An Analysis

of the

Delta Mills Bridge

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

Charles B. Gates

A Candidate for the Degree of

Bachelor of Science

June 1943

THESIS

148263

This thesis is respectfully dedicated to my parents, without whose help and consideration it would never have been written.

TABLE OF CONTENTS

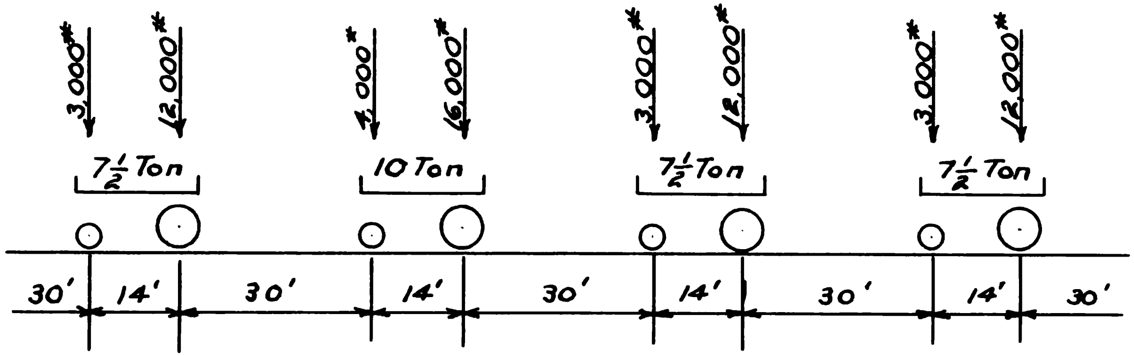
INTRODUCTION

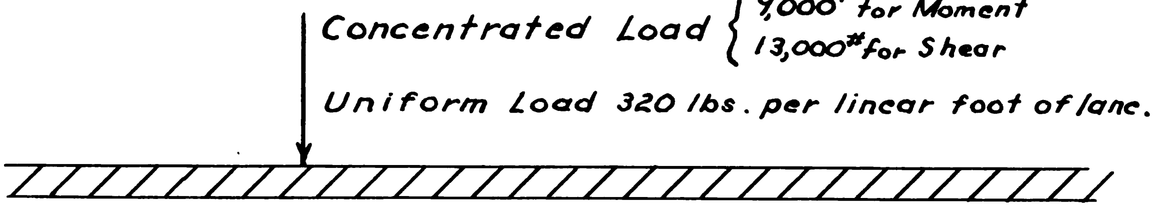
The bridge to be analyzed in this report is an old highway bridge of the through truss type. It was put up in 1891, by the R. L. Wheaton Company of Chicago, Illinois. It is located at Delta Mills, Michigan, which is a small community located about six miles to the Northwest of Lansing. The greater part of the load travelling over the bridge is composed of passenger vehicle traffic, for which the structure was undoubtedly designed. However, since the World War II, an increasing amount of heavier traffic has been using it. Gravel and equipment trucks have been re-routed over this road, and the structure gives evidence of failing.

It is a two-span bridge, and has a span length of 120 feet. The span is divided into eight panels of fifteen feet each, and the height is also fifteen feet. The roadway is twelve feet wide, and the total structure is fourteen feet wide. It is a pin-connected truss, and is supported on rock-masonary piers founded in the Grand River. The bridge has not been adequately protected against the weathering effects, and is rusting and scaling badly.

The object of this thesis is to determine whether-or-not the bridge will safely carry a loading as prescribed by the American Association of State Highway Officials as a H-10 loading. This loading is caused by a train of trucks of the 10 ton type or an equivalent loading. It will be determined just what loading is allowable if this is not safe. Recommendations will also be made for continuing the useful life of the structure, to prevent further failure.

H-10 LOADING (AASHO)





Part I

CHECK OF STRINGER SIZE AND SPACING

In this part of the check analysis, it is to be determined whether or not the stringers are of adequate size and spacing to resist the forces set up by the H-10 loading as prescribed by the American Association of State Highway Officials' Specifications.

1. Wheel Load. The stringers are 7 x 3-3/4 @ 17.5# per foot sections, and are spaced 2 feet center-to-center. The magnitude of the wheel load is equal to 8,000# per foot of stringer(20,000# x 0.4), which is reduced by the factor 2/6 because of the distribution to adjacent stringers. The wheel load is then 2/6 x 8,000 or 2670# per foot.

2. Impact. The formula that is used to determine the impact ratio is:

$$\text{Impact ratio} = \frac{50}{L - 125}$$

where (L) is the value of the length of the stringer, in feet. The impact ratio is never taken less than 30%(AASHO). Substituting the value of 15 feet for (L), the ratio comes out 35.714%.

3. Dead Load. The dead load consists of the weight of the stringer plus the weight of 3 inches of concrete roadway slab. A weight of 50# per foot is arbitrarily assumed as the weight of the stringer, and from this the needed size of stringer will be determined, which will later be corrected for the actual weight. The 3 inches of concrete is equivalent to a load of 37.5# per square foot(150 x $\frac{1}{4}$, assuming that the concrete weighs 150# per cubic foot). This makes the total dead load on the stringer 125# per foot.

4. Moments. The bending moment due to the concentrated wheel load is:

$$B.M. = \frac{8000}{2} \times \frac{2}{6} \times 7.5$$
$$= 10,000 \text{ Foot-pounds}$$

The bending moment due to impact equals 10,000 x 0.35714, or 3571.4 foot-pounds. The bending moment due to the dead load is $125 \times \frac{1}{2} \times 7.5^2$ which equals 3515.625 foot-pounds. The stringer must be designed to resist the total of these three moments, which is 17,087 foot-pounds.

5. Section Modulus. The formula used for determining the modulus is:

$$\text{Section modulus} = \frac{\text{Total Bending Moment}}{\text{Allowable stress in Steel } (18{,}000\#/in^2)}$$
$$= \frac{17{,}087 \times 12}{18{,}000}$$
$$= 11.38 \text{ Inches}^3$$

The actual section modulus of the 7 I 17.5 is 11.1 inches^3, which is not sufficient for the assumed conditions. However, by inserting the correct weight of 17.5# per foot in place of the assumed 50# per foot, the following results are obtained:

Dead load moment = 2601.56 Foot-pounds

Total moment = 16,172.963 Foot-pounds

Section modulus = 10.78 Inches^3

Since the actual section modulus exceeds the modulus required, the stringer is satisfactory at this point.

6. Web Area. The compression flange is supported laterally by the concrete so that the full unit stress is considered to be developed. The maximum shear developed(live load and impact) is equal to

$$(8{,}000 + 2{,}000 \times \frac{1}{15}) \times \frac{1}{3} \times 1.357$$
$$= 3{,}731.75 \text{ Pounds}$$

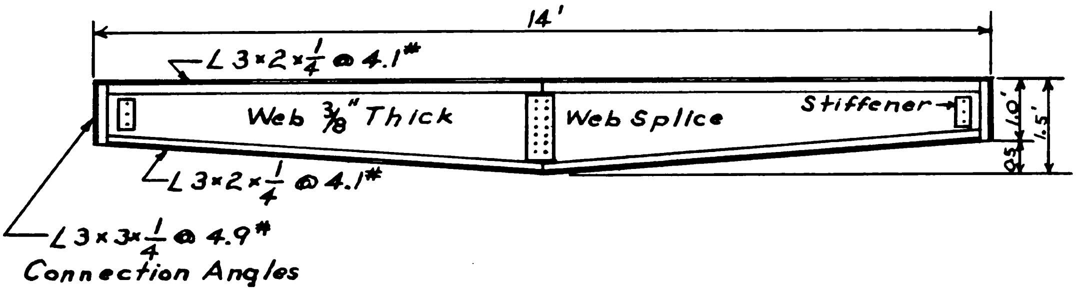
The dead load shear is equal to 694 pounds = (37.5 x 2 + 17.5) x $7\frac{1}{2}$. Since the web area necessary is found by dividing the total shear by the allowable shearing stress(11,000#/in^2), the required web area is found to be 0.401 square inches. The available web area is 5-3/8 x 0.345 or 1.854 square inches, which is more than adequate for the loading.

7. Summary. In summary, the stringers are of sufficient size and spacing to withstand the loading of a 10 ton truck or the equivalent H-10 loading.

Part II

CHECK OF FLOOR BEAMS

The floor beams are built-up members constructed as shown in the sketch. They must be checked for section modulus, web area, and adequacy of connections and splice plates.

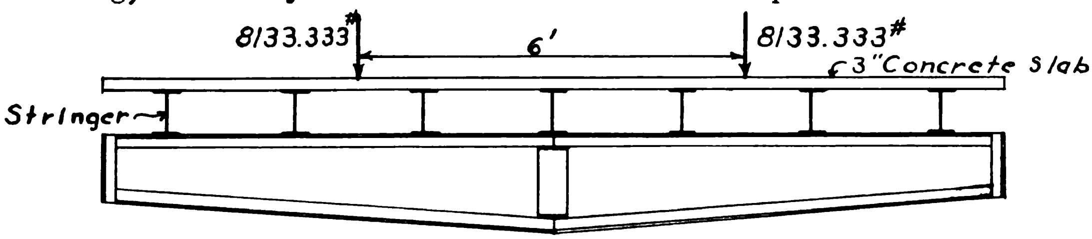


1. Depth. The minimum depth to be used is equal to 1/25 the length of the floor beam or 1/25 x 14 x 12 = 6.72 inches (AASHO).

2. Loading. The maximum concentration on the floor beams at the middle stringer comes from the 10 ton truck when one 8,000# wheel

load is directly over the floor beam and a 2,000# load 14 feet away. The concentration equals 8,000# plus 2,000# x 1/15 or 8, 133.33 pounds.

3. Bending Moment. The maximum bending moment occurs at this point when similar loads are placed as near it as the axle spacing of 6 feet and a clearance of 3 feet will permit. The result is a central loading, with only one train of trucks on the span.



The bending moment at the critical section is the same as if the wheel loads were applied to the floor beams directly, and they need not be brought down through the stringers. The bending moment due to the live load is equal to 32,533.33 foot-pounds(8133.33 x 4).

The bending moment due to impact is obtained by using the following formula:

$$\text{Impact ratio} = \frac{1}{3} \times \frac{(2000 - L)}{(1600 + L)}$$; (L) being the length of the floor beam. The ratio is never taken less than 30%(AASHO), and equals 34.562%. The impact moment is 32,533.33 x 0.34562 or 11,244.170 foot-pounds.

Assuming that the floor beam weighs 200# per foot(to determine the approximate size of floor beam needed), the other portion of the dead load on the floor beam is due to the 3 inches of concrete roadway. The equivalent dead load is

$$\left(\frac{150}{4} + 8.75\right) \times 15 + 200$$

$$= 893.75 \text{ Pounds}$$

The bending moment due to the dead load is found as follows:

$$\text{Bending moment} = 7\left(\frac{893.75}{2}\right) \times 14 - (893.73) \times \frac{7}{2}$$
$$= 21{,}896.875 \text{ Foot-pounds}$$

The floor beam must be designed to resist the total bending moment of 65,674.377 foot-pounds.

4. Section Modulus. The section modulus required to resist the above moment is 43.783 inches^3, which requires a $12 \times 5\frac{1}{4}$ I @ 40.8# per foot section. Since the actual beam is a built up section, the section modulus is calculated according to the formula:

Section modulus = $\frac{I}{c}$; where (I) is the moment of inertia of the cross section, and (c) is the distance from the neutral axis to the extreme fiber of the member in inches. The moment of inertia of the web plate is equal to 1/12 bd^3 and is 182.6 inches^4. The moment of inertia of the flange angles is obtained by use of the transfer formula with the distance between axes being 8.01 inches, and is equal to 309.802 inches^4. The distance (c) is equal to 9 inches. The section modulus available then equals (182.25 + 309.802) ÷ 9 or 54.672 inches^3, which is greater than the required modulus. The floor beam is thus safely designed at the center. Since the bending moment becomes smaller as the end of the beam is approached, the section may be reduced and still be safely designed.

5. Rivet spacing. The first check will be made of the rivets in the flange angles. The object is to determine the pitches of 7/8" rivets in a built-up floor beam with the following loadings: a uniform load of 729.291# per foot, two truck wheel loads of 8,133.33#, and a 36% impact. The wheel loads are spaced 6 feet apart.

The value of one rivet in bearing in the 3/8 inch plate is 7875#, (7/8 x 3/8 x 24,000-assuming an allowable stress of 24000#/in^2).

The distance between lines of rivets is 11.0 inches for the first three feet of the floor beam. The decrease in the dead load shear per panel is

$$729.291 \times \frac{11}{12} = 670 \text{ Pounds}$$

The decrease in the live load shear per panel is

$$\frac{8{,}133.33 \times 1.36 \times 2 \times 11}{14 \times 12}$$

$$= 1450 \text{ Pounds}$$

The vertical component of the live load is

$$\frac{8{,}133.33 \times 1.36}{14} \times \frac{2 \times 4.1}{12} \times \frac{693.75}{12}$$

$$= 850 \text{ Pounds}$$

The gross moment of inertia of the beam at a point 3 feet out from the end of the beam is equal to the moment of inertia of the web plate plus that of the flange angles. The moment of inertia of the web is $1/12\ bd^3$, which equals 95.270 inches^4, and that of the flange is

$2(\ 1.1 + 1.19 \times 6.26^2\ ) = 95.466$ Inches^4, making the total moment of inertia 190.736 inches^4.

The horizontal shear is equal to 493.2# and is obtained from the formulas

$$\text{Shear increment} = \text{Total shear} - \frac{\text{Bending Moment}(\tan a + \tan b)}{\text{Distance between Rivet Lines}}$$

where (a) and (b) are the angles of inclination of the bottom and top flanges with the horizontal. The value of the horizontal shear is

$$\text{Shear} = \frac{\text{Shear} \times \text{Flange Moment of inertia}}{\text{Dist. between Rivet Lines} \times \text{Gross I}}$$

$$= \frac{(\ 11{,}050 - 240\ ) \times 95.466}{11 \times 190.736}$$

$$= 493.2 \text{ Pounds}$$

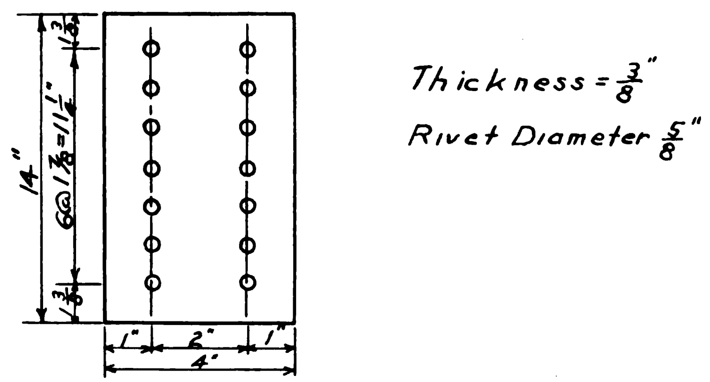
The resultant shear is equal to the sum of the squares of the component parts and equals 976 pounds. The pitch then required is found by dividing the allowable bearing by this resultant shear and equals 8 inches. The maximum pitch used is $4\frac{1}{2}$ inches for this type of member, and since the actual spacing is $3\frac{1}{2}$ inches, it is more than adequate.

The spacing is investigated at two other points, at $4\frac{1}{2}$ feet out from the end and at the mid-point, and the results are

Distance from End of Floor Beam	Required Spacing	Actual Spacing
3 Feet	4.5 Inches	3.5 Inches
4 "	4.5 "	4.0 "
7 "	4.5 "	4.5 "

The spacing in the bottom flanges is the same as that in the top flange, and both are sufficient for the loading.

6. Web Splice Plate. The plate is used at the mid-point of the floor beam to transmit the load from one web section to the other. The dimensions and rivet spacings are as shown in the sketch below.



The gross moment of inertia of the plate is 95.5 inches^4 as found by using the formula $1/12\ bd^3$. The moment of inertia of the holes in the plate is equal to 53.5 inches^4. The moment resisted by the web plate is equal to fI/c, where (f) is the allowable stress(16,000#/in^2), (I) is the net moment of inertia, and (c) is the distance from the neutral axis to the extreme fiber.

$$\text{Resisting moment} = \frac{16{,}000 \times 53.5}{7.25}$$

$$= 180{,}000 \text{ Inch-pounds}$$

The eccentricity is equal to the resisting moment divided by the summation of the vertical and horizontal distances to the rivets, squared. The summation of the vertical distances squared is 224, and of the horizontal distances 14×1^2 or 14. The eccentricity is then 180,000 ÷ 238 or 756. The horizontal component of the stress in the critical rivet is 756 x 6, which equals 4,536 pounds. The vertical component is

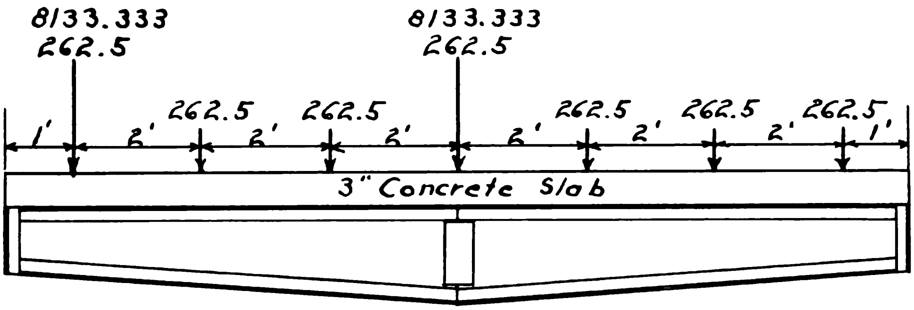
$$756 \times 4.5 + \frac{11{,}050}{4.5 \times 7} = 3753 \text{ Pounds, and the resultant}$$

stress is 5900 pounds, which is the actual stress that the most critically stressed rivet must resist. The allowable stress that the rivet is capable of withstanding is 7,031 pounds, assuming an allowable stress of 30,000# per square inch in bearing for double shear.

The web splice is of sufficient design to withstand the forces set up.

7. End Connection. The end connection is with the use of two $3 \times 3 \times \frac{1}{4}$ angle section, 12 inches long, which contain five rivets, spaced as shown; $2\frac{1}{4}$ inches between rivets and $1\frac{1}{2}$ inches for edge margin. The check is made to determine the spacing needed for the floor beam to resist the end reaction, using unit stresses of 11,000 in shear and 24,000 bearing(double shear), and using 7/8 rivets.

The maximum end reaction occurs when the truck load is placed as close to the left reaction or left end as is possible(one foot).



To obtain the magnitude of the reaction, take moments about the right end or

Left reaction = 16,480 Pounds

$$= \frac{8395.8 \times 13 + 262.5(11 + 9) + 8395.8 \times 7 + 262.5 \times 9 + 562.5 \times 98}{14}$$

The effective bearing length of one pair of angles is equal to 2 x 3 or 6 inches.. The required thickness is found by dividing the end reaction by the product of the bearing length and the allowable stress in bearing, and is equal to 0.115 inches. The actual flange or angle thickness is 0.25 inches or more than double the required 0.115 inches. The number of rivets required in the web legs of the angles is equal to the end reaction divided by the allowable stress in double shear, which is 5400 pounds, making a total of 3.03 needed in each angle. The actual number present is 5, which is an excess of about 2 rivets per angle.

The number of rivets required in the stiffener plate, as determined by bearing in the 3/8 inch plate, is 3000 x 3/8 x 7/8 or 7875# divided into the end reaction. This number is 2.14, and since 3.0 are used, the plate is safe.

This concludes the check analysis of the floor beams, and the structure is so far capable of withstanding the loading with a fairly high factor of safety.

Part III

LIVE LOAD ON THE TRUSSES

The live load for the trusses per lane is 320 pounds per linear foot of lane plus a single concentrated load of 13,000 pounds(AASHO).

1. Lanes. The number of lanes is 1.33 or 1 + 3/9.

2. Load per Panel. The live load per panel per truss is composed of the uniform loading and the concentrated load. The uniform load is

$$\frac{160 \times 1.33 \times 15}{2} = 3192 \text{ Pounds}$$

The concentrated load produces a load per panel per truss of 13,000 x 1.33/2 or 8645 pounds.

3. Impact Load. The impact ratio for the chord members is equal to

$$\frac{1}{3} \times \left(\frac{2000 - 120 \times 1.33/2}{1600 + 1200 \times 1.33/2}\right) = 26.7\%$$

Part IV

DEAD LOAD ON THE TRUSSES

The dead load brought down from each floor beam is equal to

$$(693.75 + 22) \times 7 = 5012 \text{ Pounds}$$

The weight of each truss, including bracing, per foot is approximately equal to the length in feet plus one-ninth the total load per foot from the floor system. The weight per panel is

$$120 \times 15 + \frac{5012 + \frac{(3192 + 8645)}{8} \times 1.267}{9}$$

$= 2550$ Pounds.

This makes the total dead load per panel 2550 + 5012 or 7562 pounds.

Part V

DEAD LOAD STRESSES

In this section, the computations will be made for a few members to illustrate the method of procedure, and the remaining values will be placed in tabular form.

1. Load Distribution. The dead load is assumed to be distributed between the top and bottom chords at a one-to-two ratio. The top chord is considered to take one-third of the total load per panel, and the bottom chord takes two-thirds. The magnitudes are 2520# per panel in the top, and 5040# per panel in the bottom chord.

2. Member (a). The vertical component of the stress in this member is equal to the reaction at the left end, since there is no other vertical member at this point. The reaction is equal to 7/2 x 5040 + 2520 or 26,460 pounds. Since the truss is one that is the same height as panel length, the angle at the supports or panel points is 45 degrees. The stress in (a) is then 26460 x 1.414, which is 37,500 pounds, compression.

3. Member (b). The stress in (b) is equal to the horizontal component of the stress in member (a), because these two members are the only members acting at this point that are capable of taking a horizontal loading. The stress then equals 24,460#, tension.

4. Member (c). Since the only load acting on the member (c) is the panel load of 5040 pounds, the stress in this member is 5040 pounds, tension.

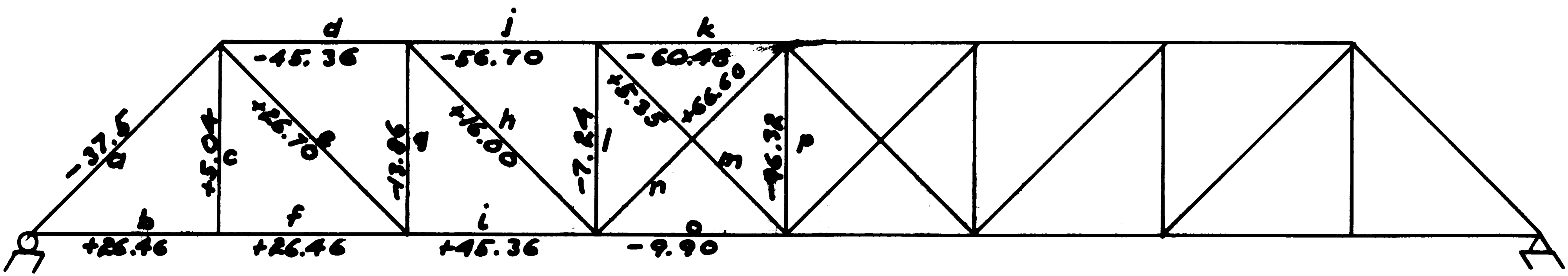
5. For the stresses in the remaining members, refer to the accompanying table of values on the following page.

Note: Member (m) is made up of two cylindrical rods with an unbraced length of 15 feet, and thus it cannot take any compression whatsoever. To take compression, the members (n) are provided and are called counters. These members do not directly take the compression, however. The force that would produce compression in the member (m) produces a tensile stress in the counters, and they are so designed to take all of this stress so that none of it will act on member (m).

DEAD LOAD STRESSES-KIPS

\+ Tension

\- Compression



8@15=120'

HEIGHT=15'

Table

of

DEAD LOAD STRESSES

MEMBER	MAGNITUDE OF STRESS		TYPE OF STRESS
a	37,500	Pounds	Compression
b	26,460	"	Tension
c	5,040	"	Tension
d	45,360	"	Compression
e	26,700	"	Tension
f	26,460	"	Tension
g	13,860	"	Compression
h	16,000	"	Tension
i	45,360	"	Tension
j	56,700	"	Compression
k	60,480	"	Compression
l	7,240	"	Compression
m	5,350	"	Tension
n	66,600	"	Tension
o	9,900	"	Compression
p	96,320	"	Compression

Part VI

LIVE LOAD STRESSES

The uniform live load of 3192 pounds per panel is treated the same as the dead load in calculating the stresses caused by it. One-third or 1064 pounds per panel is taken as acting on the top chord, and two-thirds or 2128 pounds on the bottom chord. The stresses caused by the moving live load of 8645 pounds will be determined by the use of load influence lines.

1. Uniform Live Load. Since the method of analysis of the stresses produced by this load is the same as for the dead load, it will not be necessary to repeat the procedure. The values of the stresses can be found in the table on the following page.

2. Moving Live Load. The method of influence load lines consists of placing the load of unity at various panel points and determining the stress resulting from this position in the member being investigated. The stress is then multiplied by the actual value of the moving load, 8645 pounds in our case, to determine the actual maximum stress.

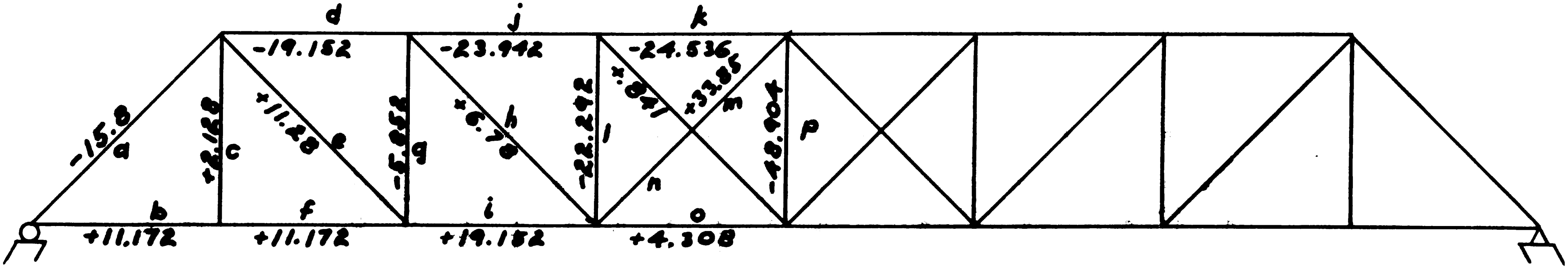
The influence line for member (a) is a sloping line going from zero at the right end to 12,224 pounds at the left end, assuming that load enters from the right.

This procedure is continued for each member in the truss and points of maximum stress and reversal of stress are noted. The maximum stress is that stress that must be used to determine the design stresses. The influence diagrams for the various members are as indicated in the following sketches and the stresses are indicated in the table.

LIVE LOAD STRESSES-KIPS

+ - Tension

− - Compression



8@15 = 120'

HEIGHT = 15'

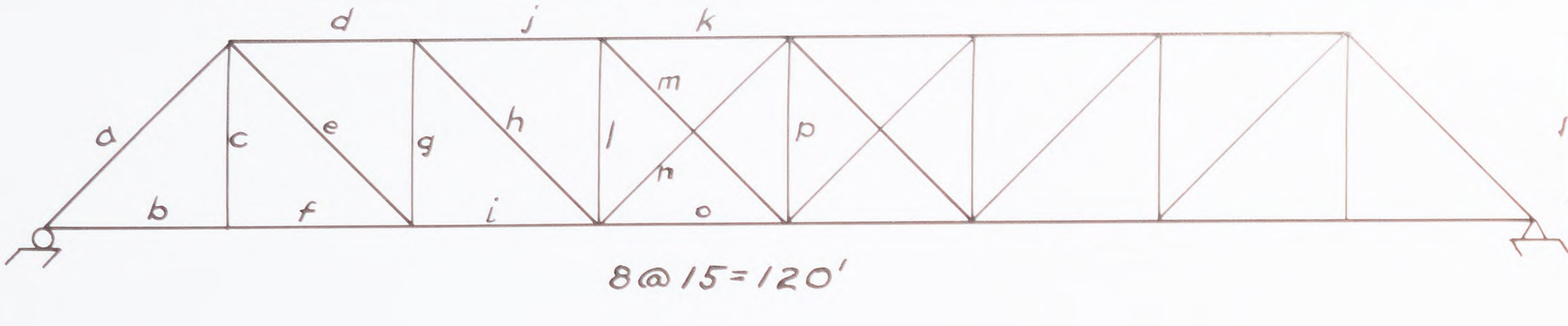
Table

of

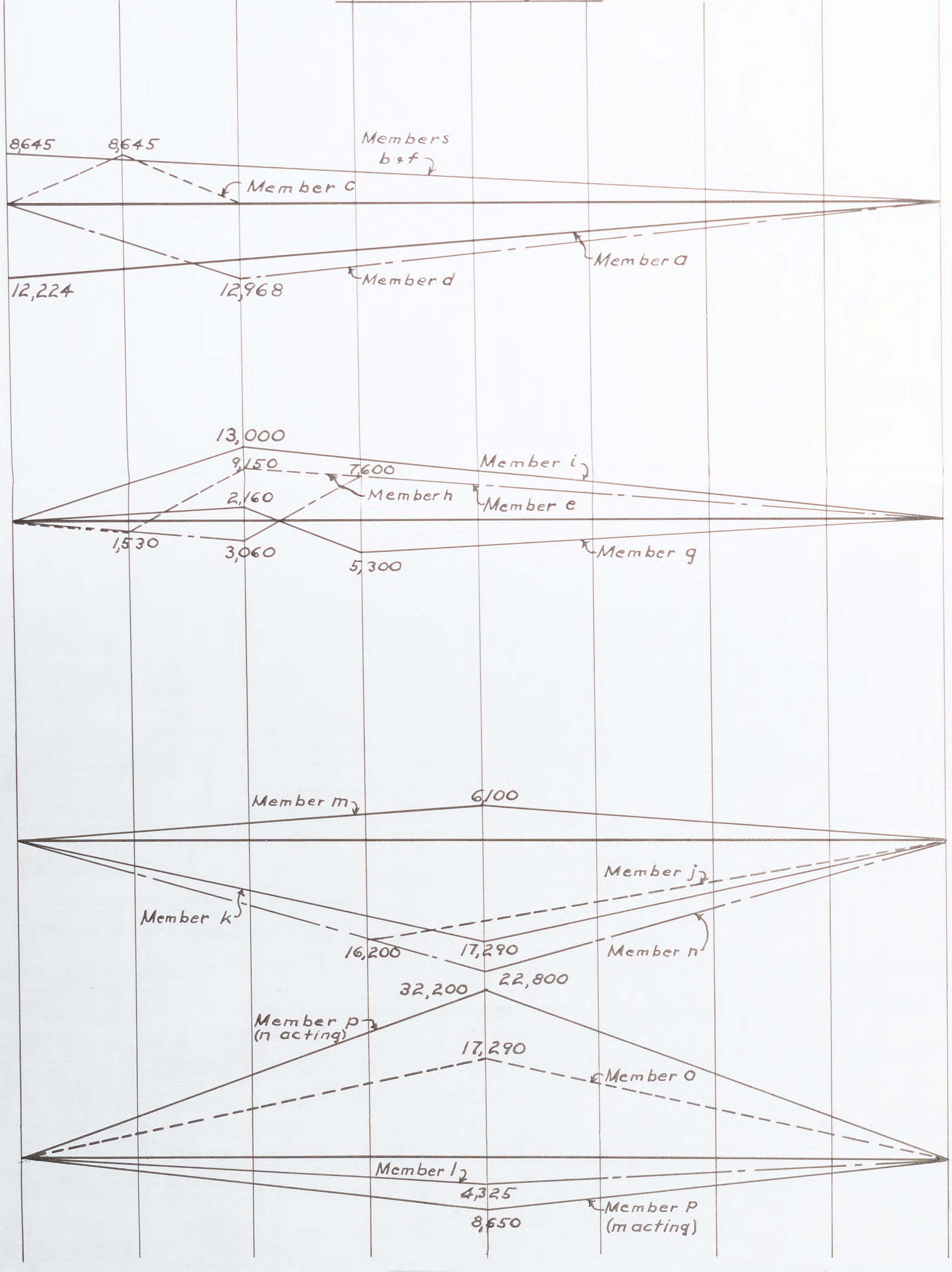
LIVE LOAD STRESSES

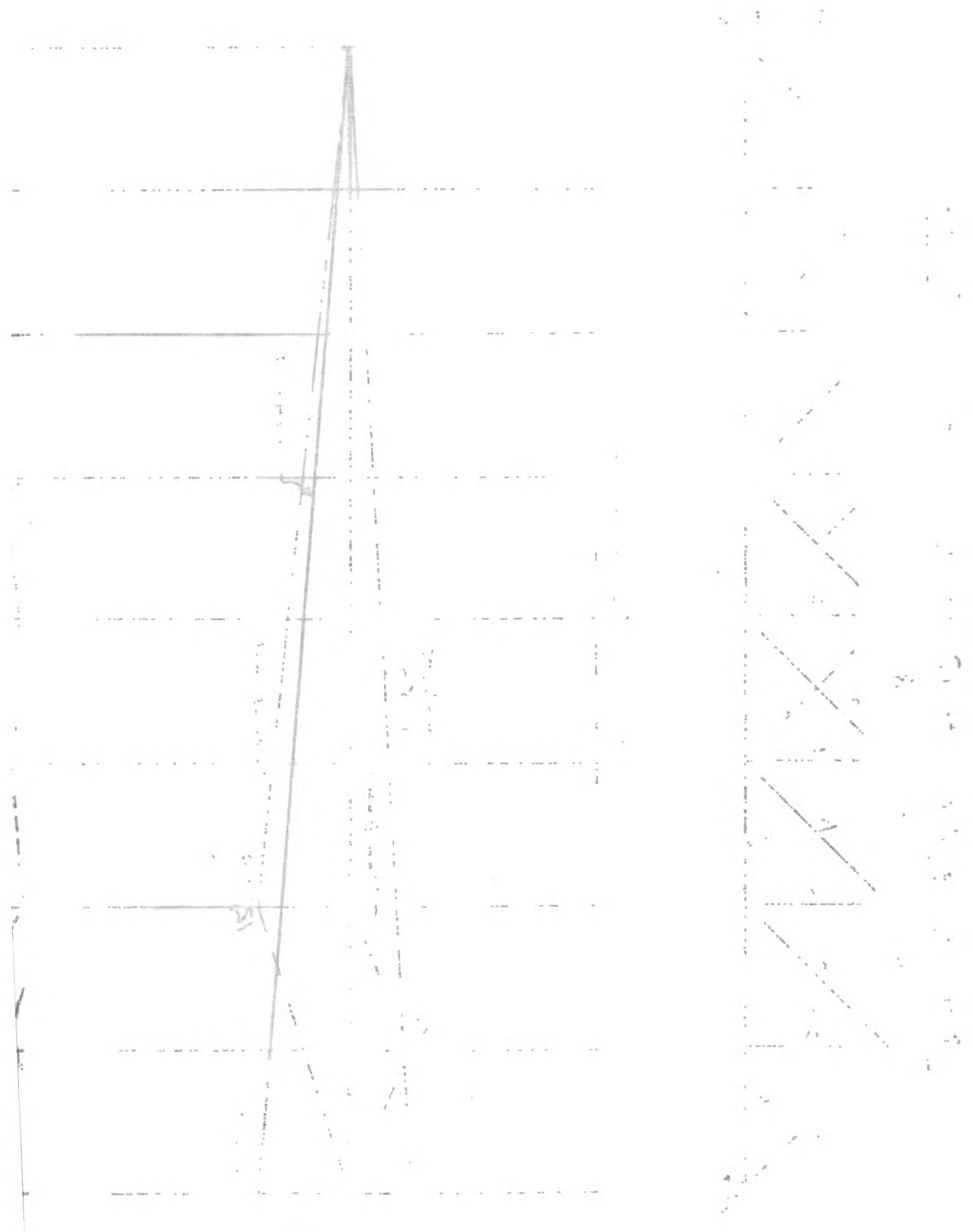
MEMBER	MAGNITUDE OF STRESS	TYPE OF STRESS
a	15,800 Pounds	Compression
b	11,172 "	Tension
c	2,128 "	Tension
d	19,152 "	Compression
e	11,280 "	Tension
f	11,172 "	Tension
g	5,852 "	Compression
h	6,780 "	Tension
i	19,152 "	Tension
j	23,942 "	Compression
k	24,536 "	Compression
l	22,292 "	Compression
m	841 "	Tension
n	33,850 "	Tension
o	4,308 "	Compression
p	48,924 "	Compression

STRESSES DUE TO CONCENTRATED LIVE LOAD-8645#



Scale-1" = 20,000#





Table

of

MOVING LIVE LOAD STRESSES

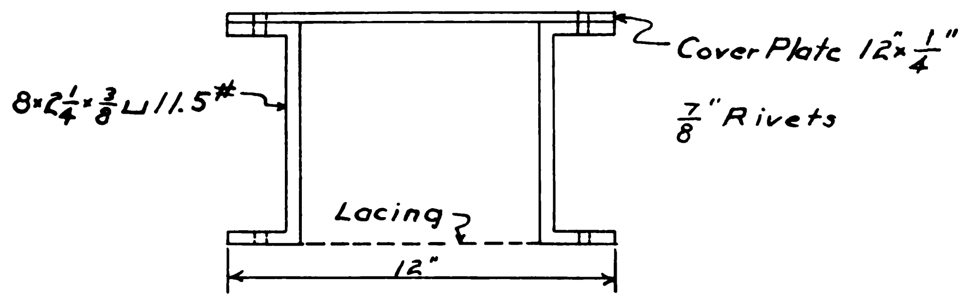
MEMBER	MAXIMUM TENSILE STRESS	MAXIMUM COMPRESSIVE STRESS
a	--	12,224 Pounds
b	8,645 Pounds	--
c	8,645 "	--
d	--	12,968 "
e	7,600 "	3,060 "
f	8,645 "	--
g	2,160 "	5,300 "
h	9,150 "	1,530 "
i	13,000 "	--
j	16,200 "	--
k	--	17,290 "
l	--	4,325 "
m	6,100 "	--
n	--	22,800 "
o	17,290 "	--
p	32,200 "	8,650 "

Part VII

DESIGN OF TRUSS MEMBERS

The total loads(dead, live, and moving live) for each individual member are shown on the following page, and are the loads that the members must be designed to resist.

1. Member (a). The member (a) is the end post of the truss and must resist a load of 65,524 pounds, compression. The cross section of the member is shown in the sketch below, and is analyzed as follows:



The area of the section is 2 x 3.36 + 12 x $\frac{1}{4}$ or 9.72 square inches, and the eccentricity of the neutral axis from the centroidal axis of the channels is (4 x 4.11) ÷ 9.72 or 1.69 inches. The moment of inertia about a horizontal axis through the center of the web is

$$32.3 \times 2 + \left(\frac{1}{12} \times 12 \times \frac{1}{4}^3 + 3 \times 4.11^2\right)$$

$$= 115.016 \text{ Inches}^4$$

The area multiplied by the eccentricity squared equals 27.72 and by subtracting this from the moment of inertia about the web axis, we obtain the moment of inertia about the centroidal axis of the section, which is 87.296 inches^4. The radius of gyration is equal to the square root of the moment of inertia divided by the area, so (r^2) will equal 87.296 ÷ 9.72 or 9.00 inches. According to the

Table

of

TOTAL LOADS ON MEMBERS

MEMBER	TENSILE LOAD	COMPRESSIVE LOAD
a	--	65,524 Pounds
b	46,277 Pounds	--
c	15,793 "	--
d	--	77,322 "
e	47,130 "	1,530 "
f	46,277 "	--
g	5,300 "	25,012 "
h	30,380 "	7,600 "
i	77,512 "	--
j	--	97,932 "
k	--	102,236 "
l	41,100 "	33,855 "
m	12,291 "	--
n	100,450 "	22,800 "
o	21,598 "	9,900 "
p	32,200 "	153,864 "

Specifications the formula to be used to determine the allowable stress in compression is governed by the slenderness ratio L/r, where (L) is the unsupported length of the member in inches, and (r) is the least radius of gyration. If the ratio is less than 120, the formula to be used is

Allowable stress = $15{,}000 - \frac{L^2}{4\ r^2}$, and if the ratio is greater than 120, the formula to use is

$$\text{Allowable stress} = 15{,}000 - \frac{L^2}{3\ r^2}\ .$$

In this case the ratio is 15 x 12 ÷ 3 or 60, which calls for the use of the first formula. The allowable stress is then 14,100 pounds per square inch. The safe load that the member can carry is equal to the allowable stress per square inch times the area of the member or

14,100 x 9.72 = 137,000 Pounds.

Since the allowable load is over twice the actual load, the member is safe.

2. Member (b). The load on member (b) is a tensile load of 46,277 pounds. The member consists of two bars of rectangular section 2" x 1" x 15 feet, spaced 6 inches apart. The cross sectional area is 2 x 2 x 1 = 4.0 square inches. Using an allowable tensile stress of 18,000 pounds per square inch, the allowable load is 72,000 pounds, and since the actual load is only 46,277 pounds, the member is safe.

3. Member (c). The load on this member is a tensile load of 15,793 pounds. The member is composed of two 15/16 inch square bars, spaced 4 inches center-to-center. The area is then 2 x 0.878 or 1.756 square inches. The allowable load is 18,000 x 1.756, and

equals 31,600 pounds. The actual load is 15,793 pounds and is less than the allowable, thus the member is safe.

4. Member (d). The member (d) is a top chord member and is composed of the same section as the end post member. The allowable compressive load on the member is 137,000 pounds, as found in the analysis of the end post, which is greater than the actual load of 77,322 pounds, making the member adequate in design.

5. Member (e). This member has both a tensile and compressive load acting on it, and is known as a reversal-of-stress member. Specifications require that for members subject to live loads producing alternate tensile and compressive stress: to the net total compressive and tensile stresses add 50% of the smaller of the two, and proportion the member to resist either of the increasing stresses resulting therefrom. The net total tensile load is 47,130 pounds and the net compressive load is 1,550 pounds. The design loads will then be 47,895 pounds and 2,295 pounds, respectively.

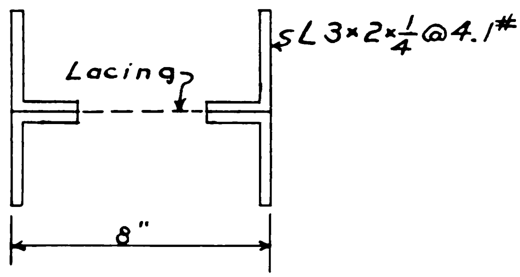
The member (e) is composed of two bars of rectangular cross section 2" x 7/8", and the total area is 3.5 square inches. The area required to resist the tensile load is 47,895 ÷ 18,000 or 2.70 square inches. This area is less than the actual, therefore the member will resist the tensile stress.

The member is also safe in compression, because the small load of 1,530 pounds is more than taken by the area of 3.5 square inches, at an allowable stress in compression of 11,720 pounds per square inch.

6. Member (f). This member is composed of two rectangular bars, 2" x 1" x 15 feet, with a center-to-center spacing of 8 inches. The total area is then 2 x 2 x 1 or 4.0 square inches. The load on the

member is a tensile load of 46,277 pounds, which is less than the allowable load of 72,000 pounds, making the member safe in tension.

7. Member (g). This member is also a member that has a reversal of stress and will be analyzed the same as member (e). The design loads are 27,242 pounds in compression, and 7,950 pounds in tension. The cross section of the member is shown in the sketch below.



The area is 4 x 1.19 or 4.76 square inches, and the moment of inertia about the centroidal axis is

$$\text{Moment of inertia} = 1.56 + 4 \times 1.19 \times 3.51^2$$
$$= 60.36 \text{ Inches}^4$$

The radius of gyration is equal to the square root of $60.36 \div 4.76$ or 3.57 inches. Since the length of the member is 15 feet, the ratio of length to radius of gyration is 50.4, and the allowable compressive stress is

$$\text{Allowable stress} = 15{,}000 - \frac{(15 \times 12)^2}{4 \times 12.7}$$
$$= 14{,}364 \text{ Pounds per square inch}$$

The allowable compressive load is then 14,364 x 4.76 or 68,500 pounds which is greater than the actual load of 27,242 pounds.

The net area in tension is 4.76 - 4 x 1/4 x 7/8 or 3.885 square inches. Using 18,000 as the allowable tensile stress, the allowable load in tension is 70,000 pounds, which greatly exceeds the actual load of 7,950 pounds. The member is safe for both loadings.

8. Member (h). This member is a member carrying a reversal of stress. The design loads are 33,443 pounds tension, and 12,973 pounds compression. The member consists of two bars of square cross section, 1" x 1" x 21.2 feet long, and has an area of 2 square inches in tension. The allowable load in tension is 2 x 18,000 or 36,000 pounds and the actual load is 33,443 pounds. The member is safe by a small margin in tension.

The radius of gyration squared is equal to

$$r^2 = \frac{0.0835 \times 2 + 2 \times 1 \times 6.25}{2 \times 1}$$

$$= 6.334 \text{ Inches}^2.$$

The ratio of the length to the radius of gyration is less than 120, so the allowable compressive stress is

$$\text{Allowable stress} = 15,000 - \frac{(21.2 \times 12)}{4 \times 6.334}$$

$$= 12,430 \text{ Pounds per square inch}$$

The allowable compressive load is 12,430 x 2.00 or 24,860 pounds, which is greater than the actual load of 12,973 pounds. The member is safe for the tensile and compressive stresses.

9. Member (i). The load on member (i) is a tensile load and is equal to 77,512 pounds. The member consists of two bars of rectangular section, $2\frac{1}{2}$" x 1" x 15 feet long. The area is equal to 5.00 square inches, and the allowable load in tension is 90,000 pounds. This load exceeds the actual load and thus the member is safe.

10. Member (j). The member (j) is of the same cross section as the end post member, and has loads of 97,932 pounds compression, and of 0 pounds tension. The allowable compressive load is 137,000 pounds, which exceeds the actual load of 97,932 pounds. The allow-

able tensile load is equal to the net area(9.72 - 4 x 1/4 x 7/8) or 8.845 square inches multiplied by the allowable stress 18,000 pounds per square inch, and is greatly in excess of the actual load of 9,150 pounds. The member is adequate for the tensile and compressive loads.

11. Member (k). The actual load on this member is 102,236 pounds, and the member is of the same cross section as the end post. The allowable compressive load that the member is capable of taking is 137,000 pounds and is in excess of the actual load.

12. Member (l). This member is of the same cross section and length as member (g), and carries a load of 33,855 pounds compression, and 41,100 pounds tension. The allowable loads are 68,500 pounds compression, and 70,000 pounds tension. The member is safe.

13. Member (m). The member (m) is stressed in tension only because of its large slenderness ratio. The load is 12,291 pounds. The area in tension is equal to 2 x 0.7854 square inches(for 1" square bars) and is 1.5708 square inches. The allowable load in tension is then 27,300 pounds, which is in excess of the actual load of 12,291 pounds. The member is then safe in tension.

14. Member (n). The loads on the member (n) 118,850 pounds tension, and 34,200 pounds compression. The member is composed of two square bars, $1\frac{1}{2}$ x $1\frac{1}{2}$ x 21.2 feet long. The area in tension is 2 x 1.5 x 1.5 or 4.5 square inches. The allowable load in tension is 4.5 x 18,000 and is 81,000 pounds. This allowable load is less than the actual load of 1 , pounds, and the member would not be safe in tension for the H-10 loading.

The allowable stress in compression is equal to 7,230 pounds per square inch, using a distance of $2\frac{3}{4}$ inches between the axes of the bars. The allowable compressive load will then be 4.5 x 7,230 and equals 32,500 pounds. The actual load on the member is 34,200 pounds, which is greater than the allowable load, and consequently the member is unsafe in compression as well as tension.

15. Member (o). The loads on the member are 9,900 pounds compression, and 21,598 pounds tension. The section is composed of two rectangular bars, 2 x 3/4 x 15 feet long, spaced 4 inches center-to-center. The radius of gyration squared is equal to

$$r^2 = \frac{2(1/12 \times 2 \times 3/4^3 + 2 \times 3/4 \times 4)}{3}$$

$$= 4.75 \text{ Inches}^2$$

The allowable compressive stress is

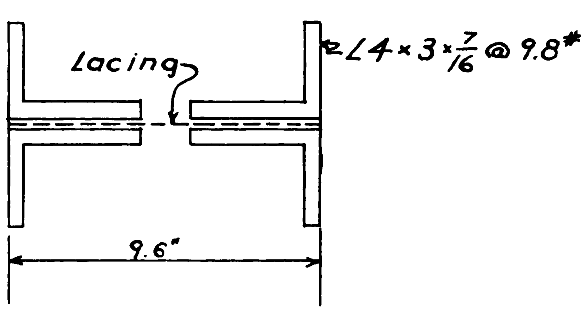
$$\text{Allowable stress} = 15{,}000 - \frac{(15 \times 12)^2}{4 \times 4.75}$$

$$= 13{,}300 \text{ Pounds/in}^2$$

The allowable compressive load is then 13,300 x 3.0 or 39,900 pounds. This member is safe in compression as the allowable load exceeds the actual load.

The allowable tensile load is 3.0 x 18,000 or 54,000 pounds, and exceeds the actual load of 21,598 pounds, making the member safe in tension.

16. Member (p). The loads on the member are 170,000 pounds compression and 48,300 pounds tension. The section sketch is below.



The area of the section is $2.87 \times 4.0 = 11.48$ square inches. The moment of inertia about the centroidal axis is

$$\text{Moment of inertia} = 4(2.2 + 2.87 \times 4.0^2)$$

$$= 192.5 \text{ Inches}^4$$

The radius of gyration squared is equal to $192.5 \div 11.48 = 16.72$ inches squared, and the allowable stress in compression is 14,515 pounds per square inch. The allowable compressive load is $14{,}515 \times 11.48$ and is 167,000 pounds. This allowable load is less than the actual load and the member is unsafe in compression.

The net area in tension is $4 \times 2.87 - 4 \times 7/8 \times 7/16$ or 9.95 square inches. The allowable tensile load is then $9.95 \times 18{,}000$ or 179,000 pounds, which is greater than the actual load. The member is safe in tension but unsafe in compression for the H-10 loading.

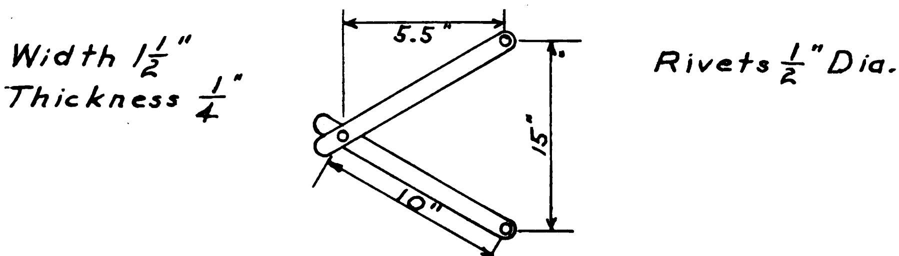
This concludes the analysis of the truss members, and except for the members (n) and (o), the truss will safely take the loading. The members that are overstressed are visibly overstressed, for they are warped out of position, and other signs of failure are evident.

Part VIII

CHECK OF LATTICE BARS AND SPACING

1. Lacing in the Verticals. The verticals are compression members and are governed by certain specifications as regards lacing members. The lacing bars shall be so placed that the ratio of the length to the least radius of gyration of the flange included between their connections shall not be over three-fourths of that of the member as a whole. For our vertical members, the slenderness ratio is 60 for the whole member and 10 for the lacing bars. This condition is satisfactory.

The lacing bars shall be so proportioned to resist a shearing stress normal to the axis of the member equal to 2% of the total compressive stress in the member. The allowable compression per lacing bar is equal to 0.02 x 157,424 or 3150 pounds. The allowable tension per member is 0.02 x 61,980 or 1240 pounds. Specifications require that the minimum width of a member in which $\frac{1}{2}$ inch rivets are used is $1\frac{1}{2}$ inches, which is the width of the members actually used. The sketch below indicates the size of members and spacings of the lattice bars.



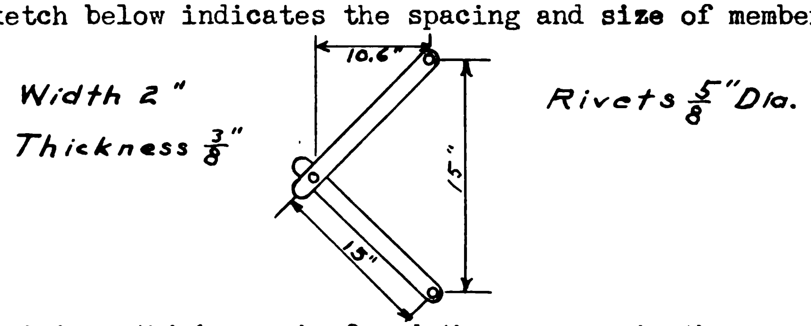
The minimum thickness of single lattice bars is 1/40 of the length from center-to-center of end holes and is 0.242 inches which is slightly less than the actual thickness of 0.25 inches. The compressive strength is equal to $(15{,}000 - L^2/4r^2)$ x the area of the member, and equals

$$(15{,}000 - \frac{10^2}{4\ r^2}) \times 1\tfrac{1}{2} \times \tfrac{1}{4}$$, where (r) for a rectangular bar in terms of the thickness is $t^2/12$. The compressive strength is then equal to 3830 pounds. This quantity represents the load that the bar is capable of withstanding, and exceeds the actual stress of 3,150 pounds, making the member safe in compression.

The tensile strength of the lacing bar is $18{,}000(1\tfrac{1}{2} - 5/8)$ or 3,940 pounds, which exceeds the actual stress of 1240 pounds, so that the member is also safe in tension.

2. Lacing the End Posts and Top Chords. These members are also compression members and are governed by the same principles as the lacing in the verticals.

The actual compression per lattice bar is 0.02 x 102,236 or 2,044 pounds, and the actual tension per bar is 0.02 x 16,120 or 322,40 pounds. The minimum width is 2 inches when 5/8 inch rivets are used. The sketch below indicates the spacing and size of members.



The minimum thickness is found the same as in the previous analysis and is 0.375 or 3/8 of an inch, which is the actual size of member used.

The allowable compressive strength in the members is

$$\text{Compression} = (15{,}000 - \frac{225}{4 \times 1/12 \times 9/64}) \times 2 \times 3/8$$

$$= 7{,}560 \text{ Pounds}$$

The allowable tensile strength is

Strength = (2 - 0.625) x 3/8 x 18,000

= 9,300 Pounds

These allowable loads exceed the actual loads and indicate that the lacing was properly designed.

Part IX

LATERAL FORCES

The American Association of State Highway Officials Specifications are quite explicit and inclusive.

The wind force on the structure shall be assumed as a moving horizontal load equal to 30 pounds per square foot on one and one-half times the area of the structure as seen in elevation, including the floor system and railings, and on one-half the area of all trusses or girders in excess of two in the span.

The lateral force due to the moving live load and the wind pressure against it shall be considered as acting 6 feet above the roadway and shall be as follows: 200 pounds per linear foot for highway bridges.

The total assumed wind force shall not be less than 300 pounds per linear foot in the plane of the loaded chord, and 150 pounds per linear foot in the plane of the unloaded chord on truss spans.

The wind pressure of 50 pounds per square foot on the unloaded structure, applied as in the first paragraph, shall be used if it produces greater stresses than the combined wind and lateral forces of the first two paragraphs.

1. Wind load. The area of the truss as seen in elevation is

21.6 x 1 = 53.2 Square inches	End posts
90 x 10/12 = 75 Square inches	Top chord
(15 x 1)2 = 30 Square inches	Panel braces
7/12 x 120 = 70 Square inches	Stringers
9/12 x 15 x 4 = 45 Square inches	Web members
3/12 x 120 = 30 Square inches	Floor slab
2/12 x 2 x 110 x 1.75 = 64 Square inches	Railings
1 x 8 = 8 Square inches	
Total = 375.2 Square Inches	

The wind load obtained by this method is 375.2 x 30 x 1.5 ÷ 90 or 188 pounds per linear foot. Since this is less than the 300 pound minimum, the latter is used.

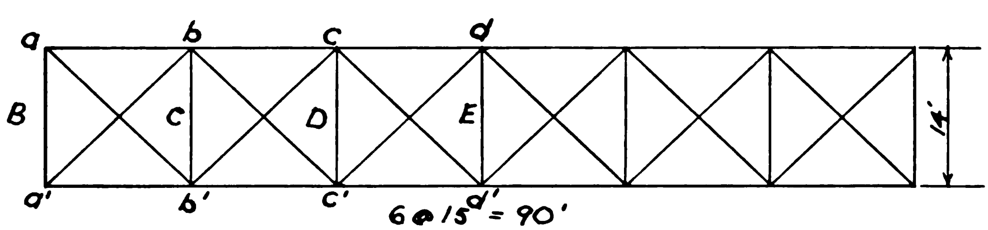
The loads to be used then are

300 - 200 - 500 pound per foot on loaded chord

150 pounds per foot on the unloaded chord.

This makes a panel load of 7500 pounds for the loaded chord and 2250 pounds for the unloaded chord.

2. Stresses in the Top Bracing. The shear in the panel (ab) is equal to 7,500 x 5/2, which is 18,750 pounds. The shear in panel (bc)



is equal to 2 x 18,750 - 7500(2 + 3 + 4 + 5 + 6)/6, or 12,500 pounds. The shear in panel (cd) is simply the panel load or 7,500 pounds. The angle that the diagonals make with the chords is 46.7 degrees, and the sine of the angle is 0.728. The stresses are found by multiplying the shear in the panel by the sine of the angle. The stresses are shown in the following table:

PANEL	SHEAR	MEMBER	STRESS
ab	18,750	ab'; a'b	± 13,640
bc	12,500	bc'; b'c	± 9,120
cd	7,500	cd'; c'd	± 5,450

The above stresses are calculated on the assumption that both diagonals in any panel are in action at the same time, one in compression and the other in tension.

The stresses in the transverse struts equal one-half of the joint load or 3,750 pounds.

3. Check of Design. The members are one inch circular bars, 21.1 feet long. The allowable load in tension is 1 x 0.785 x 18,000 or 14,100 pounds. The allowable load in compression is 1 x 0.785 x the allowable stress. The allowable stress is

$$\text{Allowable stress} = 15{,}000 - \frac{(21.1 \times 12)^2}{4 \times 6.4}$$

$$= 12{,}500 \text{ Pounds per square inch}$$

The allowable load is 12,500 x .785 or 9,800 pounds in compression. Since the allowable loads exceed the actual ones, the members are safe.

It is not generally assumed that the kind of stress changes during the passage of the live load; thus the members may be designed as simple tension or compression members, which are not subject to a reversal of stress. This is the opinion expressed in the book Steel and Timber Structures, by Hool and Kinne; Second Edition, 1942; page 313. Using this assumption, the laterals will not be investigated for reversal of stress.

4. Check of Transverse Struts. The loads on the transverse struts consist of 3,720 pounds, compression. The radius of gyration of the strut cross section is

$$r = \frac{1.10 + 1.19(3.01)^2}{2 \times 1.19}$$

$$= 2.66 \text{ Inches}^2$$

The allowable stress is

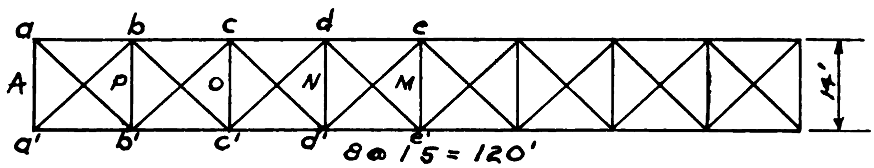
$$\text{Allowable stress} = 15{,}000 - \frac{(14 \times 12)^2}{4 \times 7.08}$$

$$= 14{,}000 \text{ Pounds per in}^2$$

The allowable compressive load is then 14,000 x 2.38 or 33,200 pounds, which is greater than the actual load of 3,720 pounds.

The assumed loading is satisfactory because the substitution of the 50 pounds per square foot load on the unloaded structure results in stresses about one-half of those now present.

4. Stresses in Bottom Bracing. The bottom bracing is arranged as shown in the sketch.



The load in the bottom bracing is 150 x 15 or 2250 pounds per panel. The shear in panel (ab) is 2250 x 7/2, and is equal to 7880 pounds. The shear in panel (bc) is 2250(2 + 3 + 4 + 5 + 6 + 7 + 8)/8, which is 9850 pounds subtracted from the shear in panel (ab). The result is 5,900 pounds.

The shear in panel (cd) is equal to 6 x 2250 - 2250 x 33/8, and is equal to 4,220 pounds.

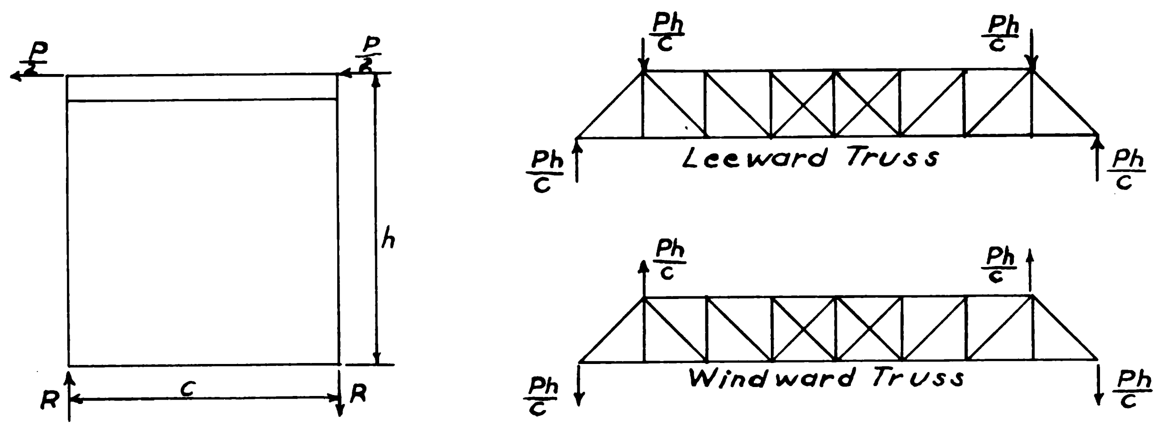
The shear in panel (ef) is equal to the load or 2,250 pounds. The angle that the diagonals make with the chords is 46.7 degrees, and the sine of the angle is 0.728. The stresses in the members are obtained by multiplying the panel shears by the sine of the angle, and are given in the following table.

PANEL	SHEAR	MEMBER	STRESS
ab	7,880	ab' ; a'b	± 5,720
bc	5,900	bc' ; b'c	± 4,290
cd	4,220	cd' ; c'd	± 3,070
de	2,250	de' ; d'e	± 1,630

The stresses of 1125 pounds are taken by the floor beams, dispensing with the use of other transverse struts. The floor beams have adequate compression areas to handle this load.

The members are the same as the top laterals, that is, one-one inch round rods. The allowable load is 14,100 pounds in tension and 9,800 pounds in compression. These values are well in excess of the maximum of 5,720 pounds, and the member is safe.

5. Overturning Stresses. The wind blowing against the side of the bridge has a tendency to overturn the structure about the supports on the leeward side, and thus introduces stresses in those members. This load is taken by the end posts and lower chord members, and the web members are not stressed. The overturning moment is equal to Ph, where (P) is the magnitude of the force brought to each portal strut, and (h) is the depth of the truss. The resisting moment is Rc, where (R) is the value of the reaction, and (c) is the width of the truss. The sketch of this condition is shown below.



The magnitude of the force brought to each portal strut is 7,500 x 2 + 3,750, or 18,750 pounds, and the equivalent loading at the portals is Ph/c or 18,750 x 15/14 + 20,100 pounds.

The stress in the end post is 20,100 x 1.414, or 28,400 pounds, compression on the leeward side and tension on the windward side. By combining this load with the load due to dead and live stresses, the total load on the end post is 79,724 pounds compression, and 42,600 pounds tension, by use of the reversal of stress theory. The member is capable of taking 137,000 pounds compression and 176,000 pounds tension, and is thus safe for the above loads.

The load in the bottom lateral (b) is 56,327 pounds tension, and

30,150 pounds compression. The allowable tensile load is 72,000 pounds and the allowable compressive load is

$$\text{Allowable load} = (15{,}000 - \frac{L^2}{4\,r^2}) \times \text{Area}$$

$$= (15{,}000 - \frac{(15 \times 12)^2}{4 \times (1/12 \times 1 \times 8 + 4 \times 2.25)/4}) \times 4$$

$$= 10{,}400 \text{ pounds/in}^2 \times 4$$

$$= 41{,}600 \text{ Pounds.}$$

These loads are greater than the actual loads and the member is thus safe.

The load in member (f) is the same as for (b), and since they are of the same section, this member is also safe.

The loads on member (i) are now 85,540 pounds tension, and 30,150 pounds compression. The allowable load in tension is 90,000 pounds, and in compression is

$$\text{Allowable load} = (15{,}000 - \frac{L^2}{4\,r^2}) \times \text{Area}$$

$$= (15{,}000 - \frac{(15 \times 12)^2}{4 \times .4(1/12 \times 1 \times 2.5^3 + 2.5 \times 1.5^2)}) \times 5.00$$

$$= 55{,}350 \text{ Pounds}$$

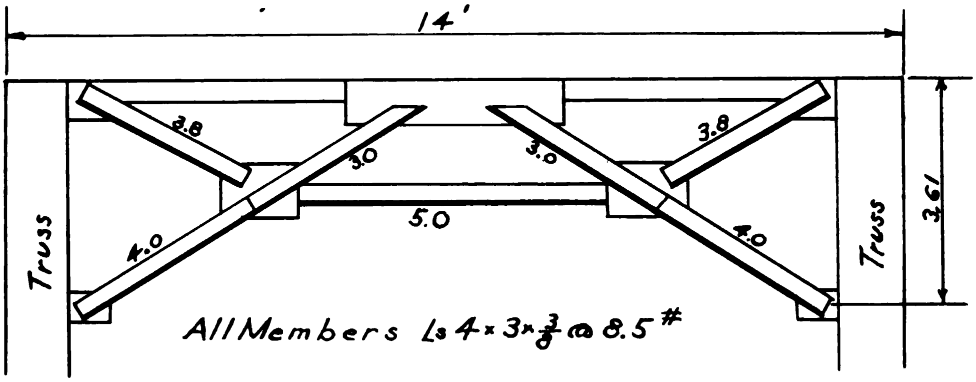
The allowable loads exceed the actual loads, making the member safe for the loading.

The member (o) has the following loads acting on it: 53,815 pounds tension, and 34,308 pounds compression. The allowable tensile load is 54,000 pounds, and the allowable compressive load is 39,900 pounds. The tensile load requirement is barely sufficient, being only 185 pounds safe. The compression requirement is adequate.

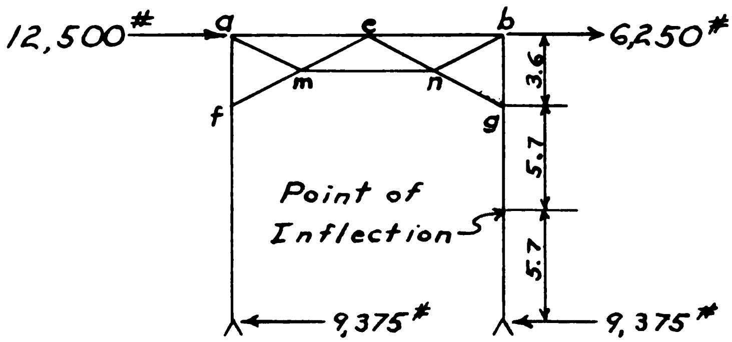
Since none of the members were over-stressed, the lateral system

is sufficiently designed to withstand the loading.

6. Portals. The construction of the portals is indicated below.



The chief function of the portals is to transfer the reactions of the upper lateral system to the supports of the bridge. In addition, it stiffens the end posts against vibration. The point of inflection of the end posts is (15 - 3.61)/2, or 5.70 feet from the base. The loading is as shown, and results in a direct stress in the end post of



(18,750 x 9.30) ÷ 12, or 14,500 pounds. To determine the vertical component of the stress in members (am) and (nb), take moments about the point of inflection.

$$\text{Stress in (mn)} = \frac{9.3 \times 9{,}375 - 6 \times 14{,}500}{1.66}$$

$$= 0 \text{ Pounds.}$$

This indicates that members (am) and (nb) are unstressed.

The vertical component of the load on (fme) and (eng) is 14,500 pounds and is tension for the former and compression for the latter. The horizontal component is 14,500 x 6 ÷ 3.6, and is 24,100 pounds.

The stress in (ae) is equal to 2/3 x 18,750 - 9,375 + 24,100, or 27,225 pounds tension. The allowable load in tension is equal to the area times the allowable stress, and is

$$\text{Allowable load} = (2.48 - 2 \times 7/8 \times 1/4) \times 18{,}000$$
$$= 36{,}800 \text{ Pounds}$$

The net section in tension was obtained by subtracting the 7/8 inch rivet holes from the area of the angle.

The stress in (eb) is equal to 1/3 x 18,750 + 24,100 - 9,375 or 20,975 pounds compression. The allowable compressive load is

$$\text{Allowable load} -(15{,}000 - \frac{(72)^2}{4 \times 0.418}$$
$$= 29{,}000 \text{ Pounds}$$

The members so far are safely designed for both tension and compression. A reversal of direction of wind load will result in a reversal of the sign of the stress in the members, and an allowance should be made for the increased stress possibly resulting therefrom. The margin of the allowable loads over the actual loading is sufficient to make this allowance.

The loads on the member (fme) are equal to the square root of the sum of the squares of the components, and are 28,000 pounds. These loads are tensile loads when the wind is from the north side of the bridge, and compression when from the south. The allowable loads are 36,800 pounds in tension and 29,600 pounds in compression. Since both are greater than the actual load, the member is safe.

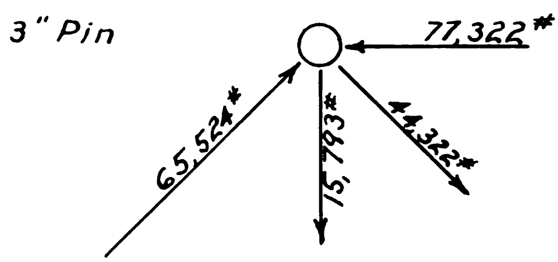
The portals are of sufficient design to carry the wind loading.

Part X

TOP CHORD PINS

The pins will be analyzed for the required thickness to resist the moment on them, and the thickness of plate required for the connections.

1. Pin at (B). The pin at this point is a 3 inch pin and is loaded as shown. The thickness of bearing plate required for each channel web of member (a) is 65,524 ÷ (2 x 3 x 24,000), or 0.456



inches, which is less than the actual thickness of 0.625 inches. The value 24,000 used in the denominator of the expression is the allowable bearing on pins, in pounds per square inch. The thickness required for member (d) is 0.536, and the actual thickness is 0.625 inches. The thickness required for member (c) is 0.11 inches and is less than the 0.94 inches available. The thickness required for member (e) is 0.306 inches, which is less than the actual 0.875 inches.

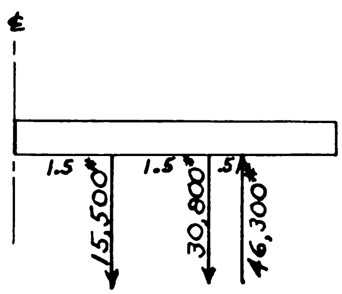
The vertical loads on the pin are arranged as shown. The maximum moment is at the center of the pin and equals 46,300 inch-pounds.

Bending Moment = 1.5 x 15,500 + 30,800 x 3 + 46,300 x 3.5

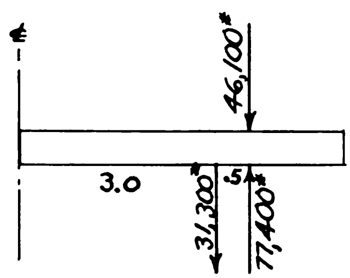
The horizontal loads on the pin are spaced as below, and the moment is equal to

Bending Moment = 46,100 x 3.5 + 31,300 x 3 + 77,400 x 3.5

= 16,100 Inch-pounds



Vertical Loads



Horizontal Loads

The total moment is equal to the sum of the squares of its components and is 49,000 inch-pounds.

The minimum diameter of pin to be used is found by use of the formula

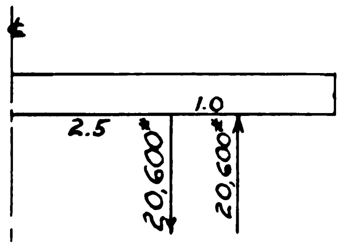
Diameter = .753 x Cube root of the Moment.

This formula is derived in Steel and Timber Structures, by Hool and Kinne, 1942, on page 350.

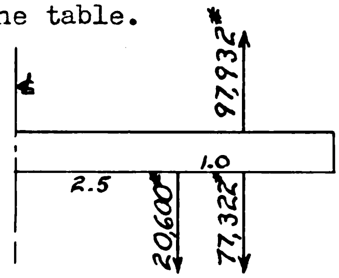
The diameter then required is 0.753 x 36.5, or 2.74 inches. The actual diameter is 3.0 inches, and is thus safe as computed. The member (c) is evidently a special eyebar, for there is no standard eyebar of this size in use at the present time.

2. Pin at (C). This pin is a 3 inch pin also and is loaded about

the same as the previous pin. The computations are carried on in the same manner as before and are listed in the table.



Vertical Loads



Horizontal Loads

Table of

REQUIRED BEARING THICKNESSES

MEMBER	LOAD	REQUIRED BEARING THICKNESS	ACTUAL BEARING THICKNESS
d	77,322 Pounds	0.536 Inches	0.625 Inches
j	97,932 "	0.678 "	0.625 "
g	24,192 "	0.168 "	0.375 "

Table

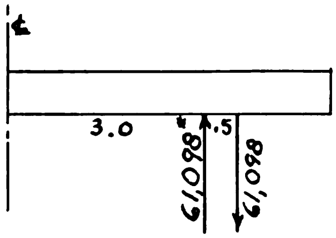
of

REQUIRED PIN DIAMETER COMPUTATIONS

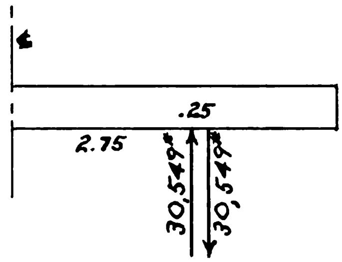
VERTICAL MOMENT	20,000 Inch-pounds
HORIZONTAL MOMENT	20,000 Inch-pounds
TOTAL MOMENT	29,200 Inch-pounds
REQUIRED DIAMETER	2.32 Inches
ACTUAL DIAMETER	3.00 Inches

The pin and connecting plates are of sufficient cross section to withstand the loading up to this point.

PIN AT (D)



Vertical Loads



Horizontal Loads

Table of

REQUIRED BEARING THICKNESSES

MEMBER	LOAD	REQUIRED BEARING THICKNESS	ACTUAL BEARING THICKNESS
j	41,232 Pounds	0.343 Inches	0.625 Inches
k	70,138 "	0.584 "	0.625 "
m	40,000 "	0.333 "	0.940 "
l	28,382 "	0.236 "	0.375 "

Table of

REQUIRED PIN DIAMETER COMPUTATIONS

VERTICAL MOMENT	28,382 Inch-Pounds
HORIZONTAL MOMENT	28,382 Inch-pounds
TOTAL MOMENT	40,000 Inch-pounds
REQUIRED DIAMETER	2.55 Inches
ACTUAL DIAMETER	2.50 Inches

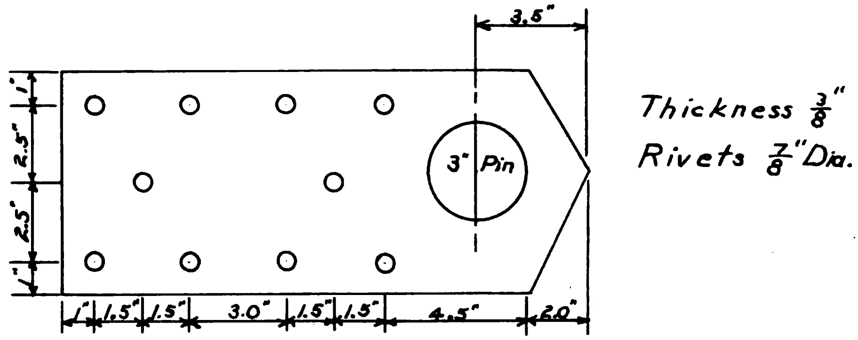
The pin is not safe for the loading as designated because the required diameter is 0.05 of an inch larger than the actual diameter. The connecting plates will transmit the stress safely, however.

Part XI

TOP CHORD PIN PLATES

These plates are used to transmit the stress from the chord members to the pins, and must be designed to resist the tensile loads thus occuring.

1. Pin Plate at (B). The plate layout is shown below, and must be



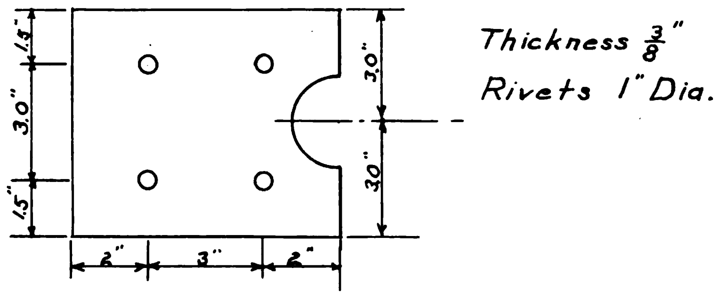
designed to carry a moment of 49,000 pound-inches.

The allowable load in shear on one 7/8 inch rivet is 11,000 x 0.601 and is 6614 pounds. The number of rivets needed to resist the moment is equal to 49,000 ÷ (6614 x 12) or 7.4. Since 10 are used, the connection is more than adequate.

The net section occurs at the section ABECD, and is (7-1.2) x 3/8 or 2.18 square inches. The allowable load in tension per plate is 2.18 x 18,000 or 39,300 pounds, and the actual load is 38,661 pounds. The plate is adequate for this joint.

2. Pin Plate at (C). The moment to be resisted is 29,200 inch-pounds, and with the use of 1 inch rivets, the required number needed is 3.37. The number of rivets is met, as 4 per plate were used. The net section is a section straight across the member, through the

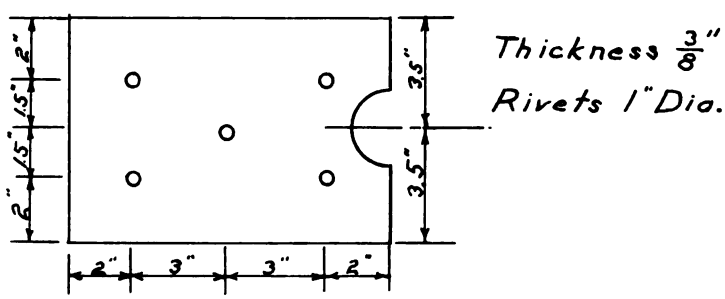
axes of the holes. It is equal to (6 - 0.87) x 1, and is 5.13 square



inches. The total allowable tensile load is 5.13 x 18,000 or 92,500 pounds per plate, as compared with an actual load of only 48,966 pounds per plate.

The plate is then adequately designed for the above loading.

3. Pin Plate at (D). The total moment on this plate is 40,000 pound-inches per plate. With the use of 1 inch rivets, the allowable stress per rivet is 11,000 x 0.7854 or 8,620 pounds. The number of rivets needed in the connection is 4.64 = 40,000 ÷ 8620. The actual number used is 5, so the requirement is met. The net section is equal to (7 - 1) x 3/8, or 2.25 square inches. The allow-



able load is then 2.25 x 18,000 or 40,500 pounds per plate, while the actual load is 35,069 pounds.

The member is safe.

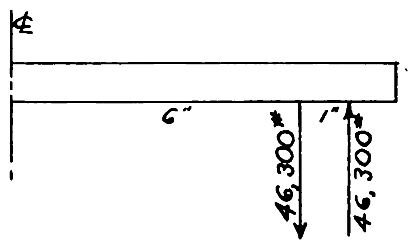
4. Pin Plate at (E). The plate at (E) is the same as the plate at (D), the allowable load being 40,500 pounds, which is greater than the actual load of 35,069 pounds.

Part XII

PIN DESIGN IN THE BOTTOM CHORDS

The procedure followed here will be the same as that followed in the analysis of the top chord pins, and reference may be made to that section for the method of procedure.

PIN AT (A)

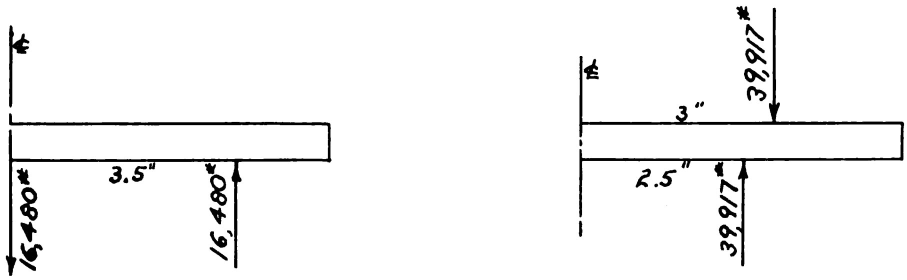


Vertical and Horizontal Loads

VERTICAL MOMENT	66,300 Inch-pounds
HORIZONTAL MOMENT	66,300 Inch-pounds
TOTAL MOMENT	94,000 Inch-pounds
REQUIRED DIAMETER	3.49 Inches
ACTUAL DIAMETER	3.50 Inches

The pin is barely safe and is very close to being overstressed.

PIN AT (P)



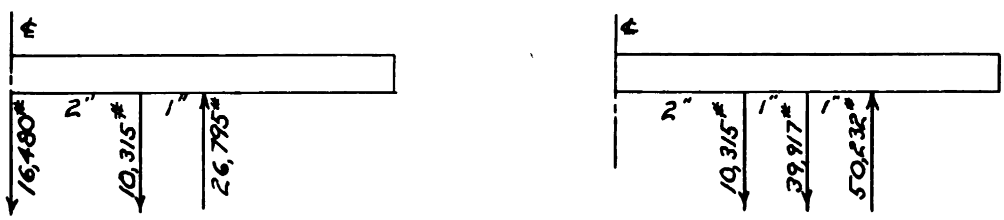
Vertical Loads

Horizontal Loads

VERTICAL MOMENT	57,700 Inch-pounds
HORIZONTAL MOMENT	19,985 Inch-pounds
TOTAL MOMENT	61,000 Inch-pounds
REQUIRED DIAMETER	2.96 Inches
ACTUAL DIAMETER	4.00 Inches

The member is safe by a safety factor of about 1.5, which is adequate for the connection.

PIN AT (O)



Vertical Loads

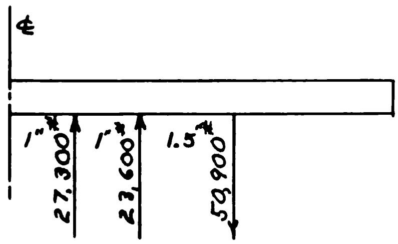
Horizontal Loads

VERTICAL MOMENT	59,755 Inch-pounds
HORIZONTAL MOMENT	60,547 Inch-pounds
TOTAL MOMENT	85,100 Inch-pounds
REQUIRED DIAMETER	3.31 Inches

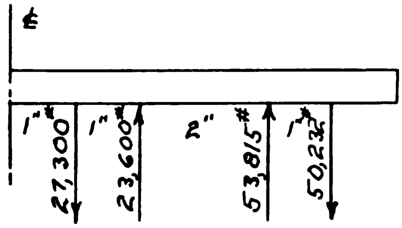
ACTUAL DIAMETER 4.00 Inches

The pin itself is more than adequate to resist the load placed upon it, and the plates are also safe by a fairly wide margin.

PIN AT (N)



Vertical Loads



Horizontal Loads

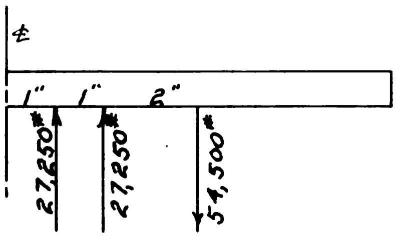
VERTICAL MOMENT	103,500 Inch-pounds
HORIZONTAL MOMENT	16,000 Inch-pounds
TOTAL MOMENT	104,800 Inch-pounds
REQUIRED DIAMETER	3.54 Inches
ACTUAL DIAMETER	4.00 Inches

The required plate thicknesses are:

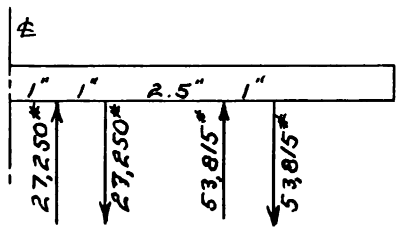
PLATE	REQUIRED THICKNESS	ACTUAL THICKNESS
1	0.264 Inches	0.375 Inches
i & o	0.280 "	1.00 "
h & n	0.201 "	1.00 "

The pin and plate are both of the required thickness, and are satisfactory.

PIN AT (M)



Vertical Loads



Horizontal Loads

The thicknesses of the member plates are sufficient to withstand the loading.

VERTICAL MOMENT	136,250 Inch-pounds
HORIZONTAL MOMENT	81,250 Inch-pounds
TOTAL MOMENT	159,000 Inch-pounds
REQUIRED DIAMETER	4.07 Inches
ACTUAL DIAMETER	4.50 Inches

The pins are safe, and will transmit the load without being overstressed.

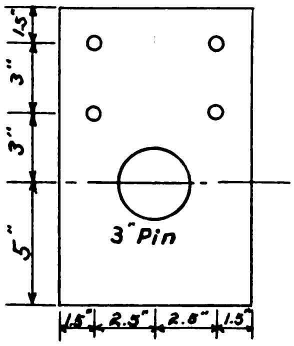
This concludes the study of the pins in the bottom chord, and there remains only the analysis of the plates that connect the vertical hangers to the bottom chord pins; the plates that connect the upper transverse members to the top chord; the pin plates that hold the lower chord and upper chord laterals; and the pedestals.

Part XIII

HANGER PIN PLATES

These plates are used to connect the truss hangers to the chords at the top and bottom. They are all of the same size, and carry maximum loads of 157,424 pounds compression, and 61,098 pounds tension. The stresses are assumed to be direct stresses and no reversal of loading takes place.

1. Rivert Required. The above loads are the actual loads per pair of plates, two being used at each connection. The allowable shear in a one inch rivet(assuming an allowable stress of 11,000 pounds per square inch) is 11,000 x 0.7854, or 8,640 pounds. The number of rivets required per plate is then 61,098 ÷ 2 x 8,640, and equals 3.55. The actual number used is 4 as shown, so the plate is adequate at this point.



Thickness $\frac{3}{8}$"
Rivets 1" Dia.

2. Tension Requirement. The net area of the plate is the smallest when taken across the axis of the 3 inch pin, and is equal to

Net area = (8-3) x 3/8

= 1.875 Square inches

The allowable tension load is then 18,000 x 1.875 or 33,800 pounds, which is less than the actual load of 30,549 pounds.

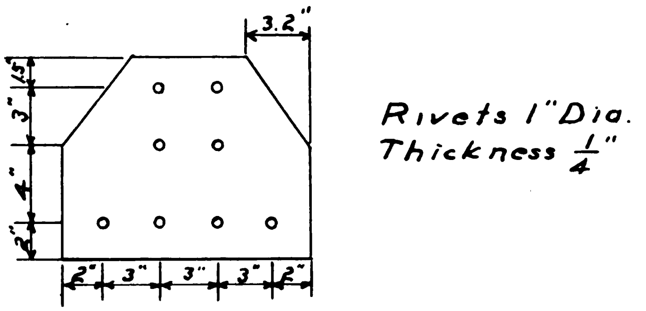
3. Compression Requirement. The compression area is equal to the gross area of the plate, which is 3.00 square inches. The allowable load that the plate will take in compression is then 18,000 x 3.00 or 54,000 pounds. This is quite a bit less than the actual load of 78,712 pounds, which would be expected upon examining the plate out at the bridge. The plates at the mid-panels are visibly failing, and the ones at the panels next to them are beginning to buckle.

The design is definitely not sufficient for this loading.

PART XIV

UPPER TRANSVERSE CONNECTION PLATES

These plates are used to connect the upper transverse struts to the chords, and are designed as shown below. Since the loads on them are so light, 3,720 pounds, it is easily seen that the



design is excessively safe. The allowable load in tension is about 60,000 pounds, far in excess of the actual load. The stresses due to the overturning moment are also so small that they will not affect the strength of the connection.

Part XV

DIAGONAL CONNECTION PLATES

These plates are the plates that hold the diagonal members of the sway-bracing system to the transverse members, and are designed as shown. The maximum load on the plate is 5,720 pounds, which is the load in member (ab') at the middle panels.

The actual shear on the plate is equal to 5,720 x sine of the angle the diagonal makes with the plane of the plate, or 5720 x 0.728 which is 4,160 pounds. The allowable shear is 4,860 pounds on a 3/4 inch rivet, which exceeds the actual load and thus is safe. The plates are more than adequate for resisting pulling out or shearing off of the diagonals.

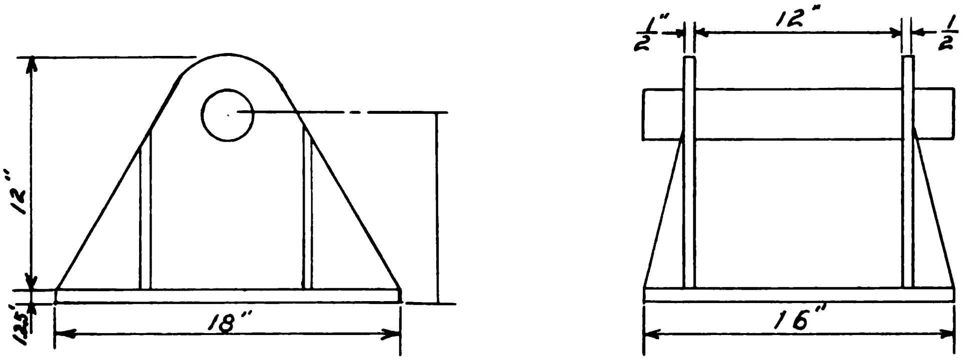
Part XVI

CHECK OF PEDESTALS AND SHOES

1. Bearing Plates. The end reaction is 66,300 pounds and assuming an allowable bearing stress on the masonary piers of 200 pounds per square inch, the required area of bearing plate is 66,300 ÷ 200 or 331.5 square inches. The actual plate is 18" x 18", giving an area of 324.0 square inches, which is not sufficient. The plate is $1\frac{1}{4}$ inches thick which is sufficient, however.

2. Shoe Design. The shoe is constructed as shown and has a bearing area of 3.5 x $\frac{1}{2}$ or 1.75 square inches. Using an allowable

bearing stress of 18,000 pounds per square inch, the allowable



bearing load is 18,000 x 1.75 or 31,600 pounds per web of the shoe. This value is less than the actual load of 33,150 pounds, and the shoe is consequently overstressed.

3. Bolt Shear. The connection to the end posts is accomplished by the use of 3.5 inch pins, set in the shoe webs. The allowable shearing stress to be used on the pin is 8,000 pounds, and the area is 9.621 square inches. The allowable shearing load on the pin is then 9.621 x 8,000 or 76,968 pounds, which is greater than the actual load of 64,300 pounds. The pin is safe.

The shoe and pedestal system is slightly inadequate to withstand the loading of the 10 ton truck or equivalent H-10 loading. The connection between spans is a 7 inch pin with double loading, and results in the same proportionate loading in excess of the allowable loading.

This concludes the analysis of the structure, and the summary and conclusions as regards the results follows on the remaining pages.

Part XVII

CONCLUSION

The bridge is, for the most part, safely designed for the H-10 loading. It will not, however, withstand the loading without causing failure in some of the members. The mid-panel truss members, and supports, and hanger pin-plates need replacing or redesigning. The lower chord pins at the mid-panels are also on the verge of being overstressed, and should be considered in the strengthening of the structure to safely carry the loading of a 10 ton truck.

It is not known just what loading was used in originally designing the bridge, but it would seem that a loading similar to the one used in this analysis was used. The members as originally designed for the bridge would, I believe, resist the loading of the 10 ton truck, but they were not adequately protected by painting or some other method. The sections have been reduced from 10 to 20 per cent because of rusting and scaling, and thus are not able to take the amount of load that they were originally designed to take. To prevent further deterioration, the bridge should be painted or rust-proofed as soon as is possible.

If the structure is to be left as it is without any strengthening or replacement of existing members, I would recommend that a load limit of seven and one-half tons-a seven and one-half ton truck-be placed on the structure to prevent any of the members being overstressed.

It would not take a great deal of extra work and expense to make the structure safe for the loading. The failing members could be removed from the truss and replaced by ones of suitable section without affecting the other members to a very great extent. The replacement of the supports will be a bit more difficult and may not be necessary. The supports are very close to being adequate and possibly they could be made satisfactory with the use of stiffeners.

In summary, the bridge will not be safe for the H-10 loading, and a maximum of $7\frac{1}{2}$ tons should be allowed on it. It should be painted to prevent further rusting and scaling, which is preceeding at an alarming rate.

BIBLIOGRAPHY

The following books were used for reference to method of procedure in the analysis of the preceding report.

The Civil Engineering Handbook by Leonard Church Urquhardt, C.E., Editor-in Chief; First Edition; 1934

Steel and timber Structures by George A. Hool, S.B. and W.S. Kinne, Editors-in-Chief; Second Edition; 1942

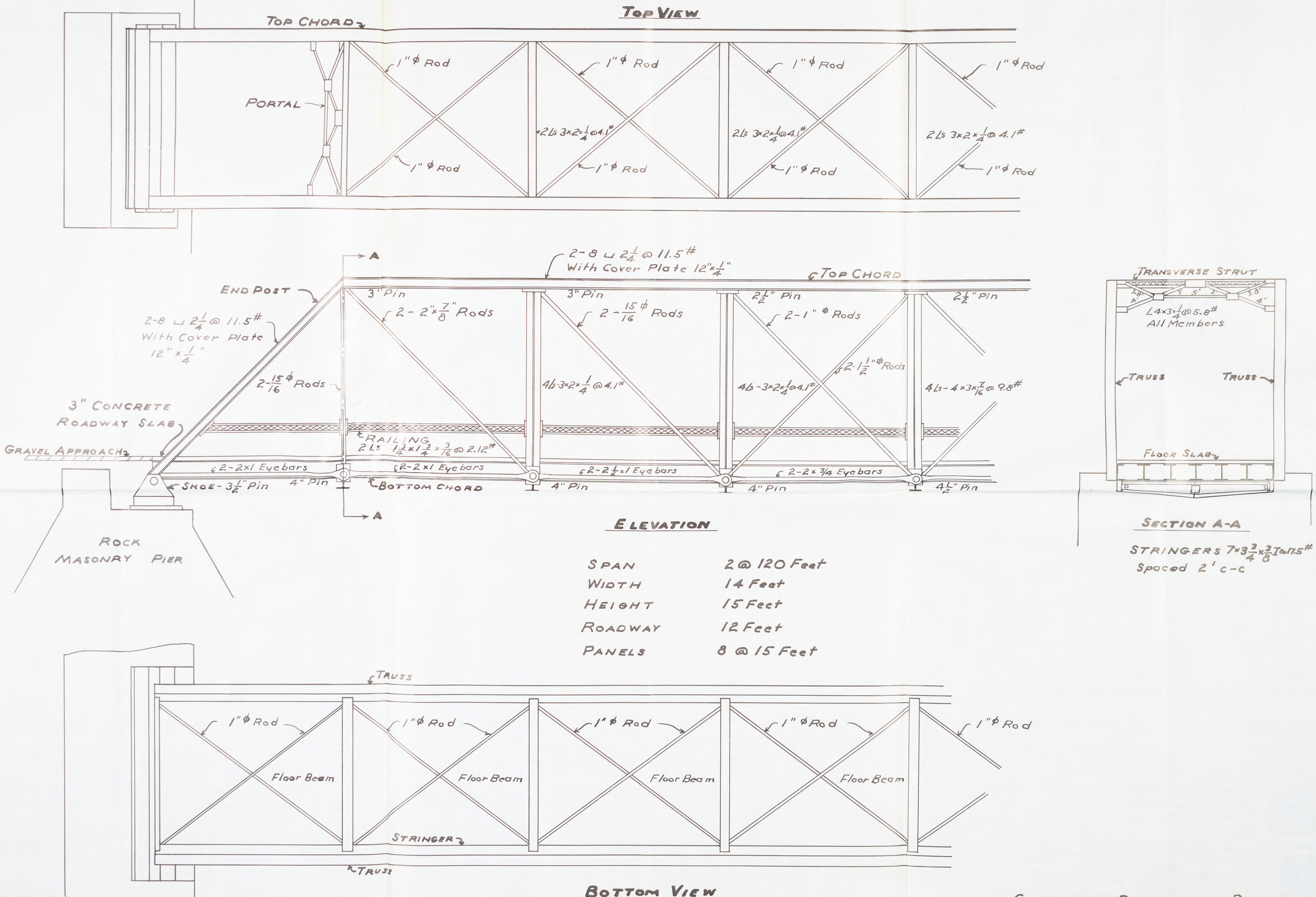
Structural Design by Carlton Thomas Bishop, C.E.; 1938

Steel Construction by the American Institute of Steel Construction, New York, N.Y.; Fourth Edition; 1941

Structural Theory by Hale Sutherland and Harry Lake Bowman; Third Edition; 1942

Mechanical Engineers Handbook by Lionel S. Marks; Fourth Edition; 1941

DELTA MILLS HIGHWAY THROUGH TRUSS



SCALE - ALL VIEWS
1 INCH = 5 FEET

GENERAL DESIGN OF DELTA MILLS, MICHIGAN THROUGH TRUSS

SCALE 1" = 5'   JUNE, 1943

Charles B. Gates