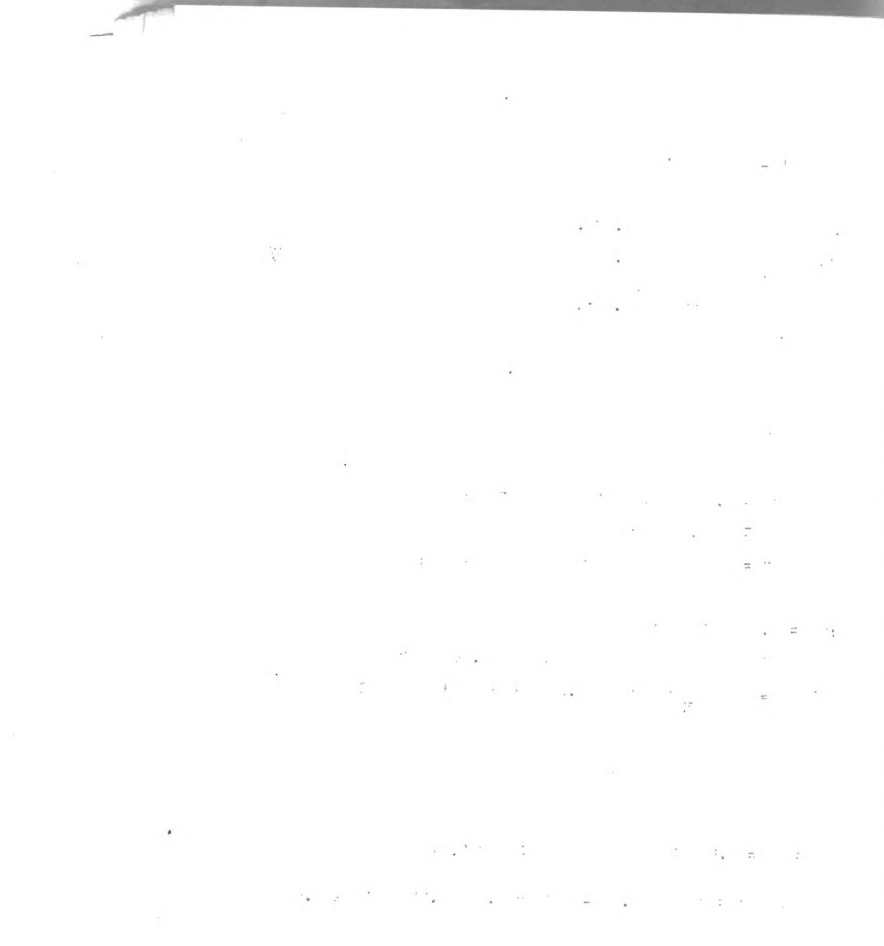
$$1516 \text{ #/ft total load assume wt. } 44 \text{ #/ft}$$

$$1475 = 7(32 + \frac{1}{6} \times 160) \times 3.58 \text{ @ each end from walls}$$



$$R_L = R_R = 2.5 \times 1560 + 1475 + 3 \times 580 = 7,115$$

$$M (\text{center}) = 7115 \times 3.00 - 1560 \times 5.5 \times 2.75 - 1475 \times 5.5$$





$M (+) = \text{None}$

$M (-) = 1475 \times 2.5 + 1560 \times 2.5 \times 1.25$

$M (-) = 103,756\text{"}\#$

$S = \frac{M}{18,000} = \frac{103,756\text{"}\#}{18,000} \quad 5.76$

$\text{Max } V = 7,115\#$

A 6" 12.5# I Beam having an S of 7.27 and an allowable shear of 16,600 will be used.



Elevator Columns

N. W. Column

Loads

1. From rear supporting beam -	9595
2. From pent house beam	7115
3. From roof beam RBK	2100
4. From floor beam FB3K	6815
5. From floor beam FB2K	<u>28367</u>
	53992

Total load on top = 18,810#

The distance between supports is not greater* than 12'

Choose an 8" Bethlehem Girder Beam

29.5# /ft

$$S = \frac{18,810}{8.69} + \frac{9595 \times .143 \times .143}{28.4} = 2,176\# / \text{sq. ft.}$$

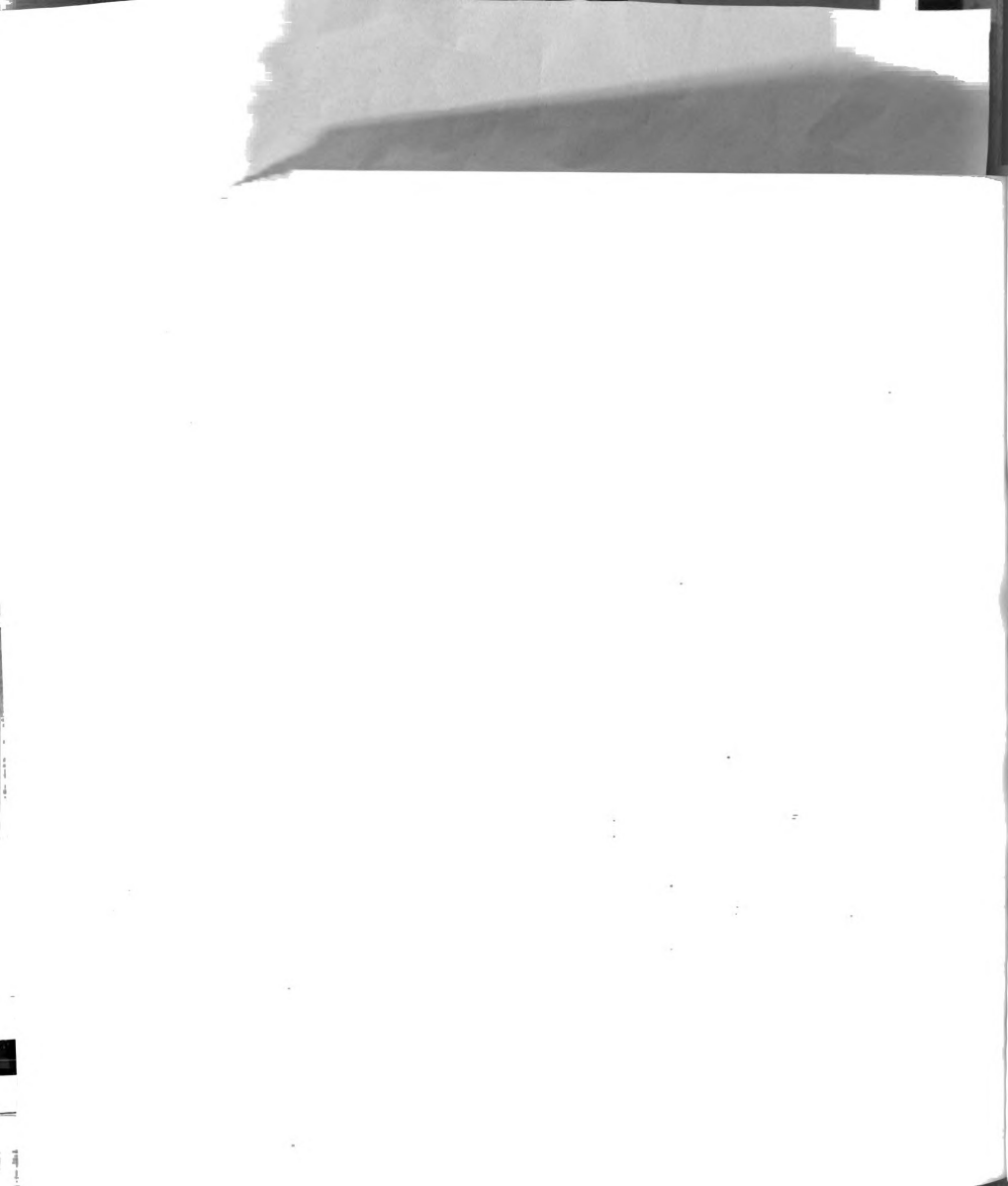
$$S @ \text{ first floor} = \frac{53,992}{8.69} + \frac{28367 \times 3.94 \times 3.94}{100.7} = 10,600$$

S allowable = 12,000 8" Col. OK

Approx. wt. of column = 40 x 29.5 = 1180#

Total load on bridge = 55,172 56,000#

* Supports for guide rails must occur at least every 12 feet.



Elevator Columns (Continued)

N. E. Column

Loads

1.	From Rear Supporting Beam -	7545
2.	" Penthouse Beam -	7115
3	" Roof Beam RBQ -	1300
4	" Floor Beam FB3Q	1750
5	" Floor Beam FB2Q	<u>1750</u>
		19460

The smallest column allowable for an unsupported length of 12' is a 6" B.S. section, 15.5# /ft.

The greatest possible bending stress would occur if the elevator load were applied without any loads on the other side of the column.

$$s = \frac{19460}{4.61} + \frac{7545 \times .12 \times .12}{9.19} = 3172\# / \text{sq. in.}$$

Full Load stress

$$s = \frac{19460}{4.61} + \frac{2745 \times .12 \times .12}{9.19} = 4254\# / \text{sq. in.}$$

Allowable stress = 12000# /sq. in.

Approx. wt. of column = 40 x 15.5 = 620#

Total load on bridge = 20,080

use 21,000#



Elevator Columns (Continued)

S. E. Column

Loads

1. From Front Supporting Beam -	2165
2. " Penthouse Beam -	7115
3. " Roof Beam RBV -	1300
4. " Floor Beam FB3V -	2765
5. " Floor Beam FB2V -	<u>2765</u>
Total	16110

Use 6" B.S. Section 15.5# /ft

Total load on bridge - 16,730#

use 18,000#

S. W. Column

Loads

1. from Front Supporting Beam -	3125
2. " Penthouse Beam -	7115
3. " Roof Beam RBT -	3725
4. " Roof Beam RBC -	750
5. " Floor Beam FB3T -	16325
6. " Floor Beam FB3C	2840
7. " Floor Beam FB2R	51540
8. " Floor Beam FB2C	<u>2840</u>
Total	88260

Eleventh Column (continued)

U. S. Column

Elevator Columns (Continued.)

S. W. Column (Cont.)

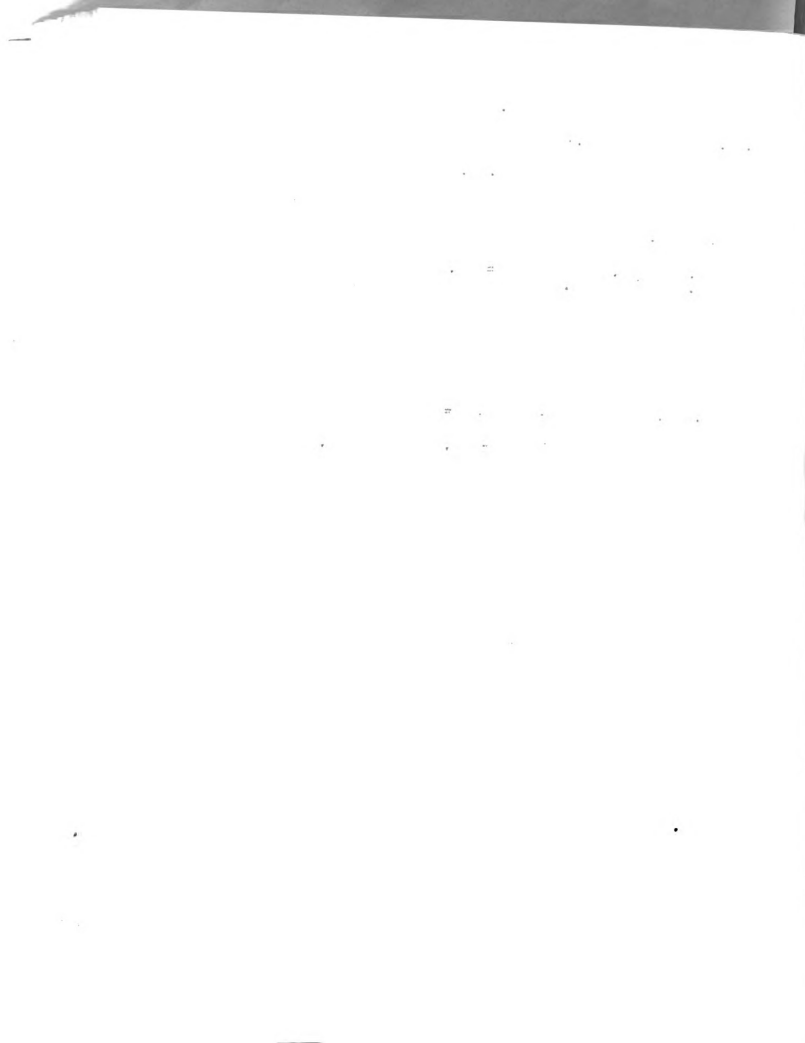
Choose A 35# 8" B. H. Section

Max stress =

$$\frac{88,260}{10.30} \quad \frac{51,540 \times 4 \times 4}{123.0} = 14,420$$

Approx. wt. of Column = 40 x 35 = 1400#

Total load on bridge = 89,660 use 90,000#



Hatchway enclosure.

For the hatchway enclosure 6" hollow tile as allowed by the Lansing Building Code will be used.

Wt. of 1' of 6 " hollow tile 40' high = $24 \times 40 = 960\#$

This wt. is carried to support beams by a beam having s =

$$\frac{960 \times 7 \times 7 \times 12}{18,000} = 31.3$$

use a 12" 31.8# std. I s = 35.97

Support Beams

West Beam (EB1A)

Length - 10'

Conc. Load 4' from R_L

1. from column	90,000
2. from front of hatchway - $960 \times 3\frac{1}{2} =$	3,360
3. from EB1C = 1968×6.5	<u>12,700</u>
	106,060#

Uniform Load (Rt. 6 Ft.)

1 from wall = 1968# /ft

Uniform Load (left 4 ft.)

1 from floor = 2500# /ft

(see EB1T)

$$R_L = \frac{6 \times 106,060}{10} + \frac{8 \times 4 \times 2500}{10} + \frac{3 \times 6 \times 1968}{10}$$

$R_L = 75,176$ $R_R = 52,692$ (Wt. not included)

$$s = \frac{(75,176 \times 4 - 10,000 \times 2) 12}{12,000} = 280,704$$

Use 27" 112# "C" Beam

s = 293.17



Support Beams (cont.)

East Beam (EBLB)

Length 10 ft.

Assume wt. = 100#

Loads

Conc. load 4' from R_L

1. Column load -	18000#
2. Front of hatchway -	3360
3. Front FBLV -	<u>2310</u>
	23670

Uniform load (Right 6 ft.)

1. Floor - (130 x 9) =	1170# /ft.
2. Hatchway -	<u>960# /ft</u>
	2130# /ft

Uniform load (left 4 ft)

1. Floor (130 x 5) =	650# /ft
----------------------	----------

$$R_L = \frac{6}{10} \times 23,670 + \frac{8}{10} \times 4 \times 750 + \frac{3}{10} \times 6 \times 2230^*$$

$$R_L = 20,622$$

$$R_R = 19,428$$

$$S = \frac{(20622 \times 4 - 750 \times 4 \times 2)}{12000} 12 = 74,488$$

use a 16", 50# "C" Beam $S = 81.95$

*Weight included



Wind Stresses

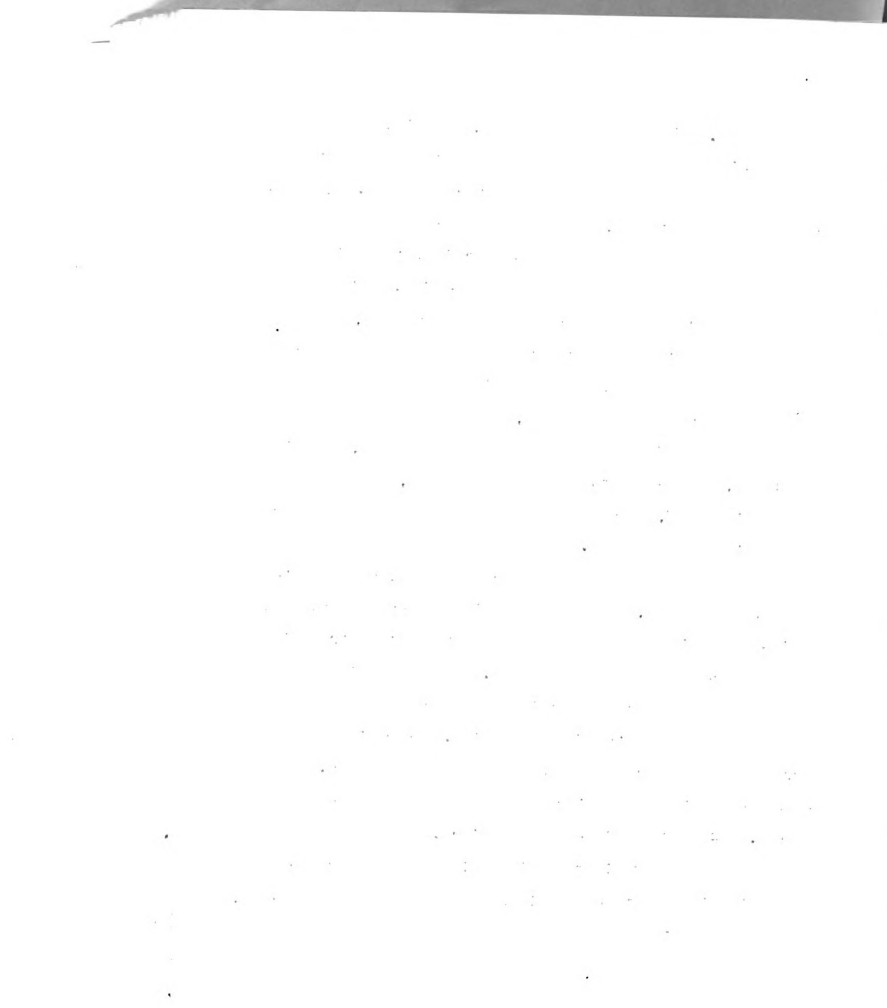
Wind Stresses,

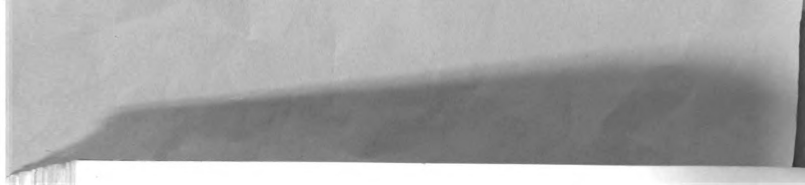
Although the building is low, and wind bracing is unnecessary; to comply with the Lansing building code the stress produced by the wind stress must be checked. According to the building code, the sum of the dead plus live plus wind stresses must not exceed the allowable stress (18000 pounds per square inch) by more than one-third. If the dead plus live stress were 18000 pounds per square inch, the allowable wind stress would be 6000 pounds per square inch.

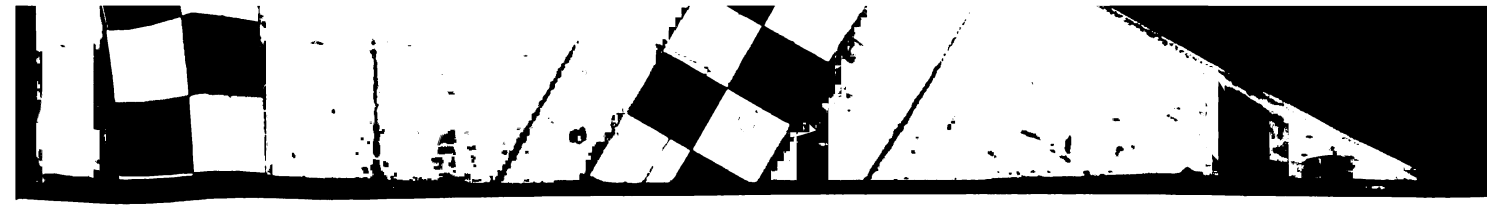
Using a panel length of fifteen feet and a height of three feet on the parapet wall, ten feet on each of the upper stories and fourteen feet on the first floor, the wind panel loads, at twenty pounds per square foot, are 2400 pounds on the roof, 3000 pounds at the third floor and 3600 pounds at the second floor.

The maximum total windstress in a column supporting the second floor, assuming the point of contraflexure at the mid-point of the column is equal to twice the stress in an exterior column on the same floor. The stress in an exterior column is found by dividing the moment of the wind panel loads by the width of the building. This gives a maximum stress of 2010 pounds over the entire section. The maximum stress in an interior column would be 4020 pounds or $4020 \div 7.89 = 510$ pounds per square inch.

Since the maximum possible windstress on an entire section is less than the allowable stress per square inch, it is evident that wind bracing is unnecessary.

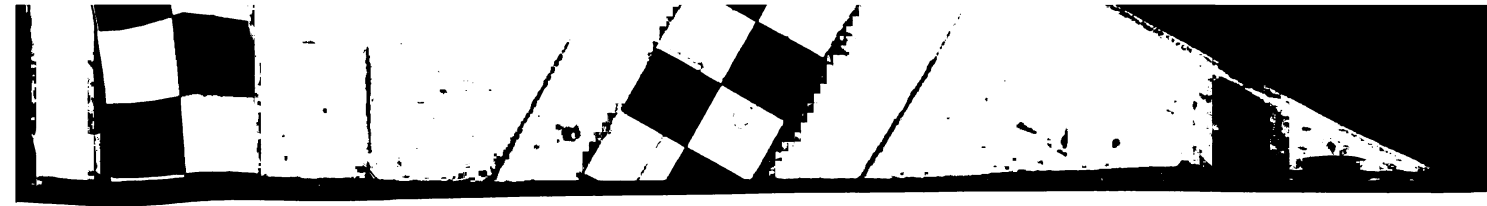






All plans are included in the pocket in the back
cover.





Conclusion



Conclusion.

An accurate estimate of the cost of this structure is impossible with the rapidly changing costs of construction and the uncertainty of the footing conditions of the location. To complete this project, however, an approximation of the cost has been made using a combination of several methods of estimating.

The cost as estimated is :

Steel frame 280,620# @ \$0.03	\$ 8,418.60
Steel erection	2,348.85
Roof - Aluminum sheets 12,600# @ \$0.43	5,418.00
Roofing surface 25,200# @ \$0.024	604.80
Installation 12,600 sq. ft. @ \$0.0125	157.50
Floors - in place 37,800 sq. ft @ \$0.75	28,350.00
(Covering and fireproofing included)	
Walls - Limestone 5080 Cu. Ft. @ \$1.60	8,128.00
Laying limestone 5080 Cu. Ft. @ \$0.71	3,606.80
8" Hollow tile 30480 @ \$0.14	4,267.20
6" " " 1040 @ \$0.11	114.40
Laying tile 31,520 @ \$0.065	2,048.80
Windows (garage) 918 sq. ft. @ \$0.76	697.68
" (ventilating) 11,260 sq. ft @ \$1.14	12,836.40
Aluminum spandrels 17,000 # @ \$0.50	8,750.00
Plaster 7758 sq. yds. @ \$0.25	1,939.50
Ceiling (Third floor)*	
Metal lath 1400 sq. yds. @ \$0.114	159.60
Partitions - Gypsum blocks	
24240 sq. ft. @ \$0.17	4,120.80

Elevator - Installed	\$ 5,000.00
Piers and Abutments#	24,000.00
Heating, Plumbing & Lighting 500,000C.F.F. @ \$0.05	<u>25,000.00</u>
	145,964.13
Add 10% for omissions, profit, etc.	<u>14,596.41</u>
Estimated cost	\$ 160,560.54

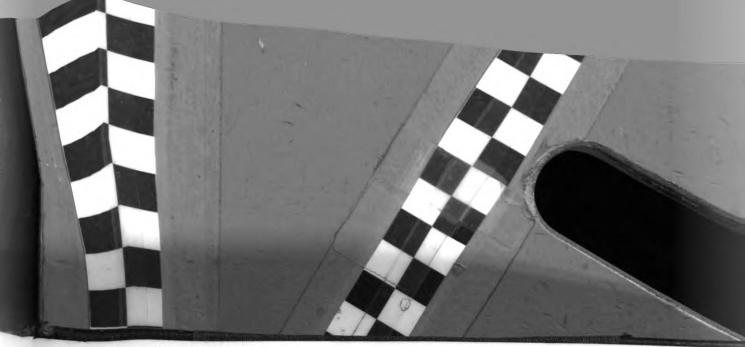
note: All items not followed by a charge for installation are given in place.

Although the above cost does not seem high as compared with the cost of other office buildings of equal volume, it must be remembered that prices are now (1932) approximately 1/3 to 1/4 lower than a few years ago, and since there is no present need for more office space in the City of Lansing, and since the bus companies of the city, since the starting of this thesis, have remodeled an existing building to suit their needs for several years in the future, the undertaking from a commercial standpoint would be unwise.

* Plaster for third story ceiling included under plaster.

Estimated from data in "Gillette" with proportionate prices.









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