LIFT-BRIDGE OVER THE OUTLET OF BLACK LAKE NEAR HOLLAND, MICH.

BY

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ARMOUR INSTITUTE OF TECHNOLOGY

1916
Deitenbeck, M.
A direct lift-bridge over the outlet of Black Lake
A DIRECT LIFT-BRIDGE OVER THE OUTLET OF BLACK LAKE NEAR HOLLAND, MICH.

A THESIS
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P R E F A C E

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M. D.
Chicago, May - 1915.
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PART I

INTRODUCTION

GENERAL DESCRIPTION AND DIMENSIONS

SPECIFICATIONS

DISCUSSION

CONCLUSION
The subject of this thesis is the design of a direct lift bridge over the outlet of Black Lake, near Holland, Michigan. The exact location of the span is shown on the accompanying map. The present width of the channel between docks is 203 feet, which clearance must be maintained for the lift span. The length of the span decided upon is 207 feet between centers of bearings.

Approach spans must be provided at each end. At the north end they must be high enough to provide passage for railroad trains underneath. For this reason the clearance of the lift span above mean low water is decided upon as 25 feet span down, and 155 feet span up.

This design will cover the lift span and towers only, with an indication of the method of operation.

The reasons for choosing this type of bridge instead of any other movable structure are contained in the following quotations:-

Mr. Waddell in his "De Pontibus" says:-

The advantages of lift-bridges in comparison with rotating draw-bridges are as follows:-

1st. A lift-bridge gives one wide channel for vessels instead of two narrow ones afforded by a center-pivoted swing-bridge.

2nd. There are no land damages in case of a lift bridge, as the whole structure is confined to the width of the street. These land damages in the case of some swing-bridges amount to a large percentage of the total cost of structure.
3d. Vessels can lie at the docks close to a lift-bridge, which they cannot do in the case of a swing-bridge; consequently with the former, the dock-front can be made available for a greater length between streets than it can with the latter.

4th. The time of operation for a lift-bridge is about 30% less than that for a corresponding swing-bridge.

The advantages of a lift-bridge in comparison with a bascule or a jack-knife draw, both of these being supposed to be without a center pier, are as follows:-

1st. The lift-bridge can be made of any desired span, while in the case of the others the span is necessarily quite limited in length.

2nd. A lift-bridge can be paved while others cannot.

3d. The lift-bridge is very much more rigid than any structure composed of two or more partially or wholly independent parts, a feature characteristic of the jack-knife bridge or the bascule without a center pier.

4th. In a lift bridge the operating machinery is much more simple; and in case that it ever should get out of order, the span can be raised or lowered either by unbalancing, or by simple hand mechanism, or by both combined.

Mr. W. L. Smith in a paper presented before the Western Society of Engineers - "The Design and Erection of the Pennsylvania Lift Bridge No. 458 over the South Branch of the Chicago River"- says:-

Among other considerations which influenced the selection of this type were:-

Its estimated cost was less than that of the three other types of movable bridges under consideration.

As the operating machinery is very simple and direct in its action, and as the ropes should not, with proper care, require renewal for a long time, the cost of maintenance would seem very low, indeed much lower than that of many other types of movable bridges, some of which have, in the past, shown and developed weaknesses and de-
terioration at certain particular points.

All stresses in the lift span and towers are fully determinate.

As rigid under traffic as a fixed span.

The adaptibility of this type of bridge to a skew crossing, such as the one under consideration, seems to require no more than passing mention.
GENERAL DESCRIPTION AND DIMENSIONS

1. Span:
Riveted truss of the Pratt type, with inclined top chords. 9 panels @ 23 feet = 207 feet.

2. Width:
Roadway - - - - - - - - - - - - 18' - 00"
Wheelguards - - - - - - - - - - - - 1' - 00"
End Posts - - - - - - - - - - - - 1' - 10 3/4"
Total C. to C. of Trusses - - - - - 19' - 10 3/4"
Total between Wheel Guards - - - - 17' - 00"

3. Height:
The economical height is between one-fifth and one-sixth of span, or between 41.4 and 34.7 feet. The height of the middle panel will be 35 feet, sloping down to 26 feet for the last panel.

4. Wheelguards:
6" x 6" timber

5. Plank Floor:
4" thick with an additional wearing surface of 1 1/2" oak.

6. Street Car Track:
Placed in center of bridge.

7. No Sidewalk

8. Specifications:
Office of Public Roads Circular #100, and Supplements.

9. Loading for Trusses:
Live Load: 35 ton street car, otherwise Class A as given in the specifications.

Dead Load: Floor including steel to be figured.

Trusses and laterals as obtained from Tyrrell's formula.
10. Towers:

Framed steel towers, one at each end of the bridge, 20 feet long by 20 feet wide and 173 feet high, carrying at their top the sheaves over which pass the steel cables sustaining the span, and the counterweights.
SPECIFICATIONS

The specifications followed in this design are:

SPECIFICATIONS FOR THE FABRICATION AND ERECTION OF STEEL HIGHWAY BRIDGES, prepared by the Office of Public Roads, Circular #100.

SUPPLEMENTS

Live Load:

On each track there shall be a concentrated load of 35 tons evenly divided between 4 axles, placed in pairs with 4' - 9" centers, and 20' centers between pairs, with wheels spaced about 5 feet center to center. This load shall be assumed to occupy a width of 6' on each side of the center line, and a length of 49'. (Bridge Department of the City of Chicago)

Bending Stresses:

Bending stresses on extreme fibers of rolled shapes, built sections and girders; net section 16000#

Unit Stresses in Vertical End Post:

The dead load unit stress in the vertical end post cannot exceed 3/4 of the allowed dead load unit stress as stated for the truss members.

Longitudinal Forces:

Provision shall be made for longitudinal forces by connecting the stringers to the lower lateral bracing.
Load for Towers:

Dead Load: from truss span, etc., and own weight.

Impact: 25% of weight of span.

Wind Load: The bracing and columns in the towers shall be proportioned to resist the following lateral forces, in addition to the strain from dead and live load: the lateral pressure specified above for the trusses and a lateral pressure of 100# for each vertical foot of the towers.

Combined Stresses:

For stresses produced by longitudinal and lateral or wind forces, combined with those from the live and dead load, the unit stress may be increased 25% over those given above, but the section shall not be less than required for live and dead load.

Splice in Columns:

Where the upper columns are spliced to the lower ones, the section of the first shall be spread so that the cover and web plates will be flush on the outside.

Unit Stresses in Cables:

Tension: 25000 pounds per square inch
Bending: 25000 " " "

The combined stresses from tension and bending shall not exceed 50000 pounds per square inch.

Unit Stresses in Timber:

The maximum unit stress allowed upon the extreme fiber of the floor plank shall be 1200 pounds per square inch for yellow pine and white oak; and 1000 pounds per
square inch for white pine and spruce.

Counterweights:

At each end of the bridge shall be a counterweight consisting of structural steel frame covered with concrete of the following proportions: 1 part Portland cement, 2 parts sand and 4 parts slag.

Machinery House:

The machinery house shall be situated on top of the lift span at its center. The floor shall be 4 1/2" plank, supported by steel beams. The walls shall be of 1 1/2" cinder concrete ("float finish" outside), on metal lath supported by a steel frame. The roof shall be of tin construction laid on steel trusses and purlins.

Operator's House:

The operator's house shall be made of the same material as the machinery house and be suspended therefrom. A ladder shall connect it with the machinery house and with the floor deck of the span.

Walkways and Ladders:

A walkway shall extend midway between the top chords the entire length of the truss span, except where it is obstructed by the machinery house. From the ends of this walkway, ladders shall lead to the top and bottom of each tower. Walkways shall also be provided around the edges of each tower at its top.
Rollers and Guides:

The truss span and the counterweights shall be provided with rollers to steady them while in motion. The guides shall be affixed to the towers.

Clearance:

The clearance of the lift span above mean low water shall be 25 feet span down and 155 feet span up.

Operation:

The span and its counterweights shall be suspended by steel cables connected to the top chord at each end of each truss, passing over a pair of sheaves of cast steel and attached by means of equalizers to the counterweights. The weight of the cables shall be counter-balanced by that of wrought iron chains, one end of each chain being attached to the span and the other to the top of the towers, so that whatever the elevation of the span, there will always be the same combined weight of sustaining cables and chain on one side of the sheaves as there is of the cables on the other. These balancing chains shall be caught in buckets placed on top of the span.

The span shall be moved by steel cables acting at each end of each truss.

The span shall be operated by electric energy. For emergency service a gasoline engine shall be installed.

The operator's house shall contain all operating levers and switches and a mechanical indicator showing the position of the span.
Provision shall be made for cutting off the power when the operating cables break at either end of the span. Limit switches shall be installed to cut off the power when the span has reached its limiting positions. There shall be solenoid brakes for automatic action and hand brakes as an additional safe-guard. Locking devices shall be provided so the span will always seat itself in exactly the same position. The maximum velocity of the span shall be 4 feet per second. Red lanterns on each side of the span and a ball signal on top of the machinery house shall be provided to indicate "clear" only when the span has been raised to its extreme height.
DISCUSSION

Floor System:

The floor planking is to be 4" thick with an additional wearing surface of 1 1/2" oak. The first is to be laid diagonally and the second crosswise. The latter shall not take any stress.

Assumptions were made as to the width of the span between centers of trusses. The design resulted in a width which is a little less than assumed; but the difference is so small that a re-calculation was unnecessary.

Lift Span:

Here, too, the same assumptions as to the width of the span were made and a re-calculation was not made, for the same reason.

The dead load for the trusses consists of the floor system as figured from the design and the weight of the steel in the trusses, as per Tyrrell's formula: \( W = 250 + 1.5 \, L \) - where \( W \) is the weight of steel per l. foot \( L \) " " length of the span in feet.

The difference between the assumed weight of the trusses and the actual weight as figured from the design is small and investigation showed that the areas of the sections are ample to take care of the additional stress.

The weight of the machinery house, the operator's house, and the steel supporting these was given by Messrs. Waddell & Harrington as 60000#. Hence the dead weight of the stringers and floor beams has not been taken
into account in the design.

Shoes:

The shoes at the expansion end of the span are provided with one roller. The lower castings are flanged out at their top to allow the shoes to settle every time in the same place. (See plate 4)

Sustaining Cables:

The sustaining cables are designed to resist the combined stresses due to the mass moved, the own weight of the cables, and the bending of the cables over the sheaves. There will be 32 cables @ 7/8" diameter, each consisting of 88 wires @ .065"diameter, 8 cables for each end of each truss. The cables will be connected to the top chord by means of pins. Equalizers are used for the connection to the counterweights. Rocker arms will be used as equalizers at the connection to the counterweights.

Sheaves:

No attempt has been made to give final detail dimensions for either the sheaves or the bearings and the least dimensions only are quoted in the design. The diameter of the sheaves must be at least 10000 times the diameter of one wire in the cables to avoid an excessive stress due to the bending of the cables over the sheaves.

The bearings are designed to bear vertically as well as horizontally. (See plates 6 and 7).
To take the pressure off the sheaves in the horizontal direction, a girder acting as a column has been placed between the bearings. This girder is supported by stringers which rest on the floor beams which take the vertical pressure of the sheaves. The floor beams in turn are fastened to the top of the tower columns. Rankine's formula has been used for the design of the girder.

Tower:

For the design of the tower an assumption was made as to the width between centers of columns, based on the width of the roadway for the span. The final width was a little more than assumed but not enough to warrant re-calculation.

The bracing in the towers has been designed to resist wind pressure only. The intermediate horizontal bracing is a member practically without any stress. For the sake of uniformity and appearance it has been given the same section as the main horizontal bracing.

4 angles have been adopted for the bracing, notwithstanding the fact that the area of the section considerably exceeds the required area for some of the members. The reason for adopting 4 angles is again that of uniformity and appearance. The idea is that the 4 angles shall be connected by lattice bracing and spaced far enough apart to cover practically the whole width of the columns.

The tower columns are designed for a length extending over two panels. The lower columns take a bending
stress as well as a direct wind stress, while the upper columns take direct wind stress only. The design of the columns was made according to Rankine's formula.

The transverse frames in the towers were not figured and nominal members were used in their design as the columns were figured to take all of the bending and direct stress in the panel. The latticing and bracing will help to take care of any bending which occurs in the upper horizontal member.

The space between the floor beams on top of the towers is filled with latticing built of nominal members.

The floor system for the towers was not figured, on the assumption that it differed so little from the floor system of the lift span that the same design could be used.

There was no need to calculate the anchorage to resist the over-turning moment of the towers, as the column footings are fastened to piers the mass of which is more than sufficient, without investigation.

Counterweights:

The counterweights are designed as consisting of a steel frame covered with concrete. The frame has only to be self-supporting before being enclosed in concrete. The design provides for the rollers to be fastened to this frame by means of castings. No attempt has been made to figure the size of these castings.

The sustaining rods go within 1' - 6" of the bottom of the counterweights and are connected to the
steel frames.

As it is difficult to calculate the exact weight of the moving mass, cables and rollers, a pocket has been provided in the upper part of the counterweights so that if necessary they can be loaded.

The design calls for a height of 16' - 6", but the exact height can only be determined after the weight of the different castings is known. (See plate 10)

Guides and Locking Devices:

To steady the lift span and the counterweights while in motion, rollers and guides are provided. At the fixed end of the lift span the rollers steady the span longitudinally and at the expansion end, transversely. The counterweights each have longitudinal rollers at their bottom and transverse rollers at their top.

The guides for the lift span are on the face of the inside columns of the towers, while the guides for the counterweights are on the face and side of the outside columns of the towers.

The locking devices are affixed to the lift span and towers underneath the rollers and guides. The lock at the expansion end allows a longitudinal movement, while the lock at the fixed end is designed to hold the span rigidly. (See plates 10 and 11)

Operation:

The maximum velocity of the span shall be 4 feet per second, but the amount of power required was in-
vestigated also for a maximum velocity of 3 feet per second with no wind acting, as well as for a maximum velocity of 2 feet per second with the greatest assumed wind pressure. The horsepower required for the first case is 36.5; for the second, 23.6 and for the third, 20.3. From this the deduction was made that a 25 horsepower motor will be sufficient for the operation of the bridge.

The operating cables were designed to resist the combined stresses due to the power transmitted, to tension from their own weight, the dimension of the curve in which they hang, and the bending over the sheaves and drums. The stress due to centrifugal force was not considered as the velocity is so low that it need not be considered.

4 cables are to be used, each of 1/2" diameter, consisting of 50 wires of .042" diameter each. One cable will act at each end of each truss.

Plate 12 shows the method of operation of the bridge.
CONCLUSION

Lift bridges on a small scale have been used for many years for crossings of canals, lifting only high enough to let the canal boats pass beneath. They have proved to be quite satisfactory and fairly economical in first cost and operation, the method of the latter being usually man-power. (Waddell: "De Pontibus")

The first direct lift bridge of any importance in this country was the Halsted Street Bridge over the South Branch of the Chicago River at Chicago, built in 1893. Since then, several have been built, for highway, electric car and railroad traffic. This type of movable bridge is coming into more frequent use, especially for railroads as it is as rigid under traffic as a fixed span.

Messrs. Waddell & Harrington, Consulting Engineers, Kansas City, Mo., hold the patents for the construction of this type of movable bridge in this country.
PART II

DESIGN

of

FLOOR SYSTEM
_FLOOR PLANKING_

The floor planking is to be 4" thick with an additional wearing surface of 1 1/2" oak. The first is to be laid diagonally and the second crosswise. The plank acts as a partially continuous beam. Concentrated load of 15 tons on 2 axles, 8'-0" centers. The narrowest plank must be 10" wide and the upper plank does not take any load.

\[ M = \frac{WL}{8} \]

Where \( M \) = the bending moment
\[ W = \text{the load} \]
\[ L = \text{the unsupported length} \]

Res. \( M = S \frac{I}{c} \)

Where Res. \( M \) = the resisting moment
\[ S = \text{the allowed stress per square inch} \]
\[ I = \text{the moment of inertia of the section} \]
\[ c = \text{the distance of the outermost fiber to the neutral axis of the section} \]

\[ M = \frac{WL}{8} = 7500 \times \frac{L}{8} \]

Res. \( M = S \frac{I}{c} = 1200 \times \frac{bd^3}{12c} = 1200 \times 10 \times 16/6 \]

BM = Res. \( M \)

\[ 7500 \frac{L}{8} = 1200 \times 10 \times 16/6 \]

\[ L = 34" \]

The widest spacing of stringers will be 2'-8"
DISTRIBUTION OF LIVE LOAD FOR FLOOR SYSTEM

15 Ton Wagon

35 Ton Street Car
LIVE LOAD MOMENTS for 23'-0" STRINGERS

35 Ton Street Car

23 R₁ = 8.75 (15.0625 + 10.3125); R₁ = 9.65 kips
M = 9.65 x 10.3125 = 99.5 kip feet
I = 99.5 x 300/(300 + 4.75) = 97.8 kip feet

15 Ton Wagon

23 R₁ = 7.5 x 8.83; R₁ = 6.62 kips
M = 6.62 x 8.83 = 58.4 kip feet
I = 58.4 x 300/(300 + 8) = 57.0 kip feet
_LIVE LOAD END-SHEAR for 23'-0" STRINGERS_

\[ 23 R_f = 8.75 (23 + 18.25); R_f = 15.7 \text{ kips} \]
\[ I = 15.7 \times 300 \div (300 + 4.75) = 15.5 \text{ kips} \]

15 Ton Wagon

\[ 23 R_f = 10 \times 23 + 5 \times 15; R_f = 13.25 \text{ kips} \]
\[ I = 13.25 \times 300 \div (300 + 8) = 12.9 \text{ kips} \]
DESIGN of ROADWAY STRINGERS

2'-0" width of road carried per stringer

Floor load per lineal foot = (4 + 1.5) x 4 x 2.0 = 44#

M = WL²/8 = .044 x 23²/8 = 3.2 kip feet

Preliminary

Moments:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live Load - 15 ton wagon</td>
<td>58.4 kip feet</td>
</tr>
<tr>
<td>Floor Dead Load</td>
<td>3.2 kip feet</td>
</tr>
</tbody>
</table>

116.6 kip feet or 1410 kip inches

Rolled I Beams to be tried

M = QS; Q = M/S

where M = Bending Moment
Q = Section Modulus
S = Allowed stress per sq. in. = 16 kips per sq. in.

Q = M/S = 1410/16 = 88.0

18" I @ 60#; Q = 93.5

Weight of stringer = 60 x 23 = 1380#

Connections 5% = 70#

Total weight of stringers = 1450#

M = WL²/8 = 1.45 x 23²/8 = 4.14 kip feet or 49.7 kip inches

Q = 49.7/16 = 3.1

Final

Total Q = 88.0 + 3.1 = 91.7 required which is less than the section modulus for an 18" I @ 60# as given in the steel handbooks; therefore this section is satisfactory and will be used for all roadway stringers, except the end stringers. The latter are usually channels. A
smaller section modulus is used as the loads will not enter upon the channels on account of the wheelguards. An 18" $L @ 50\#$ with a section modulus of 50.0 will be used.
DESIGN of TROLLEY STRINGERS

2'-6" of roadway carried by stringer

Floor load per lineal foot = \((4 + 1.5) \times 4 \times 2.5 = 55\)#
Assume track per lineal foot as \(50\)#

Total floor load = 105#

\[ M = \frac{WL^2}{8} = .105 \times 23^2/8 = 6.95 \text{ kip feet} \]

Preliminary

Moments:

- Live Load - 35 ton car = 99.5 kip feet
- \(I = 97.8" "\)
- Floor Dead Load = 6.95 " "

\[
\begin{align*}
\text{Total load} &= 203.8 \text{ kip feet} \\
&\text{or} \quad 2440 \text{ kip inches}
\end{align*}
\]

Rolled I beams to be tried

\[ M = QS; \quad Q = \frac{M}{S} = 2440/16 = 153 \]

20" I @ 95#; \(Q = 160.7\)

Weight of stringer = 95 x 23 = 2185#

Connections 5% \(= 110\)#

Total weight of stringer = 2295#

\[ M = \frac{WL}{8} = 2.295 \times 23/8 = 6.6 \text{ kip feet or} \quad 79.3 \text{ kip inches} \]

\[ Q = 77.6/16 = 5.0 \]

Final

Total \(Q = 153 + 5.0 = 158.0\) required which is less than the section modulus for a 20" I @ 95# as given in the steel handbooks; therefore this section is satisfactory and will be used for all trolley stringers.
MAXIMUM FLOOR BEAM CONCENTRATION

35 Ton Street Car
Case 1.

35 Ton Street Car
Case 2.

Case 1: 23 R = 2(17.5 x 13); = R = 19.8 kips
Case 2: 23 R = 8.75(23 + 7.75 + 5.375 + 1825); R = 20.3 kips
I = 20.3 x 300/(300 + 24.75) = 19 kips

15 Ton Wagon

23 R = 10 x 23.0 + 5 x 19.6; R = 10.75 kips
I = 10.75 x 300/(300 + 8); = 10.5 kips
**MAXIMUM FLOOR BEAM LIVE and DEAD LOAD CONCENTRATION**

---

**Live Load**

\[ 23 \times R = 2(80 \times 4.25 \times 23 \times 11.5); \quad R = 7.82 \text{kips} \]

**Dead Load**

- **Trolley Stringer**
  
  \[ 20.3 + 19.0 + 0.105 \times 23 + 2.77 = 44.25 \text{kips} \]

- **Roadway Stringer**
  
  \[ 0.047 \times 23 + 1.44 = 1.55 \text{kips} \]
For bridge fully loaded as per above, the end shear equals the sum of the loads on one-half of the floor beam.

3 1/2 roadway stringers @ 1.55 kips each = 5.43 kips
1 Trolley stringer = 43.75 "
Uniform load of 80# per sq. ft. for area outside of car = 7.82 "

Total Shear = 57.00 kips

Bending moment is maximum at the center for this maximum loading.

\[ M = 57.00 \times 10.3 - 1.55 \times (8.5 + 6.5 + 4.5) - 7.82 \times 8.125 - 43.75 \times 2.5 \]

\[ M = 387.0 \text{ kip ft. or 4650 kip in.} \]
DESIGN of FLOOR_BEAM

Shear = 57.00 kips
Q = M/S = 4650/16 = 290

Preliminary

A riveted plate girder will be tried built up of
1 web plate 26" x 5/16" and 4/3 - 6" x 6" x 5/8"
Q = 326.7; Max. end Reac. = 78.8 kips

Weight of floor beam = 124.4 x 20.5 = 2550#
10% for connections = 260
Total weight of floor beam = 2810#

M = WL/8 = 2.81 x 20.5/8 = 7.22 kip ft. or 86.5 kip in.
Q = 86.5/16 = 5.4

Final

Total Q required = 290 + 5.4 or 295.4
Total end shear = 57.00 + 2.81 kips or 59.81 kips
Both values required are less than given in the steel
handbooks for a riveted plate girder built up of
1 web plate 26" x 5/16" and 4/3 - 6" x 6" x 5/8". Hence
this section is satisfactory and will be used for all
floor beams.
 Rivet Spacing in Vertical Legs of Floor Beam Flanges

Preliminary

\[ p = \frac{r e}{S} \]

where \( p \) is the pitch in inches

\( r \) " the permissible stress in one rivet in pounds

\( e \) " the effective depth in inches

\( S \) " the maximum shear in pounds at the section under consideration.

\[ r \text{ for a 7/8" rivet} = 5470 \]  
\[ e = 22.54" \]

\( S \) at end of beam = 57.00 kips

\[ p = 5470 \times \frac{22.54}{57000} = 2.16" \]

\( S \) at 3'-10" from end of beam

\[ 57.00 - 1.55 - 7.82 - 1.55 = 46.08 \text{ kips} \]

\[ p = 5470 \times \frac{22.54}{46080} = 2.67" \]

\( S \) at 5'-10" from end of beam

\[ 57.00 - 1.55 - 7.82 - 1.55 - 1.55 = 44.53 \text{ kips} \]

\[ p = 5470 \times \frac{22.54}{44530} = 2.77" \]

\( S \) at 7'-10" from end of beam

\[ 57.00 - 1.55 - 7.82 - 1.55 - 1.55 - 1.55 = 43.75 = .78 \text{ kips} \]

\[ p = 5470 \times \frac{22.54}{780} = 15.7" \]

The final spacing may now be taken from the diagram.
PART III

STRESSES IN LIFT SPAN
**LOADING for TRUSSES**

**Live Load**

A uniform load of 1750# per 1. ft. for a 35 ton car is proportional to 1200# for a 24 ton car. This is used for a width of 12 feet, 80# per sq. ft. for the remaining surface.

**Load per panel per truss**

\[
\frac{23}{2} (1750 + (20.6 - 12) 80) = 25000# 
\]

**Dead Load**

<table>
<thead>
<tr>
<th>Item</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
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<tr>
<td>Planking per 1. foot of bridge</td>
<td>18 x 5.5 x 4</td>
<td>396#</td>
</tr>
<tr>
<td>Wheelguards</td>
<td>2 x (6/12) x 6 x 4</td>
<td>24</td>
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<td>Track</td>
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</tr>
<tr>
<td>Roadway stringers 2-18&quot;</td>
<td>L @ 50#</td>
<td>100</td>
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<tr>
<td>connections 5%</td>
<td></td>
<td>5</td>
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<tr>
<td>stringers 5-12&quot;</td>
<td>I @ 60#</td>
<td>300</td>
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<tr>
<td>connections 5%</td>
<td></td>
<td>15</td>
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<tr>
<td>Trolley stringers 2-20&quot;</td>
<td>I @ 95#</td>
<td>190</td>
</tr>
<tr>
<td>connections 5%</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>Floor beam 1 Pl. - 26&quot; x 5/16&quot;</td>
<td>4 x 6&quot; x 6&quot; x 1/8&quot; and conn.=2810/23</td>
<td>122</td>
</tr>
<tr>
<td>Steel in bridge</td>
<td>Tyrrell's formula: (2.50 + 1.5 x 207)</td>
<td>561</td>
</tr>
<tr>
<td>Total dead load per 1. ft. panel</td>
<td></td>
<td>1773#</td>
</tr>
</tbody>
</table>

Dead load per panel per truss = 1773 x 23/2 = 20200#

21000# per panel per truss will be used.

There will be in addition a load of 60000# on the top chord at the middle of the span to cover the weight of machinery, machinery house, motors and the beam supports. (Waddell & Harrington)
\[ \phi = \tan^{-1} \frac{9}{69} = 7^\circ 26' \]
\[ \tan \phi = 0.13047; \sec \phi = 1.00848; \csc \phi = 7.72962; \]
\[ \sin \phi = 0.12937; \cos \phi = 0.99160; \]

\[ \phi_1 = \tan^{-1} \frac{23}{35} = 33^\circ 19' \]
\[ \tan \phi_1 = 0.65729; \sec \phi_1 = 1.19668; \csc \phi_1 = 1.62061 \]
\[ \sin \phi_1 = 0.54927; \cos \phi_1 = 0.83565 \]

\[ \phi_2 = \tan^{-1} \frac{23}{32} = 35^\circ 42' \]
\[ \tan \phi_2 = 0.71857; \sec \phi_2 = 1.23140; \csc \phi_2 = 1.71368 \]
\[ \sin \phi_2 = 0.58354; \cos \phi_2 = 0.81208 \]

\[ \phi_3 = \tan^{-1} \frac{25}{29} = 38^\circ 25' \]
\[ \tan \phi_3 = 0.79306; \sec \phi_3 = 1.27630; \csc \phi_3 = 1.60933 \]
\[ \sin \phi_3 = 0.62138; \cos \phi_3 = 0.78351 \]

\[ \phi_4 = \tan^{-1} \frac{23}{26} = 40^\circ 30' \]
\[ \tan \phi_4 = 0.88473; \sec \phi_4 = 1.31509; \csc \phi_4 = 1.53977 \]
\[ \sin \phi_4 = 0.64945; \cos \phi_4 = 0.76041 \]
DEAD LOAD STRESSES

Dead Load Reaction

The Dead Load Reaction is \( \frac{9 \times 21000 + 30000}{2} = 109500\# \)

Chord Stresses

\[
109500 \times 4 \times 23 - \frac{(10500 \times 4 + 21000 (3 + 2 + 1))23}{35}
\]

\( ef = + 177800\# \)

\[
109500 \times 3 \times 23 - \frac{(10500 \times 3 + 21000 (2 + 1))23}{32}
\]

d, \( f \g = + 169700\# \)

\[
109500 \times 2 \times 23 - \frac{(10500 \times 2 + 21000 \times 1)23}{29}
\]

c, \( d \h = + 140500\# \)

\[
109500 \times 1 \times 23 - \frac{10500 \times 1 \times 23}{26}
\]

\( a, b, c, h, i, j = + 87500\# \)

\[
109500 \times 5 \times 23 - \frac{(10500 \times 5 + 21000(4 + 3 + 2 + 1) + 15000 \times 1)23}{35}
\]

\( E F = -177600\# \)

\[
109500 \times 4 \times 23 - \frac{(10500 \times 4 + 21000 x(3 + 2 + 1))23}{35 \cos \phi}
\]

\( DE, F G = -179300\# \)
\[
\frac{109500 \times 3 \times 23 - (10500 \times 3 + 21000 \times (2 + 1))}{32 \cos \phi}
\]

\[
CD, \ G H = -171300
\]

\[
\frac{109500 \times 2 \times 23 - (10500 \times 2 + 21000 \times 1)}{29 \cos \phi}
\]

\[
BC, \ H I = -142000
\]

\[
AB, \ I J = 0
\]

\[
aB = Ij = 109500 \sec \phi = -144000
\]

**Web Stresses**

Intersection of Chords

\[
\tan \phi = \frac{9}{69} = \frac{32}{x}; \ x = 245.33
\]

\[
\frac{(-109500 + 10500 + 21000 \times 3)153.33 + 21000(23 + 46 + 69 + 92)}{245.33}
\]

\[
Ee, \ Ff = -10200
\]

\[
\frac{(-109500 + 10500 + 21000 \times 3)153.33 + 21000(23 + 46 + 69)}{222.33}
\]

\[
Dd, \ Gg = -11800
\]

\[
\frac{(-109500 + 10500 + 21000 \times 2)153.33 + 21000(23 + 46)}{199.33}
\]

\[
Cc, \ Hh = -36600
\]

\[
Bb, \ Ii = +21000
\]

\[
Ef, \ Fe = 0
\]
\[
\frac{(-109500 + 10500 + 21000 \times 3)153.33 + 21000(23 + 46 + 69)}{(153.33 + 92) \cos \phi}
\]

\[\text{De, Gf} = +14600\#
\]

\[
\frac{(-109500 + 10500 + 21000 \times 2)153.33 + 21000 (23 + 46)}{(153.33 + 69) \cos \phi}
\]

\[\text{Cd, Hg} = -45300\#
\]

\[
\frac{(-109500 + 10500 + 21000)153.33 + 21000 \times 23}{(153.33 + 46) \cos \phi}
\]

\[\text{Bc, Ih} = +98500\#\]
**LIVE LOAD STRESSES**

**Live Load Reaction**

The Live Load Reaction is \(9 \times 25000/2 = 112500\)#

**Chord Stresses**

\[
\begin{align*}
\text{ef} & = +165000# \\
\text{de, fg} & = +162000# \\
\text{cd, gh} & = +139000# \\
\text{ab, bc, hi, ij} & = +85600# \\
\text{EF} & = -165000 \\
\text{DE, FG} & = -166500#
\end{align*}
\]
(112500 x 3 - (12500 x 3 + 25000 (2 + 1))) \frac{23}{32 \cos \phi}

CD, GH = -163500#

(112500 x 2 - (12500 x 2 + 25000 x 1)) \frac{23}{29 \cos \phi}

BC, HI = -140500#

AB, IJ = 0

aB, Ij = 112500 \sec \phi = -165000#

**Web Stresses**

Shear in Panel EF = 10/9 x 25000 = 27800#

Ef = 27800 x \sec \phi = -33200#

Ee, Ff = \frac{10/9 x 25000 x 153.33}{153.33 + 92} = -17400#

De, Gf = \frac{15/9 x 25000 x 153.33}{(153.33 + 92) \cos \phi_2} = +32200#

Dd, Gg = \frac{15/9 x 25000 x 153.33}{153.33 + 69} = -28500#

Cd, Hg = \frac{21/9 x 25000 x 153.33}{(153.33 + 69) \cos \phi_3} = +50700#

Cc, Hh = \frac{21/9 x 25000 x 153.33}{153.33 + 46} = -45000#

Bc, Ii = \frac{28/9 x 25000 x 153.33}{(153.33 + 46) \cos \phi_4} = +79000#

Bb, Ii = +25000#
Counters

Live Load shear in panel \( E_f = \frac{10}{9} \times 25000 = +26600 \# \)

Dead Load shear \( " " " - - - - - = -18000 \# \)

\( E_f = (+26600 - 18000) \sec \phi _1 = +10300 \# \) counter stress

\[
\frac{(109500 \times 5 + (6/9) \times 25000 \times 5) - (10500 \times 5 + 21000 \times 10 + 15000)}{35}
\]

\( f_g = +233000 \# \)

\[
\frac{(109500 \times 6 + (6/9) \times 25000 \times 6) - (10500 \times 6 + 21000 \times 15 + 15000 \times 3)}{32 \sin \phi}
\]

\( F_g = +8950 \# \) counter stress

\[
\frac{(109500 \times 6 + (3/9) \times 25000 \times 6) - (10500 \times 6 + 21000 \times 15 + 15000 \times 3)}{32}
\]

\( G_h = +204000 \# \)

\[
\frac{(109500 \times 7 + (3/9) \times 25000 \times 2) - (10500 \times 7 + 21000 \times 21 + 15000 \times 6)}{29 \sin \phi_2}
\]

\( G_h = -4600 \# \) No counter required.
\[
\begin{align*}
\text{ef} & = \frac{100 \times 165000}{207 + 300} = 32600# \\
de, fg & = \frac{100 \times 162000}{207 + 300} = 32000# \\
\text{cd, gh} & = \frac{100 \times 139000}{207 + 300} = 27250# \\
\text{ab, bc, hi, ij} & = \frac{100 \times 85600}{207 + 300} = 16900# \\
\text{EF} & = \frac{100 \times 165000}{207 + 300} = 32600# \\
\text{DE, FG} & = \frac{100 \times 166500}{207 + 300} = 32900# \\
\text{CD, GH} & = \frac{100 \times 163500}{207 + 300} = 32300# \\
\text{BC, HI} & = \frac{100 \times 140500}{207 + 300} = 27800# \\
\text{aB, Ij} & = \frac{100 \times 165000}{207 + 300} = 32600# \\
\text{Ef} & = \frac{100 \times 33200}{92 + 300} = 8500# \\
\text{Ee, Ff} & = \frac{100 \times 17400}{92 + 300} = 4450# \\
\text{De, Gf} & = \frac{100 \times 32200}{115 + 300} = 7760# \\
\text{Dd, Gg} & = \frac{100 \times 28500}{115 + 300} = 6875# \\
\text{Cd, Hg} & = \frac{100 \times 50700}{138 + 300} = 11500# \\
\text{Cc, Hh} & = \frac{100 \times 45000}{138 + 300} = 10250# \\
\text{Bc, Ih} & = \frac{100 \times 79000}{161 + 300} = 17150# \\
\text{Bb, Ii} & = \frac{100 \times 25000}{23 + 300} = 7750# 
\end{align*}
\]
\[
Ef \ (\text{counter}) \ -\ - = \frac{100 \times 10300}{92 + 300} \ -\ - = 2630\#
\]

\[
Fg \ (\text{counter}) \ -\ - = \frac{100 \times 8950}{69 + 300} \ -\ - = 2430\#
\]

When the bridge is lifted aA and jJ carry the total Dead Weight; hence the stress in each is \( +109500\# \).
TOP LATERALS

9 Panels @ 23'-0" = 207'-0"

BOTTOM LATERALS

9 Panels @ 23'-0" = 207'-0"

Plate 2.
WIND STRESSES

Upper Laterals
150# per 1. foot or 3450# per panel

$\sec \phi = 1.33968$

$AB' = 3450 \times 4 \times 1.33968 = 18500#$
$BC' = 3450 \times 3 \times 1.33968 = 13850#$
$CD' = 3450 \times 2 \times 1.33968 = 9250#$
$DE' = 3450 \times 1 \times 1.33968 = 4620#$

Reaction = $4 \frac{1}{2} \times 3450 = 15525#$

\[
\frac{(15525 \times 5 - (1725 \times 5 + 3450 \times (4 + 3 + 2 + 1)))}{20.5}
\]

$DG = 39700#$

\[
\frac{(15525 \times 3 - (1725 \times 3 + 3450 \times (2 + 1)))}{20.5}
\]

$CD, GH = 35600#$

\[
\frac{(15525 \times 2 - (1725 \times 2 + 3450 \times 1))}{20.5}
\]

$BC, HI = 27600#$

$EE' = 3450#$

$DD' = 6900#$

$CC' = 10350#$

$AB, IJ = 15800#$

Portal load = $4 \frac{1}{2} \times 3450 = 15525#$
Lower Laterals

150# Dead Load per 1. ft. or 3450# per panel
150# Live Load " " " 3450# " "

sec $\phi = 1.33968$

Dead Load Stresses

\[ \begin{align*}
    ab', j1' &= 18500# \\
    bc', ih' &= 13850# \\
    cd', hg' &= 9250# \\
    de' &= 4620# \\
    dg &= 39700# \\
    cd, gh &= 35600# \\
    bc, hi &= 27600# \\
    ab, ij &= 15800#
\end{align*} \]

Live Load Stresses

\[ \begin{align*}
    dg &= 39700# \\
    cd, gh &= 35600# \\
    dc, hi &= 27600# \\
    ab &= 15800# \\
    ab' &= (36/9)x 3450 x 1.33968 = 18500# \\
    bc' &= (28/9)x 3450 x 1.33968 = 14400# \\
    cd' &= (21/9)x 3450 x 1.33968 = 10800# \\
    de' &= (15/9)x 3450 x 1.33968 = 7450# \\
    ef' &= (10/9)x 3450 x 1.33968 = 5150#
\end{align*} \]
### TABLE OF STRESSES IN TRUSSES

<table>
<thead>
<tr>
<th>Member</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Impact</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Wind Load</th>
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</thead>
<tbody>
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</tr>
</tbody>
</table>

### Additional Members

- AB': +18500
- BC': +13850
- CD': +9250
- DE': +4620
- ab': +18500 +18500
- bc': +13850 +14400
- cd': +9250 +10800
- de': +4620 +7450
- ef': +5150
- EE': -3450
- DD': -6900
- CC': -10350
The posts are fixed at the shoe and the bending moment at the shoe in the posts is \( t \left( \frac{H}{2} \right) \left( \frac{L-a}{2} \right) \).

Where \( H \) is the total wind load transmitted to the abutment by one portal

\( L \) is the length of post

\( a \) " " depth of portal

The direct wind stress in the post is

\[ t \frac{H}{2} \left( \frac{L-(L-a)}{2} \right) / b \]

where \( b \) is the distance c. to c. of posts.

\[ M = t \left( \frac{15525}{2} \right) (34.83 - 12) / 2 = t \ 89000 \# \]

\[ V = t \left( 15525(34.83-11.42) / 20.5 \right) = t \ 17750 \# \]

The vertical component of the stresses in the diagonals is 8875\# on the assumption that each diagonal in the middle panel takes half the shear.

\[ \phi = \tan^{-1} \frac{6}{6.33} = 51.41^\circ; \tan \phi = 1.26545 \]

\[ \sec \phi = 1.61288; \csc \phi = 1.27454 \]

\[ \phi_i = \tan^{-1} \frac{6.33}{4} = 57.43^\circ; \tan \phi_i = 1.58286 \]

\[ \sec \phi_i = 1.87229; \csc \phi_i = 1.18285 \]

8875 \sec \phi = 14300\#

\[ JL = KM = +14300\# \]

\[ JK = LM = -14300\# \]

\[ HJ = GI = +14300\# \]

\[ MN = PO = -14300\# \]

Assuming that IF takes the horizontal component of GI;

\[ IF = 14300 \cos \phi = -8900\# \]
\[7763(34.83 - 11.42) - GI \times 8 \times \cos \phi - IF \times 8\]
\[= 12 \times \sin \phi,\]

\[FA = \pm 17950\#\]

\[=\frac{JL \sin \phi - FA \sin \phi}{\sin \phi}\]

\[JI = GH = \pm 5100\#\]

\[7763(11.81 + 4) + 15525 \times 8 - 17750 \times 6.33 + HJ \times 8 \cos \phi\]

\[EH = - 24700\#\]

\[CF = \pm 17750 - FA \cos \phi = \pm 8200\#\]

\[GC = CF - GI \sin \phi = -3000\#\]

\[LA = JI \cos \phi - JL \cos \phi + IF + FA \cos \phi = -5040\#\]

\[EK = EH + HJ \cos \phi - KJ \cos \phi = -6900\#\]

\[OQ = OF \cos \phi = \pm 8900\#\]

\[PO \times 8 \times \cos \phi + OQ \times 8 + 7763 \times (34.83 - 11.42)\]
\[= 12 \times \sin \phi,\]

\[QA = -17950\#\]

\[7763(11.81 + 4) + 15525 \times 8 - 17750 \times 6.33 + MN \times 8 \times \cos \phi\]

\[EN = \pm 24700\#\]

\[LM \sin \phi - QA \sin \phi\]
\[= \frac{\sin \phi}{\sin \phi}\]

\[MO = NP = - 5100\#\]

\[QE = -17750 + AQ \sin \phi = -8200\#\]

\[EP = QE - OP \sin \phi = \pm 3000\#\]
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<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
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PART IV
---
DESIGN
of
LIFT SPAN
Typical Section

\[ \text{abc} \]
\[ \begin{align*}
\text{DL} &= +87500# \\
\text{LL} &= +85600# \\
\text{I} &= +16900# \\
\text{Total} &= +190000#
\end{align*} \]

\[ \text{Wl} = +31600# \text{ which being less than 25\% of 190000# need not be considered} \]

\[ \begin{align*}
\text{A} &= \frac{\text{P/S}}{}; \text{ where A is the area required} \\
\text{P} &= \text{ total stress as figured} \\
\text{S} &= \text{ allowed stress per sq. in.}
\end{align*} \]

\[ \frac{\text{A} = 190000}{16000} = 11.9 \text{ sq. in.} \]

\[ \begin{align*}
4 \text{ s } 13'' = 3'' \times 5/16'' &= 7.12 - 4 \times .27 = 6.04 \text{ sq. in.} \\
2 \text{ Pl. } 14'' = 3/8'' &= 10.50 - 4 \times .33 = 9.18''
\end{align*} \]

\[ \text{Total net area} = 15.22 \text{ sq. in.} \]

\[ \text{cd} \]
\[ \begin{align*}
\text{DL} &= +140500# \\
\text{LL} &= +139000# \\
\text{I} &= +27250# \\
\text{Total} &= +306750#
\end{align*} \]

\[ \text{Wl} = +71200# \text{ which being less than 25\% of 306750# need not be considered} \]

\[ \begin{align*}
4 \text{ s } 3'' = 3'' \times 5/16'' &= 7.12 - 4 \times .27 = 6.04 \text{ sq. in.} \\
2 \text{ Fls } 14'' = 3/8'' &= 10.50 - 8 \times .33 = 7.16'' \\
2 \text{ Fls } 7 1/2'' = 5/16'' &= 4.69 - 4 \times 27 = 3.61'' \\
2 \text{ Flats } 3'' = 5/8'' &= 3.75 - 4 \times .27 = 2.67''
\end{align*} \]

\[ \text{Total net area} = 19.48 \text{ sq. in.} \]
de

\[ DL = +169700 \# \]
\[ LL = +162000 \# \]
\[ I = +32000 \# \]

Total = +363700\#

\[ A = \frac{363700}{16000} = 22.7 \text{ sq. in.} \]

\[ 4 \text{ lbs} - 3" \times 3" \times 5/16" \]
\[ 2 \text{ Pls} - 14" \times 3/8" \]
\[ 2 \text{ Pls} - 7 1/2" \times 5/16" \]
\[ 2 \text{ Pls} - 13" \times 5/16" \]
\[ 2 \text{ Flats} - 3" \times 5/8" \]

\[ = 7.12 - 4 \times 27 \]
\[ = 10.50 - 8 \times 0.33 \]
\[ = 4.69 - 4 \times 0.27 \]
\[ = 8.12 - 8 \times 0.27 \]
\[ = 3.75 - 4 \times 0.27 \]

\[ = 4.96 \text{ sq. in.} \]
\[ = 7.16 "" \]
\[ = 3.61 "" \]
\[ = 5.96 "" \]
\[ = 2.67 "" \]

Total net area = 24.36 sq. in.

ef

\[ DL = +177800 \# \]
\[ LL = +165000 \# \]
\[ I = +32600 \# \]

Total = +375000\#

\[ A = \frac{375000}{16000} = 23.5 \text{ sq. in.} \]

\[ 4 \text{ lbs} - 3" \times 3" \times 5/16" \]
\[ 2 \text{ Pls} - 14" \times 3/8" \]
\[ 2 \text{ Pls} - 7 1/2" \times 5/16" \]
\[ 2 \text{ Pls} - 13" \times 5/16" \]
\[ 2 \text{ Flats} - 3" \times 5/8" \]

\[ = 7.12 - 4 \times 0.27 \]
\[ = 10.50 - 8 \times 0.33 \]
\[ = 4.64 - 4 \times 0.27 \]
\[ = 8.12 - 8 \times 0.27 \]
\[ = 3.75 - 4 \times 0.27 \]

\[ = 4.96 \text{ sq. in.} \]
\[ = 7.16 "" \]
\[ = 3.61 "" \]
\[ = 5.96 "" \]
\[ = 2.67 "" \]

Total net area = 24.36 sq. in.
The allowable stress $S$ per sq. in. for compression is $S = 16000 - 70 \frac{L}{r}$

where $L$ is the length of the member in inches

$r""$ least radius of gyration

The approximate value for $r$ according to the A.B.C. formula will be used for the preliminary design.

$r = \left(\frac{4}{10}\right)W\text{ or }\left(\frac{4}{10}\right)14 = 5.6"$

Distance between rivet lines for web plates is

$14" - 3\ 1/2" = 11\ 1/2"; \ 1/30\ of\ 11\ 1/2" = .384"$

This is somewhat more than $3/8"$ or $.375"$ but the difference is so small that $3/8"$ web plates will be used.

Distance between rivet lines for cover plate

$.12 + 1\ 1/4 + 3\ 1/2 = 16\ 3/4"; \ 1/40\ of\ 16\ 3/4" = .42"$

hence $7/16"$ or $.4375"$ cover plate must be used.
AB

This member will be of the same section as BC

BC

Preliminary

\[
\begin{align*}
DL &= -142000# \\
LL &= -140500# \\
I &= -27800# \\
\text{Total} &= -310300#
\end{align*}
\]

\[S = 16000 - 70 \quad L/r = 16000 - 70 \times 23 \times 12/5.6\]

\[S = 16000 - 3450 = 12550#\]

\[A = 310300/12550 = 24.8 \text{ sq. in.}\]

1. Cover Pl. - 20" x 7/16" = 8.75 sq. in.
2. Pls - 14" x 3/8" = 10.50 " "
2. Ls - 3" x 3" x 5/16" = 3.56 " "
2. Ls - 5" x 3" x 5/16" = 4.80 " "

Total area = 27.61 sq. in.
Center of Gravity of Section

<table>
<thead>
<tr>
<th>Area</th>
<th>Lever Arm</th>
<th>Moment at Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.75 sq. in.</td>
<td>14.22 in.</td>
<td>124.50</td>
</tr>
<tr>
<td>10.50 &quot;</td>
<td>7.00 &quot;</td>
<td>73.50</td>
</tr>
<tr>
<td>3.56 &quot;</td>
<td>13.13 &quot;</td>
<td>46.80</td>
</tr>
<tr>
<td>4.80 &quot;</td>
<td>.68 &quot;</td>
<td>3.26</td>
</tr>
<tr>
<td>27.61 sq. in.</td>
<td></td>
<td>248.06</td>
</tr>
</tbody>
</table>

c. of gr. = 248.06/27.61 = 8.96"

Radii of Gyration

| I of Cover Pl - 20" x 7/16" = 20 \times (7/16)/12 = 0.14 |
| 8.75 x 5.26^2 - - - - - - - - - - - - - - - - - - - = 272.00 |
| I of 2 Pls - 14" x 3/8" - - - - - = 2 x 14x(3/8)/12 = 171.50 |
| 10.50 x 1.96^2 - - - - - - - - - - - - - - - - - - - = 40.40 |
| I of 2 Ls - 3" x 5" x 5/16" - - - - - - - - - - - - - - - - - = 3.00 |
| 3.56 x 4.17^2 - - - - - - - - - - - - - - - - - - - = 62.00 |
| I of 2 Ls - 5" x 3" x 5/16" - - - - - - - - - - - - - - - - - = 3.60 |
| 4.80 x 8.28^2 - - - - - - - - - - - - - - - - - - - = 329.00 |

Total I - - - - = 881.64

\( r_{1-1} = \sqrt{881.64/27.61} = 5.65" \)

| I of Cover Pl 20" x 7/16 - - - = 20^3 \times (7/16)/12 - - = 282.00 |
| I of 2 Pls - 14" x 3/8 - - - = 14x(3/8)x2/12 - = .12 |
| 10.50 x 6.19^2 - - - - - - - - - - - - - - - - - - - = 403.00 |
| I of 2 Ls - 3" x 3" x 5/16" - - - - - - - - - - - - - - - - - = 3.00 |
| 3.56 x 7.25^2 - - - - - - - - - - - - - - - - - - - = 187.20 |
| I of 2 Ls - 3" x 3" x 5/16" - - - - - - - - - - - - - - - - - = 12.60 |
| 4.80 x 8.05^2 - - - - - - - - - - - - - - - - - - - = 311.00 |

Total I - - - - = 1198.92

\( r_{2-2} = \sqrt{1198.92/27.61} = 6.58" \)

\( r_{1-1} \) of the section is the smallest and will be used.

Final

\( S = 16000 - 70 \times 23 \times 12/5.65 \)

\( S = 16000 - 3420 = 12580 \)

\( A = 310300/12580 = 24.7 \text{ sq. in.} \)
The section as tried has an area of 27.66 sq. in. 
100 r = 565", while L of member is 256"  
The section as tried will be used.

**CD, DE, and EF**

**Preliminary**

**CD**

DL = -171300#  
LL = -163500#  
I = -32300#  
Total = -367100#  

S as before = 12550#  

A = 367100/12550 = 29.20 sq. in.

**DE**

DL = -179300#  
LL = -166500#  
I = -32900#  
Total = -378700#  

A = 378700/12550 = 31 sq. in.

**EF**

DL = -177600#  
LL = -165000#  
I = -32600#  
Wind Load for all three chords  
is less than 25% of the Total  
Dead and Live Loads and need  
not be considered  

Total = -375200#  

A = 375200/12550 = 29.5 sq. in.
1 Cover Pl - 20" x 7/16" = 8.75 sq. in.
2 Pls - 14" x 3/8" = 10.50 "
2 Is - 3" x 3" x 5/16" = 3.56 "
2 Is - 5" x 3" x 5/16" = 4.80 "
2 Flats - 5" x 3/8" = 3.75 "

Total Area = 31.36 sq. in.

Center of Gravity of Section

<table>
<thead>
<tr>
<th>Area</th>
<th>Lever Arm</th>
<th>Moment at Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.75 sq. in.</td>
<td>14.60 in.</td>
<td>125.00</td>
</tr>
<tr>
<td>10.50 &quot; &quot;</td>
<td>7.38 &quot;</td>
<td>77.40</td>
</tr>
<tr>
<td>3.56 &quot; &quot;</td>
<td>13.51 &quot;</td>
<td>48.20</td>
</tr>
<tr>
<td>4.80 &quot; &quot;</td>
<td>1.06 &quot;</td>
<td>5.08</td>
</tr>
<tr>
<td>3.75 &quot; &quot;</td>
<td>.19 &quot;</td>
<td>.71</td>
</tr>
<tr>
<td>31.36 sq. in.</td>
<td></td>
<td>256.39</td>
</tr>
</tbody>
</table>

c. of gr. = 256.39/31.36 = 8.14"

Radius of Gyration

I of Cover Pl - 20" x 7/16 = .14
8.75 x 6.46^2 = 365.00
I of 2 Pls - 14" x 3/8 = 171.50
10.50 x .76^2 = 6.05
I of 2 Is - 3" x 3" x 5/16 = 3.00
3.56 x 5.37^2 = 102.50
I of 2 Is - 5" x 3" x 5/16 = 3.60
4.80 x 7.08^2 = 240.50
I of 2 Flats 5" x 3/8" = 5 x (3/8)^3 x 2/12 = .04
3.75 x 7.43^2 = 207.00

Total I = 1099.33

r = \sqrt{1099.33/31.36} = 5.93

Final

S = 16000 - 70 x 23 x 12/5.93
S = 16000 - 3260 = 12740#
CD
A = 367100/12740 = 28.8 sq. in.

DE
A = 378700/12740 = 29.7 sq. in.

EF
A = 375200/12740 = 29.4 sq. in.

The section as tried has an area of 31.36 sq. in.
100 r = 593" while the length of the members is 256"

Therefore this section will be used for CD, DE and EF.
VERTICALS

Typical Section

\[\begin{array}{c}
\end{array}\]

\[\text{CC} = -36600\#
\]

\[\text{DL} = -45000\#
\]

\[\text{I} = -10250\#
\]

\[\text{Total} = -91850\#
\]

Preliminary

Trying - 2 - 10" I's @ 20#; A = 11.76 sq. in.; \(r_2\), = 3.66

\[\begin{align*}
S &= 16000 - 70 \times L/r = 16000 - 70 \times 29 \times 12/3.66 \\
S &= 16000 - 6650 = 9350\#
\end{align*}\]

\[A = 91850/9350 = 9.82 \text{ sq. in.}\]

As channels are spaced 11" apart \(r_2\) is greater than \(r_1\).

Final

2 - 10" I's @ 20# have an area of 11.76 sq. in, which is more than required.

100 \(r = 366"\), while \(L\) is 348"; hence these two channels, connected by lattice bars, will be used.

\[\begin{array}{c}
\end{array}\]

Dd

\[\begin{array}{c}
\end{array}\]

\[\begin{align*}
\text{DL} &= -11800\#
\end{align*}\]

\[\begin{align*}
\text{LL} &= -28500\#
\end{align*}\]

\[\begin{align*}
\text{I} &= -6875\#
\end{align*}\]

\[\text{Total} = -47175\#\]
Preliminary

Trying 2 - 12" \( \ell_s \) @ 25#; \( A = 14.70 \) sq. in.; \( r = \frac{70}{4.43} \)
\[
S = 16000 - 70 \times \frac{L}{r} = 16000 - 70 \times 32 \times \frac{12}{4.43}
\]
\[
S = 16000 - 6070# = 9930#
\]
\[
A = \frac{47175}{9930} = 4.75 \text{ sq. in.}
\]

Final

2 - 12" \( \ell_s \) @ 25# have an area of 14.70 sq. in. which is more than required.

100 \( r = 475" \), while \( L = 384" \); hence the two channels connected by lattice bars will be used.

Bb

\[
\begin{align*}
 DL &= +21000# \\
 LL &= +25000# \\
 I &= +7750# \\
 \text{Total} &= +53750#
\end{align*}
\]

Preliminary

Trying 2 - 10" \( \ell_s \) @ 20#; \( A = 11.76 \) sq. in.
\[
A = \frac{53750}{16000} = 3.36
\]

Final

2 - 10" \( \ell_s \) @ 20# have a gross area of 11.76 sq. in. which is much in excess of that required, hence they will be used connected by lattice bars.
\[ \begin{align*}
\text{Ee} & \\
\text{DL} & = +10200 \\
\text{LL} & = -17400 \\
\text{I} & = -4450 \\
\text{Total} & = +10200 \\
\text{Total} & = -21850 \\
8/10 \times 10200 & = 8160\# \\
8/10 \times 21850 & = 17500\# \\
\text{Total compressive stress} & = 27700\# \\
\text{Total Tensile stress} & = 30010\#
\end{align*} \]

\underline{Preliminary}

2 - 12" \[\text{s} \] @ 25#; \( A = 14.70 \); \( r, -, = 4.43 \)
\[ S = 16000 - 70 \times \frac{L}{r} = 16000 - 70 \times 35 \times 12 \div 4.48 \]
\[ S = 16000 - 6780 = 9220\# \]
\[ A = \frac{27700}{9220} = 3.00 \text{ sq. in.} \]

\underline{Final}

2 - 12" \[\text{s} \] @ 25# have an area of 14.70 sq. in. which is more than required.

100 \( r = 443" \), while \( L = 420\# \)

As these two channels obviously are strong enough to carry the Tensile stress, they will be used.

\underline{Aa}

\[ \begin{align*}
\text{DL} & = +109500 \\
A & = \frac{-109500}{(3/4 \times 16000)} = 9.14 \text{ sq. in.} \\
2 - 10" \[\text{s} \] @ 20# have a gross area of 11.76 sq. in.
\end{align*} \]

Deducting 6 rivet holes or 1.80 sq. in., the remaining net section is 9.94 sq. in.

The two channels will be used.
DIAGONALS

Bo

DL = + 98500#
LL = + 79000#
I = + 17150#

Total = +194650#

A = 194650/16000 = 12.15 sq. in.

2 - 12" $\ell$s @ 25# = 14.70 - 4 x .35 = 13.30 sq. in.

The channels will be latticed.

Cd

DL = + 45300#
LL = + 50700#
I = + 11500#

Total = +107500#

A = 107500/16000 = 6.73 sq. in.

2 - 10" $\ell$s @ 20# = 11.76 - 4.35 = 10.36 sq. in.

These two channels latticed together will be used.

De

DL = +14600# Counter Stress
LL = +32200#
I = +7760#

Total = +54560#

8/10 of 11380 = 9100#

Total = +63660#

2 - 10" $\ell$s @ 20# will be used here also.
\[ \begin{align*}
\text{Ef} & \\
\text{LL} & = +33200# \\
\text{I} & = +8500# \\
\text{Total} & = +41700#
\end{align*} \]

\[ \begin{align*}
\text{Counter Stress} & \\
\text{LL} & = -10300# \\
\text{I} & = -2630# \\
\text{Total} & = -12930#
\end{align*} \]

\[ \frac{8}{10} \text{ of } 12930 = 10800# \]

\[ \text{Total} = +52500# \]

2 - 10\" \ell \text{ s @ 20# will be used here also.} \]
DL = -144000#  
WL = 17750# which being less
LL = -165000#  
I = -326000#

Total = -341600#

Bending moment due to wind = 1068000"#

Preliminary

The following section will be tried:-

1 Cover Pl - 20" x 7/16" = 8.75 sq. in.
2 Pls - 14" x 3/8" = 10.50 " 
2 Pls - 7 1/2" x 3/4" = 11.25 " 
2 Ls - 13" x 3" x 5/16" = 3.56 " 
2 Ls - 15" x 3" x 5/16" = 4.80 " 
2 Flats - 5" x 3/4" = 7.50 "

Total Area = 46.36 sq. in.

Center of Gravity of Section

<table>
<thead>
<tr>
<th>Area</th>
<th>Lever Arm</th>
<th>Moment at Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.75 sq. in.</td>
<td>14.97 in.</td>
<td>131.00</td>
</tr>
<tr>
<td>10.50 &quot;</td>
<td>7.75 &quot;</td>
<td>81.40</td>
</tr>
<tr>
<td>11.25 &quot;</td>
<td>7.75 &quot;</td>
<td>87.20</td>
</tr>
<tr>
<td>3.56 &quot;</td>
<td>13.89 &quot;</td>
<td>49.40</td>
</tr>
<tr>
<td>4.80 &quot;</td>
<td>1.43 &quot;</td>
<td>6.87</td>
</tr>
<tr>
<td>7.50 &quot;</td>
<td>.38 &quot;</td>
<td>2.85</td>
</tr>
</tbody>
</table>

46.36 sq. in.

\[ \text{c. of gr.} = \frac{358.72}{46.36} = 7.74" \]
Radius of Gyration (1-1)

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>I of Cover Pl - 20&quot; x 7/16&quot;</td>
<td>0.14</td>
</tr>
<tr>
<td>8.75 x 7.23</td>
<td></td>
</tr>
<tr>
<td>I of 2 Pls - 14&quot; x 3/8&quot;</td>
<td>171.50</td>
</tr>
<tr>
<td>10.50 x 0.03</td>
<td></td>
</tr>
<tr>
<td>I of 2 Pl - 7 1/2&quot; x 3/4&quot;</td>
<td>70.40</td>
</tr>
<tr>
<td>11.25 x 0.03</td>
<td></td>
</tr>
<tr>
<td>I of 2 Ls - 3&quot; x 3&quot; x 5/16&quot;</td>
<td>3.00</td>
</tr>
<tr>
<td>3.56 x 6.15</td>
<td></td>
</tr>
<tr>
<td>I of 2 Ls - 5&quot; x 3&quot; x 5/16&quot;</td>
<td>3.60</td>
</tr>
<tr>
<td>4.80 x 6.31</td>
<td></td>
</tr>
<tr>
<td>I of 2 Flats - 5&quot; x 3/4&quot;</td>
<td>0.05</td>
</tr>
<tr>
<td>7.50 x 7.36</td>
<td></td>
</tr>
</tbody>
</table>

Total I = 1439.79

\[ r_{1} = \sqrt{1439.79/46.36} = 5.58 \]

Radius of Gyration (2-2)

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>I of Cover Pl - 20&quot; x 7/16&quot;</td>
<td>282.00</td>
</tr>
<tr>
<td>I of 2 Pls - 14&quot; x 3/8&quot;</td>
<td>0.12</td>
</tr>
<tr>
<td>10.50 x 6.19</td>
<td></td>
</tr>
<tr>
<td>I of 2 Pls - 7 1/2&quot; x 3/4&quot;</td>
<td>403.00</td>
</tr>
<tr>
<td>11.25 x 6.75</td>
<td></td>
</tr>
<tr>
<td>I of 2 Ls - 3&quot; x 3&quot; x 5/16&quot;</td>
<td>3.00</td>
</tr>
<tr>
<td>3.56 x 7.25</td>
<td></td>
</tr>
<tr>
<td>I of 2 Ls - 5&quot; x 3&quot; x 5/16&quot;</td>
<td>12.60</td>
</tr>
<tr>
<td>4.80 x 8.05</td>
<td></td>
</tr>
<tr>
<td>I of 2 Flats - 5&quot; x 3/4&quot;</td>
<td>7.80</td>
</tr>
<tr>
<td>7.50 x 8.88</td>
<td></td>
</tr>
</tbody>
</table>

Total I = 2308.77

\[ r_{2} = \sqrt{2308.77/46.36} = 7.06 \]

The allowed stress in the member is

\[ S = 16000 - 70 \times L/r = 16000 - 70 \times 34.83 \times 12/5.58 \]

\[ S = 16000 - 5240 = 10760# \]

The actual stress due to the combined dead and live load is

\[ 341600/46.36 = 7360# \]

The stress due to the bending moment of the wind is

\[ S = Mc/I = 1068000 \times 7.45/2308.77 = 3440# \]
Final

The combined stress per square inch in the post is 7360 + 3440# or 10800#; the allowable stress is 10760#. The difference is so small that the section as tried will be used.
PORTAL

Max. str. = -24700# and +24700#

8/10 of 24700# = 19500#; so that the member has to be
designed for a tension and compression of 44200# on
account of the reversal of the direction of wind.

Preliminary
least r = 7.08 x 12/120 = .71
2 \( \frac{L}{s} \) = 4" x 3 1/2" x 5/16" have a least r of 1.07
S = 16000 - 10 x L/r = 16000 - 70 x 7.08 x 12/1.07
S = 16000 - 5500 = 10500#

Final
2 \( \frac{L}{s} \) = 4" x 3 1/2" x 3/8" have an area of 4.50 sq. in.
Their strength in compression is 47200# while they
obviously are strong enough in tension. The first is
more than required. These angles will be used.

IF, LA, QQ

Max. str. = -8900# and +8900#

8/10 of 8900# = 7120#, so that the member has to be
designed for a tension and compression of 16200# on
account of the reversal of wind.

Preliminary
least r = 7.08 x 12/120 = .71
2 \( \frac{L}{s} \) = 2 1/2" x 2 1/2" x 5/16" have a least r of .76
S = 16000 - 70 x L/r = 16000 - 70 x 7.08 x 12/.76
S = 16000 - 7850 = 8150#
Final

2 LE - 2 1/2" x 2 1/2" x 5/16" have an area of 2.94 sq. in., and their strength in compression is 24000#, while they obviously are strong enough in tension. These angles will be used.

ML

The stress is -14300# but on account of the reversal of the direction of the wind, this member must be designed for a compression and tension that is for 14300 + 8/10 x 14300 or 25740#.

Preliminary

least \( r = \frac{(1/2)(\sqrt{6} + 6.33^2) \times 12}{120} = .50 \)

1 \( \perp \) - 3 1/2" x 3" x 3/8" has an \( r \) of .90

\( S = 16000 - 70 \times \frac{L}{r} = 16000 - 70 \times 5.01 \times 12 \times .90 \)

\( S = 16000 - 4680 = 11320# \)

Final

1 \( \perp \) - 3 1/2" x 3" x 3/8" has an area of 2.30 sq. in., and has a compressive strength of 26100# and is obviously strong enough in tension. This angle will be used for all lattice members in the portal.

FA

The stress is +17950#, but on account of the reversal of the direction of the wind, this member must be designed for a compression and tension of 17950 + 8/10 x 17950 or 32310#
Preliminary

least \( r = \sqrt{6.33^2 - 4^2} \times \frac{12}{120} = .75'' \)

2 \( Ls - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times \frac{1}{2}'' \) have an \( r \) of .74

\( S = 16000 - 70 \times \frac{L}{r} = 16000 - 70 \times 7.5 \times \frac{12}{.75} \)

\( S = 16000 - 8400 = 7600# \)

Final

2 \( Ls - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times \frac{1}{2}'' \) have an area of 4.50 sq. in.

Their strength in compression is 34200#, while they are obviously strong enough in tension. These two angles will therefore be used for FA and QA.
Maximum stress in AB' = 18500#

\[ A = \frac{18500}{16000} = 1.15 \text{ sq. in.} \]

\[ 1 - L - 3 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times \frac{5}{16}'' = 1.78 - 2 \times 0.27 = 1.24 \text{ sq. in.} \]

This angle will be used for AB', while for all other laterals

\[ 1 - L - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times \frac{5}{16}'' \text{ will be used.} \]
LOWER LATERALS

ab'

DL = + 18500#
LL = + 18500#

Total = + 37000#

A = 37000/16000 = 2.31 sq. in.

1 - L - 5" x 3" x 5/16" has an area of 2.40 sq. in. and will be used.

bc'

DL = + 13850#
LL = + 14400#

Total = + 28250#

A = 28250/16000 = 1.76 sq. in.

1 - L - 3 1/2" x 2 1/2" x 5/16" has an area of 1.78 sq. in. and will be used for these and all further lower laterals.
_TRANSVERSE FRAMES_

CC' = 10350#

Preliminary
least \( r = \frac{L}{120} = 7.08 \times 12 = .71 \)

\[ 2 - L_8 - \text{2 } 1/2'' \times 2 \ 1/2'' \times 5/16'' \text{ spaced } 5/16'' \text{ apart, } r = 1.20 \]

\[ S = 16000 - 70 \times \frac{L}{r} = 16000 - 70 \times 7.08 \times 12/1.20 \]

\[ S = 16000 - 4960 = 11040# \]

\[ A = \frac{10350}{11040} = .94 \]

Final
2-\( L_8 \) 2 1/2'' \times 2 1/2'' \times 5/16'', spaced 5/16'' apart have an area of 2.94 sq. in. Therefore these angles will be used.

The same angles will be used for the lower horizontal frame member.

Knee Braces
The two knee braces have a length of 7.5 feet and the least \( r = .75'' \)

\[ 2 - L_8 - \text{2 } 1/2'' \times 2 \ 1/2'' \times 5/16'', \text{ spaced } 5/16'' \text{ apart, } r = 1.20 \]

This angle will be used for all the knee braces.

Latticing
The latticing in the frame EE' has an unsupported length of \( \frac{1}{2} \sqrt{1533^2 - 6.32^2} = 8.75 \text{ ft.} \)

Least \( r = \frac{L}{120} = 8.75 \times 12/120 = .875 \)

1-\( L\) - 3'' \times 3'' \times 5/16'' has \( r = .92 \)

This angle will be used for the latticing in this frame.

The latticing in the frame DD' has an unsupported
length of $\frac{1}{2} \sqrt{14^2 - 6.33^2} = 9.2$ ft

Least $r = L/120 = 9.2 \times 12/120 = .92''$

1 $\angle - 3'' \times 3'' \times 5/16''$ has $r = .92''$

This angle will be used for the latticing in this frame and in the frame at CC' and AA' as well.
WEIGHT OF STEEL IN SPAN

Bottom Chord

abc  $4 Ls - 3'' x 3'' x 5/16''$
     $2 Fl - 14'' x 3/8''$
     = 24.40#
     = 35.70#
     = 60.10# x 4 x 23  = 5520#

cd  $4 Ls - 3'' x 3'' x 5/16''$
    $2 Pls - 14'' x 3/8''$
    $2 Pls - 7 1/2'' x 5/16''$
    $2 Flats - 3'' x 5/8''$
    = 24.40#
    = 35.70#
    = 15.94#
    = 12.76#
    = 88.80# x 4 x 23  = 8150#

de  $4 Ls - 3'' x 3'' x 5/16''$
     $2 Pl - 14'' x 3/8''$
     $2 Pls - 7 1/2'' x 5/16''$
     $2 Pls - 13'' x 5/16''$
     $2 Flats - 3'' x 5/8''$
     = 24.40#
     = 35.70#
     = 15.94#
     = 27.62#
     = 116.42# x 4 x 23  = 10700#

ef  Same as de  
     = 116.42# x 2 x 23  = 5350#

Total for Bottom Chord 29720#
### Top Chord

<table>
<thead>
<tr>
<th>Part</th>
<th>Description</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>1 Pl - 20&quot; x 7/16&quot;</td>
<td>8.75#</td>
</tr>
<tr>
<td></td>
<td>2 Pls - 14&quot; x 3/8&quot;</td>
<td>35.70#</td>
</tr>
<tr>
<td></td>
<td>2 Le - 3&quot; x 3&quot; x 5/16&quot;</td>
<td>12.20#</td>
</tr>
<tr>
<td></td>
<td>2 Le - 5&quot; x 3&quot; x 5/16&quot;</td>
<td>16.40#</td>
</tr>
<tr>
<td></td>
<td>Weight: 73.05# x 4 x 23 = 6200#</td>
<td></td>
</tr>
<tr>
<td>BC</td>
<td>Same as AC</td>
<td>73.05# x 4 x 23 = 6200#</td>
</tr>
<tr>
<td>CD</td>
<td>1 Pl - 20&quot; x 7/16&quot;</td>
<td>8.75#</td>
</tr>
<tr>
<td></td>
<td>2 Pls - 14&quot; x 3/8&quot;</td>
<td>35.70#</td>
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<tr>
<td></td>
<td>2 Le - 3&quot; x 3&quot; x 5/16&quot;</td>
<td>12.20#</td>
</tr>
<tr>
<td></td>
<td>2 Le - 5&quot; x 3&quot; x 5/16&quot;</td>
<td>16.40#</td>
</tr>
<tr>
<td></td>
<td>2 Flats - 5&quot; x 3/8&quot;</td>
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<td>Weight: 85.81# x 4 x 23 = 7900#</td>
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<tr>
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<td>Same as CD</td>
<td>85.81# x 4 x 23 = 7900#</td>
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<td>EF</td>
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<td>Total for Top Chord</td>
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### End Post

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<th>Description</th>
<th>Weight</th>
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<tbody>
<tr>
<td></td>
<td>1 Pl - 20&quot; x 7/16&quot;</td>
<td>8.75#</td>
</tr>
<tr>
<td></td>
<td>2 Pls - 14&quot; x 3/8&quot;</td>
<td>35.70#</td>
</tr>
<tr>
<td></td>
<td>2 Pl - 7 1/2&quot; x 3/4&quot;</td>
<td>38.26#</td>
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<td></td>
<td>2 Le - 3&quot; x 3&quot; x 5/16&quot;</td>
<td>12.20#</td>
</tr>
<tr>
<td></td>
<td>2 Le - 5&quot; x 3&quot; x 5/16&quot;</td>
<td>16.40#</td>
</tr>
<tr>
<td></td>
<td>2 Flats - 5&quot; x 3/4&quot;</td>
<td>25.50#</td>
</tr>
<tr>
<td></td>
<td>Weight: 136.81# x 4 x 23 = 10260#</td>
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</tbody>
</table>

### Verticall

<table>
<thead>
<tr>
<th>Part</th>
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<th>Weight</th>
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</thead>
<tbody>
<tr>
<td>Co</td>
<td>2 - 10&quot; @ 20# = 40# x 4 x 23 = 3680#</td>
<td></td>
</tr>
<tr>
<td>Dd</td>
<td>2 - 12&quot; @ 25# = 50# x 4 x 23 = 4600#</td>
<td></td>
</tr>
<tr>
<td>Bb</td>
<td>2 - 10&quot; @ 20# = 40# x 4 x 23 = 3680#</td>
<td></td>
</tr>
<tr>
<td>Ee</td>
<td>2 - 12&quot; @ 25# = 50# x 4 x 23 = 4600#</td>
<td></td>
</tr>
<tr>
<td>Aa</td>
<td>2 - 12&quot; @ 20# = 40# x 4 x 23 = 3680#</td>
<td></td>
</tr>
</tbody>
</table>

Total for Verticals = 20240#
Diagonals

Bo  2 - 12" \( \ell \) @ 25#$ = 50# \( x \) 4 \( x \) 23 \( \cos \) \( \phi \) = 7080#
Cd  2 - 10" \( \ell \) @ 20#$ = 40# \( x \) 4 \( x \) 23 \( \cos \) \( \phi \) = 5900#
De  2 - 10" \( \ell \) @ 20#$ = 40# \( x \) 4 \( x \) 23 \( \cos \) \( \phi \) = 6400#
Ef  2 - 10" \( \ell \) @ 20#$ = 40# \( x \) 4 \( x \) 23 \( \cos \) \( \phi \) = 6700#

Total for Diagonals = 26080#

Top Laterals

1 \( \perp \) - 3 1/2" \( x \) 2 1/2" \( x \) 5/16" \( = \) 6.1 \( x \) 4 \( x \) 23 \( \sec \) \( \phi \) = 740#
1 \( \perp \) - 2 1/2" \( x \) 2 1/2" \( x \) 5/16" \( = \) 5 \( x \) 14 \( x \) 23 \( \sec \) \( \phi \) = 2160#

Total for Top Laterals = 2900#

Lower Laterals

ab'
1 \( \perp \) - 5" \( x \) 3" \( x \) 5/16" \( = \) 8.2 \( x \) 4 \( x \) 23 \( \sec \) \( \phi \) = 1010#
bc'
1 \( \perp \) - 3 1/2" \( x \) 2 1/2" \( x \) 5/16" \( = \) 6.1 \( x \) 4 \( x \) 23 \( \sec \) \( \phi \) = 750#

cd'
Same as bc' = 750#
de'
Same as bc' = 750#
ef'
Same as bc' = 750#

Total for Lower Laterals = 4010#
Portal

EH2  EK2  EN
2 Ls - 5" x 3 1/2" x 3/8"  = 20.8 x 2 x 19.0  = 790#

IF2  LA2  OQ
2 Ls - 2 1/2" x 2 1/2" x 5/16"  = 10 x 2 x 19.0  = 380#

Latticing
1 L - 3 1/2" x 3" x 3/8"  = 7.9 x 24 x 5.01  = 950#

FA
2 Ls - 2 1/2" x 2 1/2" x 1/2"  = 15.4 x 4 x 7.5  = 460#

Total for Portal  = 2580#
Transverse Frames

CC
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 19.0$ = 190#
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 7.5$ = 75#
1 - L - 3" x 3" x 5/16" = $6.1 \times 24 \times 6.15$ = 900#

DD
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 19.0$ = 190#
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 7.5$ = 75#
1 - L - 3" x 3" x 5/16" = $6.1 \times 24 \times 9.2$ = 1350#

EE
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $50 \times 2 \times 19.0$ = 190#
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 7.5$ = 75#
1 - L - 3" x 3" x 5/16" = $6.1 \times 24 \times 8.75$ = 1280#

AA
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 19.0$ = 190#
2 $L_s - 2 \frac{1}{2}'' \times 2 \frac{1}{2}'' \times 5/16''$ = $5.0 \times 2 \times 7.5$ = 75#
1 - L - 3" x 3" x 5/16" = $6.1 \times 24 \times 8.75$ = 1280#

Total for Transverse Frames = 5770#
Summary of Weight of Steel in Span

<table>
<thead>
<tr>
<th>Component</th>
<th>Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Chords</td>
<td>29720</td>
</tr>
<tr>
<td>Top Chords</td>
<td>32150</td>
</tr>
<tr>
<td>End Posts</td>
<td>10260</td>
</tr>
<tr>
<td>Verticals</td>
<td>20240</td>
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<tr>
<td>Diagonals</td>
<td>26080</td>
</tr>
<tr>
<td>Top Laterals</td>
<td>2900</td>
</tr>
<tr>
<td>Lower Laterals</td>
<td>4010</td>
</tr>
<tr>
<td>Portals</td>
<td>2580</td>
</tr>
<tr>
<td>Transverse Frames</td>
<td>5770</td>
</tr>
</tbody>
</table>

Add 10% for Rivets, Connections, Lattice Bars, etc. = 13370#

Total Weight = 147080#

Weight per 1. ft. per truss = 147080/207 = 724#

This weight is 120# more per panel per truss than assumed, but investigation shows that the areas of all sections are ample to take care of the additional stress.
Total Weight of Bridge

Weight of Floor System
(Floorbeams, Stringers, Planking, Wheelguards) = 
1212 x 9 = 10908#

Weight of Steel in Bridge = 147080#
" Machinery, House and Supporting Beams = 60000#

Total weight of Bridge = 217988#
DESIGN of STRINGERS and FLOORBEAMS

To CARRY the LOAD of the OPERATING HOUSE and MACHINERY
on TOP of the MIDDLE PANEL

The load due to the Operating House, Machinery, Stringers and Floor Beams, will be assumed as being uniformly distributed over an area of 23 x 15 feet. It will be carried by 6 longitudinal stringers spaced 3' - 0" apart and two transverse floorbeams. There will be a load of 60000 x 3/15 or 12000# on each of the inside, and of 6000# on each of the outside stringers.

Design of Inside Stringers

\[ M = \frac{WL}{8} = 12 \times 23/8 = 34.5 \text{ kip feet or } 414 \text{ kip inches.} \]

Rolled I beam to be tried

\[ M = QS; \quad Q = \frac{M}{S} = \frac{414}{16} = 25.8 \]

A 10" I @ 20# has a section modulus of 26.8 and will be used.

Design of Outside Stringers

\[ M = \frac{WL}{8} = 6 \times 23/8 = 17.25 \text{ kip feet or } 207 \text{ kip inches} \]

Rolled to be tried

\[ M = QS; \quad Q = \frac{M}{S} = \frac{207}{16} = 12.9 \]

A 9" Z @ 20# has a section modulus of 13.5 and will be used.
The end shear on the floor beam equals 15 kips.
The Max. Bending Moment is at the center and equals
\[ M = 15 \times 9.75 - 3 \times 7.5 - 6 \times 4.5 = 6 \times 1.5 = 89 \text{ kip feet} \]
or 1068 kip inches

\[ M = QS; \ Q = M/S = 1068/16 = 68 \]

A 15" I @ 55# has a section modulus of 68.1 and will be used.
PART V

DESIGN

of

SHOES
DESIGN of SHOE

Two cases must be considered - one with the span as a fixed truss, so that the maximum end load reaction will include the dead live load reaction, while the other is the dead load reaction plus the impact due to the lowering of the span on the shoes, which will be taken as 25% of the total weight of the span.

Case No. 1

DL React. = 109500#
LL " = 112500#
I " = 22200#
Total React. = 244200#

Case No. 2

DL React. = 109500#
I " = 54500#
Total React. = 164000#

The total reaction of Case No. 1 is the greater and governs the design.

Bearing plate:

\[ \frac{244200}{600} = 407 \text{ sq. in.} \]

Bearing on pin:

Length of lineal inches of bearing on a 4 1/2 pin = \[ \frac{244200}{4 \frac{1}{2} \times 15000} = 3.62" \]

Bending moment on pin:

\[ M = \left( \frac{244200}{2} \right) \times 1 \frac{3}{8} = 168000"# \]
The B. M. on a 4 1/2" pin as taken from the steel handbooks is 178900#, therefore a 4 1/2" pin will be used.

On one end of the truss an expansion roller has to be provided. There will be one roller only (see plate 4).

The allowed bearing stress on a roller is 500 d, where d is the diameter of the pin in inches.

Assuming a length of 24 inches for the roller,

\[
d = \frac{244200}{500 \times 24} = 18.8"\]

The roller will be of 20" diameter.
Expansion End

C.S. Casting - A - 1/2" Metal
C.S. Casting - B - " "
C.S. Casting - C - " "

C.S. Casting - D - 1 1/2" Metal
4 1/2" φ Pin
Roller - 20" D

Fixed End

SHOES

Plate 5.
PART VI

DESIGN

of

SUSTAINING CABLES
DESIGN of CABLES

The stress in the cables is

\[ S = S_m + S_w + S_b \]

where \( S_m \) is the stress due to the mass moved
\( S_w \) \( \quad \) own weight of the cable
\( S_b \) \( \quad \) bending of the cable over the sheaves

\[ S_b = \frac{E_d}{2r} \]

where \( E \) is the modulus of elasticity of the material, \( d \) the diameter of the wires in inches and \( r \) the radius of the sheave in inches.

\[ S_m + S_w = 25000\# \text{ per sq. in.} \]
\[ S_b = 25000\# \text{ per sq. in.} \]

The total weight of the bridge - - - = 218000#
Ropes, balancers, equalizers, etc. - - = 12000#

Total load to be lifted - - - = 230000#

\[ A = \frac{w}{S_m} = \frac{230000}{25000} = 9.20 \text{ sq. in.} \]

Assuming a length of 160 feet for the cables, their weight is

\[ 9.20 \times \frac{160}{144} \times 490 = 5000\#, \]

hence the assumed weight of 12000# is all right.

Using wire of \#065" diameter, the least diameter of a sheave must be 65". By using an 84" diameter

\[ S_b = 30000000 \times .065/84 = 23200\# \]

which is lower than the allowed stress.

The area of one .065" wire is .0033 sq. in., hence by using 32 cables, each cable must have
9.20/.0033 x 32 or 88 wires.

The diameter of each cable is $D = (nd/13) + 7d$

where $n =$ the number of wires in the cable
$d =$ their diameter

$D$ of each cable is $(88 \times .065/13) + 7 \times .065 = 7/8"$

Therefore 32 - 7/8" cables, each containing 88 wires
@ .065" diameter will be used, 8 at each end.

The cables will be connected to the top chord by
means of pins. At the connection to the counterweights,
equalizers must be used. They will consist of 4 rocker
arms connected in pairs to two rocker arms at right
angles to the first. The last two will be connected to
one rocker arm in the direction of the pairs. The last
arm is fastened to the sustaining rod of the counterweights.
The arrangement is shown in the sketch.
PART VII

DESIGN

of

SHEAVES

BEARINGS

and

THEIR SUPPORTS
SHEAVES

Rim

\[ \begin{align*} 
D & = \text{Diameter of cables} = 7/8" \\
M & = D + 1/2" = 1 3/8" \\
J & = .250 + 1/4" = 1/2" \\
I & = J \quad = 1/2" \\
L & = 7(D + 1/2) + D + 4J = 12 1/2" \\
H & = .8D + 3/8" = 1 1/8" \\
K & = 3/4 D \quad = 5/8" \\
\end{align*} \]

Arms

\[ \begin{align*} 
R & = \text{Radius of Sheaves} \quad = 42" \\
N & = \text{Number of arms} \quad = 12 \\
h & = \text{Width of arms} = (1/4)L + (1/10)(R/N) = 3 5/8" \\
b & = \text{Thickness of arms} = h/2 \quad = 1 13/16" \\
T_1 & = \text{Taper in arms (width)} = h/10 \quad = 3/8" \\
T_2 & = \text{Taper in arms (thickness)} = b/10 \quad = 3/16" \\
\end{align*} \]

Hub

\[ \begin{align*} 
L & = \text{Length of hub} \quad = 14 1/2" \\
T & = \text{Thickness of metal in hub} = 3/4 \quad = 2 3/4" \\
\end{align*} \]

Diameter of axle

The axle is subject to a bending and twisting moment.

The twisting moment \( T = Wr \times .05 \)

where \( W \) = the load on the axle \\
(\( r \) = the radius of the axle \\
.05 = the coefficient of friction

The bending moment \( M = WL/8 \)

To determine the diameter \((D)\) of the axle the equivalent twisting moment \( T = \sqrt{M^2 + T^2} \) is used.

\[ D = 1.72 \sqrt{T/M} \]

Using for \( W \) a value of 120000#, which includes the weight of the sheave and also assuming a diameter of 7"

\[ \begin{align*} 
T &= 120000 \times 3.5 \times .05 = 21000"# \\
M &= 120000 \times 14.75/8 = 221000"# \\
T_1 &= 221000 + \sqrt{221000^2 - 21000^2} = 464500"# \\
\end{align*} \]
D = 1.72 \sqrt{\frac{464500}{10000}} = 6.18''

Hence an axle of 7'' diameter will be used.

**Bearing**

- **D** = diameter of journal = 7''
- **L** = length of journal = 2D = 14''
- **h** = height to center = 1.05D + 1/2'' = 7 7/8''
- **L** = length of base = 3.6D + 5'' = 25 3/4''
- **W** = width of base = .8L = 8 1/2''
- **W** = width of base = .7L = 6 1/2''
- **T** = thickness of base = .3D + 5/16'' = 2 7/16''
- **T** = thickness of cap = .3D + 7/16'' = 2 9/16''
- **d** = diameter of cap bolts = .25D + 1/4'' = 2''
- **S** = distance between centers of cap bolts = 1.6D + 1 1/2'' = 12 7/8''
- **S** = distance between centers of base bolts = 2.7D + 4 3/16'' = 22''

All detail dimensions given for the sheaves and bearings are the minimum required and must be modified in the design, especially of the bearings, as there are two bearings for each sheave and each bearing will be supported vertically and horizontally (see plates 6 and 7).

**Girders Between Sheaves**

The pressure of the sheaves will bear vertically upon the columns and horizontally against two girders between the two bearings. These girders act as columns and carry a load of 57500# each. Their length will be 16' - 10 1/2".
Preliminary

Rankine's formula gives

\[ P = \frac{aS}{(1 + \phi)(L/r)^2} \]

where \( P \) = the load in pounds
\( a \) = the area
\( S \) = the unit stress
\( \phi \) = a constant
\( L \) = the length in inches
\( r \) = the least radius of gyration.

To assure proper fastening of the bearing, a deep section must be used, hence 2 15"Ls @ 33# will be tried, with their flanges in and spaced 14" apart.

\[
\begin{bmatrix}
  1 \\
  2 \\
  2 \\
  14
\end{bmatrix}
\]

\[
\begin{align*}
I \text{ of } 2 - 15" - &\quad Ls @ 33# = 2 \times 8.2 = 16.4 \\
1980 \times 6.21 &\quad = 762.0 \\
\text{Total } I &\quad = 778.4
\end{align*}
\]

\[
\begin{align*}
r_{2-2} &\quad = \sqrt{278.4/19.80} = 6.27 \\
r_{7-7} &\quad = 5.62 \\
S &\quad = 16000 - 70 \times L/4 = 16000 - 70 \times 202.5/5.62 = 13470\# \\
S &\quad = 16000 - 2530 = 13470\# \\
L/r &\quad = 36 \\
P &\quad = 19.80 \times 13470/(1 + 36^2/25000) = 259000\#
\end{align*}
\]

Final

Also this section is much stronger than required, but it will be used to secure proper fastening for the bearing as stated before.

The four columns will be connected by latticing.
Stringers

4 stringers will be used to support the columns spaced as shown in the sketch.

4 - 8" I @ 18# with a section modulus of 14.2 will be used. They are strong enough without any further investigation.

Maximum Floor Beam Moment and Shear

The shear at the point of support equals the sum of the loads on one-half the floor beam or 97.25 kips.

The bending moment is a maximum at the center.

\[ M = 97.25 \times 9.75 - 62.5 \times 9.75 - 3 \times 6.25 - 3 \times 1.75 \]

\[ M = 316 \text{ kip feet} \text{ or} \quad 3792 \text{ kip inches}. \]
**Design of Floor Beam**

The floor beams must be 2' - 6" deep in order to give enough clearance for the sheaves over the columns.

The beams will consist of 2 plates and 4 \( Ls \).

**Preliminary**

Assuming a section of 2 Pls - 30" x 1/2" and 4 \( Ls \) - 6" x 6" x 1/2", the weight of the beam is 180.4.

The B. M. due to the weight of the girder is

\[
M = \frac{WL}{8} = 0.18 \times 19.5^2/8 = 8.57 \text{ kip feet or 102.84 kip inches.}
\]

The effective depth being 26.64", the required net flange area is

\[
A = \frac{(3792 + 102.84)/26.64 \times 16}{26.64} = 9.12 \text{ sq. in.}
\]

The area required for the shear is

\[
A = 97.25/9 = 10.81 \text{ sq. in.}
\]

**Final**

Two plates 30" x 1/2" have an area of 22.50 sq. in.

2 - \( Ls \) - 6" x 6" x 1/2" have a net area of 10.00 sq. in.

Both areas are much in excess of those required as there will be a beam under each bearing; but on account of the depth these beams will be used as tried.

The beams will overhang 3'-6" at both ends - that is, to the end of the sheaves.
SHEAVE
BEARING FOR SHEAVES

Plate 7.
PART VIII

TOWERS

STRESSES and DESIGN
STRESSES in TOWERS

The towers will be 20' - 0" square and 173' - 0" high from the top of the masonry to the center line of the sheaves.

\[ \phi = \tan^{-1} \frac{28}{19.5} = 55^\circ 8' \]

\[ \sec \phi = 1.75000; \cos \phi = 1.21879 \]
\[ \sin \phi = 0.82048; \cos \phi = 0.57167 \]
WIND STRESSES

Columns

\[
\begin{align*}
350 \times 35 & = 12250 \# \\
17275 \times 28 & = 484000 \# \\
\hline
\text{A'B'} & = 496250/19.5 = 25250 \#
\end{align*}
\]

\[
\begin{align*}
350 \times 63 & = 22100 \# \\
17275 \times 56 & = 968000 \# \\
18325 \times 28 & = 514000 \# \\
\hline
\text{B'C'} & = 1504100/19.5 = 77000 \#
\end{align*}
\]

\[
\begin{align*}
350 \times 91 & = 31800 \# \\
17275 \times 84 & = 1452000 \# \\
18325 \times 56 & = 1028000 \# \\
1400 \times 28 & = 39200 \# \\
\hline
\text{C'D'} & = 2551000/19.5 = 131000 \#
\end{align*}
\]

\[
\begin{align*}
350 \times 119 & = 39200 \# \\
17275 \times 112 & = 1934000 \# \\
18325 \times 84 & = 1542000 \# \\
1400 \times 56 & = 78400 \# \\
1400 \times 28 & = 39200 \# \\
\hline
\text{D'E'} & = 3634800/19.5 = 186000 \#
\end{align*}
\]

\[
\begin{align*}
350 \times 147 & = 51900 \# \\
17275 \times 140 & = 2420000 \# \\
18325 \times 112 & = 2056000 \# \\
1400 \times 84 & = 117600 \# \\
1400 \times 56 & = 78400 \# \\
1400 \times 28 & = 39200 \# \\
\hline
\text{E'F'} & = 4763100/19.5 = 244000 \#
\end{align*}
\]
\[ \begin{align*} 
350 \times 175 &= 61400\# \\
17275 \times 168 &= 2904000\# \\
18325 \times 140 &= 2570000\# \\
1400 \times 112 &= 156800\# \\
1400 \times 84 &= 117600\# \\
1400 \times 56 &= 78400\# \\
1400 \times 28 &= 32200\# \\
\end{align*} \]

\[ F'G' = \frac{5927400}{19.5} = 304000\# \]

**Transverse Bracing**

**Horizontals**

| AA'  | =   | 16575\# |
| BB'  | =   | 17625\# |
| CC'  | =   | 19025\# |
| DD'  | =   | 20425\# |
| EE'  | =   | 21825\# |
| FF'  | =   | 23225\# |

**Diagonals**

| A'B  | = 16925 \times \text{sec} \phi | = +28800\# |
| B'C  | = 17625 \times \text{sec} \phi | = +30900\# |
| C'D  | = 19025 \times \text{sec} \phi | = +33400\# |
| D'E  | = 20425 \times \text{sec} \phi | = +35800\# |
| E'F  | = 21825 \times \text{sec} \phi | = +38300\# |

**Longitudinal Bracing**

**Horizontals**

| A''A | =   | 875\# |
| B''B | =   | 1925\# |
| C''C | =   | 3325\# |
| D''D | =   | 4725\# |
| E''E | =   | 6125\# |
| F''F | =   | 7525\# |
| G''G | =   | 8925\# |

**Diagonals**

| AB'' | = 875 \times \text{sec} \phi | = +1530\# |
| BC'' | = 1925 \times \text{sec} \phi | = +3370\# |
| CD'' | = 3325 \times \text{sec} \phi | = +5830\# |
| DE'' | = 4725 \times \text{sec} \phi | = +8275\# |
| EF'' | = 6125 \times \text{sec} \phi | = +10750\# |
| FG'' | = 7525 \times \text{sec} \phi | = +13200\# |
The greatest stress is -23225#

Preliminary

$4L_s - 2'' \times 2'' \times 5/16''$ will be tried

least $r = 20 \times 12/120 = 2$

$S = 16000 - 70 \times L/r = 16000 - 70 \times 20 \times 12/2$

$S = 16000 - 8400 = 7600#$

The strength of the angles is

$4.60 \times 7600 = 35000#$

Final

$4L_s - 2'' \times 2'' \times 5/16''$ must be spaced at least $2 1/4''$

b. to b., so that their least $r = 2.00$. Their strength is more than required. Hence these angles will be used for all transverse and longitudinal horizontals.

The angles will be connected by lattice bars.

The spacing shall be wider than $2 1/4''$ so as to conform to the column width.

Nominal lattice bars, $2'' \times 2'' \times 5/16''$ angles, will be placed between the floor beams of the sheaves.

-DIAGONALS-

The greatest stress in any diagonal is - 38300#, which requires a net area of 2.38 sq. in.

$4L_s 2 1/2'' \times 2 1/2'' \times 5/16''$ have a net area of 3.40 sq. in., and will be used for all diagonals.
Distance between rivet lines for web plate is
18" - 5" = 13"; 1/30 of 13" = .433".
Web plates must be at least 7/16" thick.
.Distance between rivet lines for batten plate is
14" + 5" = 19"; 1/45 of 19" = .422".
The batten plates must be at least 7/16" thick.
As the columns have no cover plates the thickness of the
flanges must not be less than 6/12 = .50" or 1/2".

The bending moment at the shoe of the lowest column is
\[ M = \frac{1}{2} \times 41550 \times 14/2 = \frac{1}{2} \times 291000\text{#} \text{ or } 1549000\text{#} \]
The dead load from the weight of the steel is
for the columns -- -- 40000#
" " bracing -- -- 15000#
Total weight 55000#

DL = 55000#
97000#
I = 14875#
WL = 304000#
Total = 470875#
Preliminary

The following section will be tried:-

2 Pls - 18" x 7/16" = 15.75 sq. in.
4 Pls - 18" x 11/16" = 52.26 " "
2 Pls - 9 1/2" x 7/16" = 8.31 " "
2 Pls - 9 1/2" x 5/16" = 5.94 " "
4 Ls - 6" x 4" x 1/2" = 19.00 " "

Total area = 101.23 sq. in.

Radius of Gyration (1-1)

I of 2 Pls - 18" x 7/16" = 1776.00
I of 4 Pls - 18" x 11/16")
I of 2 Pls - 9 1/2" x 7/16") = 107.10
I of 2 Pls - 9 1/2" x 5/16")
I of 4 Ls - 6" x 4" x 1/2" = 25.20

Total I = 3133.30

\[ r_{1-1} = \sqrt{\frac{3133.30}{101.23}} = 5.62 \]

Radius of Gyration (2-2)

I of 2 Pls - 18" x 7/16" = 26.00
I of 4 Pls - 18" x 11/16")

\[ 68.01 \times 6.093^2 = 2455.00 \]
I of 2 Pls - 9 1/2" x 7/16") = .51
I of 2 Pls - 9 1/2" x 5/16")

\[ 14.25 \times 7.38^2 = 789.00 \]
I of 4 Ls - 6" x 4" x 1/2" = 73.60

\[ 19.00 \times 8.99^2 = 1540.00 \]

Total I = 4884.11

\[ r_{2-2} = \sqrt{\frac{4884.11}{101.23}} = 6.95 \]

S = 16000 - 70 \times L/r = 16000 - 70 \times 28 \times 12/5.62

S = 16000 - 4200 = 11800#

Allowed stress = 11800 - 11800/4 = 14750#

The stress due to the bending moment of the wind is

\[ S = \frac{M_c}{I} = \frac{3490000 \times 13}{4884.11} = 9270# \]
Hence the allowed stress for the dead load, wind load and impact is

$$14750 - 9270 = 5480\#$$

$$P = \frac{as}{(1 + \phi (L/r)^2)} = 101.23 \times \frac{5480}{(1 + 60^2/25000)}$$

$$P = 487500\#$$

**Final**

The section as tried is stronger than required; but this difference is small. Hence this section will be used for the columns EF, E'F', FG and F'G'.

EF and E'F' will be alternately in tension and compression according to the direction of the wind. The section as tried for F'G' is large enough to take care of this additional stress.

**D'E'**

The dead load from the weight in the column is 19000#

" " " " " bracing " " " " 7500#

Total weight = 26500#

<table>
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$$\frac{318375\#}{104800\#} = \frac{8}{10} \text{ of } 131000\#$$

Total = 423175#

**Preliminary**

The following section will be tried:

2 Pls - 18" x 7/16" = 15.72 sq. in.

4 Ls - 6" x 4" x 1/2" = 19.00 sq. in.

Total area = 34.72 sq. in.
Radius of Gyration \(_{1-1}\)_

\[
\begin{align*}
\text{I of 2 Pls} & \quad 18'' \times 7/16'' = 220.00 \\
\text{I of 2 Ls} & \quad 6'' \times 4'' \times 1/2'' = 25.20 \\
& \quad 19.00 \times 8.012 = 1225.00 \\
\text{Total I} & \quad = 1470.20 \\
\end{align*}
\]

\[
r = \sqrt{\frac{1470.20}{34.72}} = 6.52
\]

\[
S = 16000 - 70 \times \frac{L}{r} = 16000 - 70 \times 28 \times 12/6.52
\]

\[
S = 16000 - 3610 = 12390#
\]

Allowed stress = 12390# - 12390/4 = 15487#

\[
P = \frac{as}{(1 + \varphi (L/r)^2)} = 34.72 \times 15487/(1 + 51.7^2/25000)
\]

\[
P = 457000#
\]

**Final**

The section as tried is stronger than needed, but it consists of the minimum members as required by the specifications. Hence this section will be used for the columns D'E', CD, BC and AB.

As the columns are figured to take all the wind stress direct and bending, the bracing in FG' will take hardly any stress and will consist of 2'' x 2'' x 5/16'' angles.

As the columns are enclosed in the pier, the mass of masonry to resist the bending moment is great enough without any investigation.

The floor system in the towers will be the same as for the truss span.
PART IX

DESIGN of COUNTERWEIGHTS
The counterweights will consist of slag concrete blocks (1:2:4) around a steel frame.

The weight of the span, including the cables, is 225000#.

Each counterweight must therefore weigh 112500#.

The counterweight will be a slab 2'-6" thick, extending over the whole width of the towers, or 21'-0".

The plate shows the construction of the steel frame and the attachment of the guide rollers.

The upper part of the frame forms a pocket so that in case the span and the weights do not balance exactly, the weight of the latter can be varied. Ordinarily this pocket should be covered by a steel plate to prohibit rain from entering.

The weight of the steel frame will be about

\[ \text{Assumed weight of castings, rollers and equalizers} = 11000\# \]

\[ \text{Weight for concrete} = 101500\# \]

At 120# per cu. ft. 848 cu. ft. of concrete are required. Considering the pocket on the top, the slab will be 16' - 6" high.

The exact weight of concrete required and the exact height must be determined after the exact weight of the castings, rollers and equalizers is known.
PART X

GUIDES

and

LOCKING DEVICES

for

SPAN
GUIDES AND LOCKING DEVICES FOR SPAN

Fixed End

Expansion End

Locking Device

Roller

Guide

Locking Device

To Ever Co/mris

LocAz/Of Den'ce

15" Roller

Batten Plates

Guide

T45° of Floor Beam

3' 7"

Tower Columns

Locking Device

Roller

Guide

Locking Device

Guide

LocAz/Of

(P/D)BS

(LD)NG

LOCK/NG OEY/CE3

FOR

LocAz/Of

Den'ce

PJcf/e

//.
PART XI

OPERATION
POWER

The amount of power required is dependent upon the coefficient of friction in the journals of the main sheaves. The value of this coefficient is assumed as .05.

The three following cases are investigated:-

1. No wind action, balanced loads and a maximum velocity of 3 feet per second.

2. No wind acting, balanced loads and a maximum velocity of 4 feet per second.

3. Greatest assumed wind-pressure and a maximum velocity of 2 feet per second. This velocity will, it is assumed, lift the span 130 feet in 65 seconds.

Case No. 1

Load on journals = 448000#

Frictional resistance of journals = 448000 x .05 = 22400#

Velocity of axle in journal = .25 feet per second

Work of friction = 22400 x .25 = 5600 foot pounds

Horse power = 5600/550 = 10.2

Inertia: assume that in 15 feet the full velocity of 3 feet per second will be developed

Mass = 448000/322 = 13950#

Kinetic energy = (13900/2) x 3² = 62600 foot pounds

The average velocity is 1.5 feet per second. The time required for development, 10 seconds.

Energy expended per second = 62800/10 = 6280 foot pounds

Corresponding HP = 6280/550 = 11.4
For bending the cables at a velocity of 3 feet per second 2 HP approximately are required.

The total HP is then:
\[ 10.2 + 11.4 + 2 = 23.6 \text{ HP} \]

**Case No. 2**

The work of friction will be proportional to the velocity; then in this case the HP for friction will be
\[ 10.2 \times 4/3 = 13.6 \]

Inertia: assuming that in 15 feet the full velocity of 4 feet per second will be developed

Kinetic energy: \( (13900/2) \times 4^2 = 111000 \) foot pounds

The average velocity is 2 feet per second, the time required for development, 10 seconds

Energy expended per second: \( 111000/10 = 11100 \) ft pounds

Corresponding HP = \( 11100/550 = 20.2 \)

For bending the cables at a velocity of 4 feet per second 2.7 HP approximately are required.

The total HP is then:
\[ 13.6 + 20.2 + 2.7 = 36.5 \text{ HP} \]

**Case No. 3**

The HP to overcome friction will be \( 13.6/2 = 6.8 \)

Inertia: assuming that in 15 feet the full velocity of 2 feet per second will be developed

Kinetic energy = \( (13900/2) \times 2^2 = 27800 \) foot pounds

The average velocity during development is 1 foot per second and the time 10 seconds.
Energy expended per second = 27800/10 = 2780 foot pounds
Corresponding HP = 2280/550 = 4.2
For bending the cables at a velocity of 2 feet per second, 1.4 HP approximately are required.

Wind pressure: Total wind pressure on span = 62100#
Diameter of rollers = 15"
Diameter of axles = 5"
Velocity of axle = 5/15 x 2 = .67 feet per second
Coefficient of friction = .05
Frictional resistance = 62100 x .05 = 3150 #
Work of friction = 3150 x .67 = 2115 foot pounds
Corresponding HP = 2115/550 = 3.9

To this should be added the rolling friction or 1 HP for each roller making a total friction of 4 HP

The total HP is then:

\[
6.8 + 4.2 + 1.4 + 3.9 + 4 = 20.3 \text{ HP.}
\]

From the above one 25 HP motor should lift the span easily 100 feet in 50 seconds with a wind force being exerted of 30# per sq. ft.
DESIGN OF OPERATING CABLES

The horse power transmitted is 25, the velocity of the rope is 4 feet per second or 240 feet per minute, hence the driving force

\[ P = 25 \times \frac{33000}{240} = 3400\# \]

Therefore the tension in the cable due to its own weight and the dimensions of the curve in which it hangs on the driving side is \( T_1 = 6800\# \) and on the slack side, \( T_2 = 3400\# \)

Now \( T_1 = 0.7854 \, d^2 \, n \, Sw \)

where \( d \) = the diameter of the wires in inches
\( n \) = the number of wires in the cable
\( Sw \) = the allowed stress due to transmission.

Using 4 ropes of wires of .042" diameter,
\[ n = \frac{T_1}{4 \times 0.7854 \times d^2 \times 25000} \]
\[ n = \frac{6800}{4 \times 0.7854 \times 0.042^2 \times 25000} = 49. \]
\[ D = (49 \times 0.042/13) + 7 \times 0.042 = 1/2" \]

With wires of .042" diameter, the least diameter of the deflection pulleys and the drums must be 42". By using for these a 54" diameter, the stress in the cable due to the bending is

\[ S_b = 30000000 \times 0.042/0.54 = 23300\# \]

The stress due to centrifugal force is negligible in this case. Therefore 4 cables of 1/2" diameter each consisting of 50 wires of .042" diameter will be used.
ELEVATION
SHOWING
OPERATING CABLES
AND
EXTREME Positions OF BRIDGe

Plate 12.
PART XII

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and

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TYPICAL SPECIFICATIONS FOR THE
FABRICATION AND ERECTION OF
STEEL HIGHWAY BRIDGES.

PREPARED BY THE OFFICE OF
PUBLIC ROADS.
LETTER OF TRANSMITTAL.

U. S. DEPARTMENT OF AGRICULTURE,
Office of Public Roads,

Sir: I have the honor to transmit herewith a manuscript entitled "Typical Specifications for the Fabrication and Erection of Steel Highway Bridges," which has been prepared in this office with the view of furnishing a suitable guide for local highway officials in fixing requirements to which bridge structures must conform.

In the past many steel bridges have been very poorly constructed, and it is believed that lack of information on the part of highway officials concerning proper specifications for this class of work has been in a large measure responsible for the unsatisfactory results. In order, therefore, to make the specifications contained in this manuscript more readily available for the use of such officials, I respectfully request that the manuscript be published as Circular No. 100 of this office.

Respectfully,

L. W. Page,
Director.

Hon. D. F. Houston,
Secretary of Agriculture.
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TYPICAL SPECIFICATIONS FOR THE FABRICATION AND ERECTION OF STEEL HIGHWAY BRIDGES.

DESCRIPTION OF PROJECT.

The contract of which these specifications form a part contemplates the construction of a steel highway bridge over ........ in township, ........ County, State of .........

The abutments and piers to consist of ........ masonry.

The total length of the superstructure to be ........ feet.

The height above mean low water to be ........ feet.

The number of spans to be ...........

The width of roadway to be ........ feet.

The assumed loading to be class ............ (The loading classification contained in these specifications is to be followed.)

The floor to be of ..........., thickness ............ inches.

Paint
[First coat to be ............
[Second coat to be ............

The structure to be entirely completed on or before ..........., 19........

GENERAL.

The Engineer.

The term “engineer,” as hereinafter employed, shall be interpreted to mean the duly authorized representative of the officials legally responsible for the work to be performed under these specifications, and the decisions of the said engineer upon all questions herein left to his discretion shall be final and conclusive.
Engineer's General Drawings.

The general drawings which form a part of these specifications are on file at .......... and consist of—

Sheet No. 1, showing ......................................................
Sheet No. 2, showing ......................................................
Sheet No. 3, showing ......................................................
Sheet No. 4, showing ......................................................

They, together with these specifications, embody all the information which will be furnished for the guidance of contractors.

Contractor's General Drawings.

If general drawings are submitted by a contractor as a part of his proposal, they shall include all stress sheets and such details as are necessary to express the general intent of the proposal. They shall show all general dimensions, such as length of spans from center to center of end bearings, clear width of roadway, width from center to center of trusses, depth from gauge line to gauge line of chords, etc. They shall indicate the assumed loads, live and dead, on which the computations are based, the sections and sectional areas of all truss and lateral members, the sizes of rivets, the thickness of gusset plates, the sizes and arrangement of floor beams and joists and their connections, and the character and quality of the materials proposed for use in various parts of the structure.

Shop Drawings.

Upon the acceptance of a proposal and the execution of the contract, all working drawings required by the engineer shall be furnished by the contractor free of cost. No work shall be commenced or materials ordered until the working drawings have been approved by the engineer in writing.

Classification.

Two classes of bridges will be considered under these specifications—those which carry suburban or interurban electric cars and those which carry highway traffic only. The former will be designated as class A and the latter as class B bridges.

Types of bridges.

It is recommended that the type of bridge employed be selected as follows:

For spans up to 30 feet—rolled beams;
For spans from 30 to 40 feet—plate girders or rolled beams;
For spans from 40 to 80 feet—riveted low trusses or plate girders;
For spans from 80 to 200 feet—riveted high trusses; and
For spans over 200 feet—pin-connected high trusses.
Materials.
All parts of the superstructure except the floor shall be of rolled steel.

Length of Span.
In computing the stresses the length of span for the trusses or girders shall be taken as the distance from center to center of the end bearings; and for the floor beams, from center to center of the trusses.

Width of Roadway.
For class A bridges the clear distance between the center line of the car track and the nearest truss shall be not less than 7 feet, and on one side the clear distance between the center line of the car track and the truss shall be at least 12 feet. The width from center to center of trusses shall in no case be less than one-eighth of the span.

Head Room.
The clear head room for a width of 6 feet on each side of the center line of the bridge shall be not less than 15 feet.

Depth Ratios.
The ratio of depth to span shall be not less than the following

<table>
<thead>
<tr>
<th>Material</th>
<th>Ratio of Depth to Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>For rolled beams</td>
<td>1/20</td>
</tr>
<tr>
<td>For plate girders</td>
<td>1/12</td>
</tr>
<tr>
<td>For trusses</td>
<td>1/10</td>
</tr>
</tbody>
</table>

Dead Loads.
The assumed dead load shall be not less than the total weight of the completed structure. The following unit weights shall be used in computing the dead loads:

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (pounds per cubic foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>490</td>
</tr>
<tr>
<td>Concrete</td>
<td>150</td>
</tr>
<tr>
<td>Brick</td>
<td>150</td>
</tr>
<tr>
<td>Macadam</td>
<td>130</td>
</tr>
<tr>
<td>Asphalt</td>
<td>135</td>
</tr>
<tr>
<td>Sand or earth</td>
<td>100</td>
</tr>
<tr>
<td>Stone</td>
<td>160</td>
</tr>
<tr>
<td>Creosoted</td>
<td>5</td>
</tr>
<tr>
<td>Oak, untreated</td>
<td>4½</td>
</tr>
<tr>
<td>Pine, untreated</td>
<td>4</td>
</tr>
</tbody>
</table>

Live Loads.
Class A.—For the floor and its supports and for the trusses of spans less than 50 feet in length the live load shall be assumed as follows:

On each car track a concentrated load of 24 tons evenly divided between two axles, spaced 10 feet center to center with wheels spaced
5 feet center to center on axles, shall be assumed to occupy a width of 6 feet on each side of the center line; on the remaining floor surface, exclusive of sidewalks, a uniform load of 125 pounds per square foot and on sidewalks a uniform load of 100 pounds per square foot.

For the trusses of spans between 50 feet and 100 feet in length, a uniform load of 1,800 pounds per linear foot for each car track (assumed to occupy a width of 12 feet) and 100 pounds per square foot of remaining floor surface, including sidewalks, shall be assumed. For the trusses of spans greater than 100 feet in length, the live load per linear foot for each car track and per square foot of remaining floor surface may be reduced, respectively, 50 pounds and 2 pounds for each additional 10 feet of span, provided that in no case shall these loads be reduced below 1,200 pounds and 80 pounds, respectively.

All class A bridges shall be assumed as also subject to the loading specified for class B bridges.

Class B.—For the floor and its supports and for the trusses of spans less than 50 feet in length, the live load shall be assumed as follows:

On any part of the floor surface a concentrated load of 15 tons on two axles spaced 8 feet center to center, with wheels spaced 6 feet center to center on axles and two-thirds of the load on one axle, shall be assumed to occupy a space 16 feet in the direction of traffic by 12 feet at right angles to that direction. On the remaining floor surface, exclusive of sidewalks, a uniform load of 125 pounds per square foot, and on sidewalks a uniform load of 100 pounds per square foot, shall be assumed.

For the trusses of spans between 50 feet and 100 feet in length a uniformly distributed load of 100 pounds per square foot of floor surface shall be assumed. For the trusses of spans greater than 100 feet in length the uniform load per square foot may be reduced 2 pounds for each additional 10 feet of span, provided that in no case shall the assumed live load be less than 80 pounds per square foot of floor surface.

Distribution of Stresses Due to Concentrated Loads.

In considering the concentrated load under class A, each wheel load shall be assumed distributed over an area of floor surface 5 feet square. In considering the concentrated load under class B, each wheel load shall be assumed distributed as follows: For reinforced concrete floors, protected by a wearing surface, 3 feet in the direction of traffic by 5 feet at right angles to that direction; for wood floors at least 3 inches thick the distribution in the direction of traffic shall be neglected in designing the joists and the distribution at right angles to the direction of traffic shall be taken as 4 feet.

Wind Loads.

The top lateral bracing in deck bridges and the bottom lateral bracing in through bridges shall be designed to resist a lateral wind
load of 300 pounds per linear foot, and one-half of this shall be treated as a moving load.

The bottom lateral bracing in deck bridges and the top lateral bracing in through bridges shall be designed to resist a lateral wind load of 150 pounds per linear foot.

**Temperature Stresses.**

Provision shall be made for stresses due to a change in temperature of 150° F.

**Longitudinal Forces.**

For class A bridges provision shall be made for a longitudinal force equal to 20 per cent of the weight of the heaviest electric train which could reasonably be expected to come upon the bridge.

**Impact.**

The maximum live-load stress in each member shall be increased to provide for impact by an amount to be determined from the formula

\[ I = \frac{100 \cdot S}{L + 300}, \]

where

- \( I \) = impact or dynamic increment due to the effect of moving loads;
- \( S \) = computed live-load stress; and
- \( L \) = loaded length of bridge in feet which produces maximum live-load stress in the member under consideration.

**Centrifugal Force.**

When curved tracks occur on class A bridges, the centrifugal force produced by two cars coupled together moving at 50 miles an hour shall be considered as an additional live load in designing the lateral bracing.

**PROPORTIONS AND UNIT STRESSES.**

All members shall be so designed that the stresses coming upon them may be accurately computed, and shall be so proportioned that the sum of the maximum stresses produced by the loads herein specified shall not exceed the following amounts in pounds per square inch.

**Tension.**

Axial tension on net section—16,000.

**Compression.**

Axial compression on gross section—16,000 – 70 \( \frac{1}{r} \), where “1” is the length of the member in inches and “r” is the least radius of gyration of its cross section in inches.

For class A bridges no compression member shall have an unsupported length exceeding 100 times its least radius of gyration for main 98925°—Cir. 100—13—2
members, or 120 times its least radius of gyration for laterals. For class B bridges no compression member shall have an unsupported length exceeding 120 times its least radius of gyration for main members, or 140 times its least radius of gyration for laterals.

**Bending Stresses.**

Bending stresses on extreme fibers of rolled floor beams, joists, and girders, 12,500; on extreme fibers of built-up girders, 16,000; and on extreme fibers of pins, 20,000.

**Shearing.**

<table>
<thead>
<tr>
<th>Shear Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pins and shop-driven rivets</td>
<td>10,000</td>
</tr>
<tr>
<td>Field-driven rivets</td>
<td>7,500</td>
</tr>
<tr>
<td>Plate-girder webs (gross section)</td>
<td>9,000</td>
</tr>
</tbody>
</table>

**Bearing.**

<table>
<thead>
<tr>
<th>Bearing Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shop-driven rivets</td>
<td>20,000</td>
</tr>
<tr>
<td>Pins and field-driven rivets</td>
<td>15,000</td>
</tr>
<tr>
<td>Wall plates on concrete masonry (1:24:5)</td>
<td>500</td>
</tr>
<tr>
<td>Wall plates on stone masonry (ashlar)</td>
<td>500</td>
</tr>
<tr>
<td>Wall plates on stone masonry (rubble)</td>
<td>400</td>
</tr>
<tr>
<td>Expansion rollers (per lineal inch)</td>
<td>500d</td>
</tr>
</tbody>
</table>

$d$ is the diameter of the roller in inches.

**Alternate Stresses.**

Members subject to alternate tensile and compressive stresses shall be proportioned to resist each kind of stress, and each stress shall be considered as increased by an amount equal to eight-tenths of the smaller stress in determining the sectional area. The connections shall be proportioned for the arithmetical sum of the stresses.

**Combined Stresses.**

Members subject to a combination of direct and bending stresses shall be designed so that the greatest unit fiber stress shall not exceed the allowable unit stress for the member.

**Counters.**

Wherever the live and dead load stresses are opposite in character, only two-thirds of the dead-load stress shall be considered effective in counteracting the live-load stress.

For class A bridges counters shall be so provided and proportioned that an increase of 25 per cent in the specified live load would not increase the unit stress in any member more than 25 per cent.

**Net Section at Rivets.**

In proportioning tension members the diameter of the rivet holes shall be taken $\frac{3}{4}$ inch larger than the nominal diameter of the rivets.
Proportioning Plate Girders.

The flanges of plate girders shall be assumed to take all the bending moment, and the web shall be assumed to take all the shear. The compression and tension flanges shall have the same gross section.

THE FLOOR SYSTEM.

Floor Beams.

All floor beams shall be rolled or riveted steel girders and shall be rigidly connected to the trusses or side girders. They shall, when practicable, be placed at right angles to the direction of traffic.

Joists.

All joists shall be rolled or riveted steel girders and shall be rigidly fastened to the floor beams. When wood floors are used, the joists shall be riveted to the webs of floor beams by means of connection angles. The spacing of joists center to center shall be not greater than 3 feet.

Wood Floors.

Wood floors shall be constructed of first-quality timber of the kind specified by the engineer in writing or indicated on his drawings. For oak floors the minimum thickness of plank used shall be 2\(\frac{1}{2}\) inches, and for pine the minimum thickness shall be 3 inches. In no case shall the thickness of the floor plank be less than one-twelfth of the distance, center to center, between joists. All plank shall be laid with the heart side down at right angles to the direction of traffic. Spaces of approximately \(\frac{1}{4}\) inch shall be left between adjacent planks.

Wood floors shall be provided with a wheel guard on each side of the roadway. Wheel guards shall be constructed of timbers having a cross section of not less than 6 inches by 4 inches, spliced with 6-inch lap joints, and shall be securely bolted to the joists at intervals not to exceed 5 feet.

Concrete Floors.

Concrete floors shall be constructed true to the dimensions shown on the drawings and in accordance with the specifications herein-after following under the heading "Concrete masonry."

Drainage.

Adequate provision shall be made for draining all parts of the floor, and the water drained off shall go clear of all metal work.
Minimum Thickness of Metal.

The minimum thickness of metal used in any part of the structure, except that used for fillers and other minor parts, shall be \( \frac{5}{8} \) inch. For class B bridges, however, this specification may be modified by the engineer in writing to permit the use of standard channels and I-beams having web thicknesses of less than \( \frac{5}{8} \) inch, provided that the modification is made as a supplementary clause to these specifications before the contract is awarded or is indicated on the engineer’s general drawings.

Camber.

Truss spans shall be given a camber by making the horizontal projection of the top chord longer than the bottom chord by \( \frac{1}{16} \) inch for each 10 feet of span.

Connections.

All connections shall be designed to develop the full strength of the connecting members and shall be of the character indicated on the engineer’s drawings or by the engineer in writing.

Angles subject to tensile stress shall be connected by both legs, otherwise only the section of the leg actually connected will be considered effective.

The neutral axes of connecting members shall meet in a point.

Rivets and Rivet Spacing.

Diameter.—Rivets shall be either \( \frac{3}{8} \) inch, \( \frac{1}{2} \) inch, or \( \frac{5}{8} \) inch in diameter, except when used in minor parts.

Pitch.—The maximum pitch in the line of stress shall not exceed 6 inches or 16 times the thickness of the thinnest outside plate. The minimum pitch shall not be less than 3 inches for \( \frac{3}{8} \)-inch rivets, 2\( \frac{1}{2} \) inches for \( \frac{1}{2} \)-inch rivets, and 2 inches for \( \frac{5}{8} \)-inch rivets. For plate girders the rivet spacing in the vertical legs of the flange angles shall be determined from the formula, \( p = \frac{rh}{s} \), where “\( p \)” is the pitch in inches, “\( r \)” the permissible stress in one rivet in pounds, “\( h \)” the distance between lines of rivets in inches, and “\( s \)” the maximum shear in pounds at the section under consideration. In no case, however, shall the pitch exceed 4 inches.

Edge Distance.—The minimum distances from center of rivet holes to the nearest edge shall be not less than \( 1\frac{3}{4} \) inches, \( 1\frac{1}{2} \) inches, and 1 inch for \( \frac{3}{8} \)-inch rivets, \( \frac{1}{2} \)-inch rivets, and \( \frac{5}{8} \)-inch rivets, respectively. The maximum distance from any edge to the center of rivet holes shall not exceed 8 times the thickness of the thinnest outside plate or 6 inches.
**Rivets in Flanges.**

Unless otherwise specified on the engineer's drawings or by the engineer in writing, $\frac{3}{8}$-inch rivets shall be used for all flanges less than 2$\frac{1}{2}$ inches wide, $\frac{3}{8}$-inch rivets for flanges between 2$\frac{1}{2}$ and 3$\frac{1}{2}$ inches, and $\frac{7}{16}$-inch rivets for all flanges over 3$\frac{1}{2}$ inches wide.

**Pitch at Ends.**

At the ends of built compression members the pitch of rivets in the line of stress shall not exceed 4 diameters for a distance equal to twice the maximum width of the member.

**Grip of Rivets.**

The grip of rivets shall, in general, be not greater than 4 diameters. When it is necessary to make the grip greater than 4 diameters, the allowable unit shearing strength shall be decreased 1 per cent for each $\frac{1}{4}$ inch of additional grip.

**Pin Connections.**

All pins shall be sufficiently long to furnish full bearing upon the turned body of the pin for all connecting parts. All pins shall be secured by chambered nuts and the screw ends shall be sufficiently long to admit of burring the threads after the nuts are set. No pin shall have a diameter less than three-fourths the width of the widest eyebar attached to it.

The several members attaching to a pin shall be so placed as to produce as little bending movement as practicable upon the pin; and they shall be held in place by means of filling rings.

All pin holes shall be so bored that when the pin is in place it shall be perpendicular to the axial plane of the truss, and each connecting member shall bear uniformly upon the pin. The diameter of the hole shall not be more than $\frac{3}{4}$ inch greater than that of the pin.

Pin holes shall be sufficiently reinforced to distribute the stresses properly over the full cross section of the members. Where "pin plates" are used, they shall contain a sufficient number of rivets to transmit their proportion of the bearing pressure, and at least one plate on each side shall extend not less than 6 inches beyond the edge of the nearest batten plate.

Riveted tension members having pin connections shall have a cross sectional area through each pin hole 25 per cent in excess of the net sectional area of the members. The sectional area of the metal between the pin hole and the end of the member shall not be less than 75 per cent of the sectional area through the pin hole.
Batten Plates and Lattice Bars.

The open sides of compression members shall be stayed by batten plates at the ends and by diagonal lattice bars at intermediate points. Batten plates shall be used at intermediate points when, for any reason, the latticing is interrupted.

Batten plates shall have a thickness of not less than $\frac{1}{4}$ inch nor one forty-fifth the distance between the lines of rivets connecting them to the flanges. They shall have a width parallel to the axis of the member not less than the maximum width of the member.

Lattice bars shall have a thickness not less than $\frac{1}{3}$ inch nor less than one forty-fifth their unsupported length. They shall be inclined to the axis of the member at an angle not less than 60° for single latticing nor less than 45° for double latticing. Double latticing shall be riveted at the intersection points.

The width of lattice bars shall not be less than 2 inches nor less than one-sixth the width of the member of which they form a part.

Abutting Ends.

Abutting ends in compression members shall be planed true to the angle of the joint and shall be sufficiently spliced on four sides to hold the connecting parts accurately in place. All joints in tension members shall be fully spliced.

Where splice plates are separated from the parts which they connect by intervening plates or fillers, the number of rivets on each side of the joint shall be increased by 33$\frac{1}{3}$ per cent of the number theoretically required for each intervening plate.

Eyebars.

The thickness of eyebars shall not be less than $\frac{1}{8}$ inch nor less than one-seventh the width of the bar. Heads of eyebars shall be formed by upsetting and forging, and never by welding. The heads shall be so proportioned as to develop the full strength of the bar.

Eyebars shall be perfectly straight at the time they are bored, and all bars which work together as one member shall be piled, clamped together, and bored in one operation.

The eyebars composing a member shall be so arranged that their surfaces are not in contact. The inclination of individual eyebars to the axis of the member which they compose shall not be greater than 1 inch in 16 feet.

Rods.

No rod shall be used which has a cross sectional area less than $\frac{3}{8}$ square inch.
All rods having screw ends shall be upset, previous to threading, so that the net sectional area at the root of the threads shall be greater by at least 17 per cent than the net sectional area of the rod.

**Compression Members.**

Compression members shall be so designed that any part or segment of a member will be proportionately as strong as the member taken as a whole.

No web shall have a thickness less than one-thirtieth the distance between the lines of rivets connecting it to the flanges, and no cover plate shall have a thickness less than one-fourtieth the distance between rivet lines.

Flanges of built members which have no cover plates shall have a thickness not less than one-twelfth the width of the outstanding leg.

**Lateral System.**

All lateral and portal bracing shall be made of shapes capable of resisting both compression and tension, and shall have riveted connections to the chords.

Laterals shall be as nearly in the plane of the axes of the chords as practicable. When eccentricity is unavoidable, however, provision shall be made for the maximum bending stresses which would be produced in the connections with the member fully loaded.

Portals for through bridges shall be as deep as the specified head room and depth of truss will permit. They shall consist of top and bottom struts and stiff intermediate bracing. All portals shall be provided with curved knee or corner braces.

Intermediate top struts in through bridges shall have a depth not less than that of the top chord, and, if the engineer so requires, they shall be provided with curved knee or corner braces.

End struts shall be provided at the ends of all bottom chords.

**Sway Bracing.**

All deck bridges shall be provided at each panel point with sway bracing made of shapes capable of resisting both tension and compression. The sway bracing shall extend the full depth of the trusses, and at the end of the trusses ample provision shall be made for transferring all wind loads to the piers or abutments.

**Hand Railing.**

A substantial hand railing not less than 3½ feet high and of appropriate design shall be constructed on the outside of footwalks, or, when footwalks are omitted, at the outside of the roadway.
Expansion and Contraction.

Provision shall be made for all bridge structures to change in length owing to temperature changes at least \( \frac{1}{8} \) inch for each 10 feet of span, and joints shall be provided at such points in the floor and pavement as may be indicated on the drawings furnished or approved by the engineer.

Expansion Bearings.

For all beam and girder bridges expansion bearings shall be designed for motion to take place by sliding. For all truss bridges the expansion bearings shall preferably be provided with rollers or rockers, though for spans less than 80 feet in length the engineer may, in his discretion, permit the use of sliding bearings. All rollers or rockers shall have a diameter of at least 3 inches. In all cases the bearings shall be so designed that motion can take place in a longitudinal direction only, and shall be so placed at the time the bridge is erected that the shoe or bolster will occupy a central position on the bearing at the atmospheric temperature specified by the engineer in writing.

Shoes or Bolsters.

Shoes or bolsters shall be so designed as to distribute the load over the entire bearing, and shall be securely stayed against lateral or upward motion by anchor bolts. Fixed bearings shall be rigidly anchored to the masonry.

Bedplates.

Bedplates shall be designed to distribute the load over an area sufficiently great to keep the pressure upon the masonry within the hereinbefore specified limits. All bed and bearing plates on masonry shall be set on sheet lead not less than \( \frac{1}{4} \) inch thick and the same size as the plate.

WORKMANSHIP.

General.

All parts entering into the structure shall be constructed in accordance with approved drawings. The workmanship and finish shall equal the best practice in modern highway-bridge construction.

Due regard shall be given the appearance of the finished structure, and any part of the structure which, in the opinion of the engineer, does not present a neat and sightly appearance shall be repaired or replaced as he may direct. The edges of sheared steel plates in main members shall be carefully faced or planed to remove defects caused by shearing.
Forging and Annealing.

All forging must be done while the steel is at red heat, and all forged parts must be annealed by heating the steel to a uniform dark-red heat and permitting it to cool slowly. No welds will be allowed.

Rivet Holes and Rivets.

In punching rivet holes the diameter of the punch used shall not be more than \( \frac{1}{16} \) inch greater than the diameter of the rivet. Material more than \( \frac{3}{4} \) inch thick shall be subpunched and reamed or drilled from the solid.

Rivet holes shall be so punched as to match accurately when the parts are assembled. Poor matching of rivet holes will constitute a cause for rejection.

Rivets shall be driven by mechanical power wherever practicable. All rivets must have neatly capped full heads and grip the assembled parts firmly. Tightening loose rivets by recupping and calking will not be allowed. All loose, burned, or otherwise defective rivets shall be removed and replaced.

Riveted members shall have their contact surfaces painted, and shall be pinned up and firmly drawn together with bolts before riveting is commenced.

Pins and Rollers.

Pins and rollers shall be accurately turned to gauge and entirely free from flaws of any character.

MATERIALS.

STEEL.

Process of Manufacture.

The steel shall be made by the open-hearth process.

Chemical Composition.

The steel shall be uniform in character and shall conform to the following requirements as to chemical composition:

<table>
<thead>
<tr>
<th>Elements considered.</th>
<th>Structural steel</th>
<th>Rivet steel</th>
<th>Steel castings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phosphorus, maximum per cent:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acid</td>
<td>0.06</td>
<td>0.04</td>
<td>0.08</td>
</tr>
<tr>
<td>Basic</td>
<td>0.04</td>
<td>0.04</td>
<td>0.05</td>
</tr>
<tr>
<td>Sulphur, maximum per cent</td>
<td>0.03</td>
<td>0.04</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Ladle Analyses.

An analysis shall be made by the manufacturer from a ladle test ingot taken during the pouring of each melt, to determine the percentages of carbon, manganese, phosphorus, and sulphur, and a record of this analysis shall be furnished the engineer free of cost.
Check Analyses.

If required by the engineer, a check analysis shall be made from finished material representing each melt. The percentages of phosphorus and sulphur shown by the check analysis may exceed the percentages specified above by 25 per cent.

Physical Properties and Tests.

The tensile strength, elastic limit, and ductility shall be determined by testing samples cut from the finished material after rolling. The samples shall be not less than 12 inches long and shall have a uniform sectional area of not less than \( \frac{1}{2} \) square inch.

Structural Steel.

Structural steel shall have an ultimate strength of not less than 60,000 pounds, and not more than 70,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength, and a minimum elongation of 20 per cent in 8 inches. For eyebar material not over 20 per cent shall show a tensile strength of less than 62,000 pounds or more than 68,000 pounds per square inch. Steel for pins may have a minimum elongation of 15 per cent. All structural steel shall withstand bending cold without cracking through 180° around a pin the diameter of which is one and one-half times the thickness of the specimen. Full size material for eyebars shall withstand bending cold without cracking through 180° around a pin the diameter of which is equal to the thickness of the material.

Pins over 7 inches in diameter shall be forged. Blooms for pins shall have at least three times the sectional area of the finished pin.

Rivet Steel.

Rivet steel shall have an ultimate strength of not less than 48,000 pounds and not more than 58,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength, and an elongation of not less than 26 per cent in 8 inches. All structural steel shall withstand bending cold without cracking through 180° so that the parts on each side of the bend come into actual contact throughout their length. When nicked and bent around a bar of their own diameter, they shall break gradually and give a fine, uniform, silky fracture.

Steel Castings.

Steel castings shall be used for drawbridge wheels, track segments, and gearing. They shall be true to form and dimensions, uniform in character, free from injurious blowholes or other defects, and finished in workmanlike manner.
When specimens having a uniform sectional area of at least \( \frac{1}{2} \) square inch for at least 2 inches of their length are tested, they shall show an ultimate strength of not less than 67,000 pounds per square inch, an elastic limit of one-half the ultimate strength, and an elongation of not less than 10 per cent in 2 inches.

**TIMBER.**

All timber used for purposes other than constructing temporary forms and false work shall be of the kind specified in writing by the engineer and of the first quality of the kind specified. It shall be free from all defects which would impair its strength or durability.

**OTHER MATERIALS.**

All other materials shall be of the kind and quality specified on the drawings or in writing by the engineer, or, in case of concrete masonry, as hereinafter specified.

**PAINTING.**

*Shop Painting.*

Before leaving the shop all steelwork shall be thoroughly cleaned and given one complete coat of such paint as is specified in writing by the engineer. Machine-finished surfaces shall be coated with white lead and tallow before being shipped or placed in the open air.

*Inaccessible Surfaces.*

All surfaces which are not accessible for painting after erection or after being riveted together shall be given before erection or before being riveted two coats of pure red lead and boiled linseed oil mixed in the proportion of 18 pounds of lead to 1 gallon of oil.

*Painting After Erection.*

After erection all metal work shall be thoroughly cleaned of mud, grease, or other objectionable material and evenly painted with two coats of paint of the kind and colors specified by the engineer. Linseed oil shall be used as the vehicle in mixing the paint for each of these coats and the separate coats shall be of distinctly different shades of color.

All recesses which might retain water shall be filled with thick paint or some waterproof material before the final painting, and the first coat shall be allowed to become thoroughly dry before the second coat is applied.

No painting shall be done in wet or freezing weather.
CONCRETE MASONRY.

Plans and Drawings.

All concrete masonry shall be built to conform with the lines and dimensions shown on the plans and drawings furnished or approved by the engineer in charge and which are hereby made a part of these specifications. In cases of discrepancies between figured dimensions and scale the figured dimensions are to govern.

Concrete.

The concrete shall be of the character and mixed in the proportions indicated on the plans or as may be indicated in writing by the engineer in charge, or as hereinafter specified. All concrete shall be prepared and placed in strict accordance with the following specifications and plans and the instructions of the engineer under them.

Cement.

The cement shall be of some standard brand of Portland cement satisfactory to the engineer in charge. No cement shall be used which, when tested, fails to conform to the United States Government specifications for Portland cement as contained in Circular 33 of the Bureau of Standards. Cement shall be delivered in sacks of 94 pounds net weight, and each sack shall be considered as having a volume of 1 cubic foot. Cement which contains lumps or has been damaged in any way by exposure to the weather or by other cause shall be rejected.

Sand.

The sand shall consist of dry, clean, quartz grains and shall not contain more than 5 per cent of clay, loam, or other foreign materials. The grains shall be well graded and of such size that all will pass a 1/4-inch mesh screen and not more than 20 per cent will pass a No. 50 sieve.

Coarse Aggregate.

The coarse aggregate may consist of either broken stone or gravel. Stone shall be sound, hard, and tough, broken to the sizes hereinafter specified, and when used shall be free from foreign material. No weathered or disintegrated material shall be used. Gravel shall be composed of hard, sound, durable particles of stone, thoroughly clean and well graded in size between the limits specified below.

Classes A, B, and C.

Unless otherwise especially provided, there shall be three classes of concrete, known as class A, class B, and class C.

Class A concrete shall consist (by volume) of 1 part of cement, 2 parts of sand, 4 parts of coarse aggregate, and water. All of the
coarse aggregate shall be retained on a ¼-inch mesh screen and shall pass a 1-inch mesh screen. Not more than 75 per cent shall be retained on a ¼-inch mesh screen, and not more than 75 per cent shall pass such a screen.

Class B concrete shall consist (by volume) of 1 part of cement, 2½ parts of sand, 5 parts of coarse aggregate, and water. All of the coarse aggregate shall be retained on a ¼-inch mesh screen and shall pass a 1½-inch mesh screen. Not more than 75 per cent shall be retained on a ¼-inch mesh screen, and not more than 75 per cent shall pass such a screen.

Class C concrete shall consist (by volume) of 1 part of cement, 3 parts of sand, 6 parts of coarse aggregate, and water. All of the coarse aggregate shall be retained on a ¼-inch mesh screen and shall pass a 2½-inch mesh screen. Not more than 75 per cent shall be retained on a 1½-inch mesh screen, and not more than 75 per cent shall pass such a screen.

Mixing.

The cement and sand shall first be thoroughly mixed dry, in the proportions specified, on a proper mixing platform. Sufficient clean water shall then be admixed to produce a pasty mortar. To the mortar thus prepared shall be added the proper proportion of coarse aggregate, previously drenched with water, and the whole shall be mixed until every particle of the coarse aggregate is thoroughly coated with the mortar. Instead of the above method a mechanical mixer satisfactory to the engineer may be employed.

Size of Batch.

Concrete shall be mixed in batches of such size that the entire batch may be placed in the forms by the force employed within 45 minutes from the time that the first water is applied. No concrete is to be prepared from mortar which has taken an initial set and would require retempering.

Placing.

All concrete shall be carefully deposited in place and never allowed to fall from a height greater than 5 feet. Concrete shall never be deposited in running water, nor in still water, except under the direction of an engineer skilled and experienced in that special work.

As fast as concrete is put into place it shall be thoroughly tamped in layers not more than 6 inches thick, and the portion next to the forms shall be troweled by using a spade, or by other means, to bring the mortar into thorough contact with the forms.

Concrete shall not be deposited when the temperature of any of the materials composing it is below 35° F.; and if during the progress of the work freezing temperature threatens or is predicted by the
United States Weather Bureau, proper precautions shall be taken to protect from freezing all concrete laid within the four preceding days.

Forms.

Forms shall be so constructed as to continue rigidly in place during and after depositing and tamping the concrete. If during the placing of the concrete the forms show signs of bulging or sagging at any point, that portion of the concrete causing the distortion shall be immediately removed and the forms properly supported before continuing the work. The amount of concrete to be removed shall be determined by the engineer, and the contractor shall receive no extra compensation on account of the extra work thus occasioned. Forms for exposed surfaces shall be constructed of dressed lumber.

All forms shall be left in place not less than 36 hours and all supporting forms not less than 10 days after the concrete has been deposited. These periods may be increased at the discretion of the engineer in charge.

It is understood that all prices for concrete masonry shall include furnishing all materials and properly constructing all necessary forms.

Joints.

When the work of laying concrete is to be interrupted for a period greater than one hour and there are no reinforcing rods projecting, provision for a joint shall be made in the following manner: Square timbers 8 by 8 inches, or some other suitable size approved by the engineer, shall be bedded in the concrete throughout the length of the course for one-half their thickness and allowed to remain until the concrete has taken its initial set. When the work of laying concrete is resumed, the timbers shall be removed and the surface thoroughly wet. No joints will be permitted in reinforced-concrete beams, and in floor slabs the joints shall be vertical and parallel to the main reinforcing bars.

Finish.

Forms covering surfaces of the concrete masonry which are to be exposed shall be removed immediately after the expiration of the period of time necessary for such forms to remain in place, as fixed by the engineer, and all crevices which may appear shall be filled with 1:2 cement mortar. These surfaces shall then be finished with 1:2 cement mortar and a wooden float, so as to present a smooth, neat appearance.

Reinforced Concrete.

All reinforced arches, beams, floors, parapets, guard rails, and all concrete masonry measuring less than 9 inches in thickness shall be
made of class A concrete, unless otherwise specified on the drawings or directed by the engineer in writing.

**Abutments and Wing Walls.**

Unless otherwise specified on the drawings or in writing by the engineer, class B concrete shall be used for all abutments and wing walls the thickness of which is not less than 9 inches.

**Footings and Cut-off Walls.**

Class C concrete shall be used for all footings and cut-off walls, unless otherwise specified on the plans or directed in writing by the engineer.

**Steel for Reinforced Concrete.**

Unless otherwise specified on the drawings, all reinforcing steel shall consist of bars which have been deformed in some approved manner. No plain bars will be permitted except as shown on the drawings or directed in writing by the engineer.

The steel bars shall have the net sectional area and shall be placed in the exact positions indicated on the drawings.

Unless otherwise specified on the drawings or in writing by the engineer, all reinforcing bars shall be of medium steel having an elastic limit of not less than 35,000 pounds per square inch, and shall be sufficiently malleable to withstand bending cold with a radius equal to twice the diameter or thickness of the bar through 180° without fracture.

When placed in the concrete, the reinforcing steel shall be free from grease, dirt, and rust, and it shall be the duty of the contractor to provide means for properly cleaning the steel.

Thorough contact of the concrete with every portion of the surface of the steel shall be obtained.

**Splicing Reinforcing Bars.**

Unless otherwise specified on the drawings or in writing by the engineer, necessary splices in reinforcing bars shall be effected by overlapping the ends of the bars a distance equal to 40 times their thickness or diameter.

**Orders for Materials.**

All materials used in connection with the work being done under these specifications shall be purchased especially for that work. The contractor shall furnish the engineer with complete copies of all orders for materials, and shall make all orders subject to the engineer's approval of the materials.
Acceptance of Material or Work.

The engineer or his representative shall stamp each accepted piece or parcel with a private mark, and any material not so stamped may be rejected at any stage of the work. The engineer's acceptance, however, does not relieve the contractor from responsibility for faulty material or workmanship and wherever such faulty material or workmanship is discovered it shall be repaired or removed and replaced, as the engineer may direct, by the contractor at his own expense.

Shop Inspection.

The contractor shall furnish the engineer or his representative all facilities for testing materials and workmanship at the shop where the material is fabricated, and shall notify the engineer well in advance of beginning the shop work.

Mill Inspection.

The contractor shall furnish all facilities for testing the weight and quality of all material at the mill where it is manufactured. He shall provide, free of cost, a suitable testing machine for making the tests and such test specimens as the engineer or his representative may require.

ERECCTION.

Contractor's Responsibilities.

Unless otherwise specified in writing by the contracting parties, the contractor shall furnish all labor, tools, machinery, and materials for erecting the bridge complete in place and ready for traffic, in accordance with these specifications and the plans furnished or approved by the engineer.

The contractor shall do all necessary hauling, set all stone or anchor bolts, remove existing structures when necessary, and perform all other incidental work for which express provision has not been made.

The contractor shall so conduct all his operations as not to interfere with the work of other contractors or close any thoroughfare by land or water except by written consent of the engineer.

The contractor shall assume all risks of damage or accident to persons or property prior to the final acceptance of the finished structure.

The contractor shall remove all false work, piling, and other obstructions produced by his operations, and shall perform any additional work necessary to produce a sightly appearance in the immediate vicinity of the structure.
Final Test.

Before final acceptance the engineer may, at his discretion, make a thorough test of the structure by passing over it the specified loads or their equivalents, or by resting the maximum load upon the structure for 12 hours.

If any part of the structure fails to return to its original position after the load is removed, the contractor shall make such alterations as are necessary to enable the structure to stand the test without permanent change of any of its parts.

SUPPLEMENTARY.

The following supplementary clauses shall also apply to the work included in the contract of which these specifications form a part:

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<tr>
<td>Wind Stresses</td>
<td>43,45</td>
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#### Upper Chord
- **Design**: 52
- **Slope**: 34
- **Weight**: 74

#### Unit Stresses
- 7,8,113

#### Velocity
- 11,16,107

#### Verticals
- **Design**: 58
- **Stresses**: 36,39,45
- **Weight**: 74

#### Walkways
- 9

#### Web Stresses
- 36,39,45

#### Weight
- **Bottom Chord**: 73
- **Counterweights**: 104
- **Diagonals**: 75
- **End Posts**: 74
- **Floor Beams**: 33
- **Laterals**: 75
- **Machinery, Machinery House, Supports**: 12,33,79
- **Planking**: 33
- **Portal**: 76
Weight (continued)

<table>
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<th>Item</th>
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<td>Steel in Bridge</td>
<td>33, 73, 78</td>
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<td>Stringers</td>
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<td>Top Chord</td>
<td>74</td>
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<tr>
<td>Total of Bridge</td>
<td>33, 79</td>
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<tr>
<td>Track</td>
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<td>Transverse Frames</td>
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<tr>
<td>Wheelguards</td>
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<td>Width of Span</td>
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<tr>
<td>Wind Stresses</td>
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