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DESIGN OF A FOUR TRACK RAILROAD BRIDGE OVER THE BRONX RIVER PARKWAY AT WOODLAWN N. Y.

Submitted by

S. M. Morosoff.

May 1933

Signature: Signature redacted

Certification of the
Department of Civil
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Head of the Department:

Professor Merrill,
Secretary to the Faculty.

I submit herewith a thesis on the design of a bridge over the Bronx River Parkway Drive at Woodlawn N. Y. as partial fulfilment of the requirements for the degree of Bachelor of Science

Very respectfully

Signature redacted

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CONTENTS

AKNOWLEDGEMENT

The writer wants to express his sincere apprecia-
tion of the help and advice received from Mr. J. B.
Wilbur during the design of a railroad bridge which
forms the subject of this thesis.

S. Morosoff

INTRODUCTION

INTRODUCTION

The problem selected to form the subject of this thesis is unique in many respects. The site of the bridge is in Woodlawn N.Y. on the Harlem division of the New York Central Railroad. There are four railroad tracks under which the proposed Bronx River Parkway must pass. These tracks are almost parallel to the Bronx River. The level of the railroad tracks is fixed because of an overbridge a little upstream of the site under consideration. On the other hand the Parkway cannot be lowered beyond the high water level of the Bronx River. As a result of these two factors the depth of the floor of the proposed railroad bridge must be a minimum. This is a controlling feature in the choice of the type of structure and the arrangement adopted finally as described herein. In addition to the above limitations, the design is further complicated by the fact that the Parkway crosses the railroad at a skew angle, but this did not prove to be a difficult requirement to satisfy.

DATA

DESIGN OF A FOUR TRACK RAILROAD BRIDGE OVER THE BRONX RIVER PARKWAY DRIVE AT WOODLAWN, N. Y.

The data for the design are:

Total opening- from centre to centre of the bearings- 80 ft.

Required width- from centre to centre of the Fascia Girder- 55 ft. $2\frac{3}{8}$ in.

Angle between the axis of the railway track and the highway- 36°-0'.

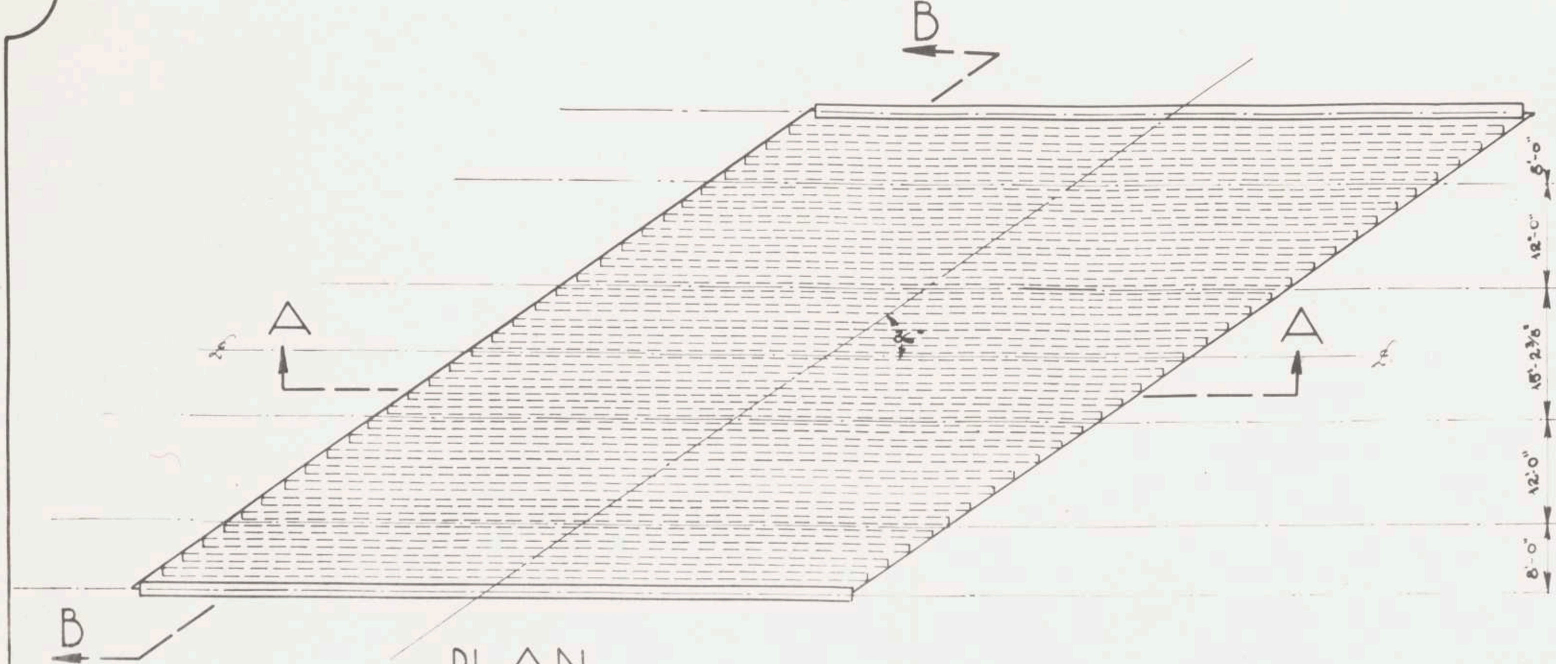
Solid Floor

Elevations:

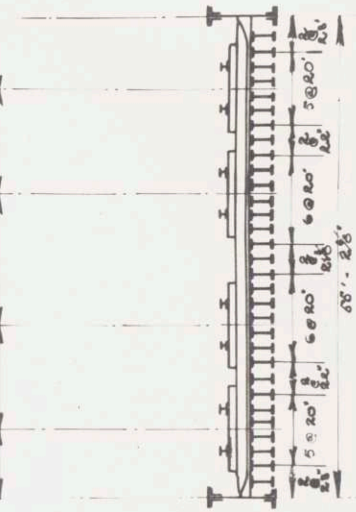
Foundations of Abutment	49.25 ft.
Top of Highway	54.75 ft.
Clearance for Highway (minimum headroom)	14.00 ft.
Proposed elevation of base of rail	73.19 ft.

Loading- Cooper E-70.

Specifications- Those of the New York Central Lines for Steel Railroad Bridges.



PLAN

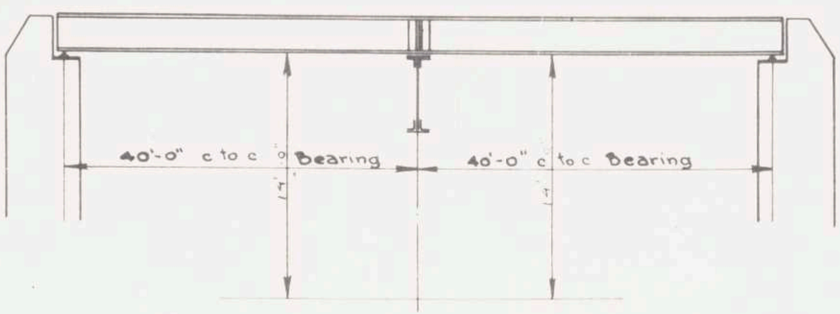


SECTION B-B

FIG-1

PRELIMINARY DESIGN

GENERAL ARRANGEMENT OF THE ELEMENTS OF THE STRUCTURE



SECTION A-A

DESIGN OF THE TRACK GIRDERS

DESIGN OF THE TRACK GIRDERS

According to the specifications, the minimum depth of the steel floor should be one-fourteenth of the length of the span. This would give us:

$$d = \frac{1}{14} L = \frac{40}{14} = 2.86 \text{ ft.}$$

The depth of the steel floor obtainable from the given elevations, however, is equal to 2.50 ft., and is computed as follows:

Elevation of the bottom of the floor: 54.75 ft. + 14 ft.=	68.75 ft.
Depth necessary for the provision of ballast, concrete slab, and asphalt-water-proofing: 2 ft.-10 in.=	2.84 ft.
The top of the floor will be at: 73.19 ft. - 2.84 ft. =	71.25 ft.
The depth of the floor obtainable will be: 71.25 ft. - 68.75 ft. =	2.50 ft.

According to the specifications, the allowable unit stress must vary directly as the ratio of the depths, and is given by the relation:

$$f_t = 18000 \frac{d_1}{d} = 18000 \frac{2.5}{2.86} = 15700 \text{ lb./sq. in.}$$

Dead load

Ballast: $2.25 \times \frac{20}{12} \times 120 =$ 450 lb./ lin.ft.

Concrete and asphalt: $\frac{6.25}{12} \times \frac{20}{12} \times 150 =$ 130 lb./ lin.ft.

Steel: 270 lb./ lin.ft.

Total: 850 lb./ lin.ft.

Live load.

The track load is distributed over six girders. The effect of such a distribution of live load on the maximum bending moment, the end shear, and the floor beam reaction is taken from specifications.

Maximum bending moment.

Dead load: $\frac{1}{8} \times 850 \times 40 \times 40 =$ 170,000 ft.lb.

Live load:(from specifications) = 382,000 ft.lb.

Impact:(from specifications) = 363,000 ft.lb.

Total : 915,000 ft.lb.

Maximum shear at the end of the girder.

Dead load: $\frac{1}{2} \times 850 \times 40 =$ 17,000 lb.

Live load: (from specifications) = 44,000 lb.

Impact: (from specifications) = 41,730 lb.

Total: 102,730 lb.

Web area.

The web area required for shear is given by:

$$A = \frac{102,730}{13,500} = 7.6 \text{ sq.in. net area.}$$

where 13,500 is the maximum allowable unit stress for shear as permitted by the specifications.

The section used will be a 27 in. CB section weighing 175 lb. per ft. When provision is made for six rivet holes, this section gives a net area of:

$$0.671 \times 27 - 6 \times 0.671 \times \frac{15}{16} = 14.23 \text{ sq. in.}$$

Section modulus.

The section modulus required at the bottom is equal to:

$$s = \frac{915,000 \times 12}{15,700} = 699 \text{ in}^3.$$

Area and moment of inertia of the cross section about an axis through 1-1

Item	Area	Moment of inertia
CB section 27 in. x 175 lb.=	51.46 in^2	6838 in^4
Deck plate 16 in. x $\frac{1}{2}$ in.=	8.00 "	1570 "
4 inside cover plates 4 in. x $\frac{1}{2}$ in.=	8.00 "	1208 "
2 bottom cover plates 14 in. x $\frac{1}{2}$ in.=	14.00 "	2840 "
Total gross area=	81.46 "	
Total gross mment of inertia=		12,476 "

Reduction in area due to the rivet holes.

$2 \times 2.71 \times \frac{15}{16} + 3 \times 0.671 \times \frac{15}{16} =$ 6.95 sq. in.

Net area.

$81.46 - 6.95 =$ 78.51 sq. in.

Displacement of the neutral axis.

$$X = \frac{8 \times 14 + 2 \times 2.71 \times \frac{15}{16} \times 13.45 + 0.671 \times \frac{15}{16}(2+6+10) - 14 \times 14.25}{78.51}$$

$= 0.16$ in.

Reduction in moment of inertia due to the rivet holes.

$$2.21 \times 2 \times \frac{15}{16}(13.45)^2 + 2 \times 2.71 \times \frac{15}{16}(13.45)^2 + 2 \times 0.671 \times \frac{15}{16}(2^2 + 6^2 + 10^2) = \quad 1914 \text{ in4}$$

Reduction in moment of inertia due to the displacement of the neutral axis.

$(0.16)^2 \times 74.51 =$ 2 in^4

Total reduction in the moment of inertia = 1916 in^4

Net moment of inertia.

$12,476 - 1916 =$ 10,560 in^4

Section modulus obtained on the bottom.

$$s = \frac{10,560}{14.9} = 709 \text{ in}^3$$

Moment of inertia of the cross section about an axis through 2-2

Item	Moment of inertia
CB section:	565.5 in^4
Deck plate:	170.7 "
2 bottom cover plates:	228.6 "
4 inside cover plates:	210.0 "
Total:	1174.8 "

Minimum radius of gyration of the cross section.

$$r = \sqrt{\frac{1174.8}{81.46}} = 3.7 \text{ in.}$$

Web diaphragm.

According to the specifications a solid web diaphragm must be provided between the track girders, and its maximum spacing is given by:12 x width of flange: i.e.

$$\frac{12 \times 14}{12} = 14 \text{ ft.}$$

This is the unsupported length of the track girders in a horizontal plane.

Allowable unit compressive stress.

The allowable unit compressive stress is now given by:

$$f_c = 15{,}700 - 70 \times \frac{40 \times 12}{3.7} = 12{,}520 \text{ lb./ sq. in.}$$

Section modulus required on the top.

$$s = \frac{915,000 \times 12}{12,520} = 877 \text{ in}^3$$

Section modulus obtained on the top

$$s = \frac{12,476}{14.1} = 885 \text{ in}^3$$

Lengths of cover plates.

The lengths of the cover plates are obtained graphically (see fig. 2)

Weight of the girder per linerar foot.

The actual weight of the girder per linear foot is as follows:

Item		
CB section:	175.	lb.
Deck plate: $\frac{20}{12} \times 22.95 =$	38.25	"
2 bottom cover plates: $\frac{14}{12} \times \frac{55}{40} \times 22.95 =$	37	"
4 inside cover plates: $\frac{4}{12} \times \frac{72}{40} \times 22.95 =$	13.7	"
Rivet heads and diaphragm:	6	"
Total:	269.95	"

Vertical flange rivet pitch.

Capacity of one rivet:

Single shear: 0.6 x 13,500 = 8,100 lb./rivet.

Bearing: $\frac{1}{2} \times \frac{7}{8} \times 27,000 = 11,800$ lb./rivet.

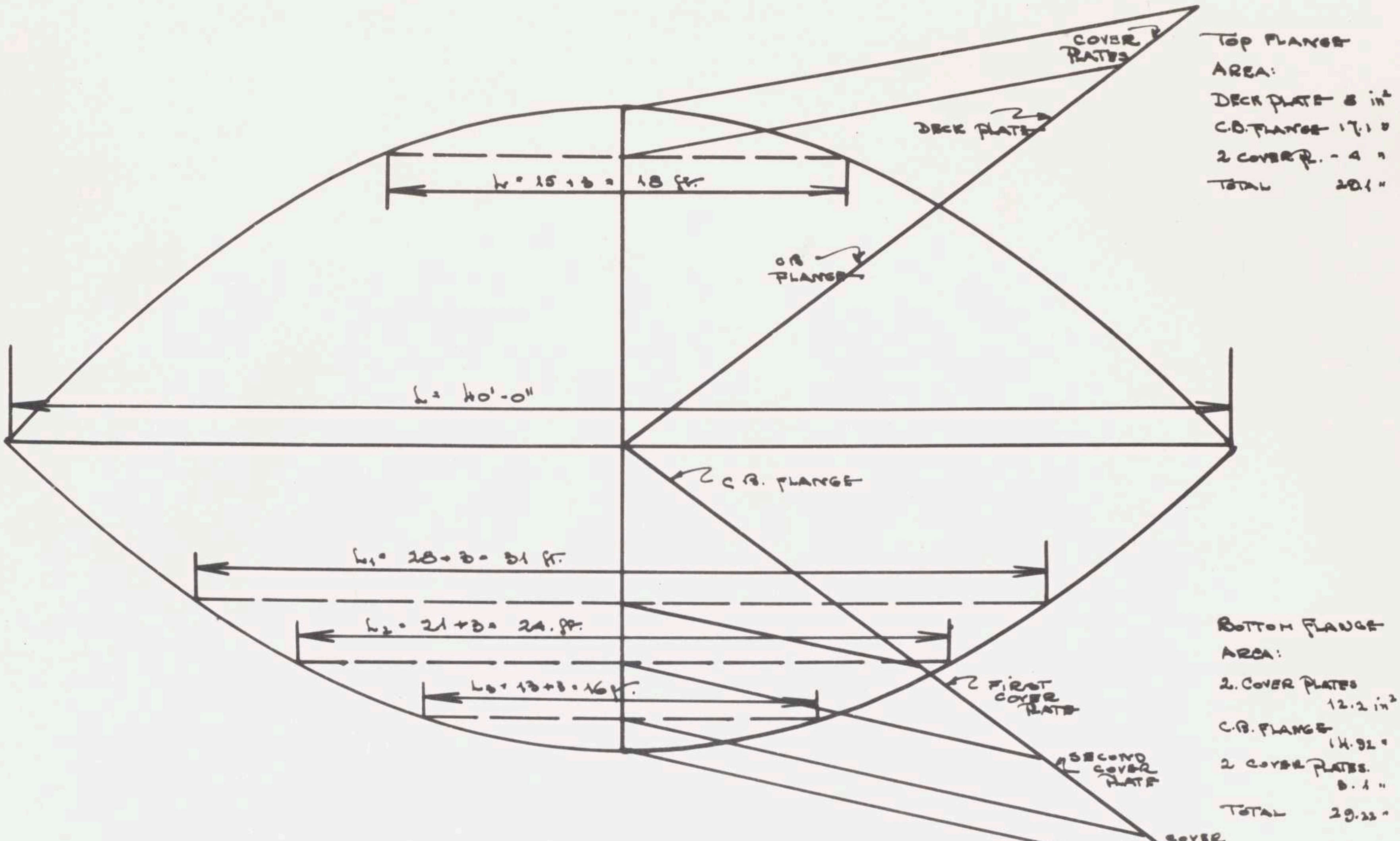


FIG-2.

GRAPHICAL DETERMINATION OF THE LENGTH OF COVER PLATES.

a) Bottom flange.

$$Z = \frac{V\ Q}{I}$$

where Z is the shear per linear inch.

V the total maximum shear.

Q the statical moment of the bottom cover plate about axis 1-1.

I the gross moment of inertia of the cross section about the centre of gravity(axis 1-1)

$Q = 7 \times 14.9 = 104.3\ in^3$

$Z = \dfrac{102,730 \times 104.3}{12,476} = 860$ lb./ lin. in.

$p = \dfrac{8,100}{860} = 9.4$ in.

A pitch of 6 inches will be used.

b) Top flange

$Q = 8 \times 14.1 = 112.8\ in^3$

$Z = \dfrac{102,730 \times 112.8}{12,476} = 930$ lb./ lin. in.

$p = \dfrac{8,100}{930} = 8.7$ in.

A pitch of 6 inches will be used.

Solid web diaphragm.

As stated above, the maximum spacing of the web diaphragms is 14 ft. Two diaphragms will be used on each span, and their location is shown in fig, Scantlings of material used for the diaphragm.

The materials used for the diaphragm shall be:

1 web plate $24\frac{1}{2}$ in. x $\frac{3}{8}$ in.

4 connecting angles $3\frac{1}{2}$ x $3\frac{1}{2}$ x $\frac{3}{8}$ in.

End stiffeners.

The bearing area required for the end stiffeners is equal to:

$$\frac{102,730}{27,000} = 3.8 \text{ sq. in.}$$

At the abutments 4 angles 5 x $3\frac{1}{2}$ x $\frac{1}{2}$ in. shall be used. The effective bearing area is then equal to:

$$4 \left(4 - \frac{3}{4} \times 5 \times \frac{1}{2} \right) = 7.5 \text{ sq. in.}$$

At the cross girders 2 angles 6 x 4 x $\frac{1}{2}$ in. shall be used. The effective bearing area here is equal to:

$$2 \left(4.75 - \frac{3}{4} \times 6 \times \frac{1}{2} \right) = 5 \text{ sq. in.}$$

Depth of steel floor obtained.

The depth of the steel floor obtained is given by:

1 CB beam:	h = 27.5 in.
2 bottom cover plates:	h = 1.0 "
1 deck plate:	h = 0.5 "

The total depth is equal to 29.0 inches.

DESIGN OF THE CROSS GIRDERS

DESIGN OF THE CROSS GIRDERS

Dimensions along the highway axis.

sine 36° = 0.5878

The width of the bridge along the highway axis is then:

$$\frac{55.2}{0.5878} = 93 \text{ ft.-11 in.}$$

A total of 7 columns (6 spaces) will be used.

Length of the cross girders.

The cross girders are coincident with the highway axis running through the centre of the bridge.Their lengths are given as follows:

2 cross girders N° 1 -	15 ft.- 0 in. each.
2 cross girders N° 2 -	15 ft.- $11\frac{3}{4}$ in. each.
2 cross girders N° 3 -	15 ft.- $11\frac{3}{4}$ in. each.
Total length-	93 ft. - 11 in.

Spacing of the track girder on a cross girder.

Under all the tracks the spacing is equal to:

$$\frac{20}{0.5878} = 34 \text{ in.} = 2 \text{ ft. } 10 \text{ in.}$$

Between the Fascia girder and track 1 (also between track 4and the corresponding Fascia girder) the spacing is equal to:

$$\frac{23}{0.5878} = 39.128 \text{ in.} = 3 \text{ ft.} - 3\tfrac{1}{8} \text{ in.}$$

Between track 1 and track 2 (also between track 3 and track 4) the spacing is equal to:

$$\frac{22}{0.5878} = 37.43 \text{ in.} = 3 \text{ ft.} - 1\tfrac{7}{16} \text{ in.}$$

Between track 2 and track 3 the spacing is equal to:

$$\frac{21\frac{3}{16}}{0:5878} = 36.04 \text{ in.} = 3 \text{ ft.} - 0 \text{ in.}$$

Live load reaction

The maximum floor beam reaction due to the live load for a span of 40 ft. is given in the specifications as 189,000 lb. per rail. Since the track load is distributed over 6 girders, the maximum live load reaction due to the track girders is:

$$\frac{189,000}{3} = 63,000 \text{ lb. per girder.}$$

Dead load reaction

a) Due to the track girders spaced 20 inches apart:

(all weights are given in lbs. per lin. ft.)

Weight of ballast: $2.25 \times \frac{20}{12} \times 120 =$ 450 lb.

Weig t of concrete: $\frac{6.25}{12} \times \frac{20}{12} \times 150 =$ 130 lb.

Weight of steel: 270 lb.

Total weight: 850 lb.

The reaction of the cross girders to the above load is equal to:

$$\frac{850 \times 40}{2} = 17,000 \text{ lb.}$$

The dead load reaction where twotrack girders come at the same point is:

2 x 17,000 = 34,000 lb.

b) Due to the track girders spaced $21\frac{3}{16}$ inches apart:

Weight of ballast: $2.25 \times \frac{21\ 3/16}{12} \times 120 =$ 476 lb.

Weight of concrete: $\frac{6.25}{12} \times \frac{21\ 3/16}{12} \times 150 =$ 140 lb.

Weight of steel: 270 lb.

Total weight: 886 lb.

The reaction of the cross girders to the above load is equal to:

$$\frac{886 \times 40}{2} = 17,700 \text{ lb.}$$

The dead load reaction where two track girders come at thesame point is:

2 x 17,700 = 35,400 lb.

c) Due to the track girders spaced 22 inches apart:

Weight of ballast: $2.25 \times \frac{22}{12} \times 120 =$	495 lb.
Weight of concrete: $\frac{6.25}{12} \times \frac{22}{12} \times 150 =$	145 lb.
Weight of steel:	270 lb.
Total weight:	910lb.

The reaction of the cross girders to the above load is equal to:

$$\frac{910 \times 40}{2} = 18{,}200 \text{ lb.}$$

The dead load reaction where two track girders come at the same point is:

2 x 18,200 = 36,400 lb.

d) Due to the track girdersspaced 23 inches apart:

Weight of ballast: $2.25 \times \frac{23}{12} \times 120 =$	520 lb.
Weight of concrete: $\frac{5}{12} \times \frac{23}{12} \times 150 =$	120 lb.
Weight of steel:	300 lb.
Total weight:	940 lb.

The reaction of the cross girders to the above load is equal to:

$$\frac{940 \times 40}{2} = 18{,}800 \text{ lb.}$$

The dead load reaction where two track girders come at the same point is:

2 x 18,800 = 37,600 lb.

Impact coefficient

The impact coefficient for all the cross girders is given by:

$$I = \frac{30,000}{30,000 + L^2} = \frac{30,000}{30,000 + 80^2} = 0.824$$

DESIGN OF CROSS GIRDER Nº 1

The loading on the girder is shown in plate

Maximum shear.

$$\text{Dead load: } (1000)\left\{\frac{37.6}{15.0} \times 3.26 + \frac{34.0}{15.0} \times (6.52+9.35+12.18) + \frac{0.25}{2} \times 15.0\right\} = \quad 73,500 \text{ lb.}$$

$$\text{Live load: } \frac{63.0}{15.0} \times (6.52+9.35+12.18)(1000) = 117,800 \text{ lb.}$$

Impact: 117,800 x 0.824= 97,500 lb.

Total = 288,800 lb.

Maximum bending moment.

$$\text{Dead load: } (1000)\left\{73.5 \times 5.66 - \frac{0.25}{2} \times 5.66^2 - 34 \times 2.83\right\} = \quad 315,000 \text{ ft.lb.}$$

$$\text{Live load: } (1000)\left\{117.8 \times 5.66 - 63 \times 2.83\right\} = \quad 488,300 \text{ ft. lb.}$$

Impact: 488,400 x 0.824 = 402,300 ft. lb.

Total: 1,205,600 ft.lb.

Maximum quarter point shear

$$\text{Dead load: } (1000)\left\{68.1 - 37.6 - 0.25 \times \frac{15}{4}\right\} = \quad 29,600 \text{ lb.}$$

Live load	71,200 lb.
Impact: 71,200 x 0.824	58,500 lb.
Total:	159,300 lb.

Shear at the centre of the girder

Dead load: $(1000)\left\{68.1 - 37.7 - 34 - 0.25 \times 7.5\right\} = \quad - 1000$ lb.

Live load: $(1000)(71.2 - 63.0) =$ 8200 lb.

Impact: $8200 \times 0.824 =$ 6700 lb.

Total 13900 lb.

Web

The web area required is equal to:

$$\frac{288,800}{13,500} = 21.4 \text{ sq.in. net.}$$

The net area obtained with a section 48 in. x $\frac{9}{16}$ in. after provision is made for 8 rivet holes is equal to:

$$48 \times \frac{9}{16} - 8 \times \frac{9}{16} \times \frac{15}{16} = 22.5 \text{ sq. in.}$$

Section modulus

The section modulus required at the bottom is given by:

$$s = \frac{1,205,600 \times 12}{18,000} = 803 \text{ in}^3$$

Area and moment of inertia of the cross section

about axis 1-1

Item	Area	Moment of inertia
Web- $48 \times \frac{9}{16}$ in.=	27.0 in^2	5184 in^4
4 angles $6 \times 6 \times \frac{1}{2}$ =	23.0 "	11800 "
2 plates $14 \times \frac{1}{2}$ =	14.0 "	8400 "
Total gross area=	64.0 "	
Total gross moment of inertia =		25384 "

Reduction in area due to rivet holes

$$\frac{15}{16} \times \frac{25}{16} + 3 \times \frac{9}{16} + 2 \times 1 = 5.05 \text{ sq. in.}$$

Net area

$64.0 - 5.05 = 58.95$ sq. in.

Reduction of moment of inertia for rivet holes

$$4 \times 24.5^2 + 2 \times \frac{25}{16} \times 21^2 + 2 \times \frac{9}{16} \times (15^2 + 9^2 + 3^2)$$
$$= 4132 \text{ in}^4$$

Displacement of neutral axis

$$X = \frac{2 \times 1 \times 24.25 + 1 \times \frac{25}{16} \times 21 + 1 \times \frac{9}{16}(15 + 9 + 3)}{58.95}$$

$= 1.37$ in.

Reduction in moment of inertia for displacement of neutral axis

$58.95 \times 1.37^2 = 100 \text{ in}^4$

Net moment of inertia about the neutral axis

$25{,}384 - 4{,}132 - 100 = 21{,}152 \text{ in}^4$

Moment of inertia of the cross section about axis 2-2

Item	
2 cover plates 14 x $\frac{1}{2}$:	228.67 in^4
4 angles 6 x 6 x $\frac{1}{2}$:	165.24 "
Total:	393.91 "

The minimum radius of gyration of the cross section

$$r = \sqrt{\frac{393.91}{64}} = 2.5 \text{ in.}$$

Allowable unit compressive stress

The allowable unit compressive stress is given by:

$$f_c = 18{,}000 - 70\frac{15 \times 12}{2.5} = 12{,}960 \text{ lb./ sq. in.}$$

Section modulus required on the top

$$s = \frac{1{,}205{,}600 \times 12}{12{,}960} = 1023 \text{ in}^3$$

Section moduli obtained

a) Section modulus obtained on the bottom

$$s = \frac{21,152}{26.14} = 809 \text{ in}^3$$

b) Section modulus obtained on the top

$$s = \frac{25,384}{23.37} = 1086 \text{ in}^3$$

Vertical flange rivet pitch

Allowable stress on rivets:

Single shear: 8,100 lb./ rivet

Double shear: 16,200 lb./ rivet.

Bearing: 13,290 lb./ rivet.

$$Q = 7 \times 24.5 = 171.5 \text{ in}^3$$

$$Z = \frac{V\,Q}{I} = \frac{288,800 \times 171.5}{25,384} = 1,950 \text{ lb./ lin. in.}$$

$$p = \frac{8,100 \times 2}{1,950} = 8.3 \text{ in.}$$

A pitch of 6 inches will be used.

Horizontal flange rivet pitch.

$$Q = 171.5 + 11.5 \times 22.57 = 331 \text{ in}^3$$

$$Z = \frac{288,800 \times 331}{25,384} = 3,700 \text{ lb./ lin. in.}$$

$$p = \frac{13,290}{3,700} = 3.59 \text{ in.}$$

A pitch of $3\frac{1}{2}$ inches will be used, and the rivets shall be located in two rows.

Horizontal flange rivet pitch at the quarter point

$$Z = \frac{161,100 \times 328.5}{25,384} = 2,090 \text{ lb./ lin. in.}$$

$$p = \frac{13,290}{2,090} = 6.35 \text{ in.}$$

A pitch of 6 inches will be used and the rivets shall be located in two rows.

Horizontal flange rivet pitch at the centre of the girder.

$$Z = \frac{11,550 \times 328.5}{25,384} = 150 \text{ lb./ lin. in.}$$

$$p = \frac{13,290}{150} = 8.86 \text{ in.}$$

A pitch of 6 inches shall be used and the rivets shall be located in two rows.

Intermediate stiffeners

The intermediate stiffeners shall be located 48 inches apart and shall be composed of angles $5 \times 3\frac{1}{2} \times \frac{3}{8}$ inches and of fillers $3\frac{1}{2} \times \frac{1}{2} \times 36$ inches.

End stiffeners

The bearing area required for the end stiffeners is given by:

$$\frac{288,800}{27,000} = 10.7 \text{ sq. in.}$$

At each end 4 angles $5 \times 3\frac{1}{2} \times \frac{1}{2}$ in. shall be used each in conjunction with a filler plate $3\frac{1}{2} \times \frac{1}{2} \times 36$ in.

The effective bearing area is then given by:

$$4 \left(4 - 3\frac{1}{2} \times \frac{1}{2} \times \frac{3}{4} \right) = 10.8 \text{ sq. in.}$$

Number of rivets required

Capacity of one rivet

a) Double shear: $2 \times 0.6 \times 13,500 = 16,200$ lb.

b) Bearing: $\frac{7}{8} \times \frac{9}{16} \times 27,000 = 13,290$ lb.

The number of rivets required is then:

$$\frac{288,800}{13,290} = 27.7$$

Increase in number of rivets for effect of filler:

$$\frac{27.7}{3} = 9.2$$

Total number of rivets = 37

A total of 38 rivets shall be used.

Actual weights

Web- 4 x 22.95	91.8 lb/. lin/ ft
4 angles- 4 x 19.6	78.4 "
2 cover plates- $\frac{28}{12} \times 20.4$	47.6 "
Stiffener-	10.0 "
Filler-	6.0 "
Rivet heads-	6.0 "
Total-	240 lb/lin/ ft.

DESIGN OF CROSS GIRDER N° 2

The loading on the girder is shown in plate

Maximum shear.

$$\text{Dead load: } (1000)\left\{\frac{36.4}{15.98} \times 8.78 + \frac{34.0}{15.98}(2.83+5.66+11.9+14.73) \; \frac{0.25}{2} \times 15.98\right\} = \qquad 98,200$$

$$\text{Live load: } (1000)\frac{63.0}{15.98}(2.83+5.66+11.9+14.73) = 138,600$$

Impact: 138,600 x 0.824 = 114,200

Total: 351,000

Maximum bending moment.

$$\text{Dead load: } (1000)\left\{76.8 \times 5.66 - \frac{0.25\,(5.66)^2}{2} - 34 \times 2.83\right\} = \qquad 334,300 \text{ ft.lb.}$$

$$\text{Live load: } (1000)\left\{113.4 \times 8.78 - 63\,(3.12+5.95)\right\} = \qquad 463,500 \text{ ft.lb.}$$

Impact: 463,500 x 0.824 = 381,900 ft.lb.

Total: 1,178,700 ft.lb.

Maximum quarter point shear.

$$\text{Dead load: } (1000)\; 98.2 - 34.0 \;\; 0.25 \times \frac{15.98}{4} = \qquad 63,200 \text{ lb.}$$

Live load: $(1000)(138.6 - 63) =$ 75,600 lb.

Impact: $75,600 \times 0.824 =$ 62,300 lb.

Total: 221.100 lb.

Shear at the centre of the girder.

$$\text{Dead load:}(1000)\left\{76.8 - 34 - \frac{0.25 \times 15.98}{2}\right\} = 6,800 \text{ lb.}$$

Live load: $(1000)(113.4 - 2 \times 63) =$ -12,600 lb.

Impact: $12,600 \times 0.824 =$ -10,400 lb.

Total: -16,200 lb.

Web.

The web area required is equal to:

$$\frac{351,200}{13,500} = 26.0 \text{ sq. in. net.}$$

The net area obtained with a section 48 in. $\times \frac{5}{8}$ in. after provision is made for 8 rivet holes is equal to:

$$48 \times \frac{3}{4} - 8 \times \frac{3}{4} \times \frac{15}{16} = 30.3 \text{ sq. in.}$$

Section modulus required at the bottom

The section modulus required at the bottom is given by:

$$s = \frac{1,178,700 \times 12}{18,000} = 786 \text{ in}^3$$

Area and moment of inertia of the cross section about axis 1-1

Item	Area	Moment of inertia
Web: $48 \times \frac{5}{8}$ in. =	$36.0\ in^2$	$6912\ in^4$
4 angles $6 \times 6 \times \frac{1}{2}$ =	23.0 "	11800 "
2 plates $14 \times \frac{1}{2}$ in. =	14.0 "	8400 "
Total gross area =	73.0 "	
Total gross moment of inertia =		27,112 "

Reduction in area due to the rivet holes.

$$\frac{15}{16} \times \frac{13}{8} + 3 \times \frac{5}{8} + 2 \times 1 = 5.5 \text{ sq. in.}$$

Net area.

$$73.0 - 5.5 = 67.5 \text{ sq. in.}$$

Reduction in moment of inertia due to the rivet holes

$$4 \times 24.5^2 + 2 \times \frac{13}{8} \times 21^2 + 2 \times \frac{5}{8} \times \frac{15}{16} (15^2 + 9^2 + 3^2)$$

$$= 4270\ in^4$$

Displacement of the neutral axis

$$X = \frac{2 \times \frac{15}{16} \times 24.25 + 1 \times 1.75 \times \frac{15}{16} \times 21 + \frac{5}{8} \times \frac{15}{16}(15+9+3)}{67.5}$$

$$= 1.41 \text{ in.}$$

Reduction in the moment of inertia of the cross section for the displacementof the neutral axis

$67.5 \times 1.41^2 = 134 \text{ in}^4$

Net moment of inertia about the **neutral** axis

$27{,}112 - 4{,}270 - 134 = 22{,}708 \text{ in}^4$

Moment of inertia of the cross section about axis 2-2

Item	Moment of inertia
2 cover plates 14 in x $\frac{1}{2}$ in=	228.67 in^4
4 angles 6 x 6 x $\frac{1}{2}$ in =	165.24 "
Total:	393.91 "

The minimum radius of gyration of the cross section

$$r = \sqrt{\frac{393.1}{73}} = 2.33 \text{ in.}$$

Allowable unit compressive stress

$$f_c = 18{,}000 - 70 \times \frac{15.98 \times 12}{2.33} = 12{,}240 \text{ lb./ sq. in.}$$

Section modulus required on the top

$$s = \frac{1.178{,}700 \times 12}{12.240} = 1{,}156 \text{ in}^3$$

Section moduli obtained

a) Section modulus obtained on the bottom

$$s = \frac{22,708}{26.16} = 868 \text{ in}^3$$

b) Section modulus obtained on the top

$$s = \frac{27,112}{23.34} = 1,162 \text{ in}^3$$

Vertical flange rivet pitch

Allowable stress on rivets:

Double shear: 16,200 lb./ rivet.

Bearing: $\frac{7}{8} \times \frac{5}{8} \times 27,000 = 14,770$ lb./ rivet.

$$Q = 14 \times \frac{1}{2} \times 26.25 = 172 \text{ in}^3$$

$$Z = \frac{351,200 \times 172}{27,112} = 2,224 \text{ lb./ lin in.}$$

$$p = \frac{8,100 \times 2}{2,224} = 7.28 \text{ in.}$$

A pitch of 6 inches will be used.

Horizontal flange rivet pitch

$$Q = 172 + 11.5 \times 24.32 = 431.5 \text{ in}^3$$

$$Z = \frac{351,200 \times 431.5}{27,112} = 5,580 \text{ lb./ lin. in.}$$

$$p = \frac{14,770}{5,580} = 2.64 \text{ in.}$$

A pitch of $2\frac{1}{2}$ inches will be used and the rivets shall be located in two rows.

Horizontal flange rivet pitch.

$$Z = \frac{221,160 \times 431.5}{27,112} = 3,520 \text{ lb./ lin. in.}$$

$$p = \frac{14,770}{3,520} = 4.2 \text{ in.}$$

A pitch of 4 inches will be used and the rivets shall be located in two rows.

Horizontal flange rivet pitch at the centre of the girder.

$$Z = \frac{16,200 \times 431.5}{27,112} = 260 \text{ lb./ lin. in.}$$

$$p = \frac{14,770}{260} = 56.8 \text{ in.}$$

A pitch of 6 inches shall be used and the rivets shall be located in two rows.

Intermediate stiffeners.

The intermediate stiffeners shall be located 48 inches apart and shall be composed of angles $5 \times 3\frac{1}{2} \times \frac{3}{8}$ inches, and of fillers $3\frac{1}{2} \times \frac{1}{2} \times 36$ inches.

End stiffeners

The bearing area required for the end stiffeners is given by:

$\frac{351,200}{27,000} = 13$ sq. in.

At each end 4 angles $5 \times 3\frac{1}{2} \times \frac{3}{4}$ in. shall be used, each in connectionwith a filler plate $3\frac{1}{2} \times \frac{1}{2} \times 36$ in.

The effective bearing area is then given by:

$4\ (\ 5.81 - 3\frac{1}{2} \times \frac{3}{4} \times \frac{3}{4}\ ) = 15.4$ sq. in.

Number of rivets required.

Capacity of one rivet

a) Double shear: 16,200 lb./ rivet.

b) Bearing: $\frac{7}{8} \times \frac{5}{8} \times 27,000 = 14,760$ lb./ rivet.

The number of rivets required is then:

$\frac{351,200}{14,760} = 23.7$

Increase in number of rivets for effect of filler:

$\frac{23.7}{3} = 8$

Total number of rivets required = 32

A total of 32 rivets shall be used.

Actual weights

Web: 4 x 22.95 =	122 lb./ lin. ft.
4 angles: 4 x 19.6 =	78.4 "
2 cover plates: $\frac{28}{12} \times 20.4$ =	47.6 "
Stiffeners, fillers and rivet heads =	22.4 "
Total:	270.4 lb./ lin. ft.

DESIGN OF CROSS GIRDER N° 3

The loading on the girder is shown in plate

Maximum shear

$$\text{Dead load: } (1000)\left\{\frac{35.4}{15.98}(5.83+3) \quad \frac{34.0}{15.98}(8.66+11.49+14.32) \quad \frac{0.4}{2} \times 15.98\right\} = 96{,}800 \text{ lb}$$

$$\text{Live load: } (1000)\frac{63.0}{15.98}(5.83+8.66+11.49+14.32) = 158{,}800 \text{ lb}$$

Impact: 158,800 x 0.824 = 130,900 lb

Total: 386,500 lb

Maximum bending moment.

$$\text{Dead load: } (1000)\left\{96.8 \times 7.24 - \frac{0.4\,(7.24)^2}{2} - 34(5.66+2.83)\right\} = 367{,}500 \text{ ft. lb.}$$

$$\text{Live load: } (1000)\left\{158.8 \times 7.24 - 63(5.66+2.83)\right\} = 614{,}800 \text{ ft. lb.}$$

Impact: 614,800 x 0.824 = 506,600 ft. lb.

Total: 1,488,900 ft. lb.

Maximum quarter point shear.

$$\text{Dead load: } (1000)\left\{96.8 - 34.0 - \frac{0.4 \times 15.98}{4}\right\} = 61{,}400 \text{ lb.}$$

Live load: (1000)(158.8 - 63) =	95,800 lb.
Impact: 95,800 x 0.824 =	79,000 lb.
Total:	236,200 lb.

Shear at the centre of the girder.

$$\text{Dead load:}(1000)\left\{84.3 - 2 \times 35.4 - \frac{0.4 \times 15.98}{2}\right\} = \quad 10,000 \text{ lb.}$$

Live load: (1000)(93.2 - 63) =	30,200 lb.
Impact: 30,200 x0.824 =	24,900 lb.
Total:	65,100 lb.

Web.

The web area required is equal to:

$$\frac{386,500}{13,500} = 28.6 \text{ sq. in. net.}$$

The net area obtained with a section 48 in. x $\frac{3}{4}$ in. after provision is made for 8 rivet holes is equal to:

$$48 \times \frac{3}{4} - 8 \times 1 \times \frac{3}{4} = 30.0 \text{ sq. in.}$$

Section modulus required at the bottom

The section modulus required at the bottom is given by:

$$s = \frac{1,488,900 \times 12}{18,000} = 992 \text{ in}^3$$

Area and moment of inertia of the cross section about axis 1-1

Item	Area	Moment of inertia
Web 48 x $\frac{3}{4}$ in.=	36.0 in^2	6912 in^4
4 angles 6 x 6 x $\frac{3}{4}$ in.=	33.8 "	14940 "
2 cover plates 14 x $\frac{3}{4}$ in =	21.0 "	12740 "
Total gross area =	90.8 "	
Total gross moment of inertia =		34,592 "

Reduction in,area due to the rivet holes.

$$\frac{15}{16} \times \frac{9}{4} + 3 \times \frac{3}{4} + 2 \times 1.5 = 7.5 \text{ sq. in.}$$

Net area

90.8 - 7.5 = 83.3 sq, in.

Reduction in moment of inertia due to the rivet holes.

$$4 \times 1.5 \times 24.5^2 + 2 \times \frac{9}{4} \times 21^2 + \frac{3}{4} \times \frac{15}{16} (15^2 + 9^2 + 3^2)$$

$= 5805 \text{ in}^4$

Displacement of the neutral axis.

$$X = \frac{2 \times 1.5 \times \frac{15}{16} \times 24.25 + 1 \times 2.25 \times \frac{15}{16} \times 21 + \frac{3}{4} \times \frac{15}{16} (15+9+3)}{83.3}$$

=1.58 in.

Reduction in the moment of inertia of the cross section for the displacement of the neutral axis.

$90.8 \times 1.58^2 = 227 \text{ in}^4$

Net moment of inertia about the neutral axis.

$34{,}592 - 5{,}805 - 227 = 28{,}560 \text{ in}^4$

Moment of inertia of the cross section about axis 2-2

Item	Moment of inertia
2 cover plates 14 in. x $\frac{3}{4}$ in. =	343 in^4
4 angles 6 x 6 x $\frac{3}{4}$ in. =	270 "
Total:	613 "

The minimum radius of gyration of the cross section

$$r = \sqrt{\frac{613}{90.8}} = 2.6 \text{ in.}$$

Allowable unit compressive stress.

$$f_c = 18{,}000 - 70 \times \frac{15.98 \times 12}{2.6} = 12{,}840 \text{ lb./ sq. in.}$$

Section modulus required on the top

$$s = \frac{1{,}488{,}900 \times 12}{12{,}840} = 1{,}390 \text{ in}^3$$

Section moduli obtained

a) Section modulus obtained on the top

$$s = \frac{34,592}{23.47} = 1,470 \text{ in}^3$$

b) Section modulus obtained on the bottom

$$s = \frac{28,560}{26.53} = 1,076 \text{ in}^3$$

Vertical flange rivet pitch

Allowable stress on rivets:

Double shear: 16,200 lb./ rivet.

Bearing: $\frac{7}{8} \times \frac{3}{4} \times 27,000 = 17,700$ lb./ rivet.

$$Q = 14 \times \frac{3}{4} \times 24.5 = 257 \text{ in}^3$$

$$Z = \frac{386,500 \times 257}{34,592} = 2,870 \text{ lb. lin. in.}$$

$$p = \frac{8,100 \times 2}{2,870} = 5.6 \text{ in.}$$

A pitch of $5\frac{1}{2}$ inches will be used from the end up to the quarter point, and a pitch of 6 inches for the remaining of the girder.

Horizontal flange rivet pitch

$$Q = 257 + 16.9 \times 22.57 = 638.4 \text{ in}^3$$

$$Z = \frac{386,500 \times 638.4}{34,592} = 7,130 \text{ lb./ lin. in.}$$

$$p = \frac{16,200}{7,130} = 2.27 \text{ in}$$

A pitch of $2\frac{1}{4}$ inches will be used and the rivets shall be located in two rows.

Horizontal flange rivet pitch at tthe quarter point.

$$Z = \frac{236,200 \times 638.4}{34,592} = 4,360 \text{ lb./ lin. in.}$$

$$p = \frac{16,200}{4,360} = 3.71 \text{ in.}$$

A pitch of $3\frac{1}{2}$ inches will be used and the rivets shall be located in two rows.

Horizontal flange rivet pitch at the centre of the girder

$$Z = \frac{65,100 \times 638.4}{34,592} = 1,200 \text{ lb./ lin. in.}$$

$$p = \frac{16,200}{1,200} = 13.5 \text{ in.}$$

A pitch of 6 inches shall be used and the rivets shall be located in two rows.

Intermediate stiffeners,

The intermediate stiffeners shall be located 48 inches apart and shall be composed of angles $5 \times 3\frac{1}{2} \times \frac{3}{8}$ inches., and of fillers $3\frac{1}{2} \times \frac{5}{8} \times 36$ inches

End stiffeners

The bearing area required for the end stiffeners is given by:

$$\frac{386,500}{27,000} = 14.3 \text{ sq. in.}$$

At each end 4 angles $5 \times 3\frac{1}{2} \times \frac{3}{4}$ in. shall be used, each in connection with a filler plate $3\frac{1}{2}$ x x 36 in.

Number of rivets required.

Capacity of one rivet:

a) Double shear: 16,200 lb./ rivet.

b) Bearing: $\frac{3}{4} \times \frac{7}{8} \times 27{,}000 = 17{,}700$ lb./ rivet.

The number of rivets required is then:

$$\frac{386{,}500}{16{,}200} = 23.8$$

Increase in the number of rivets for effect of filler

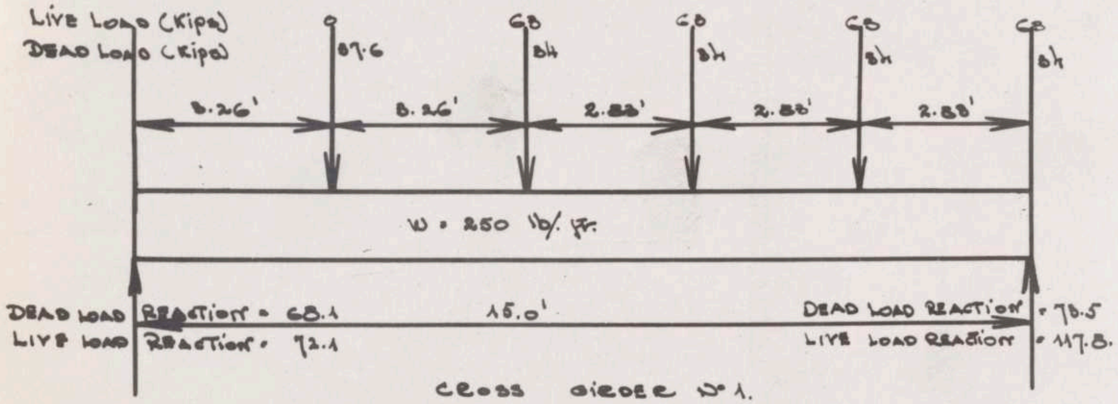
$$\frac{23.8}{3} = 8$$

Total number of rivets required: 32

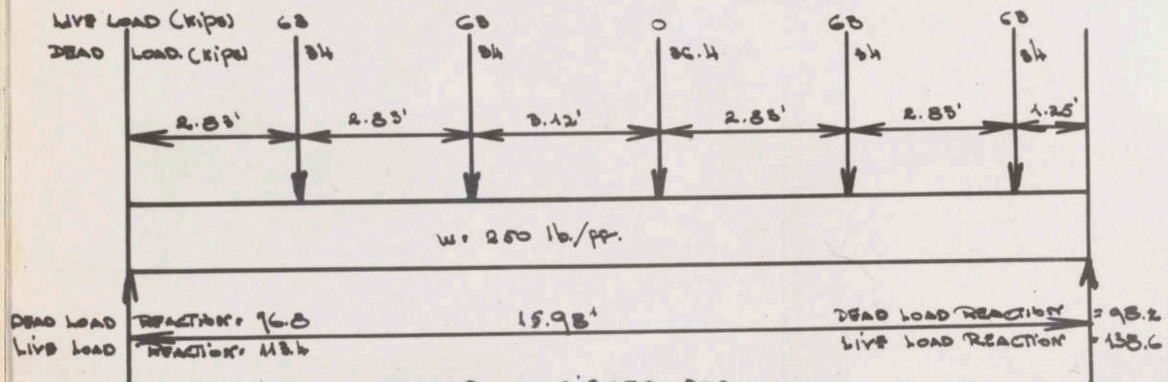
A total of 32 rivets shall be used.

Actual weights

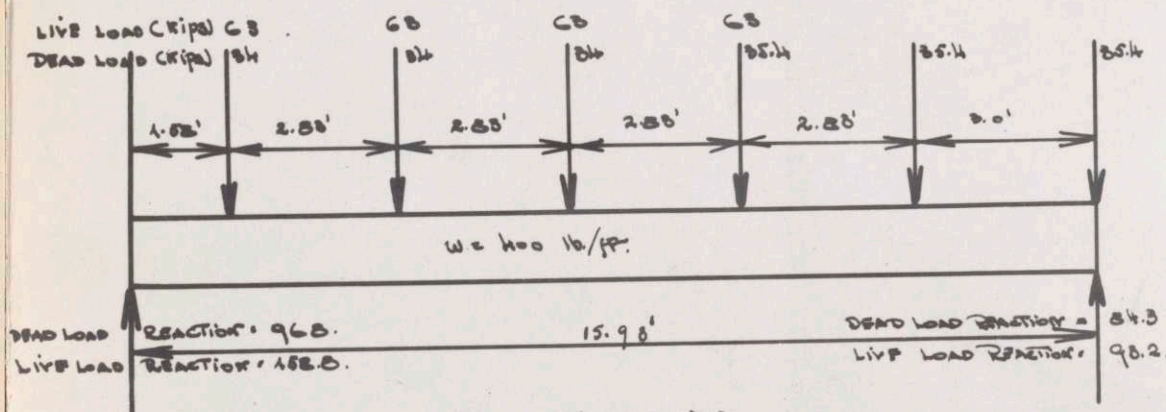
Web: 4 x 30.6 =	122.4 lb./ lin ft.
4 angles 4 x 28.7 =	114.8 "
2 cover plates $\frac{28}{12} \times 25.5$ =	59.5 "
Stiffeners, fillers, and rivet heads =	30.0 "
Total:	326.7 lb./ lin ft.



CROSS GIRDER No 1.



CROSS GIRDER No 2



CROSS GIRDER No 3

LOADINGS ON CROSS GIRDERS.

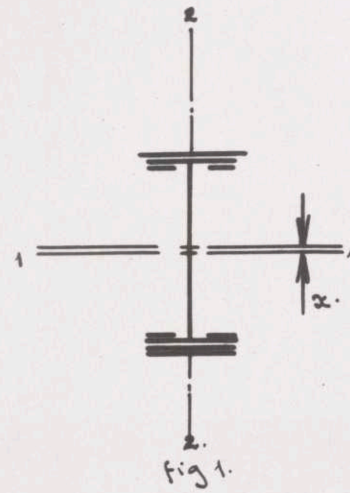


fig 1.

fig. -1-.

CROSS SECTION OF A TRACK GIRDER.

1. C.B. SECTION - 27" × 175 lb.
1. D.K. Pl. 16" × 1/2"
4. INSIDE COVER Pl. 4" × 1/2"
2. BOTTOM COVER Pl. 14" × 1/2"

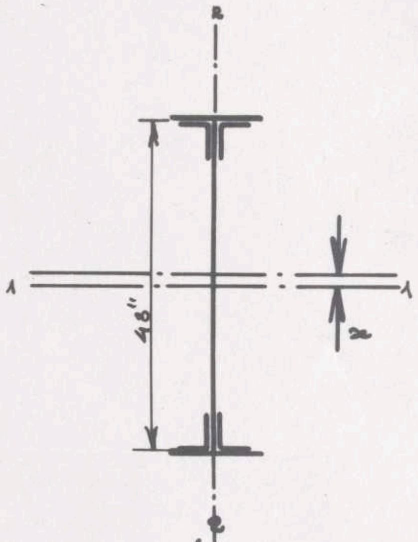


fig-2.

fig-2.

CROSS SECTION OF A CROSS GIRDER.

CROSS. GIRDER. No 1.
1. Pl. 14" × 3/8"
4. Ls. 6" × 6" × 1/2"
1. WEB. 48" × 9/16"

CROSS. GIRDER. No 2.
1. Pl. 14" × 1/2"
4. Ls. 6" × 6" × 1/2"
1. WEB. 48" × 5/8"

CROSS. GIRDER. No 3.
1. Pl. 14" × 3/4"
4. Ls. 6 × 6 × 3/4
1. WEB. 48 × 3/4.

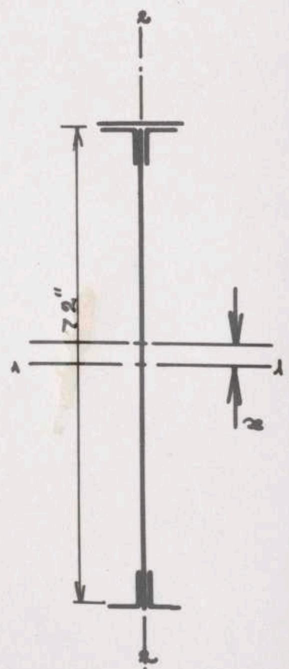


fig 3.

fig-3.

CROSS SECTION OF A FASCIA GIRDER

1. Pl 14 × 3/8.
4. Ls 6 × 6 × 3/8.
1. WEB. - 72" × 3/8"

DESIGN OF THE FASCIA GIRDER

DESIGN OF THE FASCIA GIRDERS

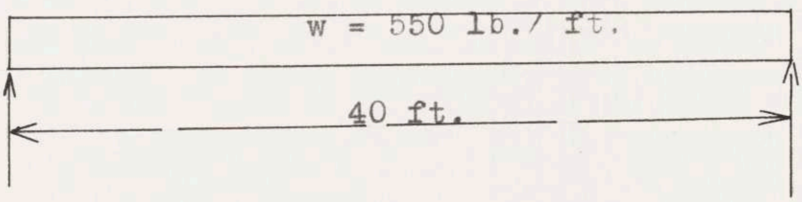
The fascia girders do not carry live load and therefore are subjected to but comparatively small stress. But they have the other purposes to keep the floor closed from the sides, to prevent the weathering, and to make the bridge of good appearance. The dimensions of the fascia girder have to be chosen accordingly.

Desirable depth of the fascia girder.

The desirable depth of the fascia girder would be given by:

Depth of steel floor	2.5 ft.
Height of concrete curb	2.0 ft.
Freeboard	1.0 ft.
Total	6.0 ft.

Dead load.



Concrete: $\frac{11.5}{12} \times 2 \times 1 \times 150 =$	290 lb./ ft.
Steel:	260 lb./ ft.
Total:	550 lb./ ft.

Maximum shear

$$\frac{550 \times 40}{2} = 11{,}000 \text{ lb.}$$

Maximum bending moment

$$\frac{55o \times 40^2}{8} = 110{,}000 \text{ ft.lb.}$$

Web area required

$$\frac{11{,}000}{13{,}500} = 0.81 \text{ sq. in.}$$

The net area obtained with a section 72 in. x $\frac{3}{8}$ in. after provision is made for 12 rivet holes is equal to:

$$72 \times \frac{3}{8} - 12 \times \frac{3}{8} \times \frac{15}{16} = 24.9 \text{ sq. in.}$$

Section modulus required

The section modulus required at the bottom is equal to:

$$\frac{110{,}000 \times 12}{18{,}000} = 73 \text{ in}^3$$

Area and moment of inertia of the cross section about axis 1-1

Item	Area	Moment of inertia
Web: 72 x $\frac{3}{8}$ in=	27.0 in^2	11,664 in^4
4 angles: 6 x 6 x $\frac{3}{8}$ =	17.4 "	20,950 "
Top cover plate: 14 x $\frac{3}{8}$ =	5.25 "	7,032 "
Total:	49.65 "	39,646 "

Reduction in area due to the rivet holes

$\frac{9}{8} \times \frac{15}{16} + 5 \times \frac{3}{8} \times \frac{15}{16} = 2.88$ sq. in.

Net area.

$49.65 - 2.88 = 46.77$ sq. in.

Displacement of the neutral axis

$$X = \frac{5.25 \times 36.5 + \frac{3}{8} \times \frac{15}{16}(3 + 9 + 15 + 21 + 27 + 34)}{46.77}$$

$= 4.9 \text{ in}^2$

Reduction of the moment of inertia due to the rivet holes

$2 \times 999 + 2 \times 1,301 + \frac{3}{8} \times \frac{15}{16}(3^2+9^2+15^2+21^2+27^2+34^2)$

$= 5,320 \text{ in}^4$

Reduction in the moment of inertia due to the displacement of the neutral axis

$46.77 \times 4.9^2 = 1,122$ in4

Net moment of inertia about the neutral axis

$39,646 - 5,320 - 1,122 = 33,206 \text{ in}^4$

Section modulus obtained. at the bottom

$s = \frac{33,206}{41.15} = 807 \text{ in}^3$

Moment of inertia of the cross section about axis 2-2

Item	Moment of inertia
4 angles:	120 in^4
1 cover plate:	86 "
Total:	206 "

Minimum radius of gyration of the cross section.

$$r = \sqrt{\frac{208}{49.65}} = 2.2 \text{ in.}$$

Allowable unit compressive stress

The unsupported length of the fascia girder will be the same as for the track girders and equal to 14 ft. Therefore:

$$f_c = 18{,}000 - 70 \; \frac{14 \times 12}{2.2} = 12{,}120 \text{ lb./ in}^2$$

Section modulus required at the top

$$s = \frac{110{,}000 \times 12}{12{,}120} = 109 \text{ in}^3$$

Section modulus obtained at the top

$$s = \frac{39{,}646}{31.86} = 1{,}245 \text{ in}^3$$

Vertical flange rivet pitch

$$Q = 5.25 \times 36.5 = 192 \text{ in}^3$$

$$Z = \frac{11,000 \times 192}{39,648} = 53 \text{ lb./ lin in.}$$

Capacity of one rivet:

a) Single shear: 8,100 lb./ rivet.

b) Bearing: $\frac{3}{8} \times \frac{7}{8} \times 27,000 = 8,850$ lb./ rivet

$$p = \frac{8,100 \times 2}{53} = 304 \text{ in.}$$

A pitch of 6 inches shall be used.

Horizontal flange rivet pitch

$$Q = 192 + 17.4 \times 34.61 = 794 \text{ in}^3$$

$$Z = \frac{11,000 \times 794}{39,648} = 220 \text{ lb./ lin. in.}$$

$$p = \frac{8850}{220} = 40 \text{ in.}$$

A pitch of 6 inches shall be used for the entire span.

Intermediate stiffeners

The spacing of the intermediate stiffeners from the support up to the centre of the girder shall be in sequence: 1 ft.-6 in., 2 ft., 3 ft., 3 ft.-8 in.,4 ft,-6 in., 6 ft. The last value is the maximum spacing of the stiffeners.

The intermediate stiffeners shall be composed of 2 angles $5 \times 3\frac{1}{2} \times \frac{3}{8}$ in. in connection with a filler plate $3\frac{1}{2} \times \frac{3}{8} \times 45$ inches.

End stiffeners

The end stiffeners shall be composed of 4 angles $5 \times 3\frac{1}{2} \times \frac{1}{2}$ inches.

Number of rivets required

The number of rivets required is given by:

$$\frac{11,000}{8,850} = 1.2$$

A total of 12 rivets shall be used for the connection of each pair of stiffener angles.

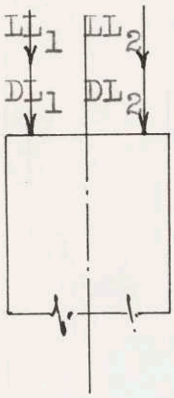
Actual weight of the girder

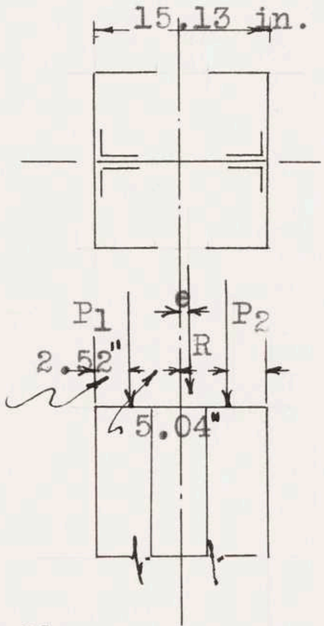
Item	Weight
Web: $\frac{72}{12} \times 17.85 =$	107.1 lb./ lin.ft.
4 flange angles: $4 \times 10.4 =$	41.6 "
Top cover plate: $\frac{14}{12} \times 17.85 =$	20.8 "
Stiffeners and fillers:	51.0 "
Rivet heads:	6.0 "
Total:	226.5 lb./lin.ft.

DESIGN OF THE COLUMNS

A) INTERMEDIATE COLUMN

The maximum live load occurs when two tracks are loaded simultaneously. Since two tracks are loaded to produce the maximum stress 95% of the maximum live load will be considered as effective.





$D\ L_1 = 98,200$ lb.

$D\ L_2 = 96,800$ lb.

$L\ L_1 = 113,400 \times 0.95 = 107,700$ lb.

$L\ L_2 = 158,600 \times 0.95 = 150,700$ lb.

$\text{Impact}_1 = 107,700 \times 0.824 = 88,700$ lb.

$\text{Impact}_2 = 150,700 \times 0.824 = 124,200$ lb.

Total maximum axial load $R = 666,300$ lb.

Trial area

The trial area is given by:

$$\frac{666,300}{15,000} = 44.42 \text{ sq. in.}$$

Trial section

2-15 in. channel angles-35 lb. at 10.23 =	20.46
1 web plate 14 x $\frac{3}{4}$ in. at 10.5 in^2 =	10.5
4 angles 5 x $3\frac{1}{2}$ x $\frac{3}{4}$ in. at 58.1 in^2 =	23.24
Total:	54.20 in^2

Loads.

P_1 = 294,600 lb.

P_2 = 371,700 lb.

R = 666,300 lb.

Eccentricity

$$e = \frac{5.04\ (371,700 - 294,600)}{666,300} = 0.58 \text{ in.}$$

Moment of inertia of the cross section about axis 1-1

2 angles(channels):	836.8 in^4
Web:	171.5 "
4 angles:	721.6 "
Total:	1,729.9 "

Moment of inertia of the cross section about axis 2-2

2 channel angles:	637.4 in^4
4 angles:	66.4 "
Total	703.8 "

Minimum radius of gyration

$$r = \sqrt{\frac{703.8}{54.2}} = 3.6 \text{ in.}$$

Allowable unit compressive stress

The unsupported length of the column is 8 ft. 6 in. Therefore:

$$f_c = 16{,}000 - 70 \frac{8.5 \times 12}{3.6} = 14{,}000 \text{ lb/ sq. in.}$$

Maximum unit compressive stress obtained

$$f_c = \frac{666{,}300}{54.2} + \frac{666{,}300 \times 0.58 \times 7.56}{1729.9} = 13{,}970 \text{ lb.}$$

Area of base plate required

The allowable bearing stress on concrete masonry is equal to 500 lb./ sq. in. The area required is then:

$$\frac{666{,}300}{5{,}000} = 1{,}333 \text{ sq. in.}$$

A plate 38 x 38 x $3\frac{1}{2}$ in. shall be used. The bearing area is then equal to:1,444 sq. in.

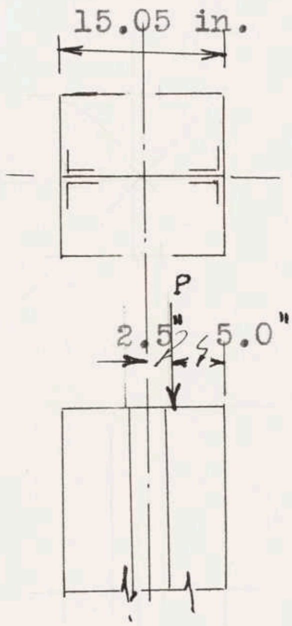
Other intermediate columns

Since the loads on the other intermediate columns are only slightly less columns of the same cross section shall be used.

B) OUTSTANDING COLUMN

Loads.

Dead load: 70,700 + 11,000 =	81,700 lb.
Live load:	71,200 lb.
Impact:	58,700 lb.
Total:	211,600 lb.



Trial area

The trial area is equal to:

$$\frac{211,600}{15,000} = 14.1 \text{ sq. in.}$$

Trial section

2- 15 in channel angles 33.9 lb. at 9.9 in^2 = 19.8 in^2

1 web: 14 x $\frac{3}{8}$ in. at 5.25 in^2 = 5.25 "

4 angles $5 \times 3\frac{1}{2} \times \frac{3}{8}$ at 3.05 in^2 =	12.20 in^2
Total:	37.25 "

Moment of inertia of the section about axis 1-1

2 channel angles:	808.4 in^4
Web:	85.75 "
4 angles:	491.6 "
Total:	1,385.75 "

Moment of inertia of the section about axis 2-2

2 channel angles:	625.2 in^4
4 angles:	70.72 "
Total	695.92 "

Minimum radius of gyration of the cross section

$$r = \sqrt{\frac{695.92}{37.25}} = 4.32 \text{ in.}$$

Allowable unit compressive stress

$$f_c = 16{,}000 - 70\ \frac{8.5 \times 12}{4.32} = 14{,}200 \text{ lb./ in}^2$$

Maximum compression obtained.

$$f_c = \frac{211{,}600}{37.25} + \frac{211{,}600 \times 2.5 \times 7.5}{1{,}385.75} = 8{,}540 \text{ lb./ in}^2$$

Area of base plate required

$$\frac{211{,}600}{500} = 423 \text{ sq. in.}$$

A plate 26 x 26 x $1\frac{1}{2}$ in shall be used, the bearing area becoming 676 sq. in.

DESIGN OF THE FOUNDATIONS FOR THE COLUMNS

DESIGN OF THE FOUNDATIONS FOR THE COLUMNS

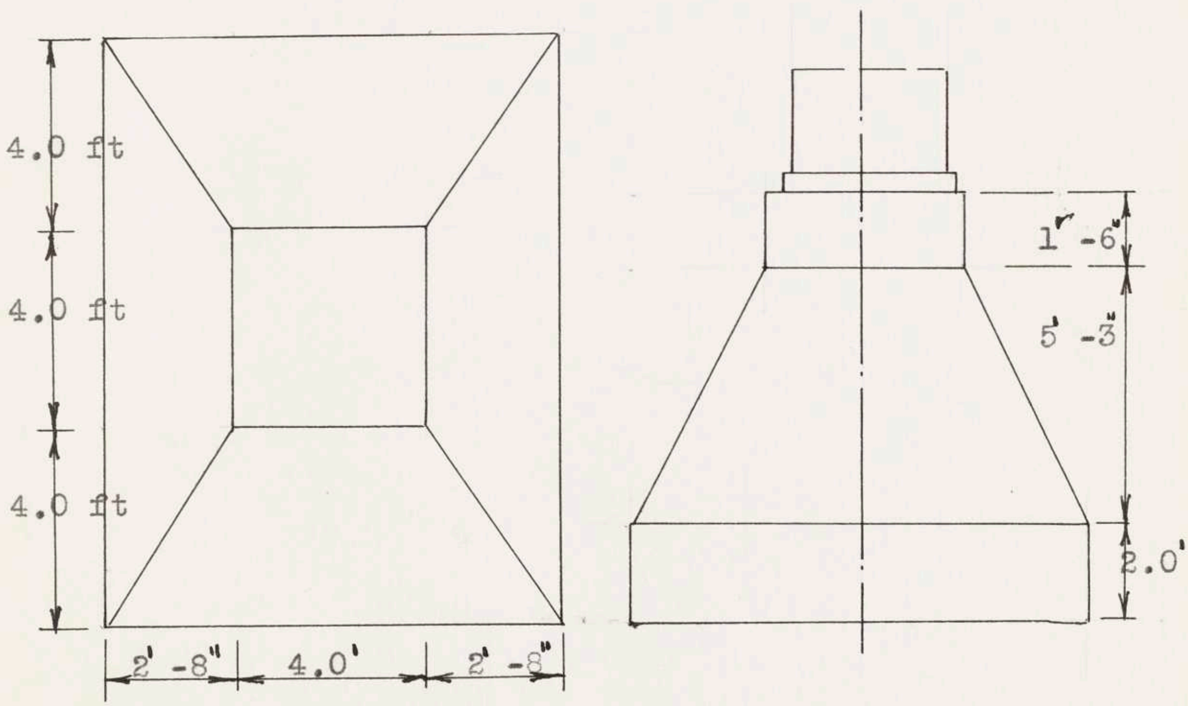
A) DESIGN OF FOOTINGS FOR THE INTERMEDIATE COLUMNS

It was found by borings that the subsoil consists mainly of medium dry sand the safe bearing capacity of which according to the New York Building Code can be taken as 3 tons per sq. ft. maximum.

Area required for footing

$$\frac{666,300}{3 \times 2000} = 111.5 \text{ ft}^2$$

A footing 12 ft. x 9 ft. x 4 in. shall be used, its bearing area being equal to 112 sq. ft.



Providing that the base plate of the column rests on a granite block 48 x 48 x 18 in., the depth of footing required for punching shear should be equal to:

$$\frac{666,300}{48 \times 4 \times 40} = 86.75 \text{ in.}$$

A depth of 7 ft. 3 in. shall be used.

B) DESIGN OF FOOTING FOR THE OUTSTANDING COLUMN

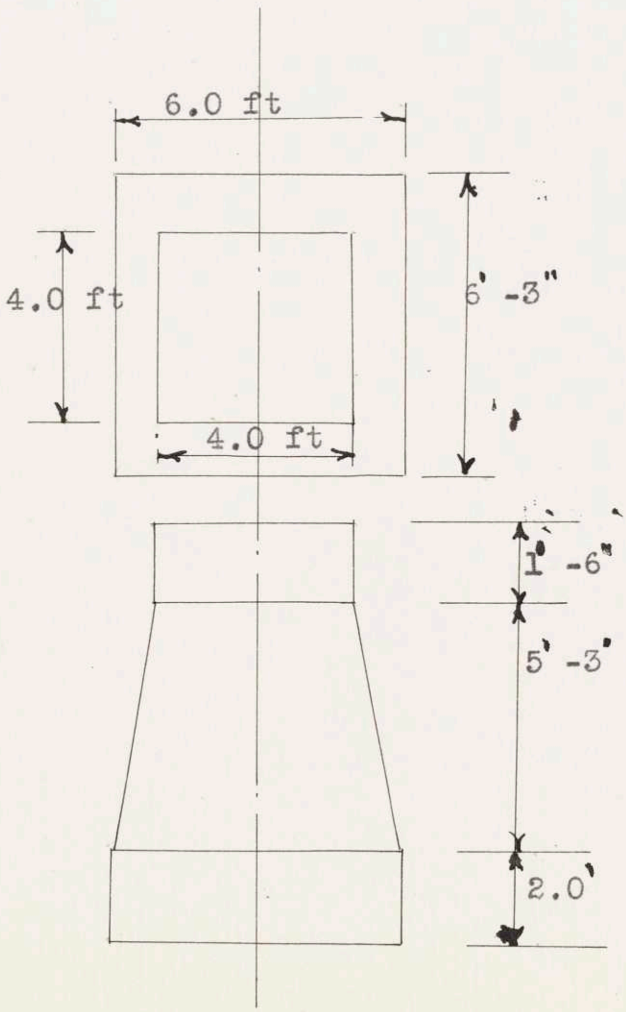
Area required

$$\frac{211,600}{3 \times 2000} = 35.25 \text{ sq. ft.}$$

Providing that the base plate of the column rests on a granite block 48 x 48 x 12 in., the depth of footing required for punching shear should be:

$$\frac{217,600}{48 \times 4 \times 40} = 2.83 \text{ ft.}$$

A depth of 7 ft. 3 in. shall be used.



In the design of the footings for the columns, the assumption is made that the column load is transferred to the base of the footing at an angle of 45° to the direction of the vertical load. Designing the footings with the sides sloping at an angle not greater than 45° to the vertical (which is the case), thus eliminates all bending moments, and we have only to consider the punching shear in the footings.

C) DESIGN OF THE ABUTMENTS

Data for the design

The weight of the earth = 100 lb./ ft^3

The angle of repose of the earth = 30°

The angle of the surface with the horizontal =0°

Considering the abutment as a surcharge retaining wall with a loading of Cooper E- 70, the surcharge will be:

$$\frac{70,000}{5 \times 12 \times 100} = 11.67 \text{ ft.}$$

where: 70,000 is the axial load of the locomotive

5 ft. is the distance between the axis of the locomotive.

12 ft. if the horizontal distance perpendicular to the track axis on which the load is distributed.

Reaction of the abutment to the track girders

The reaction of the abutment to the track girders is as follows:

The dead load of the girders which are spaced 20 in.

apart is:

$$\frac{17,000 \times 12}{20} = 10,200 \text{ lb./ lin. ft. of abutment.}$$

Under the four tracks 24 girders are carrying live load, the live load being 44,000 lb. on each.

The length of the abutment is 94 ft.

The live load on the abutment is therefore:

$$\frac{44,000 \times 24}{94} = 11,200 \text{ lb./ lin. ft. of abutment.}$$

The total load is then:

10,200 + 11,200 = 21,400 lb./lin/ft.

Effect of earth pressure on the abutment

According to the Rankine theory, the active pressure of the earth varies at different depths as:

$$P_a = w.y \cos \theta \, \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

where P_a is the intensity of the pressure on a plane parallel to the surface.

y is the distance from the surface of the earth to that of the plane.

θ is the angle between the surface of the earth and the horizontal.

ϕ is the angle of repose of the earth.

For our case:

$$\cos\theta \frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}} = 1\text{x} \frac{1 - \sqrt{1 - \cos^2 30°}}{1 + \sqrt{1 - \cos^2 30°}}$$

= 0.33

y = h + surcharge

At the top of the abutment y_o = surcharge = 11.67 ft.

P_o = 0.33 y.w = 0.33 x 11.67 x 100 = 385 lb./ft^2

At the bottom of the abutment y = 11.67 + 22 = 33.67ft.

P_1 =0.33 x 33.67 x 100 = 1110 lb./ft^2

<u>Computation for the location of the resultant of the forces</u> (see accompanying diagram)

Force(lb.)	Arm(ft.)	Moment about point 0 (ftlb)
H_1 = 8470	11	+93170
H_2 = 7980	7.32	+58400
V =21400	4.43	-95000
w = 375	9.	- 3475
W_1 = 3500	8.5	-29750
W_2 = 5000	11.5	-57500
W_3 = 6880	14.75	-101500
W_4 = 7930	18.25	-144700
W_5 =13300	22.5	-299250
W_6 = 5250	22.5	-118125
W_7 = 5770	18.25	-105400

$W_8 = 7350$	14.75	-108400
$W_9 = 7650$	11.5	- 87975
$W_{10}=12760$	6.75	- 85600
$W_{11}= 1840$	1.75	- 3220

$\Sigma V = 99,000$

$\Sigma M = 1,111,000$

$$x = \frac{1,111,000}{99,000} = 11.22 \text{ ft.}$$

Thus the resultant of the forces is within the middle third.

The eccentricity is equal to:
12.5 - 11.22 = 1.28 ft.

The pressure by the formula is equal to:

$$p = \frac{\Sigma V}{A} \pm \frac{M c}{I} = \frac{99,000}{25} \pm \frac{99,000 \times 1.28 \times 12.5}{\frac{1}{12} \times 25^3}$$

$=3960 \pm 1230$ lb./ sq/ ft.

P max. = 5190 lb./ sq. ft.

P min. = 2730 lb./ sq. ft.

<u>Investigation of the bending moment on a cantilever of the abutment.</u>

On the cantilever of the abutment a bending moment occurs which has to be considered.

The maximum bending moment on the plane a-b is equal to:

$$\frac{4,850 \times 3.5^2}{2} + \frac{340 \times 3.5}{2} \times \frac{2}{3} \times 3.5 = 31,000 \text{ ft.lb.}$$

The shear is equal to:

$$V = 4,850 \times 3.5 + 340 \times 3.5 \times \frac{1}{2} = 17,600 \text{ lb.}$$

In order to carry the above bending moment and shear the cantilever of the abutment must be reinforced with steel. The area of the cross section of the steel required for the bending moment is found by the formula:

$$A_s = \frac{M}{f_s j d}$$

where A_s is the steel area.

M is the bending moment = 31,000 ft.lb.

f_s is the allowable unit stress of the steel in tension = 16,000 lb./ in^2

d is the effective depth of the cantilever

j is the ratio of the moment arm of the resising couple to the effective depth = $\frac{7}{8}$ for this case.

Then $A_s = \frac{31,000 \times 12}{16,000 \times \frac{7}{8} \times 44} = 0,6$ sq. in./ ft of length of the abutment.

Bond stress.

For bond stress the following relation must be satisfied:

$$\Sigma\, \Theta = \frac{V}{u\ j\ d}$$

where: $\Sigma\, \Theta$ is the sum of the perimeters of the bars

V is the total shear = 17,600

u is the allowable unit bond stress which for deformed bars can be taken as 100 lb./ in^2

The other terms have the same meaning as above. Then

$$\Sigma\, \Theta = \frac{17,600}{100 \times \frac{7}{8} \times 44} = 4.57 \text{ in.}$$

$\frac{1}{2}$ in. round deformed bars will be used. The area of the cross section of a single bar is 0.1963 in^2. The spacing must therefore be equal to:

$$\frac{0.1963}{0.6} = 3.93 \text{ in.}$$

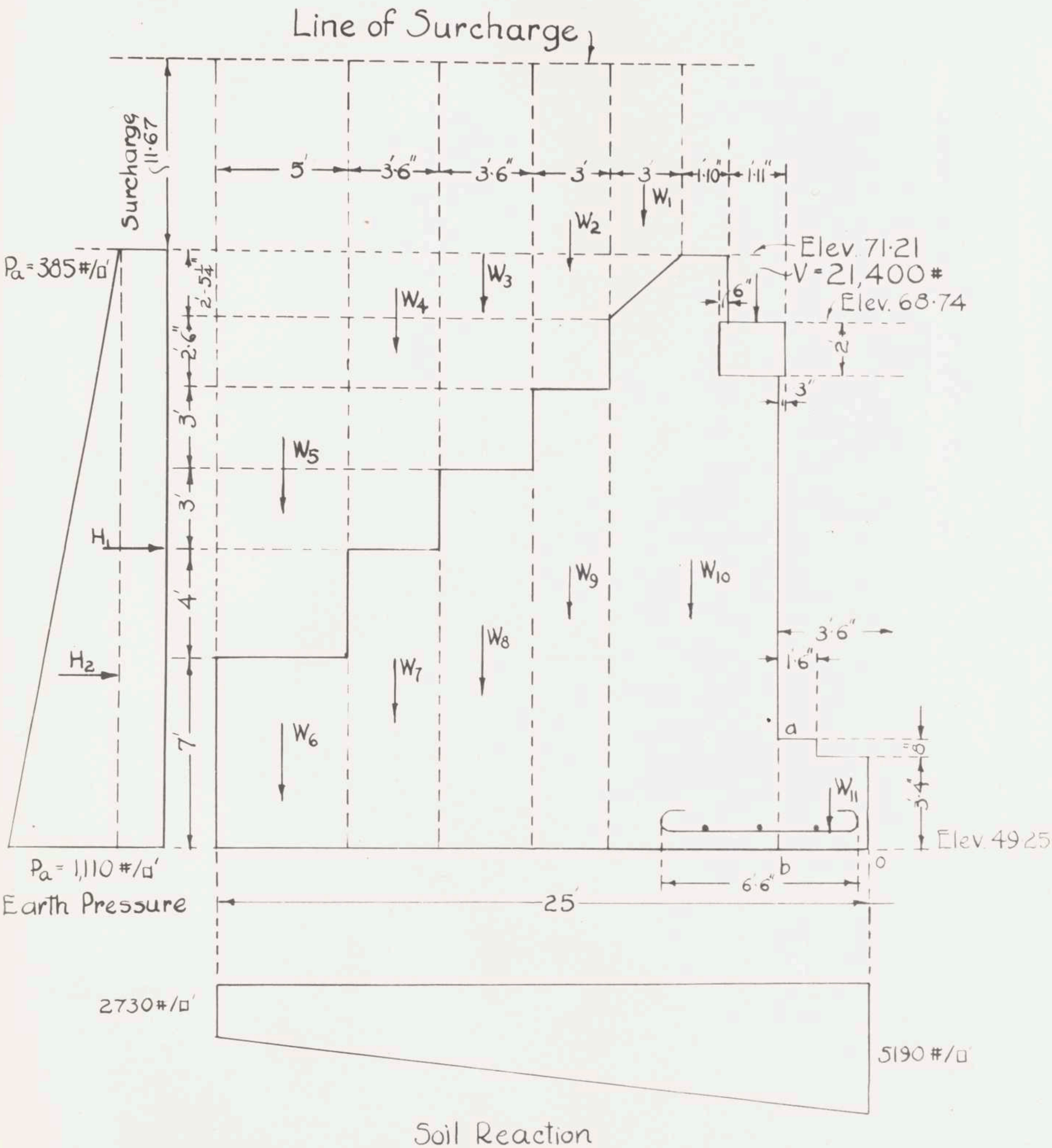
A spacing of 3 inches will be used or 3 bars per foot of length of the abutment.

The obtained A_s = 0.59 in^2 and $\Sigma\, \Theta$ = 4.71 in.

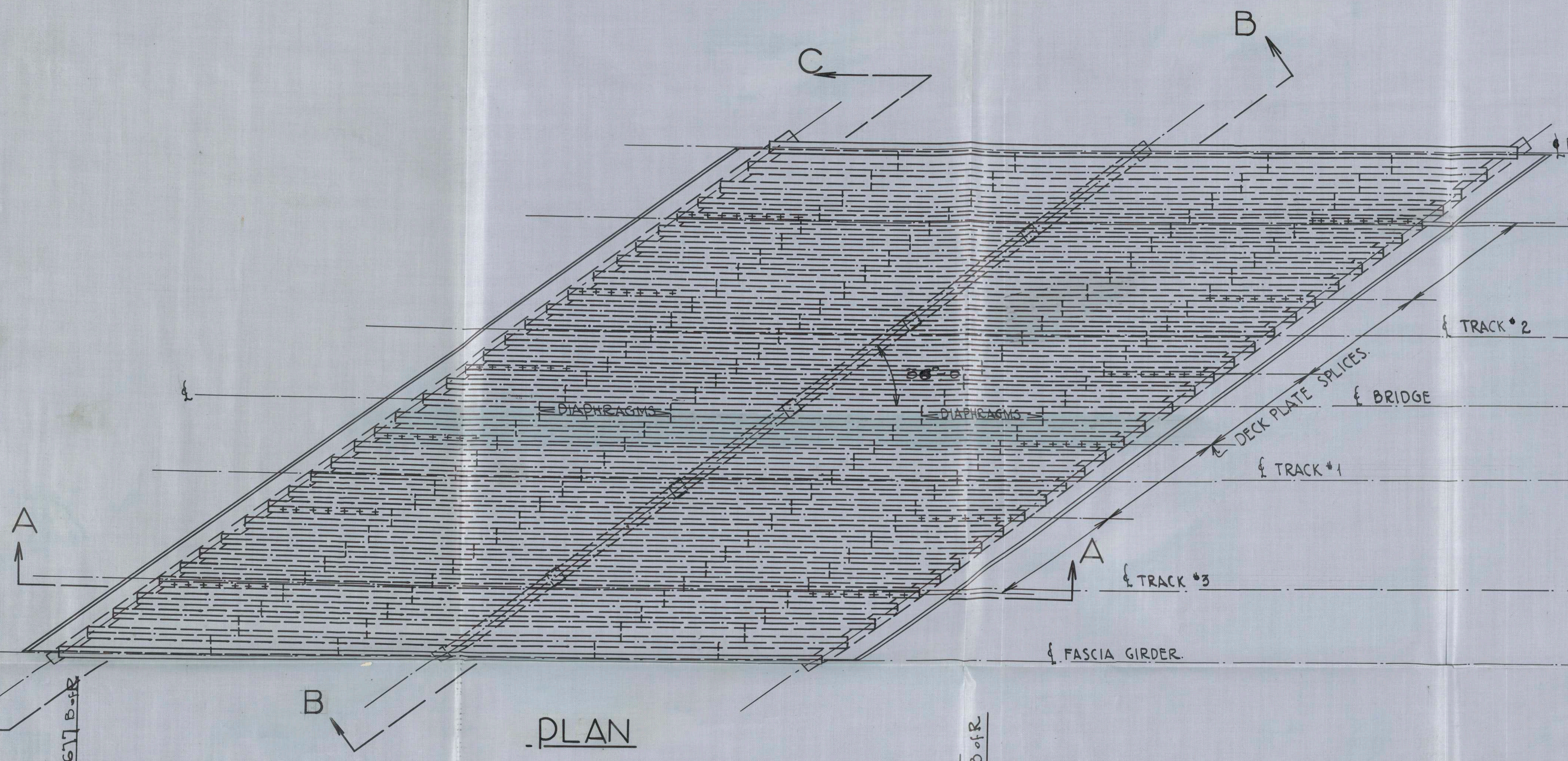
Length of bar required for anchorage in abutment: $\frac{16,000 \times 0.1963}{100 \times 1.571}$ =	20 in.
Cantilever end length:	42 "
Total length:	62 "

Bars 6 ft. 6 in. long will be used, and both ends of the bars shall be hooked.

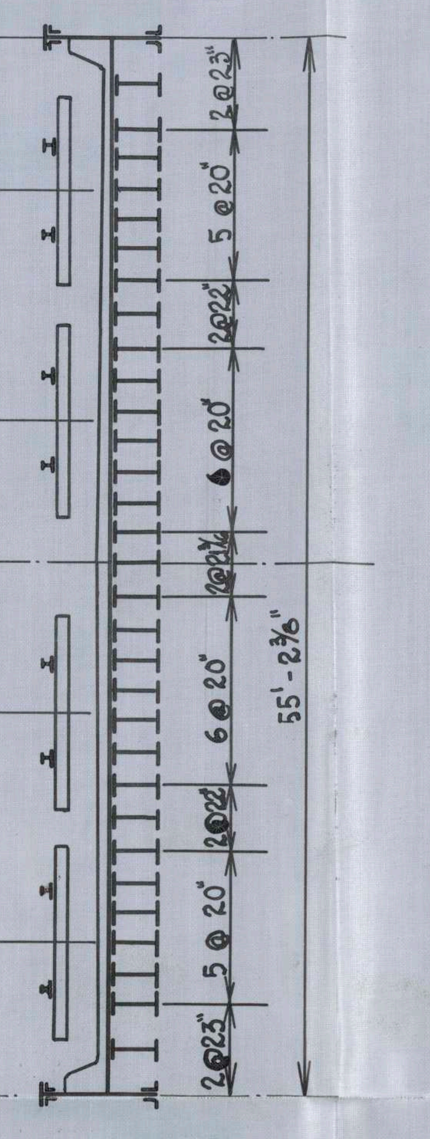
In connection with the above longitudinal bars $\frac{1}{2}$ in round and spaced 1 ft. 6 in. centre to centre shall be used.



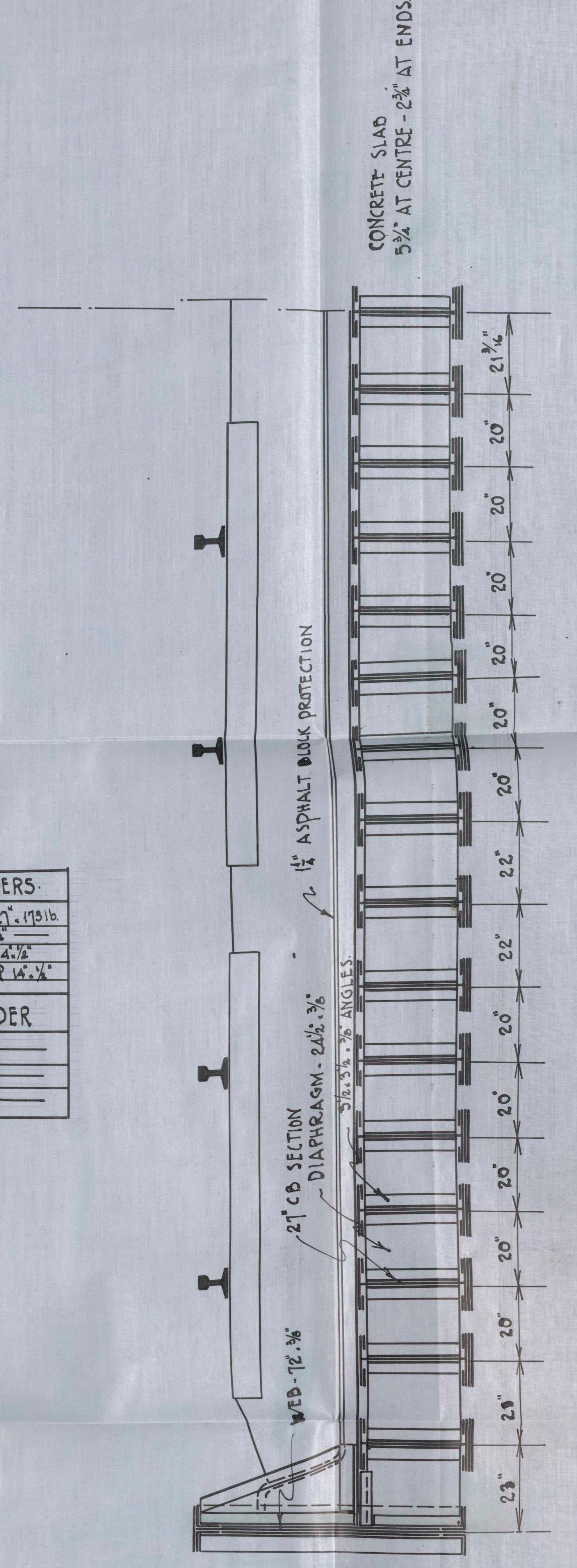
Line of Surcharge
Surcharge 11.67
5'
3'6"
3'6"
3'
3'
1'10"
1'11"
W1
W2
W3
W4
W5
W6
W7
W8
W9
W10
W11
Elev. 71.21
V = 21,400 #
Elev. 68.74
6"
2'
3"
2'-5¼"
2'-6"
3'
3'
4'
7'
H1
H2
3'6"
1'6"
a
8"
3'4"
Elev. 49.25
b
o
6'6"
25'
Pa = 385 #/□'
Pa = 1,110 #/□'
Earth Pressure
2730 #/□'
5190 #/□'
Soil Reaction



PLAN



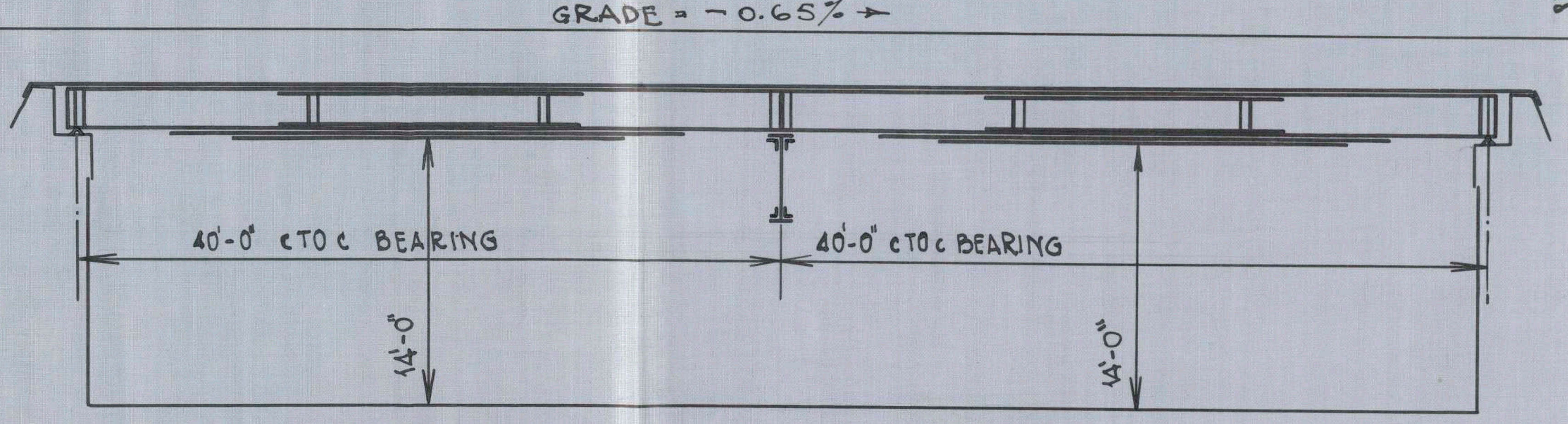
SECTION C-C



DETAIL OF DECK

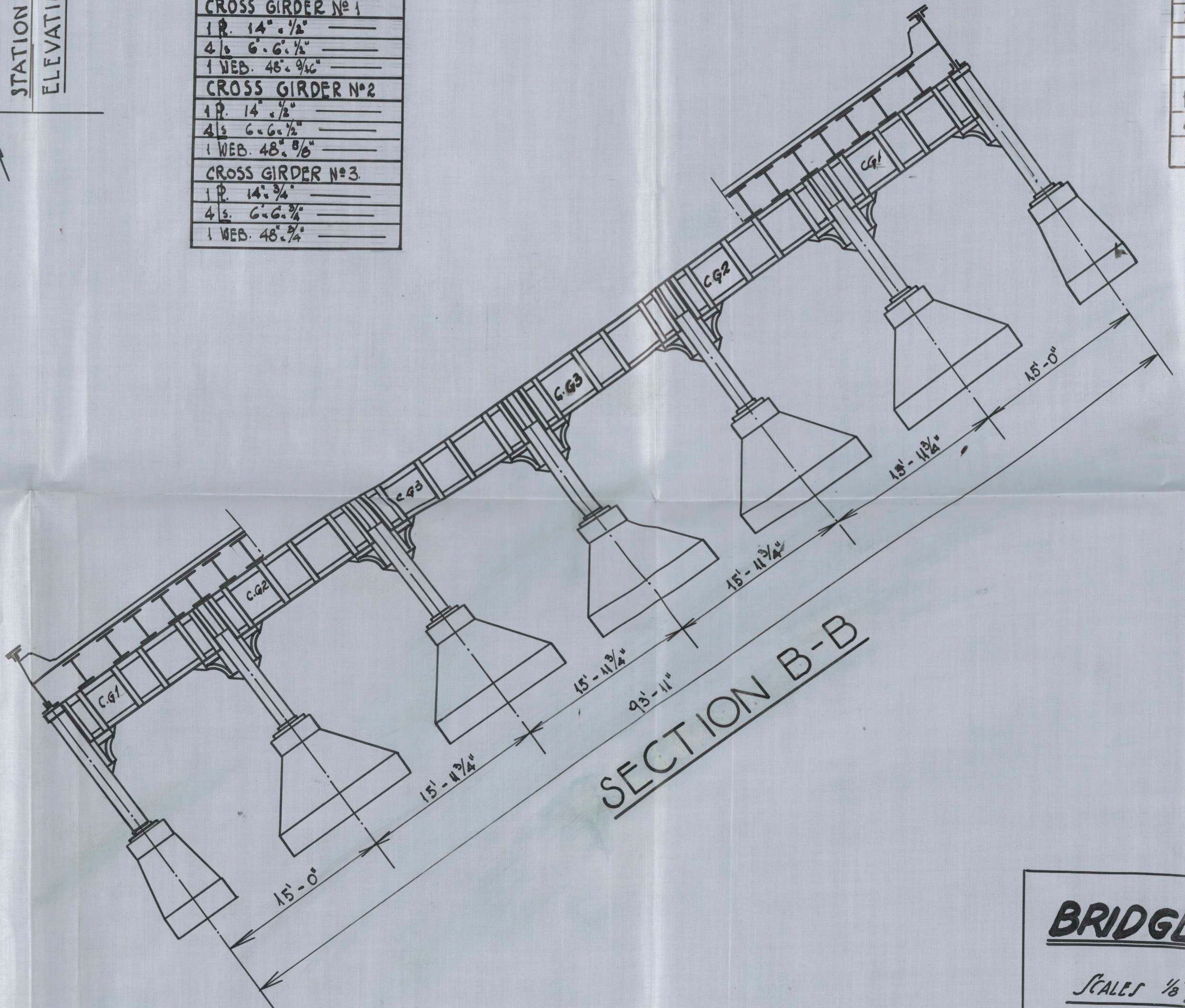
TRACK GIRDERS.
1. C.B. SECTION - 27" × 173 lb.
1 DECK ℙ. 16" × 1/2"
4 INSIDE COVER ℙ. 4" × 1/2"
2 BOTTOM COVER ℙ 14" × 1/2"
FASCIA GIRDER
1 ℙ 14" × 3/8"
4 ∟s 6" × 6" × 3/8"
1 WEB. 72" × 3/8"

CROSS GIRDERS
CROSS GIRDER No 1
1 ℙ. 14" × 1/2"
4 ∟s 6" × 6" × 1/2"
1 WEB. 48" × 9/16"
CROSS GIRDER No 2
1 ℙ. 14" × 1/2"
4 ∟s 6 × 6 × 1/2"
1 WEB. 48" × 8/8"
CROSS GIRDER No 3.
1 ℙ. 14" × 3/4"
4 ∟s. 6" × 6" × 3/4"
1 WEB. 48" × 3/4"

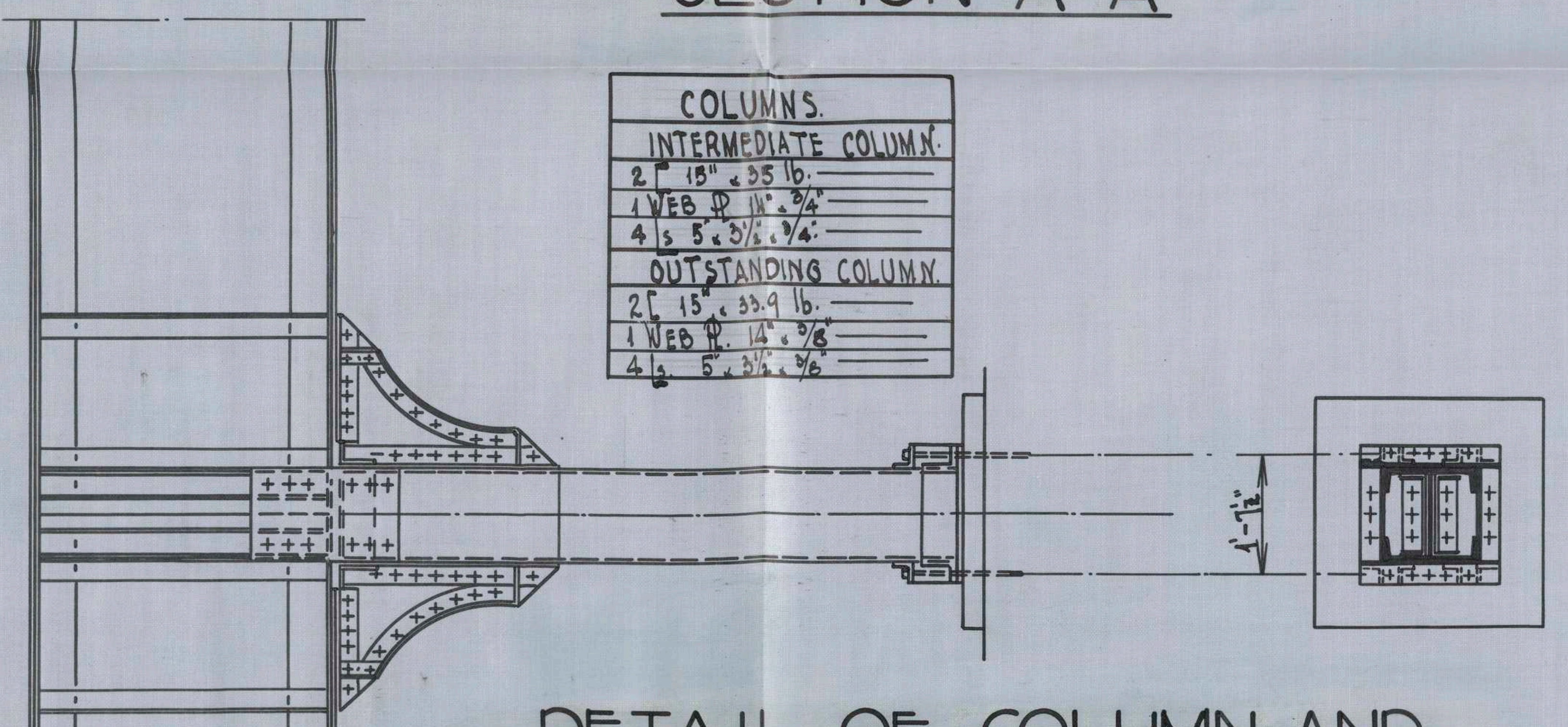


SECTION A-A

COLUMNS.
INTERMEDIATE COLUMN.
2 [15" × 35 lb.
1 WEB ℙ 14" × 3/4"
4 ∟s 5" × 3 1/2" × 3/4"
OUTSTANDING COLUMN.
2 [15" × 33.9 lb.
1 WEB ℙ. 14" × 3/8"
4 ∟s. 5" × 3 1/2" × 3/8"



SECTION B-B



DETAIL OF COLUMN AND CROSS GIRDER

BRIDGE OVER BRONX RIVER PARKWAY DRIVE
AT
WOODLAWN.
N.Y.

SCALES 1/8" & 1/2" = 1 ft.

S. M. MOROSOFF.