THE DESIGN AND CONSTRUCTION

OF THE

REED AVENUE CONCRETE BRIDGE

BOROUGH OF MONESSEN, PENNSYLVANIA

BY

MILTON FREDERICK STEIN

B. S., MUNICIPAL AND SANITARY ENGINEERING

UNIVERSITY OF ILLINOIS, 1909

THESIS

SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE

DEGREE OF
CIVIL ENGINEER

IN

THE GRADUATE SCHOOL

OF THE

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1916

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I HEREBY RECOMMEND THAT THE THESIS PREPARED BY

MILTON FREDERICK STEIN

ENTITLED The Design and Construction of the Reed Avenue Concrete Bridge, Borough of Monessen, Pennsylvania

BE ACCEPTED AS FULFILLING THIS PART ON THE REQUIREMENTS FOR THE

PROFESSIONAL DEGREE OF CIVIL ENGINEER

A N Talbot

Head of Department of Municipal and Sanitary Engineering.

Recommendation concurred in:

Ira O. Baker.
F. H. Newell
A N Talbot

Committee

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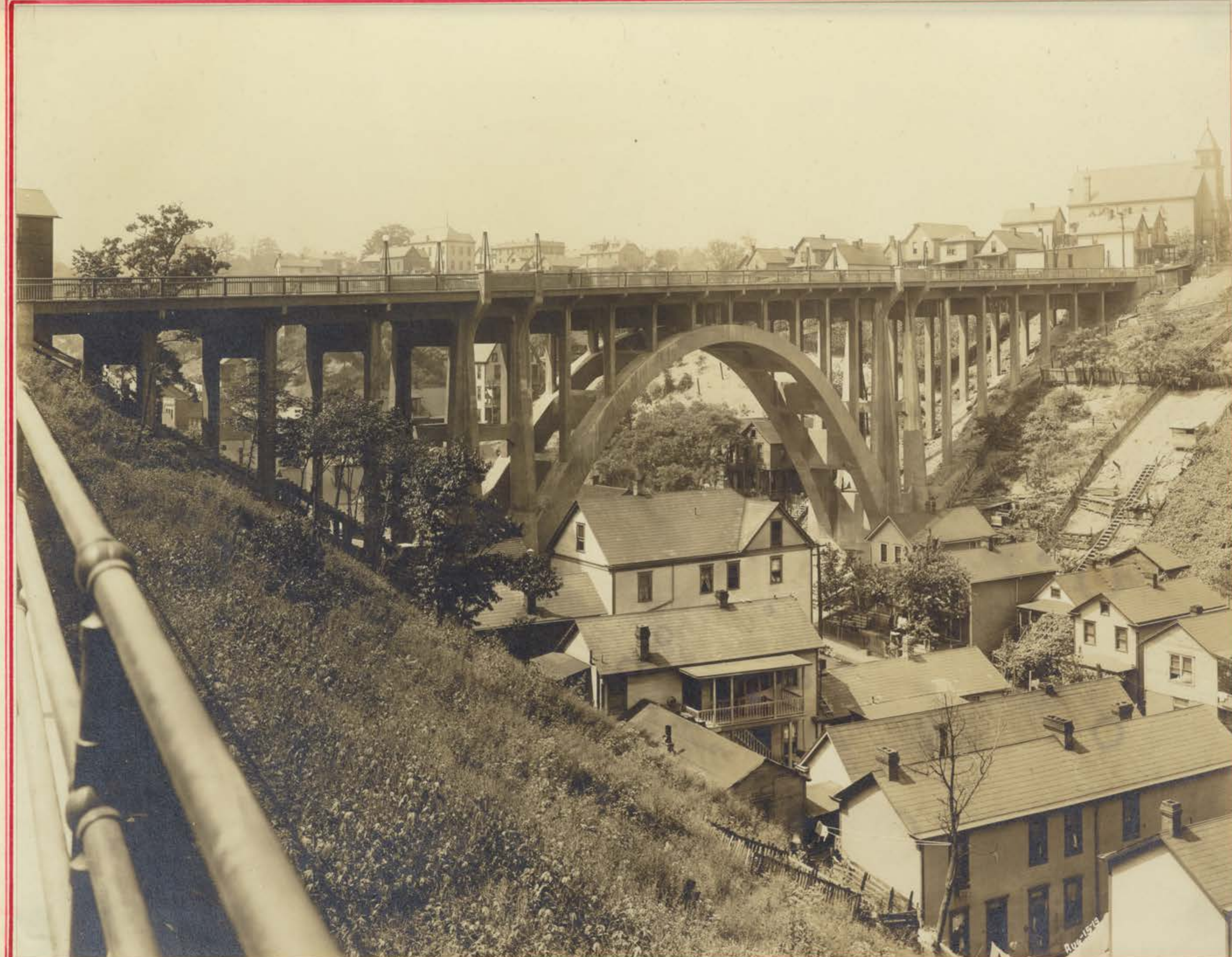

Monessen Bridge -- General View.

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The Design and Construction of the Reed Avenue Concrete Bridge, Borough of Monessen, Penna.

The Borough of Monessen is a steel manufacturing town of recent development, situated on the Monongahela River, about fprty miles above Pittsburgh. The topography of the town is peculiar to this district. Along the river is a level strip about one thousand feet wide, at an elevation of approximately fifty feet above river level (low water). This strip is devoted to steel mills, railroads and the business streets of the town. Back of this there is an abrupt rise of an hundred feet to a fairly level plateau which gradually merges into the hills behind the town. This elevated area forms the residential district of the town. The plateau is cut by several ravines through which small nameless streams flow to the river. One such ravine, whose axis is coincident with Third Street, entirely cuts off the easterly third of the residential district from the rest. In order for traffic to cross this gap it was necessary to follow a steep hillside road down to the lower level and re-ascend the other side, a detour of at least two thousand feet, whereas the direct distance across was but six hundred feet. Naturally this caused much inconvenience, and greatly retarded the growth of the easterly portion of the town.

In the autumn of 1911 the firm of Chester and Fleming, Consulting Engineers, Pittsburgh, Penna., was retained by the Borough

Council to prepare plans and specifications for a concrete bridge connecting Reed Avenue across the Third Street ravine. It was understood that the bridge should be designed with the greatest possible economy, as the appropriation had originally been made with a steel viaduct in view, and could not be increased, the Borough having practically reached its limit of bonded indebtedness. It was further stipulated that the bridge must provide for interurban traffic of the heaviest kind, and that local reinforcing steel must be used in its construction.

A survey of the site was made and test borings taken at short intervals along the axis of the proposed bridge. These showed the foundations to be of shale rock, covered only with a thin layer of disintegrated material. Obviously the design called for an arch spanning Third street and either short span arches or beam and girder construction for the two approaches. Studies showed that an arch of 150 feet span would prove most economical. A shorter span would have increased the length and size of the roadway columns and would have seriously marred the appearance of the structure. A longer span would not have decreased the cost of roadway supports materially, while making the centering and arch abutments more expensive. Concrete girders were chosen for the approaches, since they were simpler to build than arches and allowed the use of lighter piers or columns. The main piers consisted of four columns resting on the arch abutment, and well braced together.

This was a departure from the usual solid masonry main piers, and represented a considerable saving in concrete.

The unit stresses used were as follows:

a.	Concrete in compression,	500 lb. per sq. in.
b.	Concrete in diagonal tension	35 " " " "
c.	Concrete in direct shear,	150 " " " "
d.	Concrete fully reinforced for web stresses, a shearing stress of	100 " " " "
e.	Bond between Concrete and Steel - Plain bars,	80 " " " "
	Deformed "	125 " " " "
f.	Steel in tension,	16000 " " " "
g.	Steel in shear,	12000 " " " "

The floor system was designed by the usual methods for reinforced concrete. Details of the computations are given in Appendix B. The loadings were, in addition to the dead load:

A. Uniform Live Loads:

a.	For Slabs	100 lb. per sq. ft.
b.	For Beams,	100 " " " "
c.	For Girders,	80 " " " "
d.	For Whole Bridge,	80 " " " "

B. Concentrated Live Loads:

Two 50 Ton electric cars, with 25 per cent. impact allowance.

The roadway and sidewalk slab, beams and cross girders at the panel points involve no unusual design. The two columns and cross girder were considered to be rigidly connected, acting as a bent. Under this assumption some of the bending moment in the girder was transmitted to the columns. Since the columns were rigidly anchored into the rock at their bases, they were considered as fixed, and as having a point of contraflexure at their middle point. The magnitude of the horizontal force H, acting at this point, was found by the usual formula:

Should be 1/3 height

$$H = \frac{\left(\frac{l}{h}\right)^2}{12\left(\frac{2}{3} \times \frac{I_1}{I} + \frac{l}{h}\right)} \times \frac{3}{2}(2W + wl)$$

Where: l Span of girder in feet

h length of column in feet (divided by two for fixidity)

I_1 Moment of inertia of girder

I Moment of inertia of column.

W Concentrated load on girder.

w Uniform load on girder.

Then H h is the moment at junction of column and girder.

Since the slenderness ratio of the columns was large, the allowable stress was determined by the formula:

$$P'' = \frac{P'}{1 + \frac{1}{20000} \times \frac{h^2}{\frac{I}{A}}} \times (1 + .14p)$$

Where: P" Allowable Stress in pounds per square inch.

P' Unit Stress per square inch for short columns.

h Length of column.

I Moment of inertia of column.

A Area of Column.

p Percentage of steel in column.

The stresses due to the lateral force of a wind load of 25 pounds per square foot on the side of floor stringers, columns and passing car were also computed. This gave an additional compression in the leeward column, and certain stresses and moments in the girder and columns. It is probable that much of the wind load is carried directly to the main piers, abutments and shorter columns of the bridge by means of the floor slab and floor beams, which together would act as a very stiff horizontal beam.

Besides the direct load coming on the main piers, these were designed to take a longitudinal thrust of 20000 pounds acting horizontally at the top on each side. This represents 0.2 the weight of one electric car on each track. They were also stiffened laterally to take the wind load of half the floor system over the arch and an equal length of floor system of the approach.

To allow for expansion and contraction of the floor system due to temperature, expansion joints were put in the floor system on the arch side of each of the main piers, and where the floor stringers rest on the retaining walls at both ends of the bridge.

Monessen Bridge -- Arch and Floor System.

Monessen Bridge -- Floor System From Beneath.

These consisted of double bronze plates set into the floor stringers, planed smooth on the contact surfaces. A concrete slip joint was used to carry the pavement across the expansion joints, and cast iron plates to cover the joints across the sidewalk.

The arch was designed by the so called elastic theory, based on the principle of least work. In order to make the analysis less burdensome and amenable to graphical methods, several assumptions were made which seem justifiable in view of the negligible departure from the true results incident thereto. Thus the axis of the arch rings was assumed to be a parabola, although in reality a three centered arch. The width of the arch ring was kept constant and the depth made to vary as the secant of the angle with the vertical. The work due to axial and shear stress was neglected, being inconsequential as compared to that of the moment.

The essential steps in developing the elastic theory of a hingeless arch are as follows:

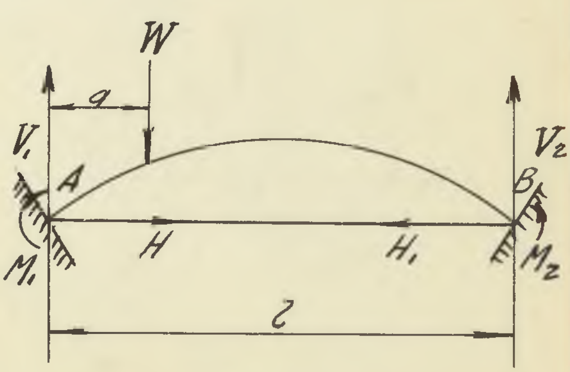


Fig. 1

Referring to Fig. 1 which shows a symmetrical arch rib loaded vertically with W, let M_1 and M_2 represent the moments at A and B respectively. H and H^1 are the horizontal components of the re-actions, and V_1 and V_2 are the corresponding vertical components of the reaction. Since the loading is vertical:

$$H - H' = 0$$

$$V_1 + V_2 - W = 0$$

For the moment at any point distant x from A,

$$m = M_1 + V_1 x - Hy , \text{ for } x < a$$

$$m = M_1 + V_1 x - Hy - W(x-a), \text{ for } X > a;$$

for the vertical shear;

$$V = V_1 \quad \text{for} \quad x < a$$

$$V = V_1 - W \quad \text{for} \quad x > a$$

Since at any section of the rib, wherever the resultant of the external forces does not pass through the center of gravity, a moment will be caused at the section, and furthermore, if the direction of the resultant does not coincide with that of the tangent to the axis of the rib, the latter, besides being axially compressed, will be subjected to a tengential or shear stress at the section. At any point x,y of the neutral axis of the rib, then, referring to Figs. 1 and 2, we have, after decomposing R into H and V:

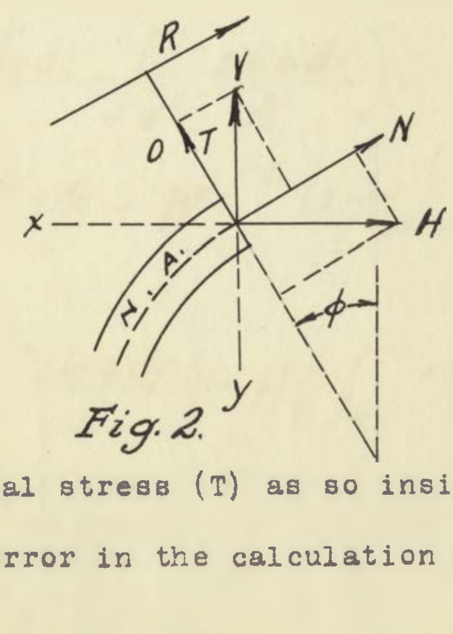


Fig. 2.

$$N = -(V \sin \phi + H \cos \phi)$$

$$T = -(V \cos \phi - H \sin \phi)$$

Neglecting the effect of the tangential stress (T) as so insignificant that it causes no sensible error in the calculation of

the internal ...

the internal work, this is

$$\omega = \int_0^{l_1} \frac{m^2 dc}{2EI} + \int_0^{l_1} \frac{N^2 dc}{2AE}$$

where A is the normal cross section of the rib at x y, and

$$dc = \frac{dx}{\cos\phi}$$

substituting in this the values of m and N,

$$\omega = \int_0^{a'} \frac{(M_1+V_1x-Hy)^2 dc}{2EI} + \int_{a_1}^{l_1} \frac{\{M_1+V_1x-Hy-W(x-a)\}^2 dc}{2EI}$$

$$+\int_0^{a'} \frac{(V_1\sin\phi+H\cos\phi)^2 dc}{2EA} + \int_{a_1}^{l_1} \frac{\{(V_1-W)\sin\phi+H\cos\phi\}^2 dc}{2EA}$$

where a' and l' are measured along the rib. Differentiating successively with respect to H, M, and V, and equating to zero for a minimum.

$$\frac{d\omega}{dH} = M_1\int_0^{l_1} \frac{y\,dc}{I} + V_1\left(\int_0^{l_1} \frac{xy\,dc}{I} - \int_0^{l} \frac{\sin\phi\,dx}{A}\right) - H\left(\int_0^{l_1} \frac{y^2 dc}{I}\right.$$

$$\left.+\int_0^{l_1} \frac{\cos\phi\,dx}{A}\right) - W\left(\int_{a'}^{l_1} \frac{(x-a)y\,dc}{I} - \int_a^{l} \frac{\sin\phi\,dx}{A}\right) = 0 \quad (1)$$

$$\frac{d\omega}{dM_1} = M_1\int_0^{l_1} \frac{dc}{I} + V_1\int_0^{l_1} \frac{x\,dc}{I} - H\int_0^{l_1} \frac{y\,dc}{I} - W\int_{a_1}^{l_1} \frac{(x-a)}{I}dc = 0 \quad (2)$$

$$\frac{d\omega}{dV_1} = M_1\int_0^{l_1} \frac{x\,dc}{I} + V_1\left(\int_0^{l_1} \frac{x^2 dc}{I} + \int_0^{l} \frac{\sin\phi\,dy}{A}\right) - H\left(\int_0^{l_1} \frac{xy\,dc}{I}\right.$$

$$\left.-\int_0^{l} \frac{\cos\phi\,dy}{A}\right) - W\left(\int_{a_1}^{l_1} \frac{x(x-a)dc}{I} + \int_0^{l} \frac{\sin\phi\,dy}{A}\right) = 0 \quad (3)$$

As to M_2 and V_2 we have:

$$M_2 = M_1 + V_1 l - W\,(l-a),$$

$$V_2 = W - V_1$$

Assuming the rib to be of uniform width and neglecting the steel, which has small effect on the moment of inertia, and that the depth of the rib varies as the secant of ϕ, so that

$$I = I_0 \sec \phi$$

$$A = A_0 \sec \phi$$

where I_0 and A_0 are respectively the moment of inertia and cross section, at the crown of the arch; and assuming the arch is a parabola whose equation is

$$y = \frac{4\,h}{l^2}\; x\,(l-x)$$

remembering, further, that $dc = \dfrac{d\,x}{\cos\phi}$; substituting in equations (2) and (3) and integrating, we get

$$M_1 l + V_1 \frac{l^2}{2} - H \frac{2}{3} h l - W \frac{(l-a)^2}{2} = 0$$

$$M_1 \frac{l^2}{2} + V_1 \left(\frac{l^3}{3} + \frac{i^2 (4hl - l^2 \phi_0}{4h} \right) - H \frac{hl^2}{3}$$

$$- W \left(\frac{(l-a)^2 (2l+a)}{6} + \frac{i^2 (8h(l-a) - l^2(\phi_a + \phi_0))}{8h} \right) = 0$$

where $i'^2 = \frac{I_o}{A_o}$ and ϕa and ϕo are the angles of the tengents at A and a respectively. Let

$$\frac{4\,h l - l^2 \phi o}{4\,h} = n \qquad \text{and}$$

$$\frac{8\,h\,(l-a) - l^2(\phi a + \phi o)}{8\,h} = m.$$

Then eliminating V_1 and M_1 successively:

$$M_1 = \frac{1}{l^4 + 12 l n i^2}\Big[H\big(\tfrac{2h l^4}{3} + 8 h l n i^2\big) - W\big((a l^2 - 6 n i^2)(l-a)^2 + 6 l^2 m i^2\big)\Big] \qquad (4)$$

$$V_1 = \frac{W}{l^3 + 12 n i^2}\big((l-a)^2(l+2a) + 12 m i^2\big) \qquad (5)$$

Neglecting the effect of axial stress, the terms containing i'^2 disappear, and

$$H = \frac{15\,a^2\,(l-a)^2}{4\,h l^3} \times W \qquad (6)$$

$$M_1 = \frac{(l-a)^2(5a^2 - 2al)}{2 l^3} \times W \qquad (7)$$

$$V_1 = \frac{(l-a)^2(l+2a)}{l^3} \times W \qquad (8)$$

In the graphical solution the reaction locus and envelope are required. Since the end moments are due to the departure of the lines of reaction from the axis of the arch at these points,

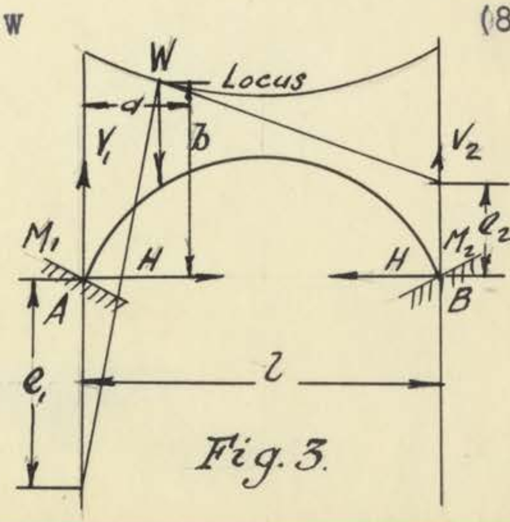


Fig. 3.

if we represent by e_1 and e_2 these departures above or below a horizontal connecting the ends of the arch, taking upward as positive and downward as negative it follows that

$$e_1 = \frac{M_1}{H} \qquad \text{and} \qquad e_2 = \frac{M_2}{H}$$

and since,

$$\frac{V_1}{H} = \frac{b-e_1}{a}, \quad , \quad b = e_1 + \frac{V_1 a}{H} = \frac{M_1 + V_1 a}{H} \qquad \text{(a)}$$

$$\frac{V_2}{H} = \frac{b-e_2}{l-a} \qquad b = e_2 - \frac{V_2}{H}(l-a) = \frac{M_2 - V_2(l-a)}{H} \qquad \text{(b)}$$

(a) and (b) are the equations of the reaction locus.

If x_1 and Y_1 are the coordinates - origin A- of a point in the line of reaction tangent to the envelope

$$y_1 = e_1 + \frac{V_1}{H}x_1 = e_1 + \frac{b - e_1}{a}x_1 \qquad \text{(c)}$$

Substituting for V_1, H, and M_1 their values from equations 6, 7, and 8 we get

$$e_1 = -\frac{2h(2l-5a)}{15a} \qquad (9)$$

$$b = \frac{6}{5}h \qquad (10)$$

e.i., the reaction locus is a straight line.

Substituting the values of e and b in (c)

$$Y_1 = -\frac{2h(2l-5a)}{15a} + \frac{4h(2a+l)}{15a^2}x_1$$

which is the equation of the tangent to the envelope at the point $x_1 y_1$. In this equation a is the independent variable, since the tangent line varies with changing positions of the load W. Differentiating with respect to a

$$a = \frac{2h(l-2x_1)}{5(2h-3y_1)}$$

Substituting this value of a in the above equation and transferring the origin of coordinates to the center of the span, $(x_1 = \frac{l}{2})$,

$$8hx^2 + 15l^2y + 30lxy = 0 \qquad (11)$$

which is the equation of the envelope.

To adapt equation (9) to graphical methods let the distance from the center of the arch to the loads be x, then $a = \frac{l}{2} - x$ to the left of the center line and $\frac{l}{2} + x$ to the right of the center line. Substituting in equation (9)

$$e_1 = \frac{2}{15}h\left(\frac{l+10x}{l+2x}\right) \qquad (9a)$$

$$e_2 = \frac{2}{15}h\left(\frac{l-10x}{l-2x}\right) \qquad (9b)$$

A change of temperature of t degrees would produce a change of tlc in the span of the arch, where c is the coefficient of expansion. Since the ends of the arch are fixed a reaction and moment is produced -- H_t and M_t -- at the support A. The general equation for internal work is:

$$\omega = \int_0^{l_1} \frac{m^2 dc}{2EI} + \int_0^{l_1} \frac{N^2 dc}{2AE}$$

$$m = M_t - H_t y \qquad N = H_t \cos \phi$$

Substituting

$$\omega = \int_0^{l_1} \frac{(M_t - H_t y)^2 dc}{2EI} + \int_0^{l_1} \frac{(H_t \cos \phi)^2 dc}{2AE}$$

Since by Castigliano's first theorem: "The displacement of the point of application of an external force acting on a body -- caused by the elastic deformation of the latter -- is equal to the first derivative of the work of resistance performed in the body, with respect to the force,"

$$\frac{d\,\omega}{d\,H_t} = tcl \qquad \text{and} \qquad \frac{d\,\omega}{d\,M_t} = 0$$

$$\frac{d\,\omega}{d\,H_t} = \int_0^{l_1} \frac{-M_t\, y\, dc + H_t\, y^2\, dc}{E\,I} + \int_0^{l_1} \frac{H_t \cos^2 \phi\; d\,c}{A\,E} = tcl$$

$$\frac{d\,\omega}{d\,M_t} = \int_0^{l_1} \frac{(M_t - H_t y)\, dc}{E\,I} = 0$$

whence

$$H_t = \frac{t\,c\,l\,E}{\int_0^{l_1} \frac{y^2 dc}{I} + \int_0^{l_1} \frac{\cos \phi\, dx}{A} - \frac{\left(\int_0^{l_1} y\, \frac{d}{I}c\right)^2}{\int_0^{l_1} \frac{dc}{I}}}$$

$$M_t = \frac{\int_0^{l_1} \frac{dc}{yI}}{\int_0^{l_1} \frac{dc}{I}}\; H_t$$

Remembering that $y = \frac{4\,h}{l^2}\, x\,(l - x)$

and $I = I_o \sec \phi$ $\quad A = A_o \sec \phi$ and integrating

$$H_t = \frac{t\,c\,E\,I_o}{\frac{4\,h^2}{45} + \frac{i^2\,l\,\phi_o}{4\,h}}$$

$$M_t = \frac{2}{3}\,h\,H_t \tag{10}$$

Neglecting axial stress,

$$H_t = \frac{45\,t\,c\,E\,I_o}{4\,h^2} \tag{11}$$

The writer claims no originality for the above analysis which follows closely that given in a number of text books.

Applying the formulas derived to the graphical analysis of the Monessen arch:

Since in a bridge of this size, the live load effects are very small as compared with those due to dead load, and it is most economical to have the arch thrust coincide as closely as possible with the axis of the arch, trial analyses were made with dead load only until an arch was found in which the line of pressures followed closely the axis of the arch.

As our theory was developed for a parabolic arch, it is necessary to choose a parabola as the axis of the arch, or else a series of circular curves which will approach closely such a parabola. In the latter case it is usually necessary to find the rise of an <u>equivalent parabola</u>, which is the parabola enclosing an area between the curve and the chord equal to the area between the arch axis and its chord. In the present instance the agreement is so close that this computation is needless. Where necessary this can be readily computed by the principles of analytical geometry.

The arch axis is drawn to a convenient scale and the thickness of the arch ring laid off for various points, remembering that this varies as secant ϕ. Next the intersection locus is drawn at a height of $\frac{6}{5}$ h above the springing line. The values of e_1 and e_2 are then computed for each panel point and laid off on vertical lines through the right and left supports. The panel loads are then computed, the weight of the arch rib for half a span length on each side of the panel point being considered as concentrated thereat.

From the intersections of verticals through the panel points with the intersection locus lines 1-1, 2-2, are drawn to the respective points e_1, e_2. It will be seen that these lines give the directions of the reactions for single loads placed at the various panel points.

The Component Diagram for the dead loads is next laid off as follows: the dead load computed for panel point No. 1 is laid off as the vertical line 0-1. From 1 a line 1-a is drawn parallel to the right reaction line in the upper diagram and a line 0-a parallel to the left reaction. The length of these lines gives the magnitude of the reactions at the abutments for the panel load 1. From a a vertical is erected equal in length to the panel load at 2, and lines are drawn from a and 2 parallel to the reactions, intersecting at b. The panel loads at 3, 4, 5, 6, 7, 8 and 9 are treated in the same manner. Similarly the right reactions V-a , a -b , b -c , etc. are laid off. The left reactions of all the panel loads have now been graphically added together. V-0 represents the resultant thrust due to dead loads at the left abutment, and may be resolved into the vertical reaction and horizontal thrust. The fact that the vertical reaction is equal to the panel loads on the left half of the span checks the graphical work so far done.

To draw the line of pressure for the dead load a pole P-DL is arbitrarily chosen and a horizontal line P-O is drawn at any point above the arch. From the intersection of P-O and component line 1-1, prolonged, a line $p-a$ is drawn parallel to P-a to an intersection with component line 2-2 prolonged. Similar lines are drawn parallel to P-b, P-c, etc., intersecting on the prolonged component lines 3-3, 4-4, etc. The closing lines P-O and P-V intersect at V_1. This diagram is called the reciprocal polygon. From V_1 a line is drawn parallel to V-O of the dead load component diagram. This line is the first portion of the dead load pressure line within the arch, and also the resultant thrust line for the dead load of the arch.

Now, the line P-a of the reciprocal polygon is continued until it intersects the closing line P-V. From this intersection is drawn a line a-V, parallel to a-V in the component diagram. This line intersects the right hand reaction of panel load 1 at r. From r a line is drawn parallel to a-a′ of the component diagram. Thus all the forces acting at a section to the right of panel point 1 have now been added together, and their resultant is given by a-a′ in magnitude and $r-s_1$ in direction and location within the arch. This line is the second portion of the pressure line and intersects the first portion on the vertical through the panel load 1.

The third portion is found by extending p-b until it intersects P-V of the reciprocal polygon, drawing b-V parallel to b-V of the component diagram to intersection with the resultant of the right hand reactions of panel loads 1 and 2, which acts through the intersection of these two reactions in a direction parallel to V-b′ of the component diagram. From this intersection a line is drawn paralleling b′-b of the component diagram, this being the third portion of the line of pressure. In the same manner the remaining portions of the pressure line are determined. The lines a-a′, b-b′, etc. in the component diagram give the magnitudes of the thrusts and by resolving these tangentially and normally to the axis of the arch the axil thrusts and shears are found. By multiplying the thrust at any point by the perpendicular distance from the line of pressure to the arch axis, the moment at that point is found.

In computing the effect of the live loads a uniform loading equivalent to the interurban cars specified was first calculated. The cars were so placed as to cause a maximum bending moment at the quarter point of the arch span, considered as a beam. For a uniform load over the entire span of a beam, the moment at the quarter point is $3/32\, p l^2$. Equating the moment at the quarter point $M_4 = 3/32 p l^2$ or $p = 32/3\, M_4 \div l^2$. Allowing 80 pounds per square foot live load on the sidewalks (the cars on both tracks completely covering the roadway) and multiplying by the roadway span (15 feet) gave a load of 39,000 pounds per panel point on each rib. This was later reduced to 22,500 pounds, being considered too severe. From these live loads a component diagram and reciprocal polygon were worked up as before.

The placing of the live load was assumed to be such as to cause the maximum stress in the outer fibers of the arch ring at each panel point. Referring to Fig. 4, since the stress in the outer fiber C is equal to the moment with respect to D divided by d, the reaction line passing through D and produced to the intersection locus will indicate the position of a load producing no stress in C. All loads to the left of O produce compression in C, and all loads to the right, tension. Similarly, a reaction line through C determines the position of load for no stress at D; all loads to the left producing tension, and to the right, compression. Applying this to the case in question, for panel point 1 or 9, panel points 5 to 8 must be loaded for maximum tension in the top fiber, and compression in the bottom fiber. In the L.L. component diagram panel components 5 to 8 inclusive are inclosed by lines P-a and P-e. Continue lines P-a and P-e to intersection and from the intersection draw a line e-a parallel to e-a of the

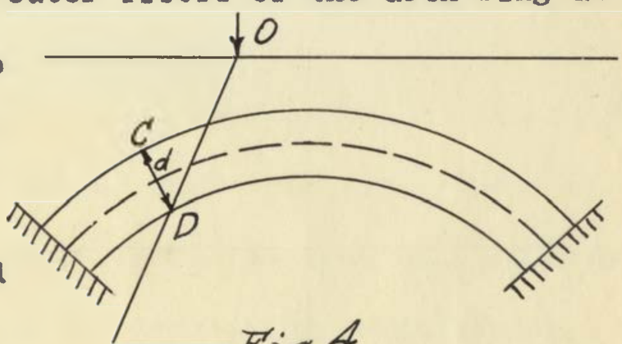


Fig. 4.

component diagram. This line is the thrust, and its distance 2.0 feet from the arch axis is the moment arm. The bending moment in the arch is 2 X 154,600 X 12 = 3,710,000 in lb., and the axial thrust, found by resolving the thrust tangential and normal to the arch axis is 145,600 lb.

The horizontal thrust due to temperature is found by the formula,

$$H = \frac{45\ E\ I_o\ t\ \theta}{4\ h^2}$$

as already explained. This horizontal thrust is first computed for $t = 1^\circ$ Fahr., and then multiplied by +20 degrees and -40 degrees to give the thrusts for increasing and decreasing temperatures. This thrust acts along a horizontal line at a height of 2/3 h above the springing line. The moment at each panel point is obtained by multiplying the horizontal thrust by its vertical distance above the axis of the arch. The axial thrust is obtained by multiplying the horizontal thrust by the corresponding factor in the component diagram for temperature and rib shortening.

The moments and axial thrusts for rib shortening are found last, after an average stress has been estimated for live, dead and temperature moments and thrusts. The formula used is:

$$H = - \frac{f\ c\ Io}{4\ h^2}$$

The computations are the same as for temperature stresses, f_c being the estimated average stress.

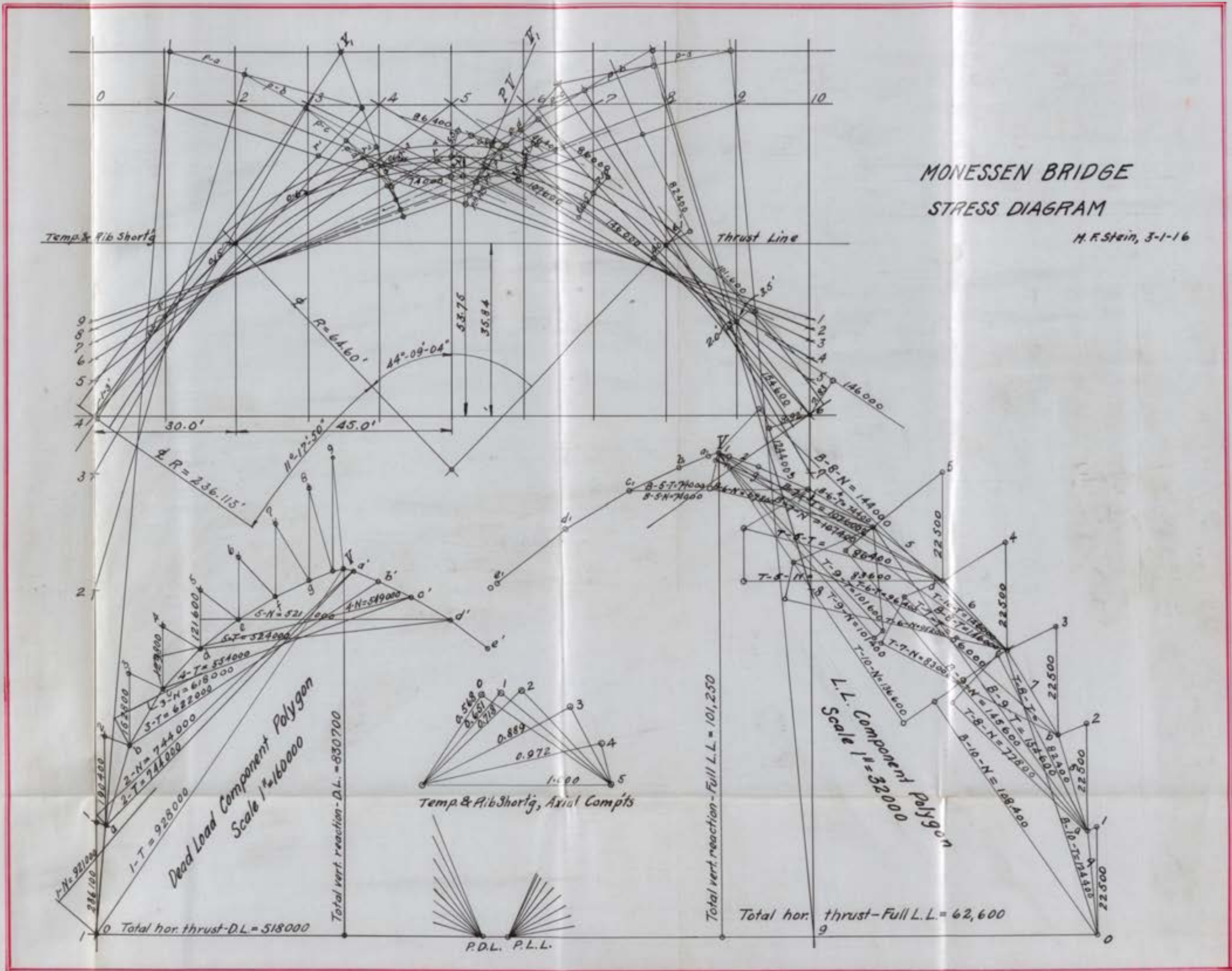
Arch Centering and Forms.

Arch After Pouring.

Centering of Main Piers and Approaches.

MAXIMUM AND MINIMUM STRESSES

Panel Point	No. bars req'd	d = Total depth, b = " breadth, A_t = Transformed area, I_t = " I		Dead Load N=Axial thrust M=Moment	Temperature N=Axial thrust M=Moment	Rib Shortening N=Axial thrust M=Moment	Live Load N=Axial thrust M=Moment	Position of L.L. for Max. Stress.	Maximum Moment and thrust	$\frac{My}{I}$	$\frac{N}{A_t}$	f_c
5	13@1½"□ =29¼ sq.in	d=54" b=60" Ac=54x60=3240	Top	N=+ 521,000	N=+ 12000 M=− 2,596,000	N=− 9 000	N=+ 84000 M=− 3996 000	− = pts 1,2,3	N=+ 608000 M=− 8 627000	−190	+150	− 40
	p=.009	As= 58½ 820 At= 4060	d/2 = 23"	M=− 4,024,000	N=− 24000 M=+ 5,192,000	M=+ 1989000	N=+ 74000 M=+5702000	+ = " 4,5,6	N=+ 572000 M=+ 8 859000	+196	+140	+336
		Ic= 787300 Is= 433800	y= 27"	N=+ 521,000	N=− 24000 M=− 2,192,000	N=− 9000	N=+ 74000 M=−5702 000	− = " 4,5,6	N=+ 572000 M=− 8 859000	−196	+140	−56
	13@1½=29¼	It=1,221,100	Bottom	M=+ 4,024,000	N=+ 12000 M=+ 2596000	M=− 1989000	N=+ 84000 M=+3996 000	+ = "1,2,3+7,8,9	N=+ 608000 M=+ 8 627000	+190	+150	+340
4 & 6	13@1½□ =29¼ sq in	d=55.6" b=60" Ac=55.6x60=3336	Top	N=+ 549000	N=+ 12000 M=− 2335000	N=− 9000	N=+ 70000 M=−3,125,000	− = " 1,2,3,4,9	N=+ 622000 M=−7920000	−166	+150	− 16
	p=.0095	As=58½ 820 At= 4156	d/2=23.8	M=−4255000	N=− 23000 M=+ 4669000	M=+1795000	N=+ 95000 M=+5900000	+ = " 5,6,7,8	N=+ 612000 M=+8109000	+170	+147	+317
	13@1½□ = 29¼	Ic= 859400 Is= 464100	y=27.8	N=+ 549000	N=− 23000 M=− 4669000	N=− 9000	N=+ 95000 M=−5900000	− = " 5,6,7,8	N=+ 612000 M=− 8109000	− 170	+ 147	− 23
		It=1323500	Bottom	M=+4255000	N=+ 12000 M=− 2336000	M=−1795000	N=+ 70000 M=−3,125000	+ = " 1,2,3,4,9	N=+ 622000 M=+ 7920000	+ 166	+150	+316
3 & 7	13@1½□" = 29¼	d= 61" b= 60" Ac=61x60=3660	Top	N=+ 618000	N=+ 11000 M=− 1246000	N=− 8000	N=+ 107000 M=−8,522,000	− = " 1,2,3,4,5	N=+ 728000 M=−13288000	− 212	+162	− 50
	p=.0085	As= 58½ 820 At= 4480	d/2=26.5	M=− 4478000	N=− 23000 M=+ 2493000	M=+ 958000	N=+ 83000 M=+ 7327000	+ = " 6,7,8,9	N=+ 670000 M=+ 6300000	+ 106	+150	+ 250
		Ic= 1134900 Is= 762600	y= 30.5	N=+ 618000	N=− 23000 M=− 2493000	N=− 8000	N=+ 83000 M=− 7327000	− = " 6,7,8,9	N=+ 670000 M=− 6300000	− 100	+150	+ 50
	do.	It=1,897,[illegible]00	Bottom	M=+4,478,000	N=− 11000 M=+ 1246000	M=− 958000	N=+ 107,000 M=+8522000	+ = " 1,2,3,4,5	N=+ 728000 M=+13288000	+ 212	+162	+ 374
2 & 8	13@1½□=29¼	d= 75.2" b=60" Ac= 75x60 = 4500	Top	N=+ 724000	N=− 17000 M=− 10000	N=− 7000	N=+ 144000 M=−7154000	− = " 1 to 6	N=+ 844000 M=−8504000	− 75	+158	+ 83
	p=.007	As= 58½ 820 At= 5320	d/2=33.6	M=− 1303000	N=+ 9000 M=+ 5000	M=− 37000	N=+ 78000 M=+6,040,000	+ = " 7,8,9	N=+ 864000 M=+ 4,705,000	+ 41	+150	+ 191
		Ic= 2,109000 Is= 925800	y=37.6	N=+ 724000	N=+ 9000 M=− 5000	N=− 7000	N=+ 78000 M=−6,040,000	− = " 7,8,9	N=+ 804000 M=−4,705000	− 41	+150	+ 109
	do	It=3,034800	Bottom	M=+ 1303000	N=− 17000 M=+ 10000	M=+ 37000	N=+ 144000 M=+ 7154000	+ = " 1 to 6	N=+ 844000 M=+ 8504000	+ 75	+158	+ 233
1 & 9	13@1½□=29¼	d= 84" b=60" Ac=84x60=5040	Top	N=+ 921000	N=+ 8000 M=− 2363000	N=− 6000	N=+ 146000 M=−3710000	− = " 5 to 9	N=+ 1069000 M=− 2884000	− 29	+184	+ 155
		As= 58½ 820 At 5860	d/2 = 38"	M=+4,454000	N=− 16000 M=+ 4726000	M=− 1265000	N=+ 101000 M=+4267000	+ = " 1 to 4 & 9	N=+ 1000000 M=+12182000	+123	+170	+ 293
		Ic= 2953500 Is= 1184100	y= 42"	N=+ 921000	N=− 16000 M=− 4726000	N=− 6000	N=+ 101000 M=− 4267000	− = 1 to 4 & 9	N=+ 1000000 M=−12182000	−123	+170	+ 47
	do	It= 4,137,600	Bottom	M=− 4454000	N=+ 8000 M=+ 2363000	M=+ 1265000	N=+ 146000 M=+ 3710000	+ = 5 to 8	N=+ 1069000 M=+ 2884000	+ 29	+184	+ 213
0 & 10	13@1½"=29¼	d= 95½" b=60 Ac=95½x60=5730	Top	N=+ 928000	N=− 14000 M=− 10389000	N=− 5000	N=+ 108000 M=− 2111000	− = " 7 to 9	N=+ 1017000 M=−30976000	−208	+147	− 61
	12@1½=27 As= 56¼	As= 85½ 1197 6927	d/2=43¾	M=−14477000	N=+ 7000 M=+ 5194000	M=− 3999000	N=+ 137000 M=+ 13724000	+ = " 1 to 6	N=+ 1067000 M=+ 472000	+ 32	+154	+ 186
		Ic= 4,354,900 Is= 2,729,100	y=47¾	N=+ 928000	N=+ 7000 M=− 5194000	N=− 5000	N=+ 136000 M=− 13724000	− = " 1 to 6	N=+ 1067000 M=− 472000	− 32	+154	+ 122
	13@1½"=29¼	I = 7,084,000	Bottom	M=+14477000	N=− 24000 M=+ 10389000	M=+ 3999000	N=+ 108000 M=+ 2111000	+ = " 7 to 9	N=+ 1017000 M=+30976000	+208	+147	+ 355


MONESSEN BRIDGE
STRESS DIAGRAM
H. F. Stein, 3-1-16
Temp. & Rib Short'g
Thrust Line
R = 64.60'
44°-09'-04"
53.75
35.84
30.0'
45.0'
℄ R = 236.115'
11°-17'-30"
Dead Load Component Polygon
Scale 1" = 160000
Total vert. reaction - D.L. = 830700
Total hor. thrust - D.L. = 518000
1-T = 928000
2-T = 744000
2-N = 744000
3-T = 622000
3-H = 618000
4-T = 554000
5-T = 524000
5-N = 521000
4-N = 519000
Temp. & Rib Short'g, Axial Comp'ts
0.568
0.651
0.718
0.889
0.972
1.000
P.D.L.
P.L.L.
Total vert. reaction - Full L.L. = 101,250
L.L. Component Polygon
Scale 1" = 32000
Total hor. thrust - Full L.L. = 62,600
B-8-N = 144000
22500
146000

The various moments and thrusts for each panel point are now added together and tabulated. From these the moment stress is computed by the usual formula $S = M\frac{c}{I}$ and the axial stress by dividing the axial thrust by the normal section of the arch at the point under consideration. Summations then yield the maximum and minimum stresses, from which the safety of the arch is estimated.

Incidental computations for the arch were the determination of stresses in the ribs, before the floor system was poured and at various stages during the latter process. For these determinations Scheffler's method sufficed. Other details of the design were the pavement,- ordinary paving brick on a sand cushion, with grouted joints--road drainage, and lighting.

The character of material and sequence of operations are indicated by the following excerpts from the Specifications:

(18) The concrete used on this work will be divided into two classes. Class (A) will consist of one part Portland Cement, 2½ parts sand, and 5 parts broken stone or gravel. Class (B) will consist of one part Portland Cement, 3 parts sand and 6 parts broken stone or gravel. These materials are to be accurately measured by volume in boxes or barrels. The proportions of ingredients may be varied slightly by the Engineers in charge but the amount of cement must remain the same.

(19) All reinforced concrete is to consist of Class (A) concrete.

(20) Abutments for arch, foundations for piers, and other plain concrete is to consist of Class "B" concrete.

(30) The following order is to be observed in building concrete sections of arch:

(1) Concreting abutments;
(2) Concreting the ribbed sections;
(3) Concreting the key-ways;
(4) Concreting the cross struts between arch ribs;
(5) Concreting the columns and cross beams; and
(6) Concreting the floor system.

(31) The ribs shall be built in sections as shown with key-ways between faces of concrete in adjoining sections. Exact length of sections concreted shall be subject to the approval of the Engineers, determined after plans for centering have been approved. The arch ribs shall be built out to the first joint at the same time as abutment and in such a manner as to effect a thorough bond between ribs and abutments. Each section of arch rib shall be concreted in one continuous operation and the work shall be done in the following order:

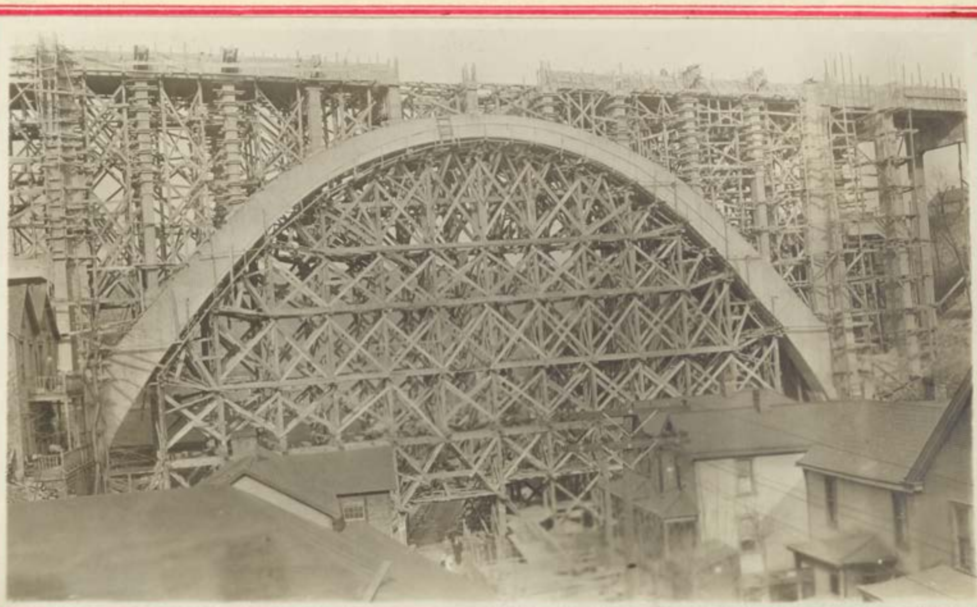
First. The two corresponding sections at the crown of the arch, marked 1-1.

Second. The two corresponding sections marked 2-2 at haunches of the arch.

Third. The two corresponding sections marked 3-3.

The bases of the spandrel columns shall be concreted at the same time as the arch section on which they rest and reinforcing material for columns is to project a bonding length above column base as shown on plans. The arch ribs shall be thoroughly braced

General View Showing
Approach Forms and Cableway.



Forms For Floor System Over Arch.

to prevent lateral deflection until after all concrete is placed in the floor system over arch span.

(32) The splices for steel reinforcement in arch ribs are to be arranged to develop full strength of reinforcement and no splices are to come within 6 feet of key-ways. After all sections of any arch rib are concreted, the Contractor shall fill the key-ways between the sections with concrete and complete the concreting of all the key-ways in one arch rib on the same day.

(42) Steel reinforcement must be made by the open hearth process and shall have an ultimate strength from sixty to seventy thousand pounds per square inch and an elastic limit of not less than 50 per cent. of the ultimate tensile strength. Minimum elongation is to be 25 per cent. in 8 inches and reduction of area at the point of fracture to be not less than 50 per cent. It shall be free from defects and contain not over 0.04 of 1 per cent. of phosphorous of 0.04 of 1 per cent. of sulphur. Steel must be protected from weather and cleaned of rust, with wire brushes,before placing. On a field test for ductility it must bend 180 degrees on a circle of its own diameter.

Bids were received as per the following tabulation, which also shows how the cost of the Monessen bridge compares with other large concrete bridges in the Pittsburgh district.

Bids Received On Monessen Bridge

BIDDERS	BRIDGE COMPL.	EXTRA EARTH EXCAVA. CU. YD.	EXTRA ROCK EXCAVA. CU. YD.	EXTRA CONCR. PER CU.YD.	COMPL. USING LOCAL STEEL.	TIME
L.J.Mensch	$53,350	$1.50	$2.50	$ 7.50	$68,350	180 days
H,C.Brooks Company	$57,000	$1.50	$3.00	$ 8.00		150 days
Bowman Bros.	$59,500	$1.00	$1.50	$12.00	$59,500	150 days
Nicola Building Co.	$52,329	$ .50	$1.00	$ 5.00	$52,329	120 days
C.B.Clark Co.	$51,000	$1.25	$3.00	$ 7.50	$51,000	150 days
Cullen-Friestedt Company.	$66,727	$ .85	$2.25	$7.40	$66,277	120 days

Comparison of Costs.

LOCATION	LENGTH	WIDTH	HEIGHT ABOVE GROUND	HEIGHT ABOVE FOUND.	SPAN OF ARCH	CUB. YDS. CONCR.	COST PER L.Ft.	COST PER Sq.Ft.	COST PER Cu.Yd
Meadow	454'	50'	78'	66'	209'	4050	$158.00	$3.15	$17.65
Larimer	670'	50'	110'	114'	300'	8525	$208.50	$4.17	$16.50
Atherton Ave. over Pgh.J.Rd.	418'	60'	95'	71'	170'	7960	$217.00	$3.62	$11.50
Atherton Ave. over Penna Rd.	380'	60'	25'	73'	90'	7400	$246.00	$4.13	$12.50
Monessen Bridge	516'	32'	94'	93'	150'	3000	$ 98.00	$3.06	$16.85

The construction of the bridge offered no serious difficulties other than those incidental to the crowded location and long haul from the railroad. Material at the bridge was handled by a stationary cableway paralleling the axis of the roadway. The contractor's shops, office, storage piles and concrete mixers were ranged along Third street. The cable-way picked up the lumber, steel, concrete, etc., from the ground and carried it to the required point on the bridge. A panel was left out of the bridge floor to enable the cable-way to be used to the last.

The arch centering was built of 8" by 8" hardwood timbers in bents on ten foot centers. It was well braced longitudinally and transversely by means of 2" by 6" planks. No bolts were used, all timbers being secured together by cleats, firmly spoked. This is the only large centering of which the writer knows, in constructing which bolts were not used. For greater security against wind pressures, the centering was guyed out with wire ropes. Wedges were used for lowering the centering after the arch was poured.

The writer, as assistant engineer for Chester and Fleming, had charge of the design of the bridge and supervised construction. Mr. A. K. Hubbard acted as resident engineer. The Nicola Building Company of Pittsburgh, were the general contractors.

APPENDIX A.

Principal Computations For Arch

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To Accompany Thesis of M.F.Stein

On Reed Avenue Bridge

Arch Computations.

I. Distance of Intersection Locus above Springing Line:

$= 6/5 \times 53.75 = 64.6I$

2. Values of e_I and e_2:

Panel point	e_I	e_2
I	- I08.62	20.II
2	- 36.2I	I8.I0
3	- I2.07	I5.52
4	0.00	I2.07
5	7.24	7.24
6	I2.07	0.00
7	I5.52	I2.07
8	I8.I0	36.2I
9	20.II	I08.62

3. Dead Panel Loads: (Panel Point I,)

Roadway Slab- Paving Brick = 50#
Sand & Cinders, 58#
Slab = I00#
208# x 6.75 x I5 = 2I000 #

Middle Beam - 28 x I6 : I44 x I50 x I5 =	7000 #
Side Beam = 3 x I50 x I5 =	6750 #
Side Walk = 6 x $3\frac{1}{2}$/I2 x I5 x I50 =	3940 #
Side Walk Stringer = 2/3 x $I7\frac{1}{2}$/I2 x I5 x I50 =	2I80 #
Curb = $\frac{3}{4}$ x I.67 x I50 x I5 =	2820 #
Cantilever = $\frac{I2 + 30}{2 \times I2}$ x 5 x I50 =	I3I0 #
Fillets = 2.33 x $\overline{I.5}^2$ x I50 =	785 #
Girder = I.5 x 5.5 x II.25 x I50 =	I3900 #
Hand Rail = I5 x 20 =	300 #
	59985 #
Column = 40 x 625 =	25000
Arch Rib (by planimeter) =	I348I5
Cross Beam = 7.25 x I0 x I.5 x I50 =	I6300
	236I00 #

3.Equivalent Uniform Live Load:

Computed for 50 ton interurban cars (trucks-28'apart) placed to produce maximum bending moment at the quarter-point.

$$p = 32/3 \times \frac{4,667,500}{(150)^2} = 2200 \text{ pounds per track and rib}$$

5'0" sidewalk loaded @ 80 # = 400 pounds per rib

Total = 2600 pounds per rib per lin. foot

4.Horizontal Thrust from Temperature:

To determine horizontal thrusts from temperature and rib shortening use section of rib at point where center line crosses the temperature and rib shortening thrust line.

$I_c = 1,492,000$

$I_s = 524,000$

$I_t = 2,016,000$

$$H_t = \frac{45 \times 2000000 \times .0000055 \times 2016000}{4 \times (645)^2} = 604\# \text{ for } 1^\circ \text{ Fahr.}$$

H_t for $+20^\circ = 12,080\#$

H_t for $-40^\circ = 24,160\#$

5.Horizontal Thrust from Rib Shortening:

$$H_s \text{ for } 1\# = \frac{45 \times 2016000}{4 \times (645)^2} = 55\#$$

H_s for 165# = 9000 #

6.Computations for Panel Points Showing Tension:

Panel Point 5,

$S = 8,627,000 \div (60 \times 50^2) = 57.5$

F_c = 350#; F_s = 7800#

p = .009; s = 60.9

Panel Points 2 and8,

$S = 8,504,000 \div (60'' \times 71.2^2) = 28$

F_c = 225; F_s = 5,800

p = .007; s = 36

Panel Points 4 and 6,

$S = 7,920,000 \div (60'' \times 51.6^2) = 49.5$

F_c = 300;F_s = 6500#

p = .0095; s = 53.1

Panel Points 1 and 9,

No tension.

Panel Points 3 and 7,

$S = 13,288,000 \div (60'' \times 57^2) = 68$

F_c= 375; F_s = 8500#

p = .0085; s = 64

Panel Points 0 and10,

$S = 30,976,000 \div (60'' \times 91.5^2) = 61.5$

F_c = 350; F_s = 7300#

p = .00 ; s = 63

APPENDIX B.

Detailed Computations for Floor System

Reed Avenue Bridge.

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To Accompany Thesis of M.F.Stein

On Reed Avenue Bridge.

File 453
Reed Avenue Bridge
Final Computations
M.F.S.

1. Roadway Slab, Arch and Approaches

Span = 5'-6" + 2 x 3"(bearing) = 6'-0"

Loading due to 1 - 55 Ton car truck,

Weight of truck	55000 lb.
Impact allowance, 25%,	13750
	68750 lb.

Depth of fill = 1.5' over slab, wheel base = 6'-0"   Ties 8' long

Area covered by load:

Width = 8.0 + 1.5 = 9.5'
Length = 6.0 + 2 + 1.5 = 9.5
Area = 9.5 x 9.5 = 90.25 sq.ft.

Uniform pressure due to live load $\frac{68750}{90.25}$	=	762 #/ sq.ft.
4" paving brick, rail, etc. @ 150#	=	50
14" sand and cinder fill @ 50#	=	58
		870 #/ sq.ft.

Moment $= \frac{1}{10} \times 870 \times 6^2 \times 12 = 37600$ in #

$d = \sqrt{\frac{37600}{72 \times 12}} = 6.6$ in.   used 6.5"

Reinforcing $\frac{5}{1000} \times 6.6 \times 12 = .395$ sq.in.   used $2\frac{1}{2}$" □ rods or 0.5 sq.in.

Shear at Support $= \frac{3 \times 870}{6.5 \times 12} = 33.5$# / sq.in.

2. Sidewalk Slab, Arch and Approaches

Span = 4'-3" + 2 x 3" = 4'-9"
Net load = 100# / sq.ft.
Requires a 3" slab, 1/4" rods 6" c.toc.
Used a $3\frac{1}{2}$" " " " 4" "   p = .57%
Conc.Load of 1000# on span 4'-3", wt.of slab neglected

M = 1/4 x 1000 x 4.25 x 12 = 12750 # in.

$T = \frac{12750}{.87 \times 2.75} = 5300, \quad fs = \frac{5300}{3 \times .0625} = 28400\# / \text{sq.in.}$

$fc = \frac{16}{3} \times 28400 \times .0057 = 865\#$

This covers worst condition of automobile running on side-walk.

3. Floor System over Arch

a. Sidewalk Stringer:

Span = 15'0"

Loading: Live Load $= 2\frac{1}{4} \times 100 =$ 225 #

Slab $= 3 \times \frac{3\frac{1}{2}}{12} \times 150 =$ 131 #

Beam $= \frac{8 \times 175}{144} \times 150 =$ 146 #

502 #

$M = \frac{1}{8} \times 502 \times 15^2 \times 12 = 170\,000 \text{ in }\#$

Let b = 8"

$d \sqrt{\frac{170000}{72 \times 8}} = 17.2"$ used 19"

Reinforcing: $\frac{1}{200} \times 17.2 \times 8 = .69$ sq.in. used .75 sq.in.

$\frac{1}{8} \times 225 \times 15^2 \times 12 = 76500 \text{ in }\#$

Equiv. concentrated load $= \frac{4 \times 76500}{12 \times 15} = 1700\ \#$

This covers worst condition of automobile running on sidewalk.

$\text{Shear} = \frac{7.5 \times 502}{17 \times 8} = 24.8\ \# / \text{in}^2$

b. Cantilever supporting sidewalk stringer:

Span = 4'-1"

Loading, Reaction of sidewalk stringers,

15 x 502	=	7530 #
Weight of railing and post	=	470 #
Beam $1\frac{3}{4} \times 4.5 \times 150$	=	1180 #
		9180 #

$M = 9180 \times 49 = 450000$ in #

$d = \sqrt{\dfrac{450000}{72 \times 12}} = 23''$ used 28"

Shears: At support, $\dfrac{9180}{28 \times 12} = 27.4$ #/sq. in.

At end, $\dfrac{8000}{12 \times 10} = 66.6$ #/sq. in.

Steel area required $\dfrac{(66.6-35) \times 12 \times 10}{12000} = .32$ sq.in

" " used = 1.53 " "

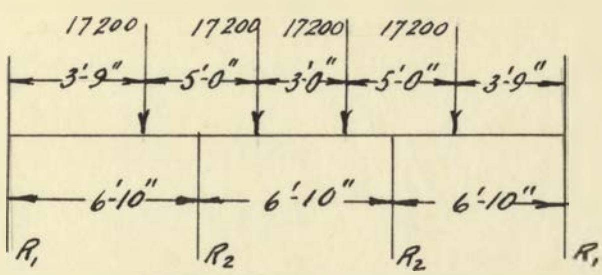
c. Side Beams:

Net span = $13'\text{-}2'' + 2 \times 5 = 14'\text{-}0''$

Loading:

Dead Load,	Inner sidewalk reaction, l.l. =	300 #
	" " " , slab =	131 #
	Beam and curb, 4 x 150 =	600 #
	Pavement and filling =	200 #
	Slab, 3 x 100# =	300 #
	Dead Load per linear foot =	1531 #

Live Load,

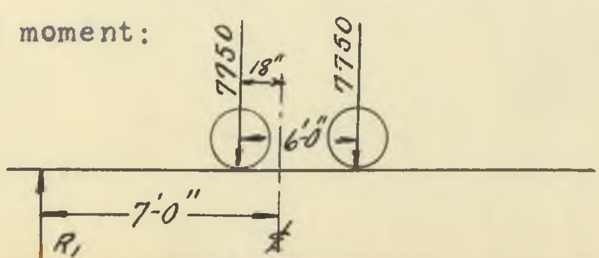


Load per wheel = $\dfrac{110\ 000}{8} \times 1.25 = 17200$ #

$\dfrac{3.08}{6.83} \times 17200 = 7750$ # $= R_1$

$2 \times 17200 - 7750 = 26650$ # $= R_2$

Live load moment:



File 453
Reed Avenue Bridge
Final Computations
M.F.S.

$R_1 = 7750 \times \frac{2.5 \quad 8.5}{14} = 6100\ \#$

$M_2 = 6100 \times 5.5 \times 12 =$ 402 000 in #

$M_d = \frac{1}{8} \times 1531 \times 14^2 \times 12 =$ 450 000 " "

852 000 in #

$b = 12"$

$d = \sqrt{\frac{852\,000}{72 \times 12}} = 31.5"$ used 34"

Reinforcing req. $\frac{1}{200} \times 12 \times 31.5 =$ 1.89 sq.in.

" used $4 \times .5625 =$ 2.25 " "

Shears: Formula for live load $R = P\,\frac{11-x}{7}$

Dist. from Support	Live	Dead	Total	Unit	Steel Req.
0	12200	10700	22900	56.2	$\frac{(56.2-35) \times 144}{12000} = .26$
1	11100	9200	20300	50.0	$\frac{(50-35) \times 144}{12000} = .18$
2	10000	7700	17700	43.5	$\frac{(43.5-35) \times 144}{12000} = .10$
3	8850	6150	15000	37.0	
4	7750	4600	12350	30.4	
5	6650	3070	9720	23.8	
6	5550	1531	7081	17.3	

Point of bending up 2 rods = .707 x 7 = 5' from Ø

Bearing required $= \frac{22900}{12 \times 250} = 7.65"$

d. Middle Beams:

Span = 14'-0" net.

Dead Load,-- Road Ballast & Paving Brick = 700 # lin.ft.

Road Slab = 700 " " "

Beam $\frac{16x28}{144}$ x 150 x 7 = 470 " " "

1870 " " "

D.L.Moment = $\frac{1}{8}$ x 1870 x 14^2 x 12 = 550 000 in #

L.L. " = $\frac{26650}{7750}$ x 402000 = 1380 000 " "

1930 000 " "

Design as T - beam - b = 16"

Web area required $\dfrac{1870x7+\frac{26650}{7750} x 12200}{100 x 16}$ = 34.4" deep.

Ratio, depth of flange to depth of web, $\frac{t}{d} = \frac{8}{34}$ = .235

R = 68

Width of flange = $\frac{1930000}{68 x 34^2}$ = 25"

Steel = $\frac{1}{200}$ x 25 x 34 = 4.25 sq. in. used 4.56 sq.in.

Shears:

Dist. from Support	Live	Dead	Total	Unit	Steel Req.
0	42000	13100	55100	101.0	1.06 sq.in.
1	38000	11200	49200	90.5	0.89 " "
2	34400	9350	43750	80.5	0.78 " "
3	30500	7500	38000	70.0	0.56 " "
4	26600	5600	32200	59.2	0.39 " "
5	22900	3700	26600	49.0	0.22 " "
6	19100	1870	20970	38.4	
7	11400	000	11400	21.0	

Bearing = $\frac{55100}{250x16}$ = 13.8"

Bearing on girder = $\frac{9.0}{4.8}$

$\frac{4.8}{13.8}$ x $\frac{55100}{100x16}$ = 12" depth of bracket

File 453
Reed Avenue Bridge
Final Computations
M.F.S.

4. Floor System over Approaches:

a. Sidewalk stringer:

Span $= 23.5'$

Loading: Live load $= 2\frac{1}{4} \times 100 =$ 225 #

Slab $= 3 \times \frac{32}{12} \times 150 =$ 131

Beam $= \frac{12 \times 23.5}{144} \times 150 =$ 294

650 # lin.ft.

$M = \frac{1}{8} \times 650 \times 23.5^2 \times 12 = 536000$ in #

Let b = 12"

$d = \sqrt{\frac{536000}{72 \times 12}} = 25"$ used 25"

Reinforcing $= \frac{1}{200} \times 12 \times 25 = 1.5$ sq.in. used 2.28 sq.in.

Shear at Support $= \frac{23.5}{2} \times 650\ 12 \times 25 = 25.5$ #/ in^2

Bearing $= \frac{23.5}{2} \times 650 \div 12 \times 250 =$ 2.55" used 6"

b. Cantilever supporting sidewalk stringer

Span = 3'-11"

Loading, Reaction of sidewalk stringers $= 24 \times 650 = 15600$ #

Weight of railing and post = 470 #

Beam $\frac{26}{12} \times 150 \times 4\frac{5}{12} =$ 1430 "

17500 #

$M = 17500 \times 47 = 824000$ in #

b = 12"

$d = \sqrt{\frac{824000}{12 \times 72}} = 31"$ used 34"

Reinforcing $\frac{1}{200} \times 12 \times 31 = 1.86$ sq.in. used 3.05 sq.in.

Shears: At support $\left(\frac{17500}{12 \times 34} - 35\right) \times \frac{144}{12000} =$ .096 sq.in.steel

used .56 " " "

At end $\left(\frac{16070}{12 \times 14} - 35\right) \times \frac{144}{12000} =$ .57 " " "

used 3.05 " " "

c. Side Beams:
Span, = 23'-0"

Loading: Inner sidewalk reaction, l.l. = 300 #
" " " slab = 131 #
Beam and curb, 7 x 150 = 1050 #
Pavement and Filling = 200 #
Slab, 3 x 100 # = 300 #
1981 #

Live Load moment (see p. 3)

$R_1 = 7750 \times \frac{13.7}{23} =$ 6750 #

$M = 6750 \times 10 \times 12 =$ 810 000 in #
$M_d = \frac{1}{8} \times 1981 \times 23^2 \times 12 =$ 1570 000 " "
2380 000 " "

Let b = 18"

$d = \sqrt{\frac{2,380,000}{72 \times 18}} =$ 43" used 46"

Steel req. $\frac{1}{200} \times 18 \times 43 = 3.9$ sq. in. used 4.6 sq. in.

Shears: Formula for live $R = \frac{P(40-2x)}{23}$

Dist. from Support	Live	Dead	Total	Unit	Steel Req.
0	13500	22800	36300	44.0	.16 sq. in.
1	12800	20800	33600	40.6	.10 " "
2	12100	18800	30900	37.3	.04 " "
3	11400	16800	28200	34.0	
4	10800	14900	23700	28.6	
5	10000	12850	22850	27.6	

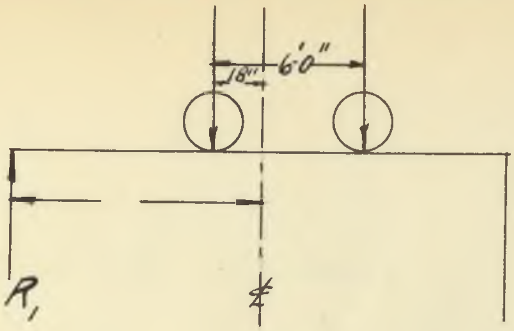
c. Side Beams, Approach Span, cont.
Point of bending up 2 rods $= \frac{1}{3} \times 11\frac{1}{2} = 6.7'$ from ℄

Bearing required $= \frac{36300}{18 \times 250} = 8.1"$ used 9"

d. Middle Beam, Approach Span, cont.
Span = 23'-0"
Dead Load, - Road Ballast & Paving Brick = 700# lin. ft.
Road Slab = 700# " "
Beam $\frac{16 \times 40}{144} \times 150 =$ 666# " "
2066# " "

D.L. Moment $= \frac{1}{8} \times 2066 \times 23^2 \times 12 =$ 1,640,000 in #
L.L. "

File 453.
Reed Avenue Bridge
Final Computations
M.F.S.



$R_1 = 26650 \times \frac{7.0\ 13.0}{23} = 23200$ #

$M_l = 23200 \times 10 \times 12 =$ $\underline{2,784,000}$ in #
4,424,000 in #

t = 8" let d = 46"

Required Web area = $\frac{2066 \times 12\ 1.75 \times 26650}{46 \times 100}$ = 15.5" wide

Ratio depth of flange to depth of web, $\frac{t}{d} = \frac{8}{46} = .174$

R = 57

Width of flange = $\frac{4,424,000}{58 \times 46^2}$ = 36.6"

Steel = $\frac{4}{1000} \times 36.6 \times 46 = 6.75$ sq. in. used 10-$\frac{7}{8}$" ▫ rds, = 7.65 ▫"

d. Middle Beam, Approach Span, cont.

Shears:

Dist. from Support	Live	Dead	Total	Unit	Steel Req.
0	46700	24792	71492	97.5	1.00 sq. in.
1	44500	22726	67226	91.5	.90 " "
2	42000	20660	62660	85	.80 " "
3	40000	18594	58594	80	.72 " "
4	38000	16528	54528	74	.62 " "
5	35600	14462	50062	68	.53 " "
6	33300	12396	45696	62	.43 " "
7	31000	10330	41330	56	.34 " "
8	29000	8264	37264	51	.25 " "
9	27000	6198	33198	45	.16 " "
10	24400	4132	28532	39	.06 " "
11	22200	2066	24266	33	.00 " "
12	20000	0000	20000	27	.00 " "

Bearing = $\frac{71492}{250 \times 16}$ = 17.8"

Bearing on girder = $\frac{12.0}{5.8}$ $\frac{5.8}{17.8} \times \frac{71492}{100 \times 16}$ = 14.6" depth of bracket.
" " Bracket =

5. Girder and Columns, Arch Span.

Design of Girder - Column Bent for Beam Action.

Girder - Net Span, 18' — 6"
Loading, Live on beam = 2 x 26650 = 53300 #
Wt of beam, slab, etc.
15 x 1870 = 28000 #
81300 #

L.L.Moment = 81,300 x 70 = 5,700,000 in #
Girder " = $\frac{1}{8}$ x 1230x18.5^{2}x12 = 635,000 " "
6,335,000 in #

$I_1 = \frac{18 \times 63^3}{12} = 375,000 \text{ in}^4$ $l = 20' = 240"$

$I = \frac{20 \times 28^3}{12} = 37,000 \text{ in}^4$ h = 37', say 40x$\frac{1}{2}$ = 20

$q = \frac{375000}{37000} = 10$

$$H = \frac{(\frac{20}{20})^2}{12(\frac{2}{3} \times 10 + \frac{20}{20})} \times (2(81900) + 20 \times 1230) = 2040 \#$$

-Hh = 2040 x 18 x 12 = 440 000 in #

Eccentric load moment of cantilever = 9180x48 = 440,000 in #

$M_{max.}$ = 7,292,000 - 440,000 - 440,000 = 6,412,000 in #

R for beam = $\frac{6412000}{18 \times 62.5^2}$ = 92.5, req. .68% reinf., fc = 595

$\frac{68}{10000}$ x 18 x 62.5 = 7.66 sq. in. = used 7.656 sq. in.

R. for column = $\frac{440,000}{20 \times 28^2}$ = 28., req. 0.2% rein. fc = 300

$\frac{2}{1000}$ x 20 x 28 = 1.12 sq. in. used 2 sq. in.

$-Hh = 1694 \times 25 \times 12 = 508,200$ in #

Eccentric load moment of cantilever $= 17500 \times 50 = 875,000$ in #

$M_{max} = 9,320,000 - 508,200 - 875,000 = 7,936,800$ in #

R for beam $= \dfrac{7936800}{24 \times 68^2} = 71.5$ req.5% reinf. $f_c = 500$

$\dfrac{1}{200} \times 24 \times 68 = 8.16$ sq.in. used 10.8 sq.in.(2 rods for wind)

R for column $= \dfrac{508,200}{30 \times 28^2}$ 21.6, req..13 % rein. $f_c = 200$#

$\dfrac{13}{10000} \times 28 \times 30 = 1.1$ sq.in. used 3 sq.in. partly for wind.

Direct Load on Column:

Middle beam, slab and max.l.l.=	103,000 #
Wt. of Girder 20 x 1800=	36,000 #
2 side beam reactions =	63,000
Cantilever and sidewalk =	17,500 #
Wt. of Column 50 x 937	47,000 #
Wind reaction (approximately) =	15,000 #
	281,500 #

$\dfrac{P}{A} = \dfrac{281,500}{900} = 313.$ #/sq.in.

Allowable stress for column action $= \dfrac{1}{1 \quad \dfrac{1}{20000} \times \left(\dfrac{50 \times 12}{\sqrt{\dfrac{55000}{28 \times 30}}}\right)^2} \times 1.14 \times 500 = 445$ allowable

Max. stress, inc. bending $= 313 + 200 = 513$ #

Direct Load on Column:

Middle beams slab amd max l.l. =	81,300 #
Wt. of girder 20x1230 =	24,600 #
2 - side beam reactions =	38,465 #
Cantilever and sidewalk =	9,180 #
Wt. of column, 40x625 =	25,000 #
Wind reaction (see p.10) =	12,650 #
	191,195 #

$$\frac{P}{A} = \frac{191195}{600} = 320\# \text{ / sq. in. max. comp.}$$

Allowable stress for column action $= \dfrac{1}{1 + \dfrac{1}{20000} \times \left(\dfrac{37\times12}{\sqrt{\dfrac{166000}{20\times28}}}\right)^2} \times 1.14 \times 500 = 440$ allowable

Max stress, inc. bending = 300+320 = 620# / sq. in.

6. Girder and Columns, Approach Span.

Design of Girder-Column Bent for Beam Action.

Girder - Net Span = 20'-0"	
Loading, Live on beam = 2 x 26650 =	53,300 #
Wt. of beam, slab, etc. 24 x 2066 =	49,700
	103,000 #

L.L. Moment = 103,000 x 80 =	8,240,000 in #
Girder " $= \frac{1}{8} \times 1800 \times 20^2 \times 12 =$	1,080,000 " "
	9,320,000 in #

$I_1 = \dfrac{24\times68^3}{12} = 630{,}000$ in^4 $l = 20' = 240"$

$h = 50\times\frac{1}{2} = 25' = 300"$

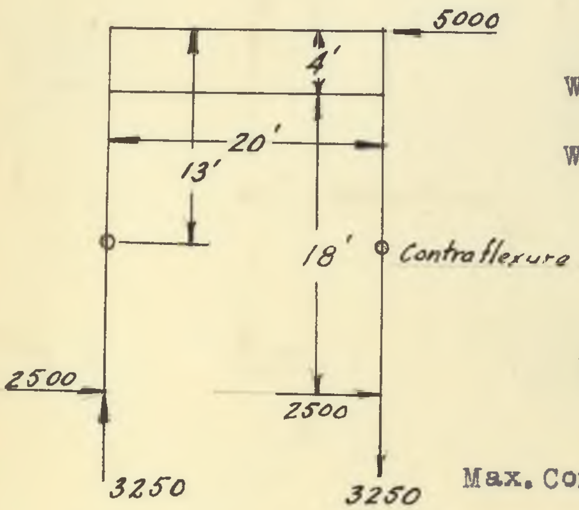
$I = \dfrac{30\times28^3}{12} = 55500$ in.^{4}

$q = 11.3$

$$H = \frac{\left(\frac{20}{25}\right)^2}{12\left(\frac{2}{3}\times11.3 + \frac{20}{25}\right)} \times (2(103000) + 20 \times 1800) = 1694\ \#$$

7. Wind Stress in Girder-Column Bents

a. Arch Bent #8 l=18'



Wind on side of stringer
5x15x25 = 1875 #
Wind on side of column
2x9x25 = 450 #
Wind on side of car
10x10x25 = 2500 #
4825 #

Use 5000 #

$$V = \frac{5000x13}{20} = 3250 \#$$

$$\text{Max. Compression} = \frac{5000x4+9x2500}{4} = 10600 \#$$

$$\text{Unit} \quad " \quad = \frac{10600}{18x24} = 25 \#/\text{sq. in.}$$

Max. Tension, upper flange: $= \frac{2500x9}{4} = 5600\#$ req. $\frac{5600}{10000} = .35$ sq. in.

Max Tension or Compression, lower flange:

$\frac{25000x13}{4} = 8140\#$ req. 0.5 sq. in.
Available .01 sq. in. @ 16000#

Max. moment in Column:
2500 x 9 x 12 = 270,000 in #

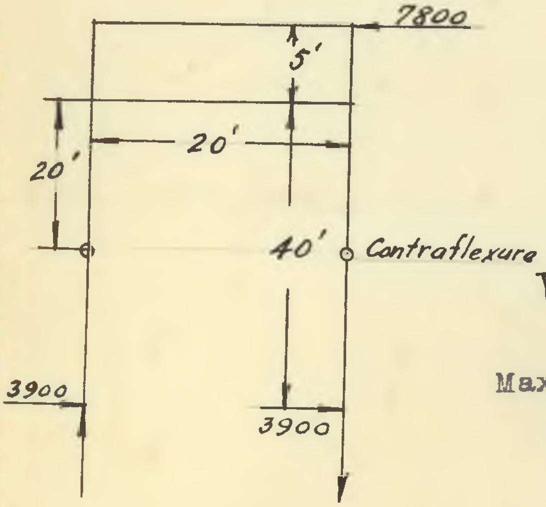
$R = \frac{270,000}{20x28^2}$ = 17, p = .1% fc = 150

$\frac{1}{1000}$ x 20 x 28 = .56 sq. in., available 2 - 1.12 = 88 sq. in.

Direct compression due to wind (for #1+9) on p.9
Shear in girder = 3250# allowed for shear reinforcing
" " column 2500# = no reinf. req.
Max compressive stress in outer fibre of column, = 300 for direct stress, 200± bending moment for load (red. for shorter column from p. 9) +150 = 650± under worst condition.

8. <u>Wind Stress in Girder O Columns Bents</u>

b. Approach Bent #2 from arch, 4th St. side, l = 40'



Wind on side of stringer
$6 \times 24 \times 25 =$ 3600 #
Wind on side of Column
$2\frac{1}{2} \times 20 \times 25 =$ 1250
Wind on side of car = 3000#
7850#

$$V = \frac{7800 \times 25}{20} = 9800\ \#$$

Max. Compression, upper flange,

$$\frac{7800 \times 5 + 3900 \times 20}{5} = 23400\ \#$$

$$\frac{23400}{24 \times 24} = 40\#/\text{sq. in.}$$

Max. Tension upper flange $= \frac{3900 \times 20}{5} =$ 15600# req. / sq. in.
used 4-$\frac{7}{8}$" sq. rods
partly for com. in conc.

Max. Tension or Compression in lower flange:

$\frac{3900 \times 25}{5} = 19500\#$ req. 1.22 sq. in.
available 2.6 sq. in.

Max. Moment in Column:
$3900 \times 20 \times 12 = 936{,}000$ in #

$R = \frac{936000}{30 \times 28^2} = 40$, $p = .25\%$ $f_c = 350\ \#$

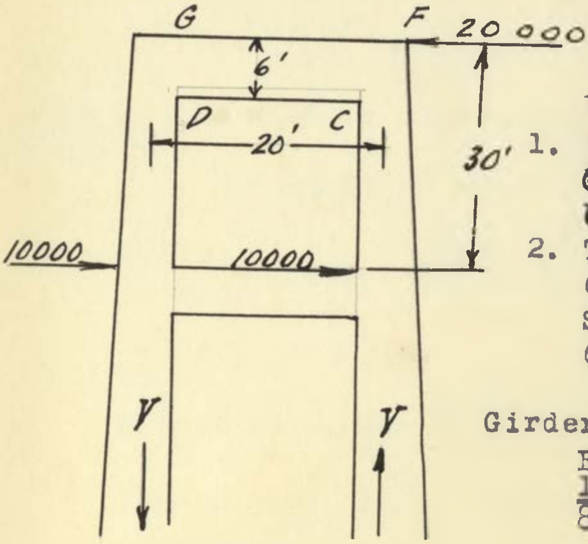
$\frac{1}{400} \times 28 \times 30 = 2.1$ sq. in; available 3@ 1.1 = 2.9 sq. in.

Shear in Girder = 9800# @ center, $\frac{9800}{12000 \times 28 \times 6} = 1\text{-}\frac{3}{8}$ double stirrup per ft.
used 2

Shear in Column = 3900 # no reinf. req.

File 453
Reed Avenue Bridge
Final Computations
M.F.S.

Lateral Stresses on Pier



W = 20,000 #

$V = \frac{3}{2} \times 20,000 = 30,000\#$

1. Compression at G
   GFx6-20,000x6 - 10,000 x 24 = 0
   G = 60,000#
2. Tension at F
   6F = 240,000, F = - 40,000 #
   Stress at C & D
   6D = 300,000 D or C = ±50,000#

Girder, Arch Side

Beam action, M = 5,746,000 in #

$\frac{1}{8}$x900x18x18x12 = 440,000 " "
6,186,000" "

$R = \frac{6186000}{24 \times 80^2} = 40$

fs = 16000, fc = 350#

Allowable fc = $1\frac{1}{4}$ x 500 = 625   625 - 350 = 275#

$\frac{60,000}{275 \times 24}$ = 9.1" proving assumed 6' O.K.

For R = 40, p = .27%

Steel req. for beam, $\frac{27}{10000}$ x 24 x 80 = 5.18

" " " tension $\frac{50000}{16000}$ = 3.12

8.30

used 12-$\frac{7}{8}$" □ rods in lower face.

Use $\frac{40,000}{61,000}$ = 2.5 sq.in. or 4-$\frac{7}{8}$" □ rods above

Shear = 85,000 + 30,000 = 115,000 #
Allowable 24x80x35 = 67,000
48,000 #

$\frac{48000}{12000} \times \frac{1}{7} \div 0.28 = 2$ - $\frac{3}{8}$" □ double stirrups per ft.

Girder, Approach Side

Beam action: $M = 8,565,000$ in #

$R = \frac{8565000}{24 \times 80} = 56$

$fs = 16000$, $fc = 420$, $p = .4\%$

Allowable $fc = 625$, $625 - 420 = 205\#$

$\frac{60000}{205 \times 24} = 12.2$ proving assumed 6' O.K.

$\frac{4}{1000} \times 24 \times 80 =$ 7.78 sq.in

Direct Tension $\frac{50000}{16000} =$ 3.12

10.90 sq.in. used 16-$\frac{7}{8}$"▫ rods.

Used 4 - $\frac{7}{8}$" rods, above.

Shear = 126,000 + 30,000 = 156,000

Allowable 67,000

89,000

$\frac{89000}{12000} \times \frac{1}{7} \div 0.28 = 4 - \frac{3}{8}$"▫ double stirrups per ft.

Pier Columns:

Normal load,

Dead Load, 12x2300 7½x1900 =	41,800
Max L.L. shear	13,300
Girder (as for 24' span)	122,400
Sidewalk reaction	20,000
Longitudinal reaction	40,000
	237,500

Area column section at top, 11.5 sq.ft.

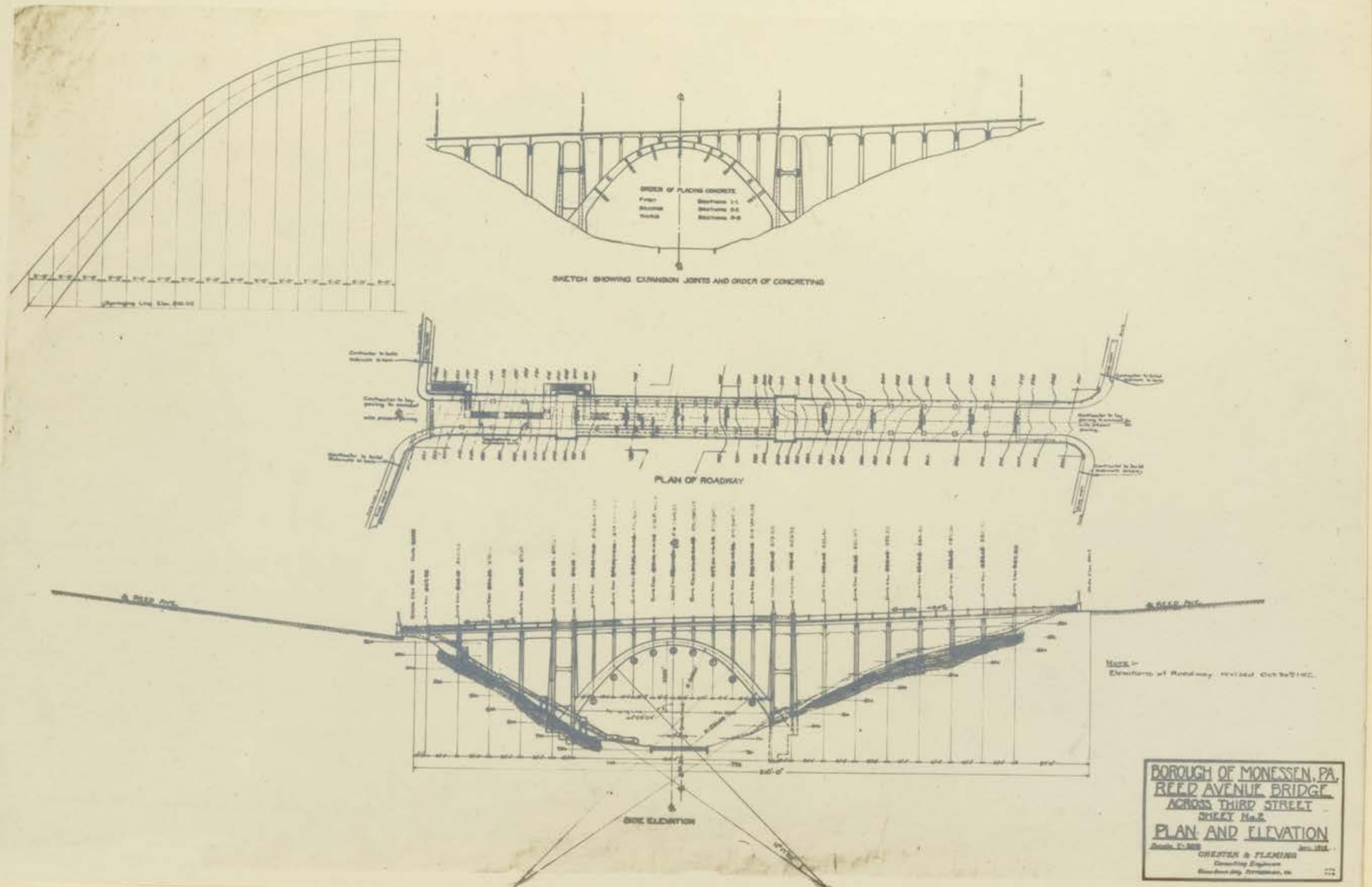
$\frac{2375000}{11.5 \times 144} = 144\#/\text{sq. in.}$

Wt. of column 160,000

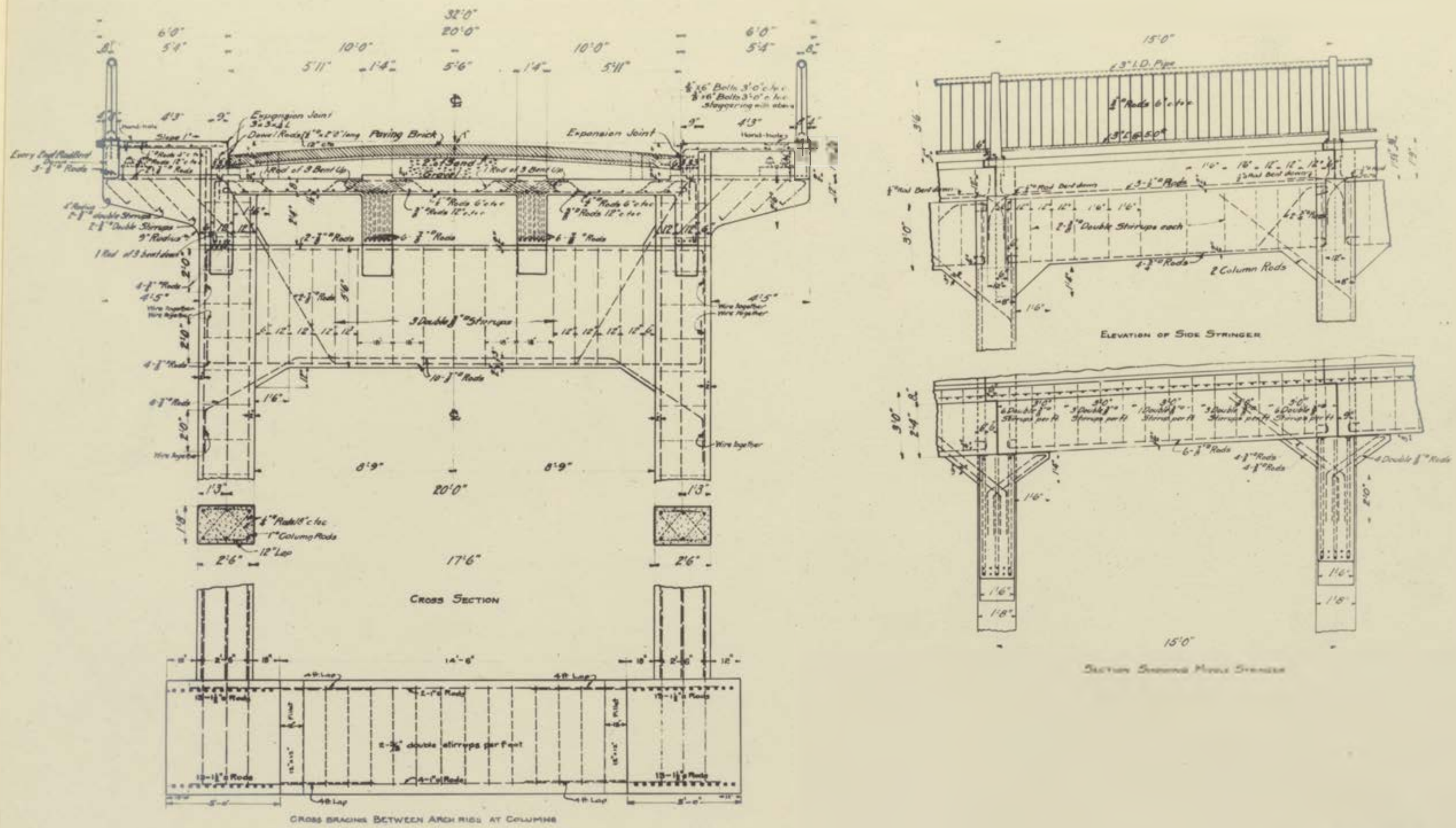
397,500

Area at bottom 28 sq.ft.

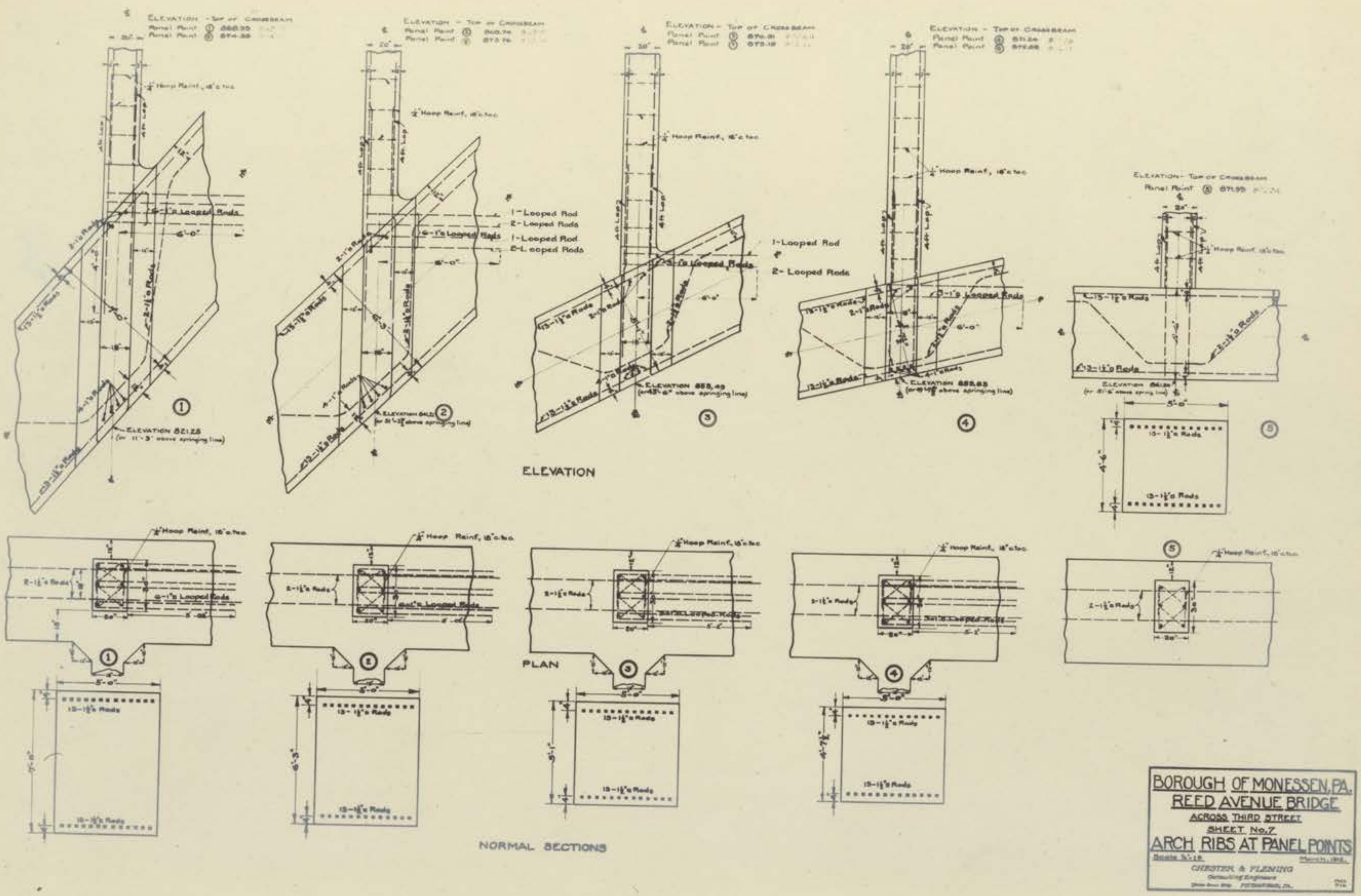
$\frac{397500}{28 \times 144} = 99\#/\text{sq. in.}$

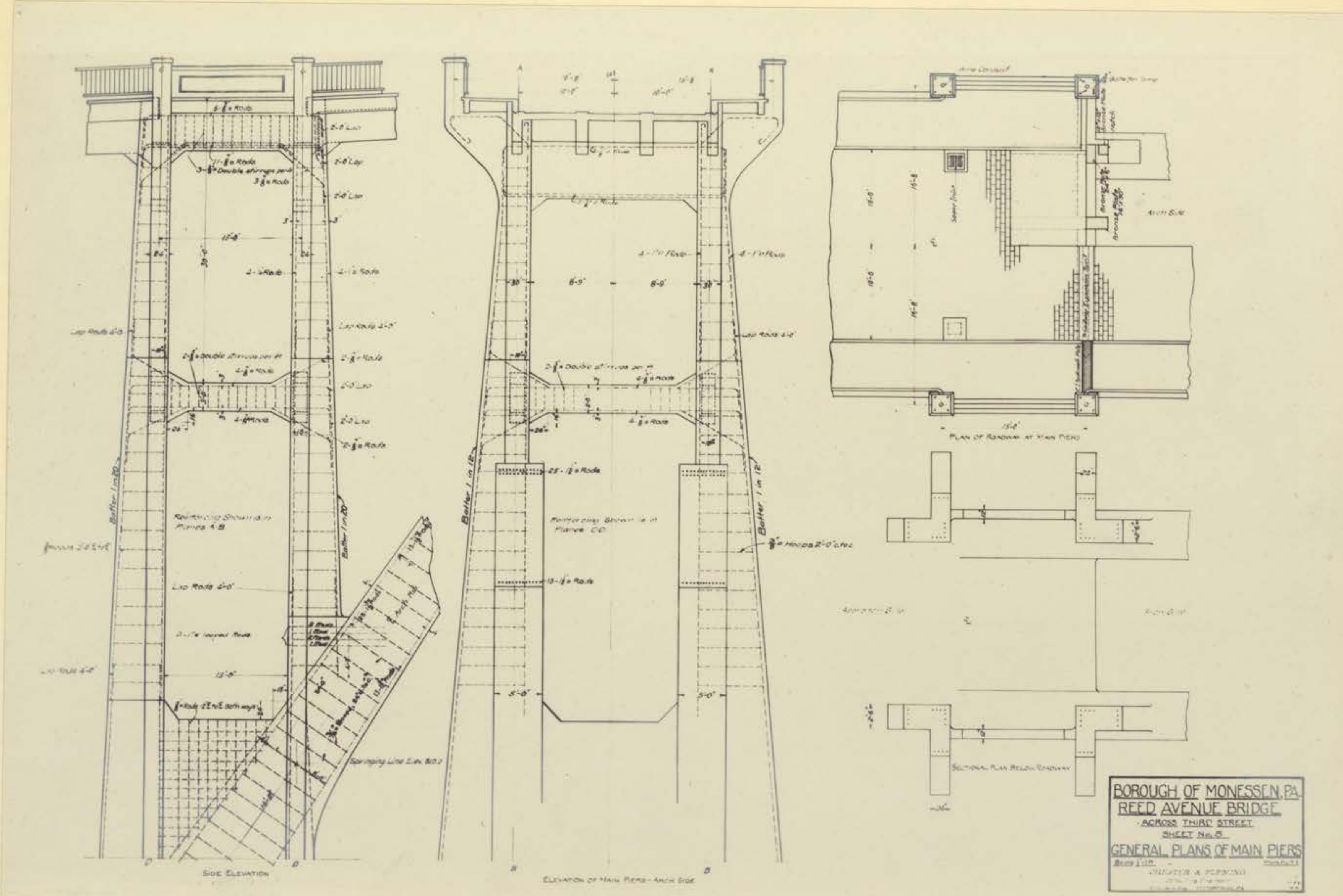


SKETCH SHOWING EXPANSION JOINTS AND ORDER OF CONCRETING
ORDER OF PLACING CONCRETE
PLAN OF ROADWAY
SIDE ELEVATION
BOROUGH OF MONESSEN, PA.
REED AVENUE BRIDGE
ACROSS THIRD STREET
SHEET No. 2
PLAN AND ELEVATION
CHESTER & FLEMING
Consulting Engineers

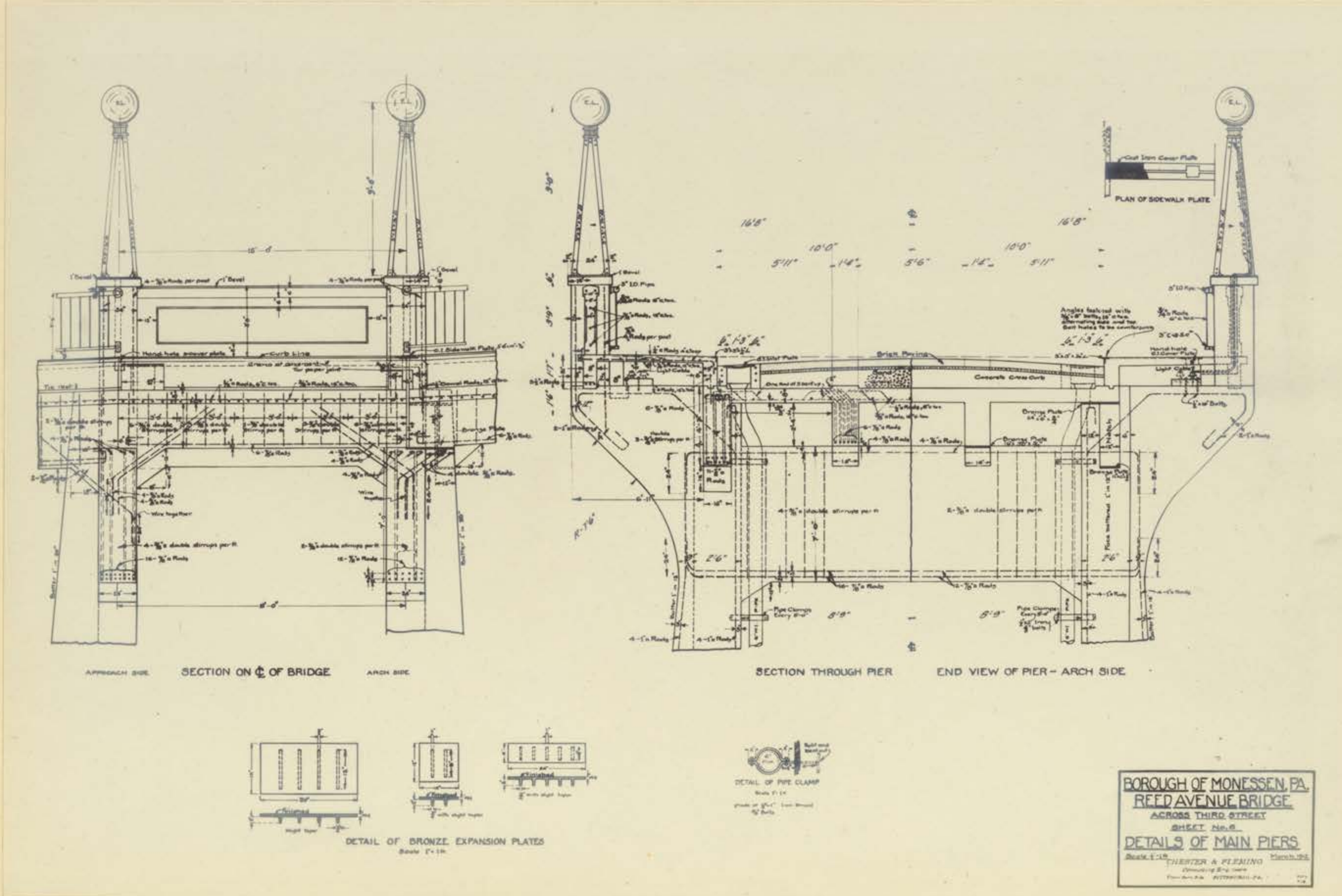


BOROUGH OF MONESSEN, PA.
REED AVENUE BRIDGE
ACROSS THIRD STREET
SHEET No. 4
FLOOR SYSTEM OVER ARCH
CHESTER & FLEMING


ELEVATION
PLAN
NORMAL SECTIONS
1-Looped Rod
2-Looped Rods
1-Looped Rod
2-Looped Rods
BOROUGH OF MONESSEN, PA.
REED AVENUE BRIDGE
ACROSS THIRD STREET
SHEET No. 7
ARCH RIBS AT PANEL POINTS
CHESTER & FLEMING



Side Elevation
Elevation of Main Piers - Arch Side
Plan of Roadway at Main Piers
Sectional Plan Below Roadway
Batter 1 in 20
Batter 1 in 12
BOROUGH OF MONESSEN, PA.
REED AVENUE BRIDGE
ACROSS THIRD STREET
GENERAL PLANS OF MAIN PIERS


SECTION ON ℄ OF BRIDGE
APPROACH SIDE
ARCH SIDE
SECTION THROUGH PIER
END VIEW OF PIER – ARCH SIDE
PLAN OF SIDEWALK PLATE
DETAIL OF BRONZE EXPANSION PLATES
DETAIL OF PIPE CLAMP
BOROUGH OF MONESSEN, PA.
REED AVENUE BRIDGE
ACROSS THIRD STREET
SHEET No. 6
DETAILS OF MAIN PIERS
CHESTER & FLEMING