# Analysis of Beal Road Bridge

A Thesis Submitted to

The Faculty of the

Michigan Agricultural College

by

F. L. Hendrick

Candidate for the Degree of

Bachelor of Science

June 1921

THESIS

copy 1

INDEX



Drawings



101169

Stress sheet for cross beams B 1 & B 2

Section through columns

Half section of bridge

# Girder Bridges

All bridge structures may be divided into three groups, Beam Bridges, Suspension Bridges, and Arch Bridges. Beam bridges exert only a vertical pressure upon the bearings or supports. Beam bridges include, simple bridges, draw-bridges, continuous bridges, and cantilever bridges. A simple bridge is one resting on two supports.

Simple bridges are of two kinds, truss bridges and girder bridges. A girder bridge has its floor supported by solid or built up beams. A wooden beam, a rolled I beam, and a plate girder formed by riveting angles and plates together are examples of girders. Girder bridges are used for short spans, usually less than 100 feet. Other kinds, however, such as the 140 feet built up plate girders are not uncommon.

About 1850, built up plate girders, formed by riveting angles to a solid web plate, were used in Europe. Plate girder bridges were not used extensively in this country until 1895. Today, the plate girder is the first choice for spans from 30 to 100 feet in length.

The advantages of a girder bridge are greater stiffness, advantage in erection, a solid floor may be used with the regular ballast and very shallow floors. The through plate girder has the added advantage of requiring very little headroom.

Beal Road Bridge

1 Mile S.W. of Grand Rapids, Mich.

1.

2

3.

1. Deal Road Bridge, looking South.

2. View from the North-West

3. View from the West.

## Introduction

The Beal Road Bridge was built in the late summer of 1913. The structure was built by the Michigan Railway Eng. Co. for the Michigan & Chicago Railway Co. The actual construction and erection was done by the Toledo Bridge and Crane Co. Mr. Wm. Fargo was the consulting engineer.

Beal Road is a highway one mile south of Grand Rapids. At the location of the bridge, the G. R. & I. R.R. crosses the highway at grade. The Michigan and Chicago Railway was forced to cross the G. R. & I. roadbed overhead. The G. R. & I. rails at the crossing has a curve of $2^{o}$ without a spiral easement. The two railways cross at an angle of $31^{o}$ 3'. This fact arbitrarily fixed the location of the supporting members. (See drawing 1)

The bridge is plate girder type throughout carried on concrete end piers and ten columns composed of buil up sections. The bridge consists of two deck spans, each sixty feet long, and three skew through spans, in addition to two short through spans which connect the skew spans and the deck spans. Due to an arbitrary placing of the columns no two girders in the through span are of the same length. The arrangment of girders is shown in Drawing 2.

The bridge is an extremely long one for an overhead crossing. This is due to the fact that the earth fill at the north end of the bridge has a depth of 28' under the pier. The approach from the south is also an earth fill and is 12' deep under the south pier. The greatest clearance over natural earth is 30'. The clearance over the G. R. & I rails is 22'.

The bridge is in strict keeping with the rest of the roadbed of this division. The maximum grade on the Kalamazoo-Grand Rapids division is 1% and the greatest curve outside of the cities is 2°.

The Michigan Railway Company operates its Kalamazoo-Grand Rapids division cars over this bridge. The weight of their cars are as follows:

Limited cars -------------76 tons.

Local cars ---------------60 tons.

Freight motors -----------73 tons.

The bridge is designed for E 40 loading with a liberal allowance for dead load.

The floor of the structure consists of 3/8" steel plates supported on 10" - 30# I beams resting directly on the flanges of the girders. Ten inch cedar ties are used on the bridge and the ballast is crushed rock.

The analysis of the structure will fall under three heads. First, the computation of the stresses in one of the deck girders, two thourgh girders, one pair of columns, the sway bracing between them, and one pair of column footings. The second step will be the actual design of the members noted above. This design will be base on A. R. E. A. specifications of 1910. The third step will be the comparison of the authors design with the design actually found in the structure. An attempt will be made to account for differences found and justify them if possible.

## General Conclusions

A close examination of the bridge in April 1921 showed the structure to be in very excellent condition. There were absolutely no signs of rust on any of the girders, I beams, or lateral bracing. There was a tendency exhibited for the paint to peel from four of the columns exposing the shop paint.

The most serious signs of depreciation were at the bases of the columns where the concrete was beginning to weather away. The concrete was poured about the column to a distance about six feet above the column footing. The concrete had failed to form a close seal to the column and water was working down between the steel and the concrete, encasing it, and upon freezing, was causing the steel to rust and the concrete to fall away from the steel. There is no reason to fear that the columns will fail before any of the rest of the structure.

The concrete in the piers seemed to have been of poor quality as large pit-holes are forming in the sloping top of the bases. Local aggregates were used in the concrete and organic matter is quite liable to have gotten into the concrete.

# Design of a Plate Girder Bridge

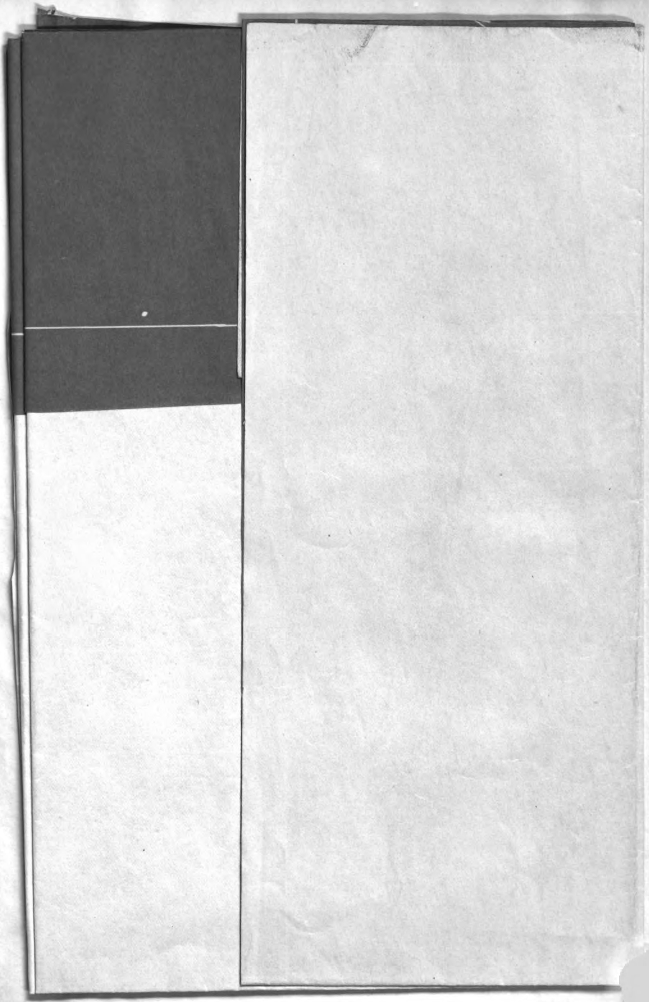
The specifications used in this design are those prepared by the American Railway Engineering Association in 1910.

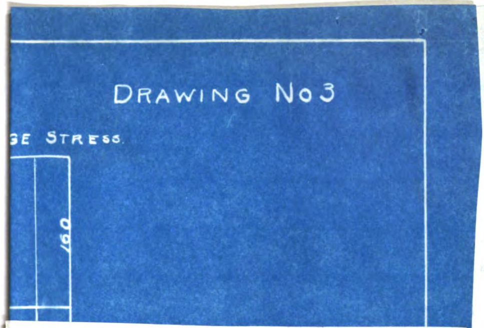
The loads that will be considered in this design are the dead load, live load, impact, and wind loads. The live load will consist of Cooper's E 40 loading. The dead load will consist of the weight of the metal in the structure, the floor, track and fastenings, ballast and all loads that are constantly applied.

Rapidly moving trains produce greater stresses in a bridge than would the same load simply standing on the structure. For this reason allowance is made in the live load stresses for the additional stress. The allowance is a certain per cent of the live load stress and varies according to the length of the bridge. This ration is found from the formula

$$I = S \left( \frac{300}{L + 300} \right)$$

The horizontal pressure exerted on bridges by the wind is called "Wind Pressure". Thirty pounds per square foot on the horizontal projection is usually allowed. Wind pressures will be found included in the lateral forces described in the specifications.




DRAWING No 3
GE STRESS.
160

Complete Design of 60' Single Track Deck Plate

Girder Span, G 10 - 11

Data:

Length 59' c-c of bearings. Dead load as computed.

Width 7' c-c of girders. Live load Cooper's E 40.

Calculations for main girder.

Dead load.

Metal only. (12.5 L - 100) .9 = 765#

| | |
|---|---|
| Tie | 150# |
| Rails and fastenings | 150# |
| Ballast 6 cu. ft. | 800# |
| I beam | 280# |
| Plates 12' long | 184# |
| Third rail and insulators | 50# |
| Two curb Ls 6 x 3 1/2" x 3/8" | 24# |
| | 1638# |

Total dead load 2400# per ft. of track.

M due to dead load = $1/8 \times 1200 \times 59^2 \times 12$ = 504,600 in #

From table in Ketcham's Handbook

Max. M. due to E 40 Loading on a 58' span = 1223,400 in #

For maximum impact $\dfrac{300}{300 + 60} = 83.6\%$

83.6% of 1223,400 in # = 1025,200 in #

Total moment at the center 2753,200 in #

The next thing to do is to determine the economic depth. No web should be less than 3/8" thick. We will assume 3/8" as thickness of this web and substitute in the formula

$$1.1\sqrt{\frac{M}{f\,t}}$$

$$1.1\sqrt{\frac{2753,200}{16000 \times 3/8"}} = 75.7 \text{ in.}$$

So we will use a web plate 75" x 3/8".

The flange area will next be determined. It is readily seen that the effective depth will be slightly less than the total depth, so assume 74" as the effective depth.

2753,200 ÷ 74 = 372000#

For the glange stress 372000 ÷ 16000 = 24.2 sq. in. required in each flange.

Use in each flange:

| | | |
|---|---|---|
| 2 Ls 6 x 6 x 5/8 - 1/2" | 11.72 sq. in. net | 9.5 sq. in. net |
| 1 pl. 16 x 1/2 x 42' | 7.00 | 7.0 |
| 1 pl. 16 x 1/2 x 31 | 7.00 | 7.0 |
| | 25.72 sq. in. | 23.5 sq. in. |
| 1/8 web | 3.37 | 3.37 |
| | 29.09 | 26.87 |

The thickest cover plate should be placed next to the angles, but in this case both cover plates are the same thickness.

For the length of the longer plate we have:

$$60\sqrt{\frac{7.00}{26.87}} = 30.6'$$

and for the length of the longer plate

$$60\sqrt{\frac{14.00}{26.87}} = 41.2'$$

According to the specifications one cover plate must cover the entire flange. The other plate will be made the same length as the longer under plate.

The next thing to determine is the stiffening angles and web. The masimum end shear is:

| | | |
|---|---|---|
| Dead Load | 34800# | |
| Live Load | 95900 | |
| Impact | 80300 | |
| | 211000# | 10,000#/sq. in. is |

allowed for shearing stress. Therefore, 21.2 sq. in. is required in the web. A web plate 72" x 3/8" gives an area of 24 sq. in.

According to the specifications, the outstanding leg of the stiffeners must not be less than 1/30 of the depth of the girder plus 2".

$$\frac{75}{30} + 2'' = 4.5''$$ The angle nearest this is

5" x 3 1/2" which will be used throughout for the intermidiate stiffeners.

The end stiffeners must be designed to carry the entire end shear and act as columns. Then assuming 5" x 3 1/2" x 1/2" used, we have for each column used:

$$r = \sqrt{\frac{8 \times 2.41^2 + 19.98}{8}} = 2.88$$

Then substituting this value of r and the column length:

$$16000 - 70\,\frac{37.5}{2.88} - 15,\ 075\#$$

and for the allowable compressive stress on the end stiffeners:

$$\frac{211,000}{15,\ 075} = 14.85 \text{ sq. in. req'd.}$$

So we will use 4 - 5" x 3 1/2" x 1/2" Ls area 16.00 sq. in.

We will use the minimum size intermidiate stiffeners and make the spacing accordingly. Using the assumed web 75" x 3/8", we have:

$$s = \frac{211,000}{28} = 7,015\#$$ for the maximum unit shearing stress in the web. From which

$$d = \frac{3/8}{40}\,(12,000 - 7,015) = 47''$$ for the required spacing near the end of the girder. Since this is less than 1/2 the depth of the girder, we will use a spacing of 45" or 3' - 9".

F

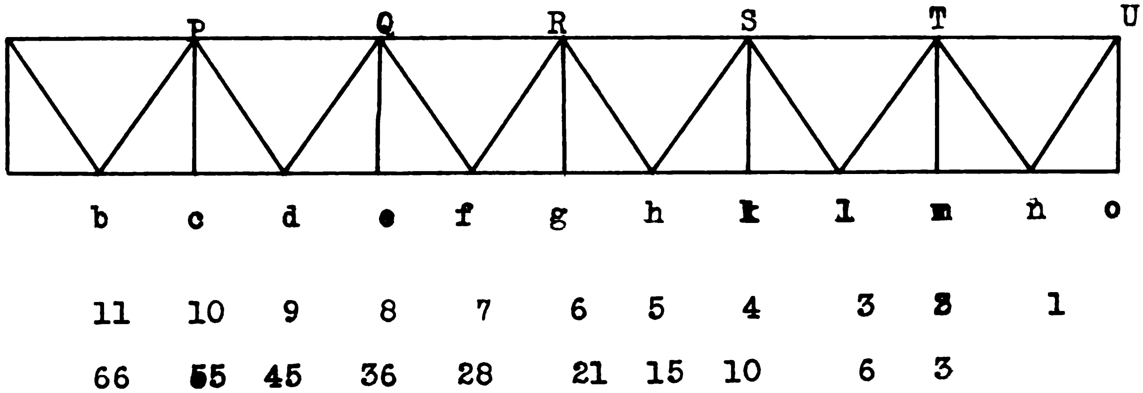
For the bearing on the masonry, $\frac{211,000}{600} = 352$ sq. in.

Use bearing plates 18" x 20". Each bearing must be designed so that there will be at least this much bearing on the masonry.

This completes the necessary calculations for the main girders, and the next thing is the lateral bracing.

The lateral bracing should be symmetrical about the center of the span. The laterals should have a slope as near $45^{\circ}$ as practicable. The distance between cross frames should never exceed 15'. In accordance with this there will be six intermediate cross frames in a 60' span.

The lateral bracing will be as shown:




According to the specifications the laterals must resist a uniform lateral force of 200 + .10( 5750 ) = 775#.
Suppose this load moves on from the right as a uniform live load. The load at each panel point will be 4.6 x 775 = 3750# (about). Then for the maximum shears in each panel, we have:

| | | |
|---|---|---|
| Shear in panel | 320 x 1 = 320# | mn |
| | 320 x 3 = 960# | ml |
| | 320 x 6 = 1920# | lk |
| | 320 x 10 = 3200# | kh |

320 x 15 = 4800# hg

320 x 21 = 6620# gf

320 x 28 = 8940# fe

320 x 36 = 11,300# ed

320 x 45 = 14,200# dc

320 x 55 = 17,600# cb

320 x 66 = 20,800# ba

Tangent of the angle $\frac{4.8}{7.0}$ = .686   sec θ = 1.213

Then for the maximum stresses in the diagonals, we have:

rf = 6620 x 1.213 = 7500 minus

qf = 8940 x 1.213 = 10,800 plus

dq = 11,300 x 1.213 = 13,620 minus

pd = 14,200 x 1.213 = 17,200 plus

pb = 17,600 x 1.213 = 21,400 minus

bo = 20,800 x 1.213 = 25,200 plus

It will be seen that the diagonals have to resist both compression and tension, but compression will probably govern. Let us try a single angle say 1-L 3 1/2" x 3 1/2" x 3/8". The radius of gyration of this angle is 1.07 ,   L = 108". Then substituting in the compression formula, $16,000 - 70 \frac{L}{r}$ ,

$$16,000 - 70\left(\frac{108}{1.07}\right) = 9,000\#$$ for the allowable stress.

Dividing the greatest compressive stress, which occurs in bo

20,800 ÷ 9,000 = 2.31 for the req'd cross section of the lateral and the assumed L has an area of 2.48 sq. in. For the area required in tension, 20,800 ÷ 16,000 = 1.30 sq. in., so the angle is quite sufficient for the end lateral. The angle selected is the smallest allowed by the specifications so it will be used throughout in the lateral bracing.

The cross frames can be fairly well analysed. The stress in the top angle of the frame can be taken equal to one-half of the lateral force per foot of span multiplied by one-hald the length of the span, and this force multiplied by the secant of the angle of slppe of the diagonals. The bottom angle of the frame has no stress(theoretically). Then according to the above we have:

$$\frac{775 \times 29}{2} = 11,300\#$$

for the stress in the top angle of the frame and as the diagonals have a slope of about $45^{\circ}$ with the horizontal the stress in the diagonals will be:

$$11,300 \times 1.4 = 15,800\#$$

A 3 1/2" x 3 1/2" x 3/8" is plenty large for this position, so a 3 1/2" x 3" x 3/8" angles will be used in the cross frames. This concludes the preliminary computations for the design of the girder.

| 60' Girder | | No. | wt. per ft. | Total | |
|---|---|---|---|---|---|
| Web | 60' x 72" x 3/8" | 1 | 91.8 | 5508# | 11,016# |
| Flange | | | | | |
| Angles | 60' x 6 x 6 x 5/8 | 4 | 24.2 | 5808# | 11, |
| Plate | 60' x 16" x 1/2" | 1 | 27.2 | 1632# | |
| Plate | 32' x 16" x 1/2" | 2 | 27.2 | 1741# | |
| Plate | 42' x 16" x 1/2" | 1 | 27.2 | 1138# | |
| Rivets | 7/8" x $\frac{14}{8}$" | 800 | | 675# | 21,988# |
| Stiffeners | 6" x 3 1/2" x 1/2" x 5" | 8 | 13.6 | 652# | |
| | 6" x 3" x 5" x 3/8" | 12 | 9.8 | 704# | |
| Fillers | 3 1/2" x 5" x 5/8" | 8 | 4.46 | 778# | |
| Rivets | 7/8" D | 110 | | 74# | 3,216# |
| Cross Frames | 3" x 3" x 3/8" x 7'-5" | | 7.2 | 216# | |
| Angles (end) | 3 1/2" x 3" x 3/8" | | 7.2 | 144# | |
| Plates | 12 1/2" x 17" x 1/2" | 4 | 21.25 | 126# | |
| | 14 1/2" x 16" x 3/8" | 4 | 24.65 | 131# | |
| | 8" x 8" x 3/8" | 2 | 13.60 | 18# | |
| Rivets | 7/8" D | 104 | | 56# | 871# |
| (Intermediate Angles | 3" x 3" x 3/8" x 7'6" | 12 | 7.2 | 648# | |
| | 3" x 3" x 3/8" x 5'11" | 12 | 7.2 | 518# | |
| | 12 1/2" x 17" x 1/2" | 12 | 21.25 | 378# | |
| | 14 1/2" x 16" x 3/8" | 12 | 24.65 | 499# | |
| | 8" x 8" x 1/2" | 6 | 13.60 | 55# | |
| Rivets | 7/8" D | 104 | | 55# | 2,513# |
| Lateral Bracing | | | | | |
| Plates | 17" x 29" x 3/8" | 6 | 21.04 | 840# | |
| | 17" x 11" x 3/8" | 6 | 21.04 | 116# | |

| | | | | | |
|---|---|---|---|---|---|
| Angles | 3" x 3" x 3/8" x 8'6" | 12 | 7.2 | 750# | |
| Rivets | 7/8" D | 260 | | 141# | 1,807# |
| Web Splice | 9" x 30" x 3/8" | 2 | 9161 | 48# | |
| Plates | 13" x 42" x 3/8" | 2 | 16.58 | 116# | |
| Rivets | 7/8" D | 42 | | 45# | 418# |
| | | | Total | | 41,469# |

Conclusions for the Deck Girder Span

Considerable difficulty was encountered at first in discovering the dead load allowed which would produce the stress which the web plate would sustain. The span is designed for a dead load of 2400#/ft. of track which makes a liberal estimate of the weight of ballast.

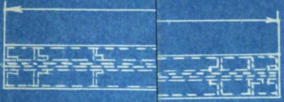
Using the E 40 live load and the dead load as stated, the total shear is found to require a web area of 21.18 sq. in. A web 75" x 3/8" gives an area of 28 sq. in.

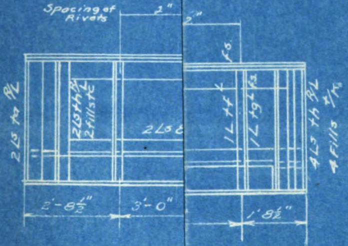
The economic depth is 75.7 sq. in., but a depth of 75" was found in the girder.

The flanges as designed were found to be exactly the same as those found on the girder. The flange areas are 2.5 sq. in. in excess of the required areas. The cover plates as designed were also the same length as found in the flange.

The end stiffeners as designed were the same as used on the girder. The stiffeners were also amply strong, having an area 2 sq. in. in excess of that required. The spacing was also very liberal, and the entire web is sufficiently stiffened.

The minimum sized laterals used in the specifications are 3 1/2" x 3/8" x 3", but the angles found in the laterals were 3" x 3" x 3/8". With this one exception, everything else about the girder is well above that required by the specifications.






## 35' GIRDE

Assumed dead lo
Eff. Span 35'
End Shear
Dead Load
Live Load
Impact
Total 1
Reqd Area of We
Use Web Plate 42"
Reqd Pitch at end
Moment
Dead Load
Live Load
Impact
Total 11
Eff. Depth 39"

BEAL ROAD BRIDGE
MICHIGAN CHICAGO RY.
FARGO ENGINEERING CO.
Scale $\frac{3}{8}" = 1'$

Complete Design of a 35' Through Plate

Girder, G 8.---(Johnson, Bryan, and Turneau)

Length 35' c-c of bearings.

Width 16' c-c of girders.

Dead Load

(metal only) 0.9(12.5 L + 100) = 485#.

Total dead weight 2000#/ft. of track.

Maximum moments occur at the center of the span

| | |
|---|---|
| Dead Load | 153,125 |
| Live Load | 523,000 |
| Impact | 468,300 |
| | 1,144,425 in # |

Maximum shear occurs at end

| | |
|---|---|
| Dead Load | 17,500 |
| Live Load | 69,200 |
| Impact | 62,000 |
| | 148,700 # |

For the depth(economical)

$$h = h_f \sqrt{\frac{V}{62.5} + h_f^2}$$

$$= 6 - \left(\frac{148,700}{62.5} - 36\right) 1/2 \qquad h = 49"$$

$$t = \frac{49 - 12}{160} = .232 \text{ in.}$$

3/8" is the minimum thickness used so a web plate 3/8" will be used.

For the economical depth $h = 1.1\sqrt{\frac{1,144,425}{14,000 \times 3/8}} = 51.6"$

Reducing this by 20%, h = 40.5".

We will use a web 42" x 3/8", area 15.75 sq. in.

The flange area will next be determined. The effective depth will be slightly less than the total depth, so an effective depth of 41" will be assumed.

1,144,425 in.# ÷ 41 = 280,000# for the flange stress.

280,000 ÷ 16,000 = 17.5 sq. in. in lower flange.

For the upper flange

$$A = \frac{P}{16{,}000 - 70\frac{l}{b}}$$

$$= \frac{280{,}000}{16{,}000 - \frac{70 \times 35 \times 12}{14}}$$

$$= 20.2 \text{ sq. in.}$$

Use in the lower flange

| | | |
|---|---|---|
| 2 Ls 8 x 8 x 5/8" | 19.22 | 16.72 |
| 1 pl 6" x 5/8" | 3.75 | 2.50 |
| 1 L 4" x 3" x 3/8" | 2.48 | 1.73 |
| 1/8" web | 1.97 | 1.97 |
| | 24.22 | 22.92 sq. in. |

Use in the upper flange

| | |
|---|---|
| 2 Ls 6" x 6" x 5/ 8" | 11.72 |
| 1 pl. 14" x 7/16" | 5.25 |
| 1 pl. 14" x 3/8" | 4.50 |
| 1/8" web | 1.97 |
| | 23.44 sq. in. |

The thicker cover plate should be placed next to the angles and in the upper flange one cover plate must cover the entire length of the cover plates.

The length of the shorter plate is given by the formula:

$$L \sqrt{\frac{a_1 + a_2}{A}}$$

from which

$$\sqrt{\frac{4.5 + 5.25}{23.44}} = 22.6'$$

and for the length of the cover plates on the lower flange no cover plates are used.

The next thing to calculate are the stiffeners. End stiffeners must be carried on fillers and carry the entire load at the end of the girder. The maximum end shear was 148,700#. According to the specifications the outstanding leg of the angle must equal 2" + 1/30 of the depth of the girder.

$2" + \frac{62}{30} = 3.4"$. We will try Ls 3 1/2" x 3 1/2" x 3/8" for the end stiffeners. Then assuming each two of these angles to act as a column we have:

$$r = \sqrt{\frac{4.96 \times 1.88^2 + 5.8}{4.96}} = 2.18$$ , using this value of "r" and the column length of 21"

$$16{,}000 - 70\,\frac{21}{2.18} = 15{,}320\#$$ for the allowable compressive stress in the stiffeners.

$$\frac{148{,}700}{15{,}320} = 9.62$$ sq. in. for the required area in the end stiffeners. We will use 4 Ls, 3 1/2" x 3 1/2" x 3/8". Area 9.62 sq. in.

Using the angles as calculated, we will have to find the required spacing of the intermediate stiffeners.

$s = 148{,}700 \div 15.75 = 9{,}450\#$ maximum unit shearing stress in the web from which

$$d = \frac{66}{40}(12{,}000 - 9{,}450) = 23.5"$$ since this is less than the depth of the web, the spacing will be considered satisfactory.

35' Girder

| | | | |
|---|---|---|---|
| Assumed dead load 2000#. | Use in bottom flange | | |
| Effective span 35'. | 1/8 web area | 1.97 | 1.97 |
| | 2 Ls 8 x 8 x 5/8 F.L. | 19.22 | 16.72 |
| | 1 pl. 6 x 5/8 | 3.75 | 2.50 |
| | 1 L 4 x 3 x 3/8 | 2.48 | 1.73 |
| | | 24.22 | 22.92 |

Maximum end shear

| | |
|---|---|
| Dead Load | 17,500# |
| Live Load | 69,200# |
| Impact | 62,000# |
| | 148,700# |

Required web area 14,87 sq. in.

Use 42" x 3/8" , 15.75.

Use in top flange

| | | |
|---|---|---|
| 2 Ls 6 x 6 x 5/8 | 14.22 | 11.72 |
| 1 pl 14 x 7/16" | 6.12 | 5.25 |
| 1 pl 14 x 3/8 x 18" | 5.25 | 4.50 |
| 1/8 web | 1.97 | 1.97 |
| | 27.56 | 23.44 |

Maximum momentum at center

| | |
|---|---|
| Live load | 153,125 in# |
| Dead load | 523,000 in# |
| Impact | 468,300 in# |
| | 1,144,425 in# |

Stiffeners

Weight of 35' Girder

| | Size | No. | wt. per ft. | Total | |
|---|---|---|---|---|---|
| Web | 35' x 42" x 3/8" | 1 | 53.55 | 1880# | 1,880# |
| Flange Angles | 6 x 6 x 5/8" x 35' | 2 | 24.2# | 1700# | |
| Angles | 8 x 8 x 5/8" x 35' | 2 | 32.7# | 2280# | |
| Plate | 6" x 5/8" x 35' | 1 | 12.75 | 445# | |
| Angle | 4" x 3" x 3/8" x 35' | 1 | 8.5# | 297# | |
| Plate | 14 x 7/16" x 35' | 1 | 20.83 | 728# | |
| Plate | 14 x 3/8" x 18' | 1 | 17.85 | 322# | |
| Rivets | 7/8" D | 720 | | 420# | 6,192# |
| Stiffeners Angles | 6" x 6" x 5/8" x 3'6" | 2 | 24.2 | 169.4 | |
| Angles | 5" x 3" x 3/8" x 3'6" | 2 | 9.8 | 68.6 | |
| Fillers | 3" x 5/8" x 2'4" | 2 | 1.87 | 88.0 | |
| Angle | 4" x 3" x 3/8' x 4' | 1 | 8.5 | 34.0 | |
| Filler | 6" x 5/8" x 3' | 1 | 3.75 | 11.25 | |
| Fillers | 3" x 5/8" x 4' | 7 | 1.87 | 52.2 | |
| Angles | 5" x 3" x 3/8" x 3'6" | 7 | 9.8 | 241.0 | |
| Angles | 5" x 3" x 3/8" x 2'4" | 7 | 9.8 | 178.0 | |
| Angles | 5" x 4" x 3/8" x 2'4" | 3 | 11.0 | 77.0 | |
| Angles | 5" x 3 1/2" x 3/8" x 3'6" | 3 | 10.4 | 110.0 | |
| Angles | 5" x 3 1/2" x 1/2" x 3'5" | 4 | 13.6 | 191.0 | |
| Fillers | 3 1/2" x 5/8" x 2'4" | 4 | 2.18 | 205.0 | |
| Fillers | 3 1/2" x 5/8" x 9' 1/4" | 3 | 2.18 | 61.2 | |
| Rivets | | 118 | | 55.8 | 1,362.4# |
| Gussets | | 2 | 150.0 | | 300.0 |
| | | | | Total | 9,738.4# |

Summary of Analysis of 35' Through Girder

Length 35'

Depth 16' c-c of girders.

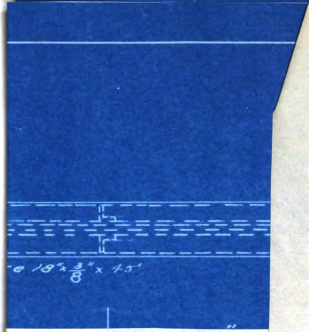
The weight of the metal in this girder by empirical formula is 485#. The other dead load consisting of track, ties, etc., was 1638#. The total dead load is 2123#/ft. of track, but Mr. Fargo's figures allowed 2900#/ft. of track.

The maximum shear was 148,700# and required a web plate area of 14.87 sq. in. The web plate used was 42" x 3/8" = 15.75 sq. in.

The maximum moment was 1,144,425 in # and with an effective depth of 41", a flange area of 17.5 sq. in. was required. Due to the construction of the larger girders, the same size angles were used in the 35' girder as in the larger girders. The lower flange used had an area of 22.92 sq. in and the upper flange an area of 23.44 sq. in. The shorter cover plate as determined from the empirical formula was 15.2'. The cover plate used on the girder was 18' long. No cover plates were used.

The spacing of the intermediate stiffeners was found to be quite satisfactory as all spacings were very conservative.

The weight of 9,738# is in excess of that given by the formula 9(100 + 12.5 x 35). Since 2000# ft. of track was allowed for the dead-load weight, the increase in weight is taken care of.


e 18" x 3/8" x 4.5'

Completed Design of an 86' Through Plate Girder

Data:

length 86'-0" c-c. of bearings Dead Load 3000#/ft. of track

Width 16'-0" c-c. of girders Live load Cooper's class E 40

Calculations:

| | |
|---|---|
| Weight of track, ballast and etc. | 1636# |
| Weight of metal in girder $.9(12.5\ L + 100) = .9(1075 + 100)$ | 1060# |
| | 2696# Total weight |
| M due to dead load $= 1/8 \times 1500 \times 86^2 \times 12$ | 1,354,600 in # |
| From table in Ketcham's handbook Max. M due to E 40 loading on 86' span | 2,407,000 in # |
| Impact = 85.6% of live load moment | 1.875,500 in # |
| Total moment at the center span | 5,637,100 in # |

The first thing to do is to find the economic depth. The specifications require a web thickness of at least 3/8" so that thickness of web will be assumed in these calculations. This thickness is at once used in the formula:

$$h = 1.1\sqrt{\frac{M}{f\ t}}$$

$$= 1.1\sqrt{\frac{5,637,100}{16,000 \times 3/8}} = 107"$$

A web plate 114" x 3/8" was necessary in the 101' span and for the sake of appearance, all of the longer through spans will use the same size web plate, 114" x 3/8".

Since the flange takes all the moment, the flange area will next be determined. The effective depth will be slightly less than the depth of the web plate so we will assume an effective depth of 112". 5,637,100 ÷ 112 = 50,300 in # for the flange stress. 50,300 in # ÷ 16,000 = 32.6 sq. in required flange area.

| Use in the lower flange | Gross | Net |
|---|---|---|
| 2 Ls 8" x 8" x 5/8" | 19.22 | 15.47 |
| 1 pl 6" x 5/8" | 3.75 | 2.25 |
| 1 pl 18" x 7/16" | 7.88 | 7.00 |
| 1 pl 18" x 3/8" | 6.75 | 6.00 |
| 1 L 4" x 3" x 3/8" | 2.48 | 1.73 |
| 1/8" web | 5.34 | 5.34 |
| total | 45.42 | 38.04 |

| Use in upper flange | |
|---|---|
| 2 Ls 8" x 8" x 5/8" | 19.22 |
| 1 pl. 18" x 5/8" | 11.25 |
| 1/8 web | 5.34 |
| 1 pl 18" x 9/16" | 10.13 |
| Total | 44.94 |

Required area in upper flange is found from the formula

$$A = \frac{P}{16{,}000 - 70\frac{L}{b}}$$

from which $A = \dfrac{50{,}300 \text{ in } \#}{16{,}000 - \dfrac{70 \times 86 \times 12}{18}} = 42.6$ sq. in.

The thickest cover plate should be placed next to the angles. For the lower flange and the longer plate, we have:

$$86\sqrt{\frac{7.00}{38.04}} = 37'$$

and for the shorter plate:

$$86\sqrt{\frac{6.00 + 7.00}{38.04}} = 50.2'$$

According to the specifications one of the upper cover plates must extend the full length of the girder.

The next thing to determine is the shear in the web and the necessary stiffeners. The masimum shear is at the end of the span and is made up as follows:

| | |
|---|---|
| Dead load | 63,750 |
| Live load | 130,700 |
| Impact 83.6% of live load | 101,900 |
| Total | 296,350 # |

The allowable unit shearing stress is 10,000# per sq. in. so 29.6 sq. in. of web area is required. A web plate 114" x 3/8" has an area of 42.75 sq. in. and gives the required area plus. From the specifications the outstanding leg of the stiffening angle must equal 2" + 1/30 of the depth of the girder.

$\frac{114"}{30} + 2" = 5.8"$. An angle 6" x 31/2" x 3/8" will be selected and used throughout for the intermediate stiffeners. The end stiffeners carry the full and shear and act as columns. Angles 6" x 6" x 1/2" will be selected for the first trials. We have then for each of the two angles acting as a column:

$$r = \sqrt{\frac{11.5 \times 2.67^2 + 39.8}{11.5}} = 3.16$$

Using this value of "r" and the column length of 57"

$$16{,}000 - \frac{57}{3.16} - 70 = 14{,}760\# \quad \text{for the allowable}$$

compressive stress in the end stiffeners.

$$\frac{269{,}350}{14{,}760} = 18.2 \text{ sq. in.} \quad \text{We will use 4 Ls}$$

6" x 6" x 1/2"  Area 23 sq. in.

Using the stiffeners as calculated, we will make the spacing accordingly. $s = 269{,}350 \div 42.75 = 6{,}320\#$ for the maximum unit shearing stress from which:

$$d = \frac{3/8}{40}(12{,}000 - 6{,}320) = 54.2" \text{ or } 4'\ 6"$$

Since this is less than the depth of the girder and less then 6', this arrangment of stiffeners is satisfactory.

In a through plate girder bridge stays for the top flanges are necessary at distances not greater than every 12'.

Weight of Girder 61'

| Part | Size | No. | Wt. per ft. | Total | |
|---|---|---|---|---|---|
| Web | 114" x 3/8" x 86' | 1 | 166.7 | 14400# | 14,400# |
| Flange Angles | 8 x 8 x 5/8" x 86' | 4 | 32.7 | 11,040 | |
| Plates | 18" x 5/8" x 86' | 1 | 38.25 | 3,290 | |
| " | 18" x 7/16" x 52' | 1 | 28.26 | 1,480 | |
| " | 18" x 3/8" x 36" | 1 | 22.95 | 821 | |
| " | 6" x 5/8" x 86' | 1 | 8.5 | 731 | |
| " | 18" x 9/16" x 43' | 1 | 34.43 | 1,480 | |
| Angle | 4" x 3" x 3/8" x 86' | 1 | 3.75 | 323 | |
| Rivets | 3/4" D | 972 | | 455 | 19,620# |
| Stiffeners | | | | | |
| Angles | 6" x 6" x 1/2 x 9'6" | 8 | 19.6 | 1,481 | |
| | 6" x 3 1/2" x 3/8" x 98" | 34 | 11.7 | 3,820 | |
| Fillers | 6" x 5/8" x 8' 6" | 8 | 3.75 | 262 | |
| Rivets | | 342 | | 160 | 5,723# |
| Gussets | | 7 | 284.0# | | 1,988# |

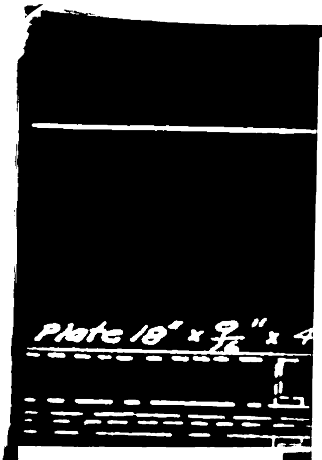


Plate 18"

Summary of Analysis of 86' Through Plate Girder

Length 86' c-c. of bearings.

Width 16' c-c. of girders.

The total dead load was 2696#/ft., but the figure allowed by Mr. Fargo was 3000#/ft. of track.

The total shear was 296,350# requiring an area of 29.63 sq. in. The economic depth was 107"; but for the sake of appearance a web plate 114" x 3/8" was used, giving an excess web area of 42.75 sq. in.

The required flange area for an effective depth of 112 sq. in. was 42.6 sq. in. for the upper flange and 32.6 sq. in. for the lower flange. The area in the upper flange used was 44.94 sq. in. and in the lower flange, 38.04 sq. in.

The intermediate stiffeners used are 6" x 3 1/2" x 3/8" angles and the end stiffeners are 4 6" x 6" x 1/2" angles. The end stiffneers have an area in excess of that required of 5 sq. in.

The spacing of intermediate stiffeners is very conservative and no criticism can be expressed of them.

Complete Design of an 83' Through Plate Girder, G3

Data: Length 88' c-c. of bearings Dead load 2900#/ft.

Width 16' c-c. of bearings Live load class E 40.

Calculations for the main girder:

Dead load(metal only) .9(12.5 L + 100) = 1020

Total dead load 2900#/ft. of track.

Max. moment occurs at the center of the span.

| | |
|---|---|
| Dead Load | 1,250,000 in # |
| Live load (from Ketcham's Handbook) | 2,306,400 in # |
| Impact 83.6% of live load | 1,800,530 in # |
| Total | 5,356,930 in # |

The effective depth is:

$$h = 1.1\sqrt{\frac{M}{f\,t}} \qquad t = 3/8''$$

$$h = 1.1\sqrt{\frac{5,365,930}{6000}} = 104''$$

We will use the same size web as in the 101' girder, namely, 114". The effective depth will be assumed as 112". Then 5,365,930 ÷ 112 = 478,000# for the flange stress.

478,000 ÷ 16000 = 29.2 sq. in. net area required in the lower flange.

For the upper flange $fs = 16,000 - \frac{70 \times 83 \times 12}{18} = 11,900$

Then an area of 40.2 sq. in. is required in the upper flange

| Use in the lower flange | Gross | Net |
|---|---|---|
| 1/8 web | 5.34 | 5.34 |
| 2 Ls 8" x 8" x 5/8" | 19.22 | 15.47 |
| 1 Pl 6" x 5/8" | 3.75 | 2.25 |
| 1 Pl 4" x 3" x 3/8" | 2.48 | 1.73 |
| 1 Pl 18" x 3/8" x 51' | 6.75 | 6.00 |
| 1 Pl 18" x 3/8" x 35' | 6.75 | 6.00 |
| | 41.29 | 36.79 |

Use in the upper flange

| | |
|---|---|
| 1/8 web | 5.34 |
| 2 Ls 8" x 8" x 5/8" | 19.22 |
| 1 Pl. 18" x 9/16" F.L. | 10.10 |
| 1 Pl 18" x 9/16" x 42' | 10.10 |
| | 44.76 |

The shorter cover plate in the upper flange equals

$$83\sqrt{\frac{10.1}{44.76}} = 39.5'$$

For the length of the cover plates in the lower flange

$$83\sqrt{\frac{6.00}{36.79}} = 34' \text{ for shorter plates}$$

$$83\sqrt{\frac{12.00}{36.79}} = 47' \text{ for longer plates}$$

The end shear in the web is:

| | |
|---|---|
| Dead Load | 60,100# |
| Live Load | 128,200# |
| Impact | 100,400# |
| | 288,700# |

Hence 28.87 sq. in. of web area is required. A web plate 114" x 3/8" has an area of 42.75 sq. in. The outstnading leg of the stiffener angle must equal 2" + 1/30 of the depth of the web.

$$2'' + 1/30 \text{ of } 114 = 5.8''$$

Since 6" x 6" x 1/2" angles are used as end stiffeners on all the large girders, they will be used on this girder.

From the analysis of the 86' girder, this allowable compressive stress is 14,760#/sq. in. Four Ls have an area of 23 sq. in., making the total shear which could be safely carried 338,000#, but we have only 288,700# shear. For the intermediate stiffeners 6" x 3 1/2" x 3/8" Ls were used. The spacing is

$$\frac{3/8}{40}\left(12,000 - \frac{288,700}{42.75}\right) = 49'' \text{ or } 4' \text{ at the ends.}$$

In a through plate girder, stays must be provided at distances not exceeding 12'. The gusset or knee brace will be analyzed under a separate heading.

Conclusions on Analysis of 83' Girder

The web area and slope is determined by that required in the largest girder. The web in the 105' girder was 114" deep and for appearance all the longer through spans were made the same depth. This gives an excess area in the web of 14 sq. in. or about 50% greater than is needed.

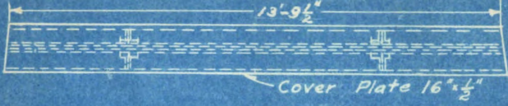
The flange composition except for the cover plates is also determined by the size of the flanges used in the largest girders. The upper flange area is 10% larger than is needed. The lower flange is arbitrarily chosen because of the connection between the floor sections and the girder. The lower flange area is 26.5% larger than is necessary.

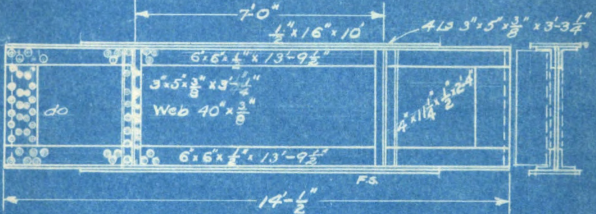
The end stiffeners are 6" x 6" x 1/2" Ls and are standard throughout the through girder spans. The intermediate angles are likewise standardized, 6" x 3 1/2" x 3/8" Ls being use. According to Kirkham there is no theoretical way of spacing the intermediate stiffeners.

The analysis 83' girder is exactly the same as that of the 86' girder except for the cover plates in the flanges.

Estimated Weight of Girder G3

| Part | Size | No. | Wt. per ft. | Total | |
|---|---|---|---|---|---|
| Web plate | 114" x 3/8" x 83' | 1 | 166.7# | 13,823 | 13823# |
| Flange | | | | | |
| Angles | 8" x 8" x 5/8" x 83' | 2 | 32.7 | 5,430 | |
| " | 6" x 8" x 5/8" x 83' | 2 | 32.7 | 5,430 | |
| Plate | 18" x 9/16" x 83' | 1 | 34.43 | 2,840 | |
| " | 18" x 9/16" x 42' | 1 | 34.43 | 1,428 | |
| " | 6" x 5/8" x 81' | 1 | 3.75 | 305 | |
| " | 18" x 3/8" x 51' | 1 | 6.75 | 345 | |
| " | 18" x 3/8" x 35' | 1 | 6.75 | 237 | |
| Angle | 4" x 3/8" x 5/8" x 81' | 1 | 11.25 | 911 | |
| Rivets | 3/4" D | 2312 | | 1,081 | 18,007# |
| Stiffeners | | | | | |
| Angles | 6" x 6" x 9' 6" | 8 | 19.6 | 1,481 | |
| " | 6" x 3 1/2" x 3/8" x 9' 9" | 38 | 11.7 | 4.320 | |
| Fillers | 6" x 5/8" x 3 1/2" | 8 | 3.75 | 262 | |
| Rivets | 3/4" D | 416 | | 196 | 6,259# |
| Web splices | | 4 | | | |
| Gussets | | 6 | 248# each | | 1,688# |




## 2 Cross Beams B1

Load: 2 concentrated loads 7' apart at
= 11000# each
Max Shear 211,000#
Reqd Area 21.1 sq. in
Total Moment 4,431,000 in#
Eff Depth 39"
Reqd flange Area 7.32 sq. in.
Use for flanges

| | | | |
|---|---|---|---|
| 2 Ls | 6" x 6" x ½" | 11.50 | 9.0 |
| 1 PL | 16" x ½" x 10' | 8.00 | 7.25 |
| | | 19.50 | 16.75 |

BEAL ROAD BRIDGE
MICHIGAN & CHICAGO RY. CO.
FARGO ENGINEERING CO.
SCALE ⅜" = 1'-0"

Analysis of Cross Beams B1 and 2

Length 14 - 1/2" say 14'. See Photograph 2.

2 concentrated loads 211,000# each 7' apart about center.

Calculations:

Total shear 213,000#

Total moment 4,447,300 in #

Require. area for shear 21. sq. in.

The plate used is 40" x 3/8" = 15 sq. in.

2 plates 1/2" x 28" = 28 sq. in.

43 sq. in.

The shear in the flange equals

4,447,300 ÷ 40 = 111,825 in #.

111,825 ÷ 16,000 = 7.34 sq. in.#

The flange used was

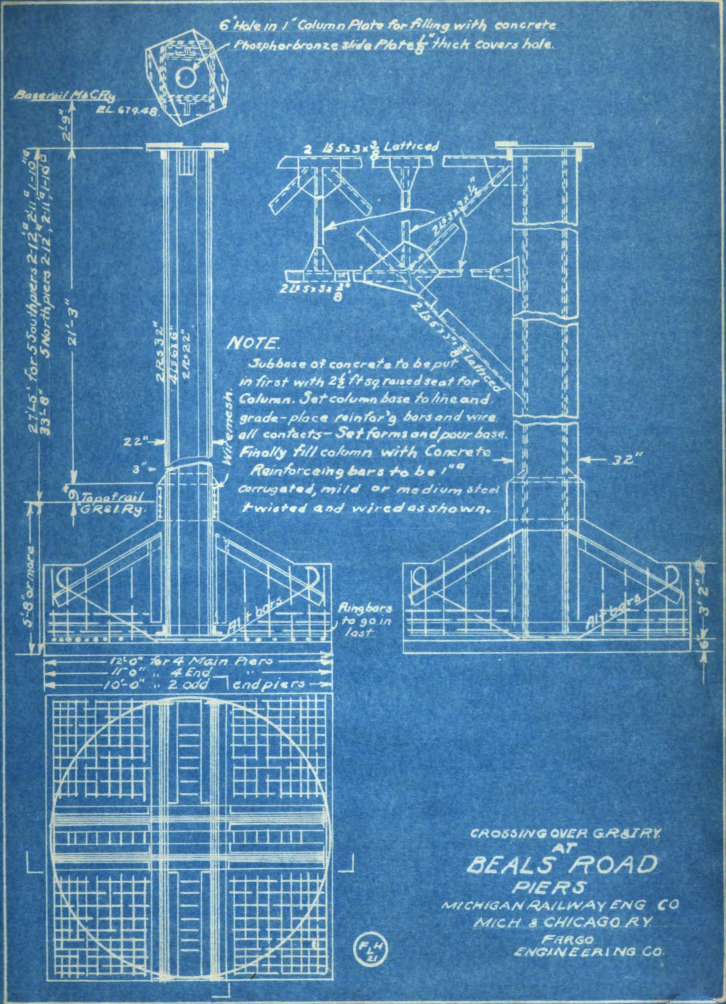
| | Net area |
|---|---|
| 2 Ls 6" x 6" x 1/2" | 9.5 |
| 1 pl. 16" x 1/2" x 10" | 7.25 |
| | 16.75 sq. in. |

Length of cover plates $14\sqrt{\frac{7.25}{16.75}} = 9.5"$

Stiffeners under concentrated loads 212,000#

212,000 ÷ 15 = 14,100 # sq. in.

This shows the allowable unit shearing stress in the web directly underneath the concentrated loads to be exceeded by 4,100# sq. in. With this exception, the beams are amply strong. This is the first number in the entire structure which failed to come up to specifications.



6" Hole in 1" Column Plate for filling with concrete
Phosphorbronze slide Plate 1/8" thick covers hole.
Baserail M&CRy
EL 679.48.
2'-9"
2 Ls 5x3x3/8 Latticed
2 Ls 3x3x1/4"
2 Ls 5x3x3/8
2 Ls 5x3x3/8 Latticed
21'-3"
27'-5" for 5 South piers 2-12", 2-11", 1-10"
33'-8" 5 North piers 2-12", 2-11", 1-10"
2 Ps 32"
4 Ls 6x6
2 Ps 22"
22"
3"
Wiremesh
Top of rail G.R.&I.Ry.
9
NOTE.
Subbase of concrete to be put in first with 2½ ft sq raised seat for Column. Set column base to line and grade - place reinfor'g bars and wire all contacts - Set forms and pour base. Finally fill column with Concrete
Reinforceing bars to be 1" corrugated, mild or medium steel twisted and wired as shown.
32"
5'-8" or more
All bars
Ringbars to go in last
3'-2"
6"
12'-0" for 4 Main Piers
11'-0" " 4 End "
10'-0" " 2 odd end piers
CROSSING OVER G.R.&I.RY.
AT
BEALS ROAD
PIERS
MICHIGAN RAILWAY ENG CO
MICH. & CHICAGO RY.
FARGO
ENGINEERING CO.
FLH 21

Analysis of the Concrete Footing under Column. CL1.

The total dead load from the girders to the column is 269,000# + 210,000# = 480,000#. The weight of the column is approximately 27,000#. The pier is 10' square and 3' 6" deep. At 150#/cu. ft., the pier weighs 60,000#, making the total weight on the soil underneath the pier 567,000# spread over an area of 100 sq. ft. The unit bearing stress resulting is 5.67 tons per sq. ft. or 2.83 tons. The allowable pressure on the foundation is from 2 to 3 tons for ordinary clay and dry sand. The foundation here rests on a gravelly soil underlaid by a layer of hard blue clay over a deep bed of muck. Hence the footing will be considered safe for bearing pressure.

Piers commonly fail from pinching shear, that is the section directly underneatch the column bearing plate being forced down through the footing. The column rests on a plate 24" x 36" having a perimeter of 10 ft.

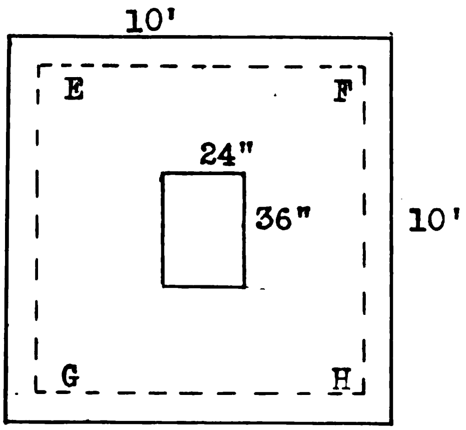
The concrete in the piers was made of local ungraded aggregate in a ration of 1 : 2 : 6. The concrete was carelessly mixed and placed so the allowable compressive stress will be taken as 1500#/sq. in. The pier has both vertical and horizontal reinforcement so 6% of the compressive stress will be allowed for shear.

The load producing punching shear is:

$$\frac{100 - 6}{100} \times 507{,}000\# = 475{,}000\#$$

The minimum depth then for punching shear is

$$\frac{475{,}000}{2 \times 10 \times 12 \times 90} = 22''$$




Since $d = 36"$

the total shear on EFGH

$$= \frac{100 - 8 \times 9}{100} \times 507{,}000 =$$

142,000#

$$v = \frac{V}{b\ j\ d} = \frac{142{,}000}{388 \times .875 \times 32} =$$

13.4# so no

are needed.

Bending moment on each set of rods is

$M = C_2 dp \qquad c_1 = .615$

$.615 \times 507{,}000 \times 120 = 3{,}160{,}000$ in #

$$As = \frac{3{,}160{,}000}{16{,}000 \times 36 \times 0.875} = 6.4 \text{ in of steel.}$$

Fourteen - 3/4" round bars. Area 6.21 sq. in.

Hence from this analysis it will be seen that the pier has sufficient area for bearing and are deep to prevent punching by the column. The fourteen round bars give ample reinforcement in the bottom of the pier. In addition there are ring bars and angles attached to the column itself which increases the allowable stress allowable in the concrete. The piers will probably carry fully 50% more load than will ever come upon them.

**Analysis of One Gusset**

Consisting of 2 Ls 3 1/2" x 3" x 3/8" x 5' 3" and 1 plate 16" x 3/8" x 8' 2". The radius of gyration of these angles is 0.91 and the length 98". Substituting in the compression formula:

$$16{,}000 - 70 \frac{98}{0.91} = 15{,}250\#$$

The area of these two angles is 4.22 sq. in. or the total compressive stress that these angles will take is 4.22 x 15,250#

= 64,100#.

The flange stress due to overturning is .10(5200) + 200 = 720# applied 7' above the rail.

As the rail is about 2' above the lower angles and the girders are 16' apart, the overturning moment is:

$$720 \times \frac{9}{16} = 406\#$$

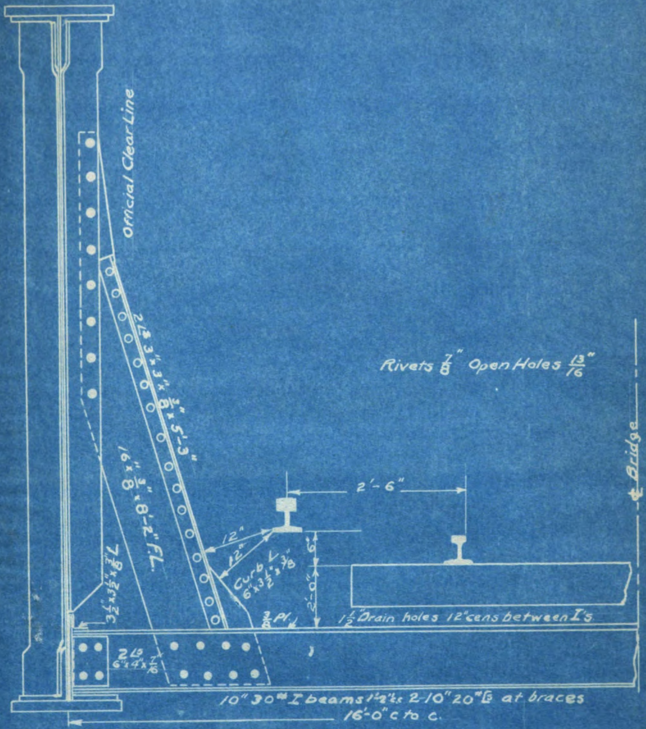
The bending moment at the center of the span is:

$$\frac{406 \times 100^2}{8} = 457{,}000 \text{ ft.\#}$$ and the rewulting flange stress is $$\frac{457{,}000 \times 12}{112} = 48{,}100\#.$$

It is hard to determine the exact stress passing from the girder to the gusset plate, but standard practice requires 2 Ls 3" x 3" x 3/8" and a 3/8" web plate between them. This is the arrangment used in the gussets on this bridge.

Weight of Gussets

| | |
|---|---|
| 2 Ls 3" x 3" x 3/8" x 5' 3" | 76.1# |
| 1 pl. 16" x 3/8" x 8' 2" | 192.4# |
| 30 3/4" Rivets | 14.1# |
| | 248.6# |



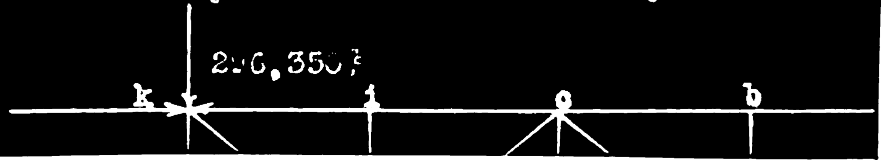

HALF SECTION OF BRIDGE

CROSSING OVER G.R.&I. RY
AT
BEALS ROAD
SECTION of BRIDGE
MICHIGAN RAILWAY ENG. CO.
MICH. & CHICAGO RY.

TRACED APRIL 21, 1921 F.H.

Scale 3/4"=1'0"

Analysis of Columns and Sway Bracing.



## Final Conclusions

The Beal Road Bridge is a very good example of the best practice in recent bridge building. Bridge designing like all other forms of structural designing is an art.

To design a bridge, one must do more than to compute the various stresses in the bridge and to make all members amply strong. He must so proportion all the parts as to make an economical design to build, but also a structure easily erected and when once erected, capable of resisting not only the loads it carries, but the elements as well until a time until newer structural practice makes it advisable to replace the structure.

This bridge is designed to carry what is known as Cooper's Class E 40 loading. This loading consists of a standard locomotive of a certain weight followed by loaded cars also of definite weight. Since no standardized loading has been provided for electric railway structures, steam road loadings have to be used. A careful analysis of the forces caused by an electric car passing over a bridge shows that those forces are from 50 to 75% of those caused by # 40 loading. Hence Beal Road Bridge will be able to carry interurban cars from 33 1/3 to 50% larger than are being now used. Although it is hard to prophesy the future growth and development of electrical railways, it is safe to assume that the Bridge will not require replacement for 50 years.

A comparison of the results obtained from my computations of the design of several members of the structure with the actual conditions found in the bridge show the bridge to be entirely safe. Extreme conditions were watched out for and these conditions amply provided for.

The greatest stresses in the bridge occurr in the girders. Every girder shows excess material.

If a bridge was not stiffened and braced throughout, there would be exessive vibration when the cars passed across the bridge. Due to the rock ballast used and the thorough bracing of the entire structure, a car passing over the bridge will not jar a coin placed on a horizontal surface on the bridge. Lack of vibration makes riding on the Kalamazoo Interurban cars much more pleasant than is commonly attributed to electric railway cars.

No structure can stand for long unless it has secure foundations. All piers are of heavily reinforced concrete with broad bases. The piers rest directly on gravelly soil mixed with clay which forms the best bracing support offered by any soils.

The steel columns supporting the girders appear to be mammoths to the layman. These columns are filled with concrete to prevent rusting of the metal on the inside of the columns and to give added stiffness to the structure.

The layman may fail to see the economy of an overhead crossing at this point. The structure cost very nearly $50,000 and the approaches several thousand dollars more. But the state Railroad Commission ruled there must be a separation of railroad grades. The reason for this is quite apparent when one considers the amount of traffic on each. The Michigan Railway Company operates 36 passenger trains a day and the G. R. & I. operates about 20 trains a day on this line. The roads cross at very acute angles which forms the same conditions as was present at the recent wreck at La Porte, Indiana. A grade separation

prevents the greater number of accidents and has a value which cannot be measured in dollars and cents.

The Beal Road Bridge is by all means a monument to modern engineering foresight and skill. A bare plate Girder bridge is never a thing of beauty, but it serves its purpose well. The designers of this bridge have made the best of the conditions present and have produced a structure which is stable and thoroughly suited to its location.

## Miscellaneous Data

Rivet spacing in flange angles at end of girder

| | | | | | |
|---|---|---|---|---|---|
| Girder | G 10 | G 8 | G 2 | G 8 | G 3 |
| Spacing | 2.5" | 2.0" | 2.5" | 2.5" | 2.5" |

Maximum moments for remaining girders not analyzed:

| | | | |
|---|---|---|---|
| Girder | 45' | 101' | 96' |
| Moment | 1,805,000 in # | 7,510,000 in # | 6,989,000 in # |
| Shear | 180,150# | 337,530# | 326,500# |

Greatest web area required 33.75 sq. in.

Economical depth 114"

# BIBLIOGRAPHY

The following books were used throughout the preparation of this thesis:

Ketcham's "Structural Engineer's Handbook".

Kirkham's "Structural Engineering".

Johnson's, Bryan, and Turneau's "Modern Framed Structures".

Hool and Johnson's "Concrete Engineer's Handbook".

The author is greatly indebted to Mr. Wm. G. Fargo of the Fargo Engineering Company and the Engineering Department of the Michigan Railway Company for the ready and willing assistance they have given him in the preparation of this thesis.