# THE DESIGN AND CONSTRUCTION OF THE LOGAN STREET VIADUCT, LANSING, MICHIGAN

THESIS FOR THE DEGREE OF C. E.

Collins Thornton

1933

THESIS

Viaducts

The Logan street Viaduct

SUPPLEMENTARY MATERIAL IN BACK OF BOOK

Civil engineering Bridges & roofs

THE DESIGN AND CONSTRUCTION

of the

LOGAN STREET VIADUCT, LANSING, MICHIGAN

Submitted to

Michigan State College

as a

Thesis for the Degree of Civil Engineer

by

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Spring 1933

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MAPS OR DRAWINGS

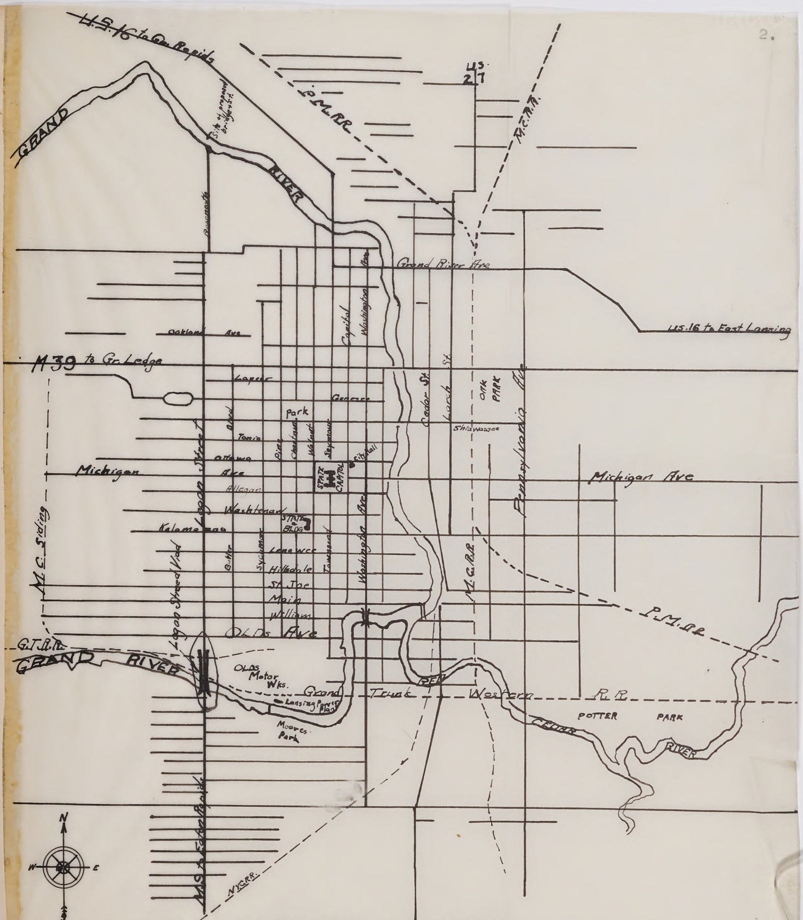
# The Design and Construction of the Logan Street Viaduct

The contract for the erection of the Logan Street Viaduct was awarded October 8,1928, and the viaduct was accepted by the City of Lansing, June 30,1930, as a completed structure. The cost to the city was about $400,000.00 . The Folwell Engineering Co. of Chicago, Ill. was the general contractor. Mr. E. Sebern was general superintendent and in active charge of the work for the Folwell Co.

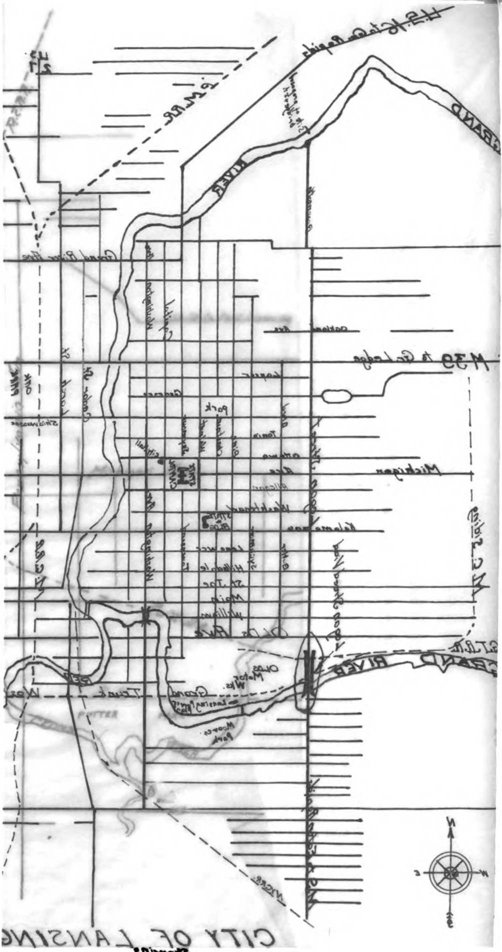
The viaduct was designed by the bridge department of the City of Lansing, Michigan. Mr. R.F.Rey being Bridge Engineer and Collins Thornton, his assistant. Mr. E.G.Eddy was City Engineer and Mr. Laird J. Troyer the Mayor, at that time. The Bridge Committee of the City Council consisted of Mr. F.S.Vandervoort, chairman, Mr. Geo. Hagamier, and Mr. Wm. McComb.

The construction was supervised by the City of Lansing, which was represented by the Bridge Engineers. All work, materials, and workmanship, were performed in accordance with the COntract and Specifications of the Logan Street Viaduct.

In order to designate the location of the project in the mind of the reader, the following sketch is presented.




# CITY OF LANSING
Showing
Location of Logan Street Viaduct




showing
Location of Logan Street Viadu

By means of this sketch, the setting of the proposed structure can be visualized. The importance of the crossing, and the inadequacy of the existing structure to present day traffic were principal factors in the promotion of the improvement. As can be seen, Logan Street is a North and South street in the City of Lansing, Michigan, ten blocks West of the Capitol. It extends North to the Grand River, where a proposed bridge would connect it to U.S. 16. To the South of the city it becomes State Trunk No. 9 and is the paved road to Eaton Rapids and Albion. Three quarters of a mile South of the Capitol it crosses the Grand River, the Grand Trunk R.R. main lines and sidings, and the Michigan Central R.R. switches to the Oldsmobile plant. Previous to 1928, a steel Pratt truss, erected in 1898, served the public. The steel was in fair condition but was designed for lighter traffic than that to which it was being subjected. Moreover, the maintenance costs were increasing, the planking in the roadway not being able to withstand increasing traffic vibration. In 1926, a five ton load limit was imposed upon it.

Bordering the river on the North, lie the main tracks of the Grand Trunk R.R. Just to the N.E. of this is the huge plant of the Oldsmobile Co., which is served by four Grand Trunk sidings and three Michigan Central switches. This aggregation of railroad switches and main lines, which are in almost constant use when the Oldsmobile Co. is running at peak production, caused a serious traffic impediment. Not only this but the situation is dangerous, the bottle neck bridge and steep grade to the North having caused several accidents, a number of which were fatal. The last one in 1926, when four persons were instantly killed by a fast Grand Trunk flier, so aroused public sentiment that the elimination of the grade crossing was demanded. Nevertheless, the problem of convincing the city council of the necessity was not so easy, but the following excerpt from the council preceedings are self explanatory.

Excerpt- City Council Proceedings
October 17, 1927
Public Improvement, Resolution No. 1
by Ald. F.S.Vandervoort

"It is hereby determined to be a public necessity and a necessary public improvment to separate the grades and build a bridge over the Grand River and the Grand Trunk R.R. on Logan Street, City of Lansing, and to be paid for by the city at large, be it"

"Resolved,- That the City Engineer be and hereby is ~~is~~ directed to estimate kind and quantity of materials to be used therefor, and estimate in detail the probable expense of such work, and of all materials and labor to be used therein, and to cause to be prepared the necessary plans and specifications for such work, and to report to the City Council as soon as possible, the expense to be paid from the contingent fund."

This resolution authorized the City Engineer to proceed with a preliminary survey for the purpose of determining the type of structure and probable expense to the city, and incidently, the economic need of a new

Among the first steps, was the location survey and mapping the details of the area concerned. By inspection of the profile, it is easy to discern that a viaduct spanning the railroads and river is the most appropriate. The greatest difficulty in this plan was that of providing outlets for the Olds Motor Works and the Lansing Municipal Power Plant. Consultation with the Lansing Board of Water and Light showed a plan involving the construction of another bridge, the Island Avenue Bridge, to be the most feasible. This, because it made the plant more accesible to the downtown area and at a very nominal increase in cost, also, it was desirable that they have an outlet during the construction of the viaduct. The cost arrangement was a fifty-fifty proposition, and was erected at a cost of $75,000.00, of this, $25,000.00 of the city at large's share was to be paid from the funds for the viaduct, it being considered incidental and necessary to its construction. $12,500.00 was paid from the contingent fund, and the Board of Water and Light furnished the remainder. This bridge was built during the summer of 1928 and the roadway was ready for use Dec.1,1928. The outlet for the Olds Motor Works was provided for by the construction of a new drive, paralleling the new viaduct and on city property, connecting the old outlet with Olds Avenue on the North.

The desirability of a new structure is evidenced by the promptness in which the officials of the Olds cooperated enabling a tentative agreement to be signed by I.J.Reutter, President of the Oldsmobile Co., Oct.31,1927, providing for the new outlet and waiving damages.

Before consulting either of the railroads, it was decided upon to go ahead with the preliminaries and have a clear cut, workable plan or plans when the time came to consult with them, it being realized, that while the railroads have capable engineering organizations, it is usually necessary for the public authorities to take the initiative. With this in mind, several plans were prepared and considered.

Late in 1927, the railroad officials were consulted and after several conferences and much correspondence between the parties, tentative agreements were signed with the Grand Trunk R.R. on Nov. 16,1927, and as their share, agreeing to pay $80,000.00, and the Michigan Central Jan. 16,1928, in which they agreed to pay the sum of $20,000.00 and to lower their tracks to elevation 144.00 feet, an average of five feet at their own expense.

This form of agreement, commonly called the "Lump Sum Agreement", was based upon the preliminary estimate of cost. These agreements are representative of a fifty-fifty division of costs as per the Public Acts of 1893, Act 92, as amended, which is a part of the Compiled Railroad Laws.

Property damage was limited to those lots fronting on Logan between Olds Ave. and Albert St., and was settled by the agreement to build new sidewalks at the property lines abbutting and at the new elevations at the cities expense.

In order to make a fairly accurate estimate of the probable cost, it was necessary to decide on the details of the proposed structure. For instance, although the clearances over the railroads is standardized at 22 feet over

main lines and from 18 to 21 feet over switches and sidings, it was necessary that the general city plan of grade separations be thoroughly considered. In 1923, the City of Lansing employed as engineer, Harland Bartholmew of St. Louis Mo. to make a city plan for the City of Lansing and in it he recommends an underpass at Washington Ave. separating it from the Grand Trunk R.R., this being not quite a mile from the proposed structure at Logan St. In his survey, however, he did not establish definite controlling grades but established a general plan and probable system of grade separation. After due consideration, it was found that even a considerable elevation of the tracks would not materially affect the grade at Logan Street.

The general profile was determined by using a 22 foot clearance over the Grand Trunk main lines and it was found that a five percent grade would meet Moores River Drive nicely. The Michigan Central insisted on a 21 ft. clearance over its switch tracks, and in order to limit the maximum grade to 5.5%, it would be necessary to raise the intersection at Albert St. This would be a place for the excess dirt from the excavations and was adopted.

The entire portion of the viaduct north of the river was planned and estimated as concrete beam and girder, resting upon concrete columns as being the most fitting and economical type of structure. However, it must be understood that these plans were only tentative.

Because of the position and size, the river crossing cannot be of the same type of construction. The old steel bridge was 203 ft. long, but the banks were curved to the

old abutments and the proper span was about 240-250 feet, and as the water elevation was maintained at elevation 127.00 feet by the dam belonging to the Lansing Municipal Power Plant, it was planned to widen the river to the width required by this elevation.This elimated the collection of trash on the upstream side that had made the old bridge such an eyesore.

In considering a bridge of this size, it is necessary that the architectural lines be pleasing to the eye and a source of pride to the community. Economically, it may have been cheaper to bridge the river with steel, encased in concrete, using two or perhaps three spans, but the depth of the beams necessary to span these distances as compared to the depths of the beams over the railroad were so different as to give a very displeasing appearance. Also, the clearance above the water on the south side would have been very small which was undesirable. Test borings or earth soundings had been made and the soil conditions found suitable for arch foundations. In view of the above, three arches of progressing span lengths were adopted as most suitable and fitting in this preliminary design.

The general cross section was a roadway which could carry four lanes of traffic, 38 ft. between the curbs. A six foot sidewalk on the West and a two and one-half foot sidewalk on the East, with 1ft. 3in. on either side for coping and handrail, giving an overall width of 49 ft. The walk on the East was necessarily narrowed to provide for an 18 ft. drive for the Oldsmobile Outlet on city property. By actual traffic and pedestrian count, it was found that the foot traffic was comparatively light, and although the designers would have liked equal width walks, they felt justified under the conditions in making one narrower than the other. The handrails were to be of concrete spindles set on a concrete plinth and surmounted by a concrete coping. A 4 in. crown and brick wearing surface were used in the design of the roadway. The depth of the beams throughout the viaduct were maintained constant, varying the reinforcing steel to fit the span lengths.

From this preliminary design, quantities could be estimated, and an approximate idea of the cost obtained, but as a check, the costs of the previous bridges and viaducts built by the city recently were tabulated. Reducing these to costs per square foot of roadway gives a quite accurate method of comparison and estimate. For instance, the Kalamazoo St. Bridge, completed in 1926, was of a similar type and cost nearly $200,000.00, and the Saginaw St. Bridge, completed early in 1928, cost about$91,000.00. When reduced to costs per square foot, we find a figure of approximately $9.50 per square foot, and checking with the other method and making due allowance for the deep water piers and abutments, it is found that it will require approximately $400,000.00 to build the structure as designed. Of this amount, as before stated, $100,000.00 would be paid by the two railroads, leaving $300,000.00 to be raised by the city. On Jan. 23,1928, the city clerk of Lansing was instructed by vote of the council to prepare the ballot for the spring election. The form of the ballot was questioning whether or not the city should bond itself for the sum of $300,000.00

for a period of thirty years, paying a maximum of 4 1/2% interest.

In order to stir up public interest in the project, a series of articles were published in the local newspapers, describing conditions as they existed. An architectural sketch of the proposed structure was prepared and shown in the papers as the type of bridge contemplated. Mayor Troyer threw his influense behind the project, describing it as a necessary improvement and a public necessity. Both newspaper editors commended the project and all the civic leaders and clubs were loud in their acclaim. Consequently the bond issue carried by a vote of three to one.

On april 30, 1928, the City Engineer was instructed by the City Council to prepare specific and detailed plans and specifications for the erection of a viaduct at Logan Street.

Design of the Logan Street Viaduct

By the order of the City Council of Lansing April 30, 192[illegible] the bridge engineers were definitely ordered to go ahead with the detailed design of the structure. In presenting this design, it will be necessary to condense the material to a considerable degree. For instance, where only span lengths vary but the rest of the section remains the same, a typical beam will be designed illustrating method and proceedure. The results obtained will be tabulated so that a comprehensible view of the designed and viaduct is obtained. Only the design of the viaduct as constructed willbe detailed in this analysis; however, its relative merits in comparison with various other designs will be discussed.

As was mentioned before, certain controlling factors are encountered in the design of any structure. Tabulation of these is of first importance, and the best possible method is by a drawing, so the drawing showing the proposed layout and profile was prepared. Upon examination of these drawings, we find the following details:

1. Using 0+00 as the South property line of Moores River Drive and Logan Street, it is 224.1 ft. to the old bridge abutment and between the abutments is 200 feet. There is a pier midwaywhich is built of concrete set upon a timber cribbing. It is inadequate for the foundation of a new, wider, and heavier structure. The removal of this pier and both abutments shall form a part of the contract. In order to widen the river to boundaries more nearly in conformity with the established banks, this span was increased to 240 feet, with the face of the South Abutment at station 2+03 and the North at station 4+43.

2. The next controlling feature was the clearance over the Grand Trunk main lines. These are located at station 5+58.3 south rail of the south tracks, and station 5+78.5 the north rail of the north tracks. The elevation of the center of the road at station 0+00 is 138.03 feet, station 0+66, the north side of Moores River Drive, is elevation 137.54 feet. Inoorder to maintain the existing grade on Moores River Drive, station 0+66 is the beginning of the rise and consequently the P.C. (point of curvature) of a vertical curve. The elevation of the south rail is 134.55 ft. Allowing the 22foot clearance over main lines and estimating the beam to be of probable span of 40 ft. and a depth of 24", with a slab thickness of 8" or 10", we find that the elevation of the deck on the center line of roadway approximately elevation 159.30 ft. From elevation 137.54 at station 0+66 to elevation 159.30 ft. at station 5+58.3 is a rise of 21.76 ft. in 492.3 feet. Assuming a 100foot vertical curve, the P.I. (point of intersection) would be at station 1+16 leaving a distance of 442.3 feet. This gives a grade of 4.9%, very close to the 5% grade adopted.

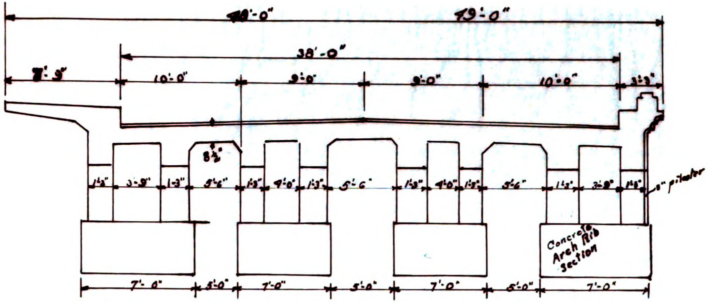
3. Using the center line grade at station 5+58.3 as elevation 159.30 ft., the next controlling point is that of providing the necessary clearance over the Michigan Central switches. The north rail of the south tracks at station 9-00 is at elevation 147.59 ft. According to the agreement between the Michigan Central R.R. and the City of Lansing, the railroad would lower their tracks to elevation 144.00 ft. Adding the required clearance of 21 feet and the 2.75 ft. for the depth of structure, we get an elevation of 167.75ft. which is a rise of 6.45 ft. in 342 feet or approximately

a 2% grade. This was later revised because of necessary vertical curves to a 2.5% grade.

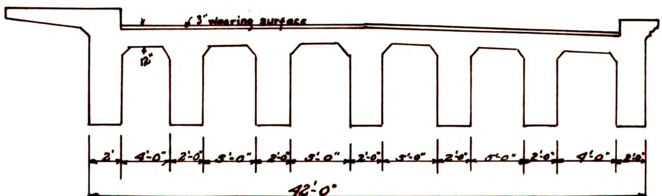
4. The center line intersection of Albert Street with Logan Street on the north end of the viaduct was originally at elevation 154.9 at station 11+30, but by raising the grade of the intersection it was possible to use a grade of 5.5%.

5. By certain revisions and the insertion of vertical curves at the breaks in grades, the controlling grade was selected. Using these grades it was necessary that the design be made to fit and correspond with precision. In several cases the design was altered to fit as will be shown in the railroad spans, whereby steel beams were used in the place of reinforced concrete so as to provide the clearance required.

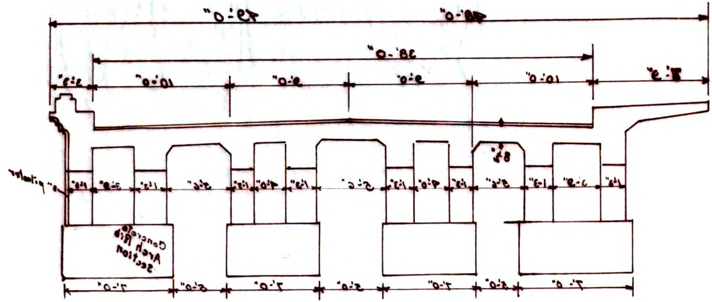
It is necessary in the design of any structure that the limiting factors be recognized, plans made to conform with them, and then designed to fit. This will eliminate many conflicting details that have to be ironed out later in the design or in the construction work.



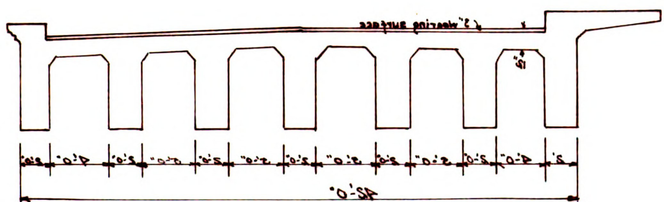

Arch Section




Beam and Girder Section




Arch Section




Beam and Girder Section

The Arch Design

The general form of the superstructure is shown upon the cross section drawing. As constructed, the arch section consists of a 38 ft. roadway, a 7.75' walk, and a 3.25' walk, supported by beams resting on columns, which are in turn supported by four arch ribs. The spandrel columns are spaced in conformity to the arch spans, so that an equal number of columns are used in each arch span.

The loading system used in the design is that used by the Michigan State Highway Bridge Department, namely that of a system of 20 ton trucks and 15 ton trailers, so placed as to give the maximum stress in the member concerned in any given section. Influence lines were used when there was any question of arrangement, as for example, the arches. Impact was provided for by a twenty five percent allowance. On the sidewalks, a liveload of 100 lbs. per square foot plus one front wheel load was used. The wearing surface was designed as 3" brick with 1/2" sand cushion. Both curbs were designed as 10" and a 4" roadway crown was provided.

In designing the pavement for the arch section, the span lengths, as shown in the cross section, are of two different dimensions, 3' and 5'. The 5' span will govern and is here given.

Span 5'-0"

Assume, d as 8"  D.L. 150# per sq. foot

L.L. 7 ton wheel load, or 14,000 x 5/4= 17,500#

Moment distribution ( O'Rourkes Empirical Formula )

E  $2/3(l - t) = 2/3(5 - 1.25) = 4.166$

$\frac{17{,}500}{4.166} = 4{,}200$ #/ft. of width

Shear Distribution (O'Rourke)

$E = 4/3\,(x + t) = \frac{4}{3}\,(1.45 + 1.25) = 3.60$

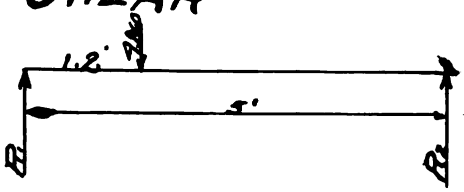
$\frac{17{,}500}{3.60} = 4{,}860$ lbs.

$M_{Dead\,Load} = \frac{wl^2}{12} = \frac{150 \times 5^2}{12} = 312.5$ ft lbs.

$M_{(truck)} = \frac{Pl}{4} = \frac{4200 \times 5}{4} = \underline{5{,}250}$ ft lbs

$M_{Total} =$ 5,563 ft. lbs

SHEAR




$R_{1\,L.L.} = 4{,}860 \times \frac{3.80}{5} = 3560$#

$R_{1\,D.L.} = 150 \times 2.50 = \underline{375}$#

$V = 3{,}935$#

$v = \frac{V}{bjd} = \frac{3935}{1.2 \times \frac{7}{8} \cdot 7} = 53.5$#

$\Sigma_o = \frac{V}{jdu} = \frac{3{,}935}{\frac{7}{8} \cdot 7 \cdot 12} = 5.35''$

Use ½"# @ 4" c.c. Lower and 8" in Upper. Bend up alternate bars over beams for Negative Moment.

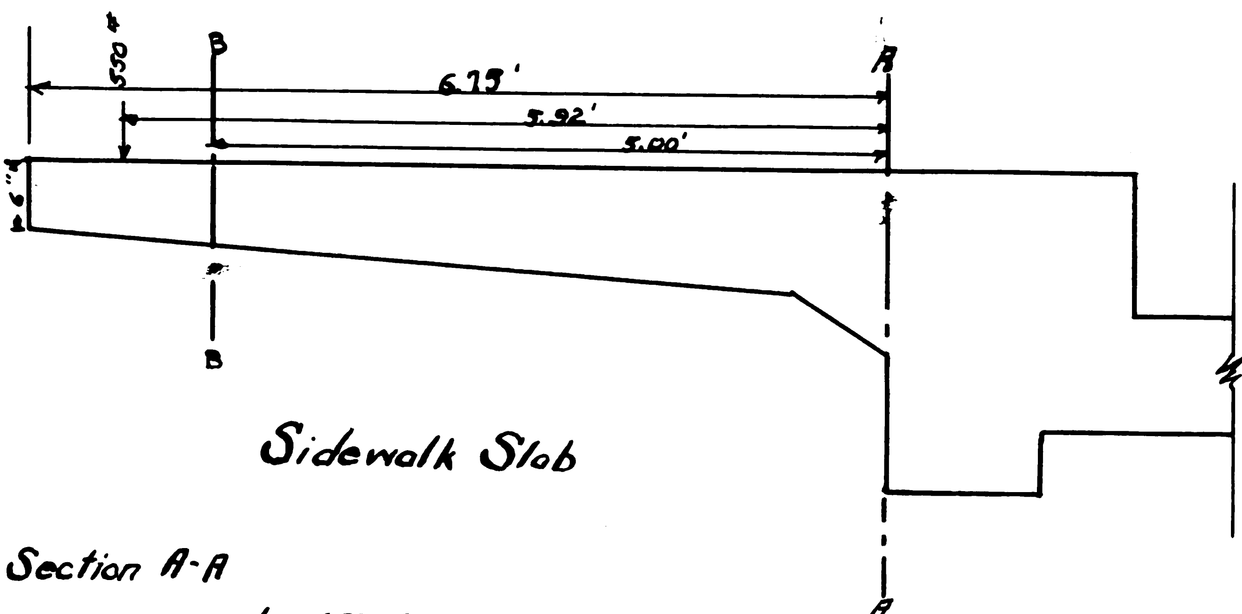
$A_s = .75\,\square''$ $\Sigma_o = 6.00''$ $W = .85$

Longitudinal bars for temperature reinforcement ½"φ not more than 2'- center to center, using one at every bend in the cross bars.

Total D of slab = 8" space main lateral steel 1" from surface.

The slab thus designed is in conformity with the method recommended by the Joint Committee on Concrete and Reinforced Concrete in the Aug. 1924, report. The empirical distribution of loads for the slab design is given by O'Rourke in his book on reinforced concrete design and conforms to accepted practice.

In connection with the deck, the sidewalk must be designed so that the reinforcing steel will match. It is of cantilever style, partially supported by the outside beam. The railing weighs 550# per lineal foot, is 1' wide, and is centered 9" from the outside edge of the walk. Assuming the section drawn below, the design is worked out,




Sidewalk Slab

Section A-A

$$M_{slab} = \frac{\frac{1}{2}\cdot 6.75\cdot 6.75\cdot 150}{2} = 1710'\# \quad \frac{wl^2}{2}$$

$$\text{and} = \frac{\frac{1}{4}\cdot 6.75\cdot 6.75\cdot 150}{3} = 570'\# \quad \frac{wl^2}{3}$$

$$2{,}280'\#$$

$$M_{rail} = 550\cdot 5.92 = 3{,}250'\#$$

$$M_{uniform\,L.L.} = \frac{wl^2}{2} = \frac{200\times 5^2}{2} = 2{,}500'\#$$

$$M_{Total} = 8{,}030'\#$$

4 Ton Truck Wheel
Spread over 4'
$M_r = 2000\times 4 = 8{,}000'\#$

Sidewalk Slab Design (cont.)

$M_{total} = 8{,}030'^{\#}$

from table

$d_s = 9\frac{1}{4}''$ ($A_s = .85^{\square''}$ $M = 9{,}200'^{\#}$)

$\frac{8{,}030}{9{,}200} \times .85 = .74^{\square''}$ Use $3 \cdot \frac{1}{2}''\phi$ spaced at 4"

Shear

Slab $= .75 \times 6.75 \times 150 = 760^{\#}$
rail $= 550^{\#}$
L.L. $= 5 \times 200 = 1{,}000^{\#}$
$2{,}310^{\#}$

$p = \frac{A_s}{bd} = \frac{.75}{12 \times 9.25} = .00675$
$f_c = \frac{2f_s p}{k} = 2 \cdot 18000 \times .00675 =$
$f_c = 675^{\#}$ OK.

$U = \frac{2310}{9.25 \times \frac{7}{8} \times 12} = 238^{\#}$ Must Use stirrups and hook bars for Web reinforcement (table shows $\frac{3}{8}''\phi$ at 6" ok.)

$\Sigma_o = \frac{2{,}310}{Ujd} = \frac{2310}{120 \times \frac{7}{8} \times 9.25} = 2.38''$ ($3 \times 2 = 6''$ Used)

Section B-B

$M_{slab} = 1.75 \cdot 6 \cdot 150 \cdot .9 = 142'^{\#}$
$M_{rail} = 550 \cdot .92 = 506'^{\#}$
$M_{total} = 648'^{\#}$

Shear

slab $= 157^{\#}$
rail $= 550^{\#}$
$707^{\#}$

$U = \frac{707}{9 \times \frac{7}{8} \times 12} = 16.9^{\#}$

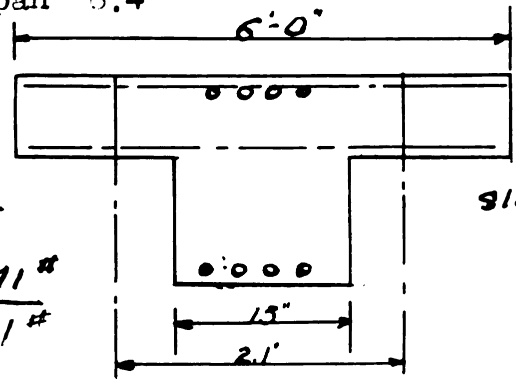
$\Sigma_o = \frac{707}{4 \times 120 \times \frac{7}{8}} = 1.69''$ Using 3 bars we have 6"

<u>Therefore</u> $3 \cdot \frac{1}{2}''\phi$ bars will be used at 4" centers

The slab and sidewalk being designed, the design of the Tee-Beams in the arch section is next. As has been stated, the spans vary in accordance with the arches, those over the south arch having a span of 4.5', 7.0' over the central arch, and 8.4' over the north arch. In the cross section, two beams were designed, an interior, and an exterior or sidewalk beam. As the typical beam design, an interior beam of the 8.4' span is selected and presented.

Tee-Beam Continuous

Span 8.4'




Slab = 8" Concrete
3" Wearing surface
½" Sand Cushion

$b = 15''$

$b_f = 2.1'$

Assume $d_c = 18''$

D.L. slab $= 6 \times 150 = 900^{\#}$

beam $= .75 \times 1.25 \times 150 = 141^{\#}$

$1041^{\#}$

L.L. + Impact $= 17{,}500^{\#}$

$M_{d.l.} = \dfrac{wl^2}{10} = \dfrac{1041 \times 8.4^2}{10} = 7{,}350'^{\#}$

$M_{L.L.} = \dfrac{Pl}{4} = \dfrac{17{,}500 \times 8.4}{4} = 36{,}700'^{\#}$

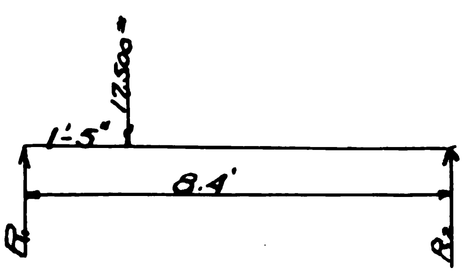
$M_{Total} = 44{,}050$ ft. lbs.

from table

$d_s = 15\frac{1}{2}''$ (Moment = 25,800 ft. lbs. Area Steel = 1.43 sq. in.

Area Steel $= \dfrac{44{,}050}{25{,}800} \times 1.43 = 2.44$ square in.

Shear




$R_1$ dead load $= 4.2 \times 1041 = 4{,}370$ lbs

Live Load $= 17{,}500 \times \dfrac{6.9}{8.4} = 14{,}400$ lbs

$V = 18{,}770$ lbs.

Continued

$V = 18,770$ lbs.

Unit Shear $v = \frac{V}{bjd} = \frac{18,770}{15 \times 7/8 \times 15.5} = 92.2$# ($.03 f_c = 120$ lbs. 120 lbs. allowable)

$\Sigma_o = \frac{V}{dju} = \frac{18,770}{15\frac{1}{2} \times \frac{7}{8} \times 120} = 11.5"$

Use 4 - 1 in round = 3.14 square inches

$\Sigma_o$ = 12.56" (summation of perimeters for bond stress)

Note: this is considerably more steel than is required by the total Moment stress.

## Web Reinforcement

$a_s$ = .461 sq. in. (6" spacing of 3/8" bars assumed at support.

$s = \frac{3}{2} \left( \frac{3,926 \times 16,000 \times .875 \times 15.5}{18,770} \right) = 6.8$ inches

Beyond point of inflection

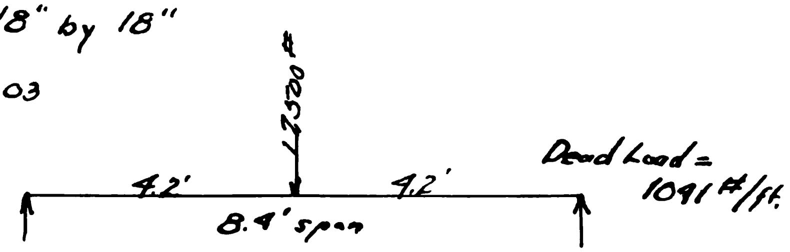
$s = \frac{3}{2} \times \frac{3,926 \times 16,000 \times .875 \times 15\frac{1}{2}}{14,400} = 8.9$ inches

The next step is the design of the spandrel columns over the arches. Since the beam having a span of 8.4' was presented, the column for this beam is designed.

Assume a column 18" by 18"

percentage of steel $p = .03$

Load




Moment of Live Load $\quad 17,500 \times 4.2 \times 12 = 881,000$ inch pounds

Moment of Dead Load $\quad 1041 \times 8.4 \quad = \quad 8,750$ ft. pounds

Axially Loading $\quad 8,750 + 17,500 \quad = 26,250$ pounds

$t = 18'' \qquad e = \dfrac{881,000}{26,250} = 33.7$ inches

$\dfrac{e}{t} = \dfrac{33.7}{18} = 1.87$

From the Column Formula in accordance with the Joint Committees Standard Building Code – $\dfrac{e}{t} = \dfrac{1 + 12np_o\left(\frac{a}{t}\right)^2}{6(1 + mp_o)}$

$$\frac{e}{t} = \frac{1 + 15 \cdot 12 \cdot .03 \times \left(\frac{7}{18}\right)^2}{6(1 + 15 \times .03)} = .21$$

from table No. 55 (tables by A.R. Lord)

$K = .41$

$L = .19$

Stress in Concrete

$$f_c = \frac{Ne}{bt^2L} = \frac{881,000}{18\,(18)^2\,.19} = 795 \text{ pounds}$$ (Use 3000 pounds Concrete)

Tensile stress in steel

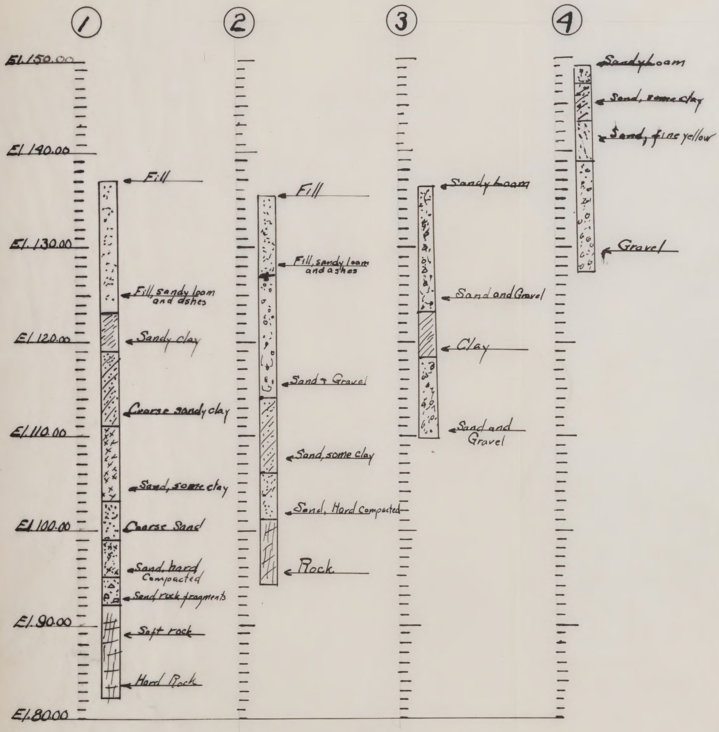
Area steel $= .03 \times \overline{18}^2 = 9.7$ sq. in. Use 12 - 1" round bars symmetrically placed.

$$f_s = mf_c\left(\frac{d}{kt} - 1\right) = 15 \times 785\left(\frac{14}{.41 \times 18} - 1\right)$$

$= 10,700$ # per sq. in stress in steel

Place web reinforcing as per code – $\frac{3}{8}$" bars around longitudinal steel at 18" centers

# Logan Street Earth Soundings



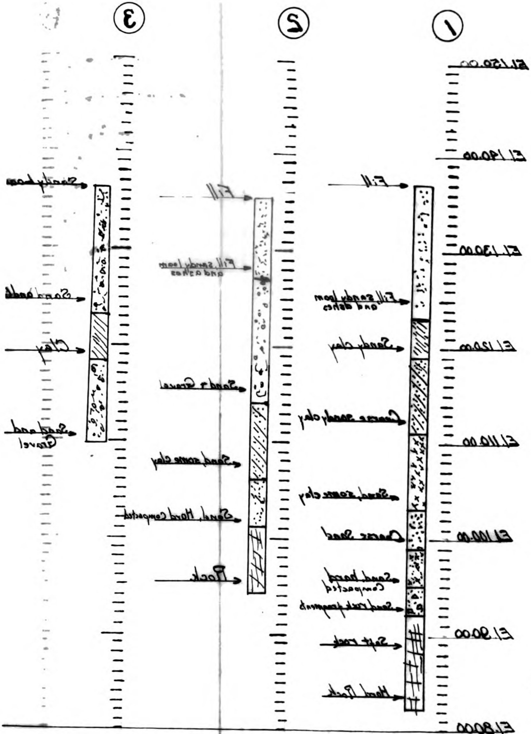

No. 1 was located at the South Abt. site.

No. 2 was located at the North Abt. site

No. 3 was located at the S.W. Corner of the Olds. property

No. 4 was located at the North End of the Viaduct.

# Logan Street Earth Soundings




No 1 was located at the South Abt. site.

No. 2 was located at the North Abt. site

No. 3 was located at the S.W. Corner of the Oblong

No 4 was located at the North End of the Area

With the superstructure designed, the next step is the arch design. In this structure, four arch ribs were planned. With the face of the South Abutment at station 2+03 and the face of the North Abutment at station 4+43 and governed by the 5% grade for the roadway, it is plain that a single arch of 240 foot span was impossible, and that two symmetrical arches were also unsuitable. Three equal arches could have been used but the appearance would not be as satisfactory as the system used, that of three arches of progressing spans and rises. This conforms to the grade closely and gives a composite structure.

As earth soundings had been made at the abutment sites, it was known at about what depth the foundations would be placed in order to rest the structure upon rock. The elevation as designed called for the South Abutment foundation at elevation 90.00 feet, the South Pier at elevation 93.00 feet, the North Pier at elevation 96.00 feet, and the North Abutment at elevation 100.00 feet. Going to this depth meant the penetration of several feet of soft rock and two feet into hard rock.

The arch piers were designed of the elastic type with a quite heavy cross section, in order that the unbalanced thrusts might be transmitted to the foundation more nearly vertically.

As designed, the arches consist of three clear spans, 61 feet, 76 feet, and 90 feet. The South Pier is 6 feet and the North Pier is 7 feet wide, making a total of 240 feet. The elevation of the springing line is the same as the normal water level, or elevation 127.00 feet. In each span, four arch ribs, 7.25 feet wide and 5 feet apart are used.

Each arch is a circular segment having the following properties,

Intrados Radius

61' span, Rise 11.3'

$R^2 = (1/2 \text{ span})^2 + (R - \text{rise})^2$

R 46.818'

76' span, Rise 14.79'

R 56.21'

90' span, Rise 16.03'

R 62.25'

The radius of the extrados was obtained by passing a circle through the three points which are the assumed thickness at the crown and at each springing line. In order to estimate the thickness with any degree of accuracy for the trial analysis, the most useful aid is familiarity with the dimensions of similar existing structures. In this bridge the assumptions were fairly close, and with the exceptions of the springing lines no changes were made. In each arch, however, it was found that the assumed size at the springing was too small to carry the stress placed upon it.

The final designed thicknesses at the crown and springing and the radius of each extrados, are,

| Span | Crown t | Springing t | Radius of Extrados |
|---|---|---|---|
| 61' | 1'-3" | 2'-4 1/2" | 53.99' |
| 76' | 1'-5 1/2" | 2'-11" | 64.51' |
| 90' | 1'-8" | 3'-6" | 71.18' |

In the open spandrel type of structure, the loads are transmitted to the arches through the columns which definitely fixes their point of application. In order to obtain the loading that will produce the maximum stress in the arch or piers and abutments, influence lines were drawn. Using this data and the influence lines for the various loadings the complete analysis for each arch and combination of arches was made. The attempt to present the complete analysis will not be made as this was the subject of a graduating thesis in 1928 by the author and is covered in full there. However, since the thesis was presented, the analysis was checked by the Cissell Engineering Company of Ann Arbor, Michigan, and approved with a few minor changes which will be noted and explained. The steps will be shown and results obtained through the analysis in as brief a manner as possible. The method of design used is that presented by Hool in his book "Reinforced Concrete Construction" Volume 3. Since the construction of the viaduct a book by McCollough of the Oregon State Bridge Department gives a method of analysis which is believed to be more in accordance with present day design and construction and should be used in the place of the method used here.

This brief of the analysis is prepared for the 90 foot arch.

The arch, from springing line to crown, is separated into eleven divisions having constant s/I, and the following data computed,

L of 1/2 axis 51.3'

$P_o$ .00733

| | |
|---|---|
| $a_s$ | .0122 square feet |
| $na_s$ | .183 square feet |
| s/I | 3.7 feet |

In this arch the loads are at the columns, there being ten columns per arch and as the arches are symmetrical, unit loads are applied at five points for 1/2 the arch. By placing this unit load at the column points in order the thrusts and moments at the crown are determined and are,

| Unit load at | $L_1$ | $L_2$ | $L_3$ | $L_4$ | $L_5$ |
|---|---|---|---|---|---|
| $H_c$ | .0708 | .306 | .643 | 1.105 | 1.29 |
| $V_c$ | .0113 | .0497 | .121 | .277 | .363 |
| $M_c$ | .114 | -.48 | -.725 | -.422 | 1.39 |

The moments and thrusts at each of the eleven sections are computed with the unit load at the five points and the results tabulated as above.

After all points have been loaded for each section the data for the influence lines is prepared. To do this the thickness at each section is computed or scaled from a large drawing. The data for the crown section is here given, with the load at $L_5$, $L_4$, $L_3$, $L_2$, and $L_1$. t 1.66' $p_o$ .0073

| Pt. | $\frac{x_o}{t} = \frac{M}{Nt}$ | K | NK | K' | NK' |
|---|---|---|---|---|---|
| crown | .645 | 4.10 | 5.29 | 2.30 | -2.97 |
| at $L_4$ | -.257 | 2.25 | -2.25 | .3 | .3 |
| at $L_3$ | -.668 | 4.15 | -2.70 | 2.30 | 1.50 |
| at $L_2$ | -.933 | 5.50 | -1.70 | 3.60 | 1.11 |
| at $L_1$ | -.574 | 3.62 | - .272 | 1.90 | .131 |

From this data the influence line for the crown section is drawn.

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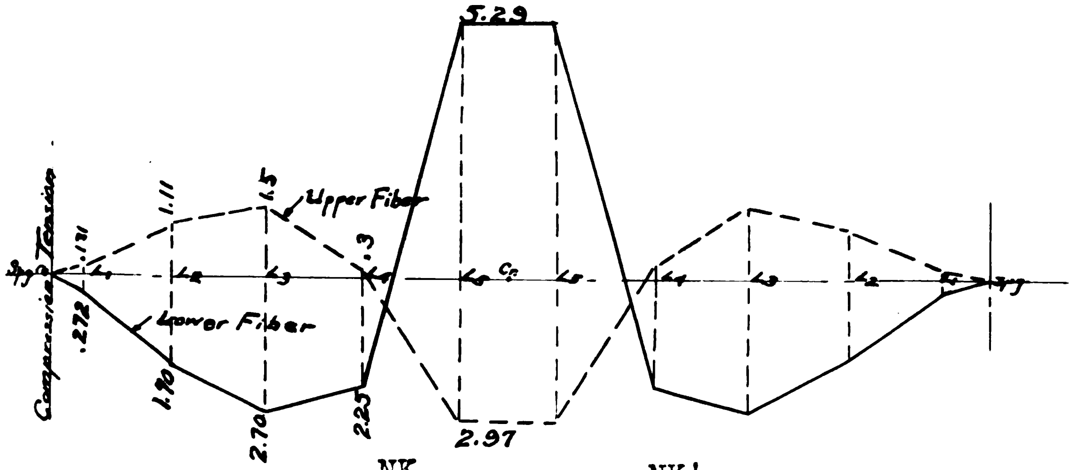
From this data the influence line for the crown section is drawn.




$f_c$ $\frac{NK}{bt}$ $f'_c$ $\frac{NK'}{bt}$

From this, the unit load placed at column no. 5 produces the maximum stress and so the dead load will remain on points 1, 2, 3, and 4, while the combined live and dead loads will be placed on column no. 5.

Dead Loads 10,690# at L1, 6,050# at $L_2$, 4,980# at $L_3$, 4,370# at $L_4$

Combined Live and Dead loads, 8,610# at $L_5$

Unit stress

Upper fiber

-2 x .131d.l. -2 x 1.11d.l. -2 x 1.5d.l. -2 x .3d.l. +2 x (-2.97 d.l.+l.l.) = $f_c$= 2(28,650)/1.66x144= 240 lbs. per sq.in. compression

Lower fiber

-2x.272 d.l.-2x1.70 d.l.-2x2.7 d.l.-2x2.25 d.l.+2x5.29 d.l.and l.l.= $f_c$= 2(11,068)/1.66x144 = 92 lbs. per sq. in. tension

Reversing the loads and loading the first four columns with the live plus dead loads and,

the stress in the upper fiber is equal to 65 pounds per square foot tension, and the stress in the lower fiber is equal to 447 pounds per square foot compression.

These influence lines are drawn for each section and the stresses due to the loading determined, and tabulated.

The stresses due to temperature and to rib shortening must next be determined for the eleven sections and tabulated. These are computed by formula given by Hool, and only the results of the work are given below,

Fall of temperature and rib shortening,

stress in crown section

upper fiber 137 pounds compression

lower fiber 167 pounds tension

Rise of temperature and rib shortening,

upper fiber 31 pounds tension

lower fiber 3u pounds compression.

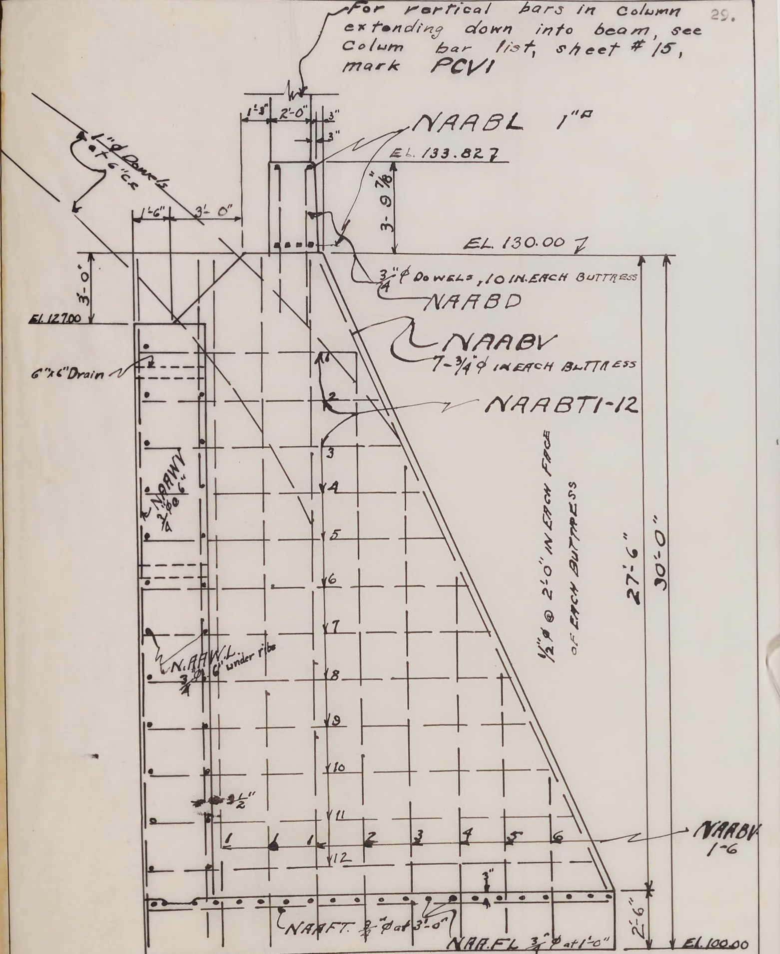
The maximum combination of these stresses is that obtained by using the second system of loading, combined with that due to a rise of temperature and rib shortening and is,

upper fiber 96 pounds tension

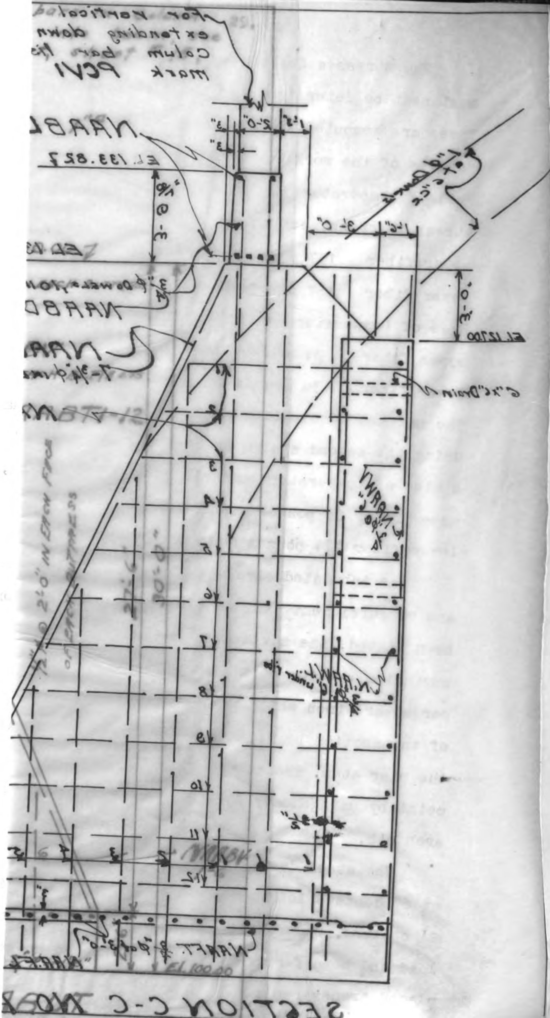
lower fiber 484 pounds compression

The tabulated stresses show whether the designed sections are of sufficient size to withstand the loads or not. As has been stated, the maximum stress in this design occured at the springing section with a compressive stress of 850 pounds per square inch which was resisted by the increase of the size of the section by using a curve of short radius in meeting the pier stem. The amount of steel was also increased at this point by using heavy dowels and extending them well into the arch rib.

The steel designed was provided by using 3/4" round bars at 6" centers longitudinally, with stirrups, 3/8" round, at 2' centers. Under each set of columns, to distribute the load and bond for shear, four 3/4" round hooked rods were placed transversely.




SECTION C-C NO. ABUT.


For verticals extending down Colum bars His mark PCV1
EL. 133.827
2'-0"
3'-0"
EL. 127.00
6"x6" Drain
EL. 100.00
SECTION C-C

After the arch analysis is completed, the resultant stresses are known and the foundation piers and abutments may be designed. Inasmuch as the piers are designed as elastic, thereby permitting the horizontal component of the resultant to the abutments, where they must be transmitted to the rock foundation, the vertical loading must be such as to produce the maximum resultant. In the North Abutment design, this was obtained by loading the north arch alone as determined from the influence lines for maximum stress at the springing as the thrusts produced by loading the other arches was counteracted by the dead load of the 90' arch. The amount of this stress at the springing being known from the analysis, the resultant was computed and together with the dead load of 1/2 the span which rested upon the abutment and by assuming several sections, the final abutment design was determined. Similarly with the South Abutment, with the exception that the three spans were loaded to produce maximum stresses at the springing, which left a residual thrust to be carried by the south abutment, the same method of design was followed. In the pier design, the larger arch was loaded and the resultant computed. Strictly speaking, it would have been permissable to use a slightly smaller pier stem with a greater percentage of steel and certain variations in the footing design so as to cut down the concrete stresses, but according to general practice in this type of design, the tendency is to be on the safe side and to endeavor to keep the resultant within the middle third of the footing. In order to do this the footing was offset in each pier from the centerline of the stem. These footings were also designed to rest upon solid rock.

A considerable portion of the viaduct design has been discussed in the preliminaries. A complete resume' of the design of the viaduct section will be given here.

In order to facilitate in the description, a cross section has been drawn of a typical section. See the tracing in pocket.

From this, it is seen that the slab span governing remains at 5' and that the slab as designed for the arch section holds as well for the viaduct section. In most places, the span lengths are determined by the location of railroad tracks and the angle of the foundations are also determined. From the general plan the station of the foundations, and their angle with the centerline of the road may be tabulated.

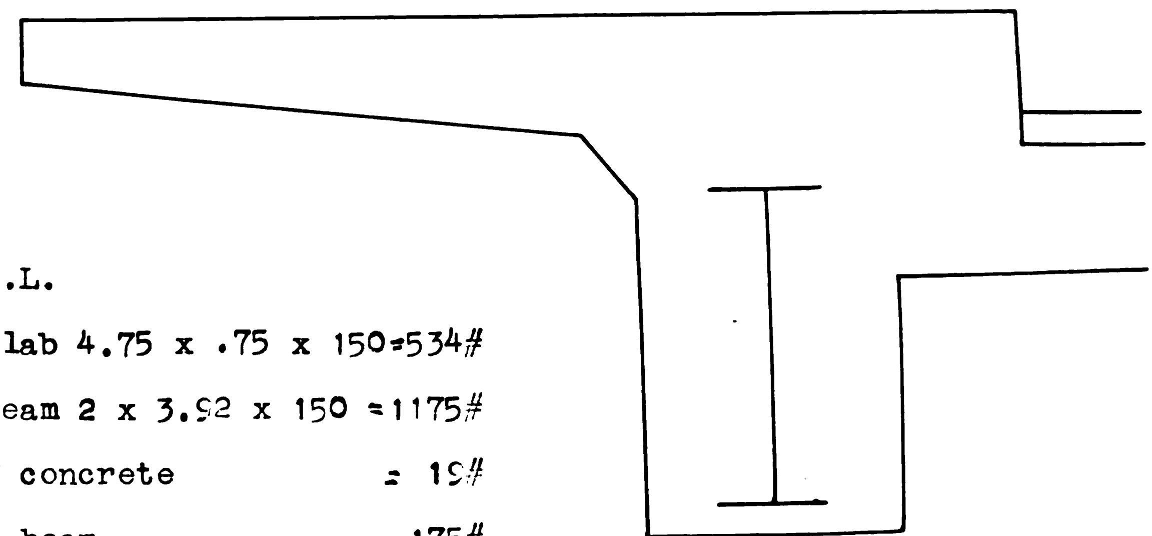
| Station | Type of column | Angle with roadway |
|---|---|---|
| 4-43 | Face of North Abutment | 90 degrees |
| 4-82.5 | Standard | 80 de. 35 min. |
| 5-16 | Standard | 71 de. 38 min. |
| 5-50 | Expansion | 63 " 32 " |
| 5-90 | Standard | 63 " 32 " |
| 6-34 | Expansion | 63 " 32 " |
| 6-76 | Standard | 63 " 32 " |
| 7-03 | Standard | 90 " |
| 7-40 | Expansion | 90 " |
| 7-77 | Standard | 90 " |
| 8-14 | Standard | 90 " |
| 8-51 | Expansion | 90 " |
| 8-79 | Standard | 65 " |
| 9-13 | Standard | 65 " |
| 9-35 | Expansion | 90 " |
| 9-64 | Standard | 90 " |

| Station | Type of column | Angle with roadway |
|---|---|---|
| 9-86 | Standard | 115 degrees |
| 10-20 | Expansion | 115 " |
| 10-54 | Standard | 115 " |
| 10-88 | Expansion | 115 " |
| 11-19.5 | North bridge abutment | 115 degrees |

From this it can be seen that the spans vary considerably and that the length of the beams vary in some of the spans. The spans over the Grand Trunk R.R. were at first planned as reinforced concrete beams but upon investigation it was found that as the spans were long it would be better to use steel sections. For the remainder of the viaduct, five interior and two exterior beams are used. These rest upon girders supported by three columns, which in turn rest upon concrete footings. As a speciman of the type of design used, a steel beam and a typical concrete beam will be designed here. In the sections over the railroads, blast plates of cast iron were designed to be placed above the tracks as protection for the concrete. These plates were four feet wide, one inch thick and made in interlocking sections three feet long. Bolts concreted in, held the plates in place.

Design of a 42' steel outside beam

In each steel section, 9 interior and 2 exterior beams were used.



D.L.

slab 4.75 x .75 x 150 = 534#

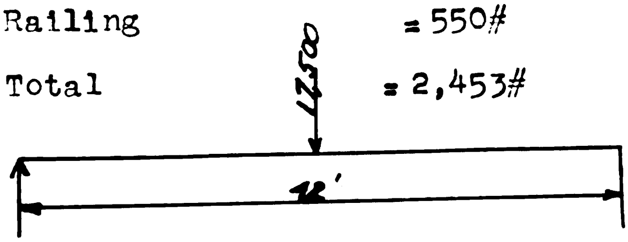
beam 2 x 3.92 x 150 = 1175#

V concrete = 19#

I-beam = 175#

Railing = 550#

Total = 2,453#




Moment

Dead Load $wl^2/8$ = 2453x42x42/8 = 540,000 foot pounds

Uniform live load = 100x6x42x42 = 132,000 " "

Live load Pl/4 = 17,500x42/4 = 183,500 " "

Total Moment = 728,500 " "

Carnegie C. B. 272 27" 175# beam

M 738,750 foot pounds

Maximum shear 229,500 pounds

AS a typical interior reinforced concrete beam, that having a span of 37' is here designed completely.

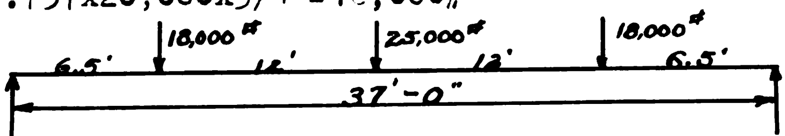
Negative moment at end supports,

$M = wl^2/16$ for dead load

Live Load = 7/38 of 4 trucks or .737 of 1 truck

.737x28,000 = 25,000#

.737x20,000x5/4 = 18,000#




M(truck) = (30,500x6.5 v12,500x12) .5 = 174,000 foot pounds

M(dead load) = 1703x37x37/16 = 145,000 " "

Total moment = 319,000 " "

$A_s = M/f_s(d-12t)$ = 319,000x12/16000x24.75 = 9.66 square inches

Bend up six 1 1/8" square bars

Investigation over support

d'/d = 4/29 = .138 p' = 7.62/24x29 = .0077 p = 15.18/24x29 = .0218

from diagram

k = .461 j = .848

$f_s$ = 319,000x12/15.18x.848x29 = 10,300# per square inch

$f_c$ = 10,300x .461/15(1- .461) = 585# per square inch

$f_s'$ = 10,300x(.461-.138)/.539 = 6,150# per square inch

Investigation at the center of the beam

Dead Load slab 7x150 = 1,050#

beam 2.21x2x150 = 663#

cant. 1/3x1/3x150 = 17#

total = 1,730#

Moment (truck) = (30,500x6.5 + 12,500x12)8 = 278,000ft.pounds

(dead load) 1,703x37x37/10 = 233,000 " "

Total Moment = 511,000 " "

Shear R = 45,130# Dead Load = 29,000#

V = 74,130#

$v = 74,130/24x.892x29 = 121\#$ requires stirrups

$zo = 74,130/100x.892x29 = 29.2''$

$K = 1/(1-16000-15x650) = .378$

$A_s = 511,000x12/16,000x(29-4.25) = 15.5$ square inches

Use 12 11/8" square bars in two rows

$f_s = 511,000x12/15.18x.892x29 = 15,650\#$ persquare inch

$f_c = 15,650x.378/15(1-.378) = 586\#$ per square inch

This works well with the steel over the supports and is satisfactory in other respects.

Stirrups will be spaced at 7" for 3' 3/8" round bars

9" for 3'

13" to center

The columns are of two different types, the standard and the expansion type. The standard type is three foot square, except in the steel spans. The expansion type consists of two columns, each 4' by 18" spaced 2 inches apart for expansion purposes. The columns are designed to line up on the outside faces, and to line transversly in each bent. In the bents forming a right angle with the road, the sides form the same angle as does the bent. In each bent, except the railroad steel spans, three columns are used to the bent. Under the steel spans, four columns are used in each bent. The method of design is the same as that used on the arch columns, and all columns of the same type are equally reinforced and are sufficiently strong to carry the loads imposed upon them.

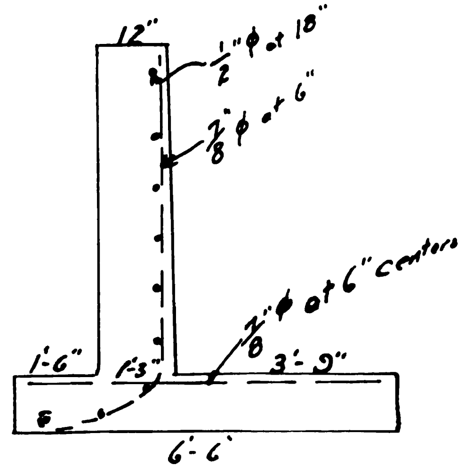
The girder and foundations were designed by the same methods. In the place of bending up bars for negative moment in the girders, double reinforcement was used. At the expansion joints the girders of the bent were separated by 1" expansion felt and steel plates were set 1" apart with their top edge forming the parabolic curve of the roadway crown and set to that elevation. The foundations were designed to be shallow as only direct bearing was transmitted to them. In most places this was at a depth of from 6 to 10 feet and placed them upon a gravel base. The footings were made one foot wider than the columns and extended under the entire bent. They were designed as three feet deep and with the required steel necessary under the loading.

From the north bridge abutment to a point northward 150' where the Olds outlet parallels the viaduct, it was necessary to design a small retaining wall. The maximum height being 11' and sloping to 5' at the north end.

In the design of this wall the loads were considered as surcharge being equal to 4'. The design of the 11' section,

$h' = 4'$  $h = 11'$  $c = 1/3$  $P = w/2 \times h(h + 2h')c$

$P \quad 50 \times 11(11 + 8).333 = 3{,}500\#$




$x = \dfrac{h^2 + 3hh'}{3(h+2h')} = 4.44'$

$M = 3{,}500 \times 4.44 = 15{,}400$ foot pounds

$V = 3{,}500\#$

$d = \sqrt{\dfrac{15{,}400}{107.4}} = 12" + 3 = 15"$

$v = \dfrac{3{,}500}{12 \cdot .875 \times 12} = 27.8\#$

Steel

$A_s = .0077 \times 12 \times 12 = 1.11$ sq.in.

Use $\frac{7}{8}$"φ bars at 6" centers = 1.26 sq.in.

Bond $U = \dfrac{3500}{2.95 \times 12 \times .875 \times 12} = 56.6\#$ O.K.

11' Retaining Wall Design

Stability against sliding

$P = \frac{1}{2} wh^2C = \frac{1}{2} \times 100 \times \overline{15}^2 \times \frac{1}{3} = 3,750^{\#}$

Resisting Force

$.45\,W = R.F.$

| | | |
|---|---|---|
| Concrete Wall | $1.25 \times 13' \times 150$ | $= 2440^{\#}$ |
| Footing | $1 \times 6.25 \times 150$ | $= 940^{\#}$ |
| Rail | | $= 550^{\#}$ |
| Walk | | $= 255^{\#}$ |
| Fill | $17 \times 3.75 \times 100$ | $= 6,375^{\#}$ |
| Total | | $= 10,570^{\#}$ |

$.45 \times 10,570^{\#} = 4,750^{\#}$ which will resist $3,750^{\#}$

Stability against overturning

Moment about toe

$$x = \frac{2440 \times 2.12 + 550 \times 2.12 + 940 \times 3.25 + 255 \times 3.0 + 6375 \times 4.62}{10,570}$$

$x = 3.64'$ - point of application of resultant is inside middle third

Soil Pressure

$$P = \frac{10,570}{6.5}\left(1 \pm \frac{6(3.25 - 2.07)}{6.5}\right)$$

$P = 3,400^{\#}$ or $-146$ pounds which is satisfactory

Design of Rear Cantilever

Downward pressure $= 561^{\#}$ concrete $+ 6,375^{\#}$ fill $= 6,936^{\#}$

Upward pressure $= \frac{1900}{2} \times 3.47 = 3,290^{\#}$

$V = 6,936 + 41 - 3,290 = 3687^{\#}$

$M = 6,936 \times 1.87 + 41 \times 3.47 - 3290 \times 1.73 = 7,442$ ft pounds

$d = \sqrt{\frac{7442}{107.4}} = 6.9''$ or $7''$

$\text{Shear} = \frac{3687}{12 \times .875 \times 7} = 51^{\#}$ too great

$d = \frac{3,687}{12 \times 40 \times .875} = 8.75''$ or $9'' + 3'' = 12''$ Depth of Rear Cantilever

Steel $A_s = .0077 \times 12 \times 9 = .83\,\square''$ Use $\frac{7}{8}''\phi$ at 6" centers.

Design of Toe Cantilever

depth and steel check with rear cantilever and is so used.

The outlet for the Olds Motor Works was planned to be built adjacent to this retaining wall on a 5.5% grade. It was to be 18' wide, with a concrete base and brick wearing surface. It was reinforced with 3/4" round bars at 9" c.c. as it was in part over that area that had been excavated for foundations,and it was not to be considered a part of the contract for the viaduct construction.

The intersection of Moores River Drive and Logan St. was discussed in the preliminaries as being subject to change. It was found in the final design that the grade of Moores River Drive would not be materially affected, but that it would be necessary to widen the Drive considerably. This meant a few minor changes in the crown of the intersection. At Albert Street it was found necessary to raise the intersection about 5' in order to conform with the desired established grade of the viaduct. Therefore it would be necessary to fill the roadway for 200' from the intersection and as this was a suitable place for the excess excavation fromthe foundations, it was so decided to arrange for the fill and grading of this street to be made a part of the contract. By means of the vertical curves the old pavement on north Logan St. was met very smoothly.

The wearing surface was designed as paving brick, using a row of white brick in marking each of the four traffic lanes. Brick was used because of the steep grades, asphalt having a tendency to creep and wrinkle on steep bridge grades. Bricks are also easier to replace and have better wearing qualities than a concrete wearing surface.

As has been stated before, one full sized and one small sidewalk was designed for two reasons, first in order that the required Olds Outlet might be constructed on City property, and second, that as there is so little pedestrian traffic and so much motor vehicle traffic, it was felt that, rather than decrease the roadway width it would be better to build a narrow sidewalk.

The handrail is of reinforced concrete plinth and coping with concrete spindles spaced at about 4". Over every bent a concrete pilaster, corresponding to the width of the bent, is placed. In the expansion bents the 1" felt was carried up through the pilaster. Experience shows that while the expansion joint is supposed to be concealed, all attempts to hide it were entirely unsatisfactory.

Concrete lamp posts were designed to be cast in place, spaced at about equal intervals. The city furnished the lamp post form as well as the spindle forms. The light wires are carried in a 2" conduit cast in the plinth.

The architectural treatment has been mentioned in connection with the arches. Throughout the entire structure the same thought was carried out as far as possible, that the completed structure should be pleasing to the eye and an asset to the city architecturally as well as economically. Throughout the viaduct, the viaduct section beams were all made of the same depth, breadth, and of equal number. The lines were made as continuous as possible, for instance, the columns all line up on the outside faces, as do the arch ribs and spandrel columns. All columns and the girders of the bent are of the same width which makes a uniform line transversely.

Curved brackets at the ends of the beams are used, both for the appearance and for the extra area of concrete available for stress in shear. Brackets are used under the sidewalks at the pilaster points for the same reasons. The vertical curves are carried out in detail, showing no distinct breaks of grade. Throughout, the effect desired was was one of a plain, pleasing, reinforced concrete design.

In this design also simplicity in form work and identical sections were sought in so far as possible. This held true in the detailing of various parts as for instance, the reinforceing steel placing and spacing.

After having completed the design, the exact quantities of excavation, concrete, structural and reinforceing steel were computed and placed in tabulated lists on the plans.

The total quantities are:

| | |
|---|---|
| Excavation | 10,250 cubic yards |
| Concrete | 7,422 cubic yards |
| Reinforcing Steel | 1,000,000 pounds |
| Structural Steel | 233,635 pounds |

The bar lists were detailed and marked with letters and numerals indicating their place in the structure, as, S.A.W.W.1. meaning, South Abutment Wing Wall.

The contract and specifications were modeled on those used in the state highway bridge department. The contract calls for the erection of the Logan Street Viaduct and the gist of it is given as follows, " Whereby the contractors shall furnish all the necessary equipment, machinery, tools, apparatus, and other means of construction, in order that the work may be performed in accordance with the plans and specifications, and to furnish all materials and labor except, cement. The work to be done under the authorized representatives of the City of Lansing".

The contract stated that work shall commence on Sept. 18, 1928, and be prosecuted in such a manner as to insure completion on or before October 1, 1929. The compensation to be paid upon completion and acceptance of the structure by the representatives of the city in the full amount as bid. Estimates from time to time as work progresses and based upon 80% of the work completed will be paid. The retained 20% retained will be paid upon final acceptance.

Construction

On August 6, 1928, the City Clerk was authorized to advertise for bids for the construction of the Logan Street Viaduct. The bids were opened Sept.6, 1928, and the low bid was that of the Folwell Engineering Co., one of the nineteen bids received and was for the amount of $279,940.04. The next low bid being that of the Stein Construction Co. of Milwaukee and was for the amount of $280,900. Anderson and Co. being third with a bid of $281,300. All except three of the bids were below $300,000.

Three alternate designs and their accompanying bids for construction were received. Two of these were the Luten design and the other a design by a Detroit firm.

One of the Luten designs was a series of three earth fill arches, the remaining section of the viaduct belonging to the City was used. The other was a structure of very similar design to that prepared by the city. The design by the Detroit firm used four river piers with structural steel beams of 80' spans. Of these alternates, only one was bid at a lower price than that bid for the City design, that one being the earth filled arches.

After thorough investigation by the City Bridge Committee, the bid of the Folwell Engineering Co. was accepted as being the lowest and best bid and upon the recommendation of the committee, the contract was awarded Sept.10, 1928 and formally approved by the City Council after all parties had signed on October 8, 1928. The agreements between the City and the Grand Trunk R.R., the Michigan Central R.R., and the Olds Motor Works were executed October 29, 1928.

Accompanying the bid as requested by the City, was a list of equipment which the contractor had available and intended to use in the erection of the bridge. This list follows and while more equipment was purchased, the bulk of the equipment used is contained in the list.

2 - Erie steam cranes, 25 tons capacity.

1 - Concrete mixer, 1 cubic yard capacity.

2 - Concrete mixers, one-half yard capacity.

1 - Concrete tower, complete with hoisting apparatus and spouts, height, 150'

1 - Air compressor (large size Ingersoll-Rand)

2 - 8" Centrifugal pumps complete with electric motors

4 - 6" Centrifugal pumps complete with motors

1 - 4" Centrifugal pump complete

1 - Barber-Greene material hoist complete with motors and belt

1 - Blaw-Knox bins and batching equipment.

350 - Pieces of steel sheet piling, 40' long.

2 - McKiernan Terry No. 7 steam hammers

1 - " " No. 9 " "

1 - Upright frame band saw.

2 - Skilsaws

1 - Buzzsaw

2 - Floor sanders

1 - G.M. C. 7 yd. truck

2 - Ford 1 yd. trucks

3 - temporary buildings and office.

Miscellaneous assortment of small tools and timbers.

The Folwell Engineering Co. presented at the City's request, a schedule of work and the method and manner of proceedure. It is briefly:

1. To erect a temporary foot bridge for pedestrians. A number of wood piling were to be driven 75' east of the bridge and a wooden bridge erected.

2. Using the bridge as support, piling were to be driven to act as a working trestle, later to act as arch centering supports, and to remove the old bridge as the trestle was built.

3. By December 1, to start excavation and set sheeting in the South Abutment and South Pier.

4. At the same time the trestle was being built, the excavations for the viaduct footings were to be dug, the footings concreted, and the columns built in the entire viaduct by December 1, and by this time formwork was to be sufficiently advanced to permit steel placing and pouring of the first viaduct deck section.

5. The construction was to continue through the winter so that the viaduct section would be completed with the exceptionof the handrail by May 1,1929.

6. The completion of the arches by July 1, and the arch superstructure by August 1.

7. The entire structure to be complete by September1, 1929, or a month ahead of the completion date as set by the City in the contract.

This schedule was to be accepted as a guide to the method of proceedure in the prosecution of the work and was prepared by the Folwell Co. officials. As soon as the contract was awarded they hired their superintendent for the for the job, Mr. E. Sebern of Detroit, Michigan. He was not an engineer, but had considerable experience in the building line, having worked on several important projects in varying capacities more notably, the Book Cadillac Hotel in Detroit, as general superintendent, and for the O.W.Jenkins Co. of Detroit on a power house project in Ohio, also as superintendent, and as carpenter foreman for the Christman Co. of Detroit, on the Fisher Building in Detroit. In his first appearances in Lansing, he spoke at several luncheon clubs and promised to do better than the prepared schedule, by having the viaduct ready for traffic by July 4,1929. Naturally the City Bridge Committee and the engineering department were glad to have a statement that the work would be pushed with so much vigor and that traffic could be resumed so much sooner than had been anticipated.

Mr. Sebern employed from 200 to 250 men and the work followed closely on schedule. In ten days the foot bridge was completed and the work on the trestle and viaduct progressing rapidly. The first foundations dug were those at stations 5-50 and 5-90 and then the north bridge abutment and retaining wall. One crane was used on these foundations while the other worked on the trestle. The viaduct footings and columns from the North Abutment to station 5-50 were not to be constructed at once as this would hamper the activities on the arch construction.

As the footings were shallow for the viaduct section, no difficulty was experienced and they progressed satisfactorily. About the middle of October the first difficulty was experienced. The forms for the first set of columns at station 6-34 were set and lined up perfectly. In bracing the columns, Universal column clamps were used with 2x6 plank holding the columns in their respective positions. Before the permit to proceed with the concreting, the superintendent was informed that due to the shape of the columns, it was believed that the bracing was not sufficient, and that, under pressure the columns would tend to form a square, but he stated that he had poured a considerable number of such columns and that they would be all right. Concreting was permitted, but after about two thirds had been poured, three of the column forms broke. They failed by tending to assume a square shape and as they were held securely at the base, they were badly twisted. These columns were ordered removed and replaced with columns of true shape as designed. The removal of these columns was difficult because in order to save the reinforcing steel, it was necessary to proceed with care and even though a continuous shift with air hammers worked on it, the concrete, which had been poured slowly, was well set up before all had been removed. In the rebuilding of these columns, certain of the suggestions offered by the city were accepted for the most part and the columns were satisfactory on the second trial. However, several were later removed for the same reason, and it was not until the forms were constructed as the city suggested that complete success was obtained in their construction. This method being that of setting the forms in their proper positions and building the bracing square and blocking back to the forms in the triangular space on either side.

A little difficulty was encountered in the expansion columns of maintaining true perpendicular expansion joints, but by filling the 2" space between the forms with dry sand and maintaining equal heights of the concrete on either side of the expansion joint during pouring, this obstacle was overcome.

Honeycombing was almost entirely eliminated after the superintendent learned that the city would not tolerate it after one complete column was ordered removed because of an excessive amount that exposed the reinforcing steel. The superintendent strenously objected to removing it, as he stated that the patch could be as strong as the rest of the concrete, but it was felt that he had had sufficient opportu-ity to produce acceptable concrete, and that it was bad prac-tice to allow patchwork as inevitably spalling results after weathering, leaving an unsightly structure. So in the interest of the city and in accordance with the plans and specifications it was ordered that any honeycombed work would not be permitted or accepted. This proved to be a wise move, for after that, special care was taken during concreting to prevent the accumulation of stone pockets.

At this time, in connection with the preceeding statements, it might be well to discuss the concrete designed and used in this structure for it might be misunderstood and thought that the city was unduly severe in its inspection and control. The contract and specifications state that the City of Lansing will furnish all the cement for the structure. This was intended to eliminate any claims on the contractors part if certain changes were made in the mix. Under this policy, it was to the cities advantage to produce a strong concrete with the minimum amount of cement. Although the plans were designed using 2,000# and 2,500# concrete, the concrete used was designed by Abram's water cement ratio for 3,000# and 3,500# concrete. The slumps varied according to the location and spacing of the steel, from a minimum of 4" to a maximum of 7" which is generally considered quite workable in a properly designed mix. The aggregates were measured differently, the gravel being measured by volume and the sand by weight. The sand hopper was of the scales type and permitted rapid adjustant. For all the superstructure work approximately 6.5 sacks of cement per yard of concrete were used. On each pour, concrete test cylinders were made, cured in the same manner as the concrete they represented and broken by the testing laboratory at Michigan State College. Some being broken at each of 3,7, and 28 days. In the neighborhood of 1000 test cylinders were made and tested for accurate control of the concrete. Some of the specimans tested represented the worst conditions encountered in the pouring or curing. For instance, during freezing weather, if a certain place did not recieve as much heat as the average, a cylinder was placed there to cure and the results noted, not as being a test of the average section but more as an indication of the strength of the weakest spot or link in the structure. Therefore, it is useless to tabulate the results of these tests as they refer in part to specific conditions and are not the indication of the average concrete strength.

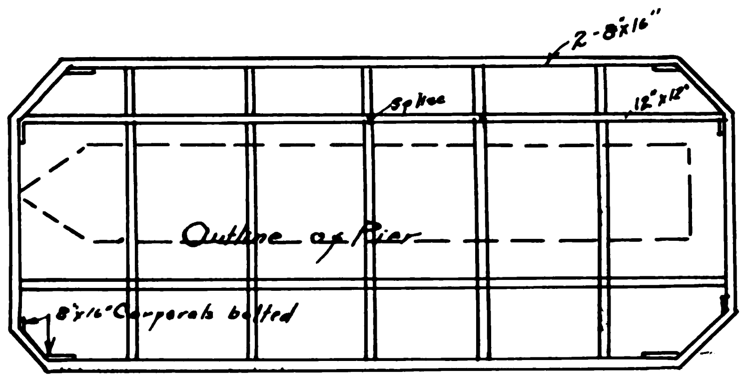
Due to the slump maintained, it was impractical to use the spouting for any appreciable distance, as concrete having a slump of 5" or 6" will not slide on much less than a 45 degree angle. From the termination of the spouts it was necessary to convey it to the destination by other methods. In this manner the contractors cost for placing concrete was unreasonably high. Although they had a fine general concrete plant, the rehandling caused an increase in cost. The plant was permanently located at about station 7-00 being in a central position for the viaduct pouring. In order to concrete the columns and superstructure of the arches, the concrete was mixed at the plant, hoisted to the top of the tower, automatically tripped and dumped to the spouting hopper, there a man controlled its flow into the spouts. This conveyed the concrete about 150' to a hopper, there another man controlled the flow into a minature railway and it was transported from 400' to 700' into a third hopper and from thence to the concrete buggies and into the forms. This was elaborate but impractical. In order to save money, concrete handling must be contained in one or two operations. In the construction of other bridges, the methods used were not as elaborate and yet produced concrete of excellent quality and the control was just as efficient for a great deal less money, which is the point the contractor must watch. Of course the same methods are not always applicable to all conditions. The concrete pours were quite large and as it was necessary to have a large amount of aggregate on hand to permit continuous pouring, it is understood that it called for special consideration.

While the concreting of the columns and footings of the viaduct section was in progress, the temporary footbridge and trestle was completed. About the first of December, 1928, the South Abutment was started, the sheeting set and driven to refusal and excavation commenced. The cofferdam was built 3' from each face of the neat footing line which made the cofferdam 25' x 51' as required by the plans and specifications. The excavation proceeded at a fair rate, the material corresponding very closely to that shown by the soundings. The elevation of the roadway at the site of the South Abutment was 137.00 feet and the footing of the South Abutment was designed to be placed at elevation 90.00 feet. At elevation 112.00 feet, the first variation from the material indicated was encountered. The plan showed that sand, some clay, and hard compacted sand could be expected. The material as excavated proved to be a laminated, sandy shale. It varied greatly in hardness and in places became very soft and crumbly. This shale overlaid the hard rock upon which the foundation was placed and is the basis of the law suit against the city, it being the contention of the contractor that the city engineers knew or should have known of the existance and true nature of this material, and that it was their intention to conceal this for the purpose of obtaining a low bid upon the plans, and that they having been wilfully misled, made such low bid and that due to the true nature of the material and of the greater difficulty of excavation for which they were unequipped, they were forced to make large expenditures in order to perform the work as required by the city engineers. They claim as due them an amount equal to this added expense with a fair profit and for which amount, the city having refused to allow any such claim, they are suing in the court of equity for such claim as a just and reasonable reward. The city maintains that its engineers were uninformed as to the true conditions of the material. It maintains that from the samples obtained by the method of sounding employed, it would have been impossible to ascertain a difference in the material as actually encountered because the sample when pumped to the surface would appear as the material described on the plans. Further, that the soundings were made for the use of the city and were not guaranteed as correct but were thought to be so and were offered only as an aid to the bidder. However as this lawsuit is still pending in the United States District Court at Detroit, awaiting its turn on the court calender, this remains a controversial subject.

In the excavation of this material, air spades were used to good advantage. The shale was easily shelled off in layers and loaded into boxes which were hoisted out by the crane. The excavation of this material proceeded even more rapidly than the previously encountered material. It was at this elevation(112.00 feet) that the steel sheeting had stopped penetration and in view of the character of the material, the superintendent decided to work below the sheeting and did so, even in the face of contrary advice from the engineers, but in this case he was successful and the footing was poured in a fairly dry hole on January 23,1929. The footing was poured at the elevation planned, the rock being of suitable hardness and free from laminations. The rock was a hard sedimentary limestone. Forms for the abutments took two weeks to build, during which time the footing was heated to a suitable temperature. Cold weather that year made it necessary that all materials should be thoroughly heated before pouring. As the abutment consisted of about 800 cubic yards of concrete, considerable difficulty was experienced in reaching this proper temperature and so when the temperature of the concrete fell below 60 degrees consistantly, the pour was stopped until the temperature of the material warranted starting again. Thus the abutment was concreted in three pours of about equal volumns and they were heated to a temperature of 70 degrees for fourteen days after which time backfilling was started and evenly distributted over the area. The sheeting was pulled and moved to the north side of the river on about March 15,1929.

During January, the steel sheeting was being set for the South Pier cofferdam and after a delay of about 30 days this was completed. This is the cofferdam that collapsed and especial care will be taken in the description of its construction and excavation.

In the South Abutment the cofferdam was rectangular 26' x 51' with wood walers and struts. There were six 12"x12" transverse struts and one 12"x12" longitudinal strut with 8"x16" double walers in each set. The spacing of the sets varied from 10' at the top to 6' on the bottom, there being five sets used. There was an open space not timbered below the sheeting. In the South Pier, the plan was a rectangular cofferdam, 26' x 54.5', but in trying to close this section it was found that five sheet piles were lacking, and in order to avoid purchasing or renting the piling the corners were cut on a 45 degree angle with the consent of the engineer in charge. The top set of timbers was placed 1' above the springing line and water surface, or at elevation 128.00 feet and the next at elevation 120.00 feet, the next at elevation 114.00 feet, another set at elevation 108.00 feet, and at the time of the collapse, another was being set at elevation 103.0 feet. Four sets of timbers were already in place and a fifth being placed. The following sketch gives a general idea of the timbering used.




The top set is entirely of wood, while the remaining sets contained 12" x12" concrete struts which were to pass through the pier and eliminate the boxing around them while concreting. These were held in place by cables, bolts and nailed cleats. They appeared in every way structurally sound.

In the first attempt to dewater the cofferdam it became apparent that a leak of quite some size existed. After driving the sheeting again and adding two more 6" pumps, the water was gradually lowered and excavation started. The first material was silt and debris and several large boulders were encountered. In removing them the cofferdam flooded. The sheeting was again driven but ice forming on the interlock held them open and again made dewatering difficult. After struggling for several days the water was again lowered and the cofferdam again ready for excavation but there had been little progress made due to subsequent floodings and adjustment of timbers. The excavated material was being dumped into the river on the south west corner of the cofferdam, and as the leaks were washing in a great deal of material, the engineer suggested that a berm be placed around the cofferdam, but the superintendent flatly refused to do this because of the cost. A little money spent at this point would have saved, money, time, and lives as it later developed. The stability offered to a cofferdam by a berm is considerable and should not be overlooked in the work. The surface of the water was at elevation 127.00 feet and the excavation was carried to elevation 102.5 feet which meant a head of 24.5 feet of water. The material being excavated was a mixture of clay and the shale. The sheeting had been driven to about elevation 101.00 feet and this was the state of conditions prevailing April 12,1929, when the cofferdam collapsed, trapping five men and seriously injuring two others. Five minutes previous to the collapse, nothing was noticed to indicate the impending disaster, except that the dam was beginning to leak a little more freely.The foreman had just called 12 or 14 men from the bottom to put cinders around the outside to stop the leak and had it not been for this, the loss of life would have been much greater. The crash was almost instantaneous and was entirely unexpected even by the men working in the bottom as indicated by their positions when they were recovered. Only one man escaped from the bottom and he was seriously injured. He said that he was in a different part of the cofferdam from the rest and was carried up by the surge of water. The other survivor was standing on the first set of timbers and was thrown high in the air and fell back into the maize of timbers and was permanently crippled. This upheaval seems to be an indication of the method of collapse, the timbers being broken and heaved upward and the sheeting closing at the surface.

At the time of the collapse, the superintendent who was in Chicago, returned immediately and offered several theories as to the cause of the collapse. These, being of a defensive nature were later disqualified by actual findings. His principal theory was that the excavation had reached a layer of quick sand and that the bottom had heaved up and disrupted the timbering. Others gave as their opinion that the method of cutting corners was not structually sound, while still others said that the concrete struts gave way first, causing the collapse. A great many other people expressed their ideas and probably still have faith in them. Mr. Rey and the author of this thesis did not express or involve themselves in any way as they only approved the methods of construction and were not responsible for the loss, as the superintendent only presented plans for their approval and resented any further control of their work.

A Grand Jury was called upon by the People to investigate the disaster and place the blame if any. They requested a competant engineer to examine the plans of the cofferdam and to see if they were strong enough for the loads imposed. Professor C.L.Allen of the Civil Engineering Department of Michigan State College made the analysis and stated that sufficient bracing and timbering were used if properly placed and fixed to withstand the pressures. After all

available testimony was heard, the Grand Jury decided to wait until the previous level had been reached and all the evidence then available studied.

Accordingly, a new cofferdam was constructed in the same position as the first one. After the sheeting was set, the task of removing the twisted cofferdam was attempted. Most of the sheeting could not be pulled and had to be burned off in pieces which made a slow job as only that portion above water could be burned. Shifts worked night and day in an attempt to recover the bodies as quickly as possible but progress was necessarily slow and it was four weeks until the first body was recovered and six until the last corpse was reached.

The new cofferdam was constructed with double struts spaced one foot apart to allow a concrete strut to be poured between before concreting. Mamy more precautions were taken in the building of the second, such as bolted x-bracing and instead of using the cut corners, the cofferdam was made rectangular. Also the sets were held in position by spacers between and bolts tying them together ridgidly. At this time the city bridge engineer ordered a berm placed around the cofferdam. The sheeting used in this cofferdam was the same as that used and recently pulled from the South Abutment.

On May 25,1929, the last body was recovered and excavation was ready to continue. Upon examination, it was found that the material in the bottom at the time of the collapse was the shale mentioned and showed no signs of upheaval. Of the 20 concrete struts used, only two were found broken. While the sheeting was crushed completely together at the surface, the general outline of the sheeting at elevation 102.00 feet was close to the original lines, the south wall having shifted north about two feet and the north wall at the corners almost intact with a 3' or 4' curve between. The sheeting was dragged south. Very little dirt had washed in. The Grand Jury received this evidence but failed to place any blame for the accident, terming it an unavoidable accident and an act of God. In this Thesis, no attempt will be made to continue this investigation or offer a solution, but the facts are given the reader so that he may draw his own conclusions.

In the process of excavating from elevation 102.00 feet to the elevation at which the footing was placed at elevation 93.12 feet, practically the same material was found at that elevation as in the South Abutment, namely, sandy shale. The pier was founded on the limestone rock and the forms and concreting were simple after that.

The South Pier was completed about the middle of July, which was considerably behind the contemplated schedule, although Mr. Sebern assured the city that the bridge would be completed on the date called for in the contract, October 1, 1929. At this time it was suspected by the engineers that the job was just a little beyond Mr. Sebern's ability to handle and the Folwell Co. was so informed but they seemed to have great confidence in him and the protest was not pressed.

The North Abutment was being excavated at this time, or as the South Pier was being poured. Excavation moved along fairly well and the materials conformed to the sounding sheet until the shale was reached as before, but in this case the rock at the point at which the footing was to be poured was found to be a wedge shaped slab, about 7' thick at the point where the sounding had been made and vanishing completely toward the east.

This rock was underlain with soft clay and would have been a treacherous foundation, so the engineer ordered the contractor to go deep enough to get a suitable rock foundation. This was found after excavating 10' of shale which was paid for as an extra at the price bid for this depth of excavation. Incidently, it was generally admitted by the Folwell Co. that this excavation was the only foundation that they made any money on, which discounts the claims made in the law suit. The foundation was placed at elevation 90.00 feet and work proceeded so that the abutment was completed at about the time time that the contract called for the completion of the bridge. In figuring the completion date when extra work is added, the proportion of the cost of the extra work to the total contract was applied to the time allowance as provided in the specifications, which added about another month, or November 1,1929.

After the sheeting had been released in the South Pier, it was moved to the North Pier and set. In setting this sheeting and driving the piles slanted out of plumb and the superintendent was so informed, but he maintained that it was to do any damage and did not change them at that time. After excavating about 10 feet it was very evident that the sheeting would be inside of the footing line. At this, Mr. Sebern asked the engineer to change the design which he refused to do and for this reason it cost the Folwell Co. a lot of money to pull and reset the sheeting and when driving again, it was most difficult to maintain them perpendicular. It was in December before this pier was completed, the same type of material being excavated as before and the footing set upon solid rock at elevation 97.00 feet which was a foot higher than planned.

In August 1929, the removal of the old foundations in the river was finally commenced and was found more difficult than they had anticipated. The foundations were built upon timber cribs filled with boulders and it was almost impossible to get a tight cofferdam around them. The old South Abutment was removed first, the pier next and the old North Abutment last. This was completed in January and held up the rest of the work a month. After their removal the substructure was complete and ready to receive the arches.

In describing the work on the river foundations, the description of the work being carried on at the points has been postponed with the idea of giving a clear picture of the river work, which in reality was the key to the whole structure. For while the viaduct work presented difficulties of a kind, it was in no manner as difficult as the river foundations. Having disposed of this and resuming at the point where the columns and footings of the viaduct had been poured and certain forming had been completed by December 1928, a brief description of the viaduct construction from that point on will be given. As has been stated in the design , expansion joints were placed at about 100' intervals, and the pours consisted of beams, girders, deck, and sidewalk slabs between these joints. All the items named were considered in the design as monolithic concrete which must be poured without construction joints, except as specifically provided for or permitted by the engineer in charge. This meant that in cold weather, all material for the pour must be on hand and heated to the proper temperature and that the provisions for heating the concrete when poured should be fully taken care of.

The first section of superstructure to be formed was that from station 6-34 to 7-40 and contained 386.1 cubic yards of concrete and 70,000# of reinforcing steel. This pour then included 2 girders , 2 expansion girders ,3 spans of 7 beams each, and 106' of deck and sidewalk slabs.

The formwork was simplified as much as possible by the preparation of beam sides and bottoms on the work benches, as were the girder sides and bottoms. All forms for exposed concrete were sanded, using floor sanders, and were oiled with parafin oil. The form work progressed rapidly in this section. The beam sides and bottoms, were set on the true grade and then blocked up to give a construction camber and allow for settlement. As beam sides were set to position the decks were boarded up. The falsework consisted of upright timbers, supported by bases and with longitudinal caps upon which wooden wedges were used for blocking. The main difficulty in forming came in supporting the curb forms to grade and line. The method employed was that of placing stakes under it and through the deck concrete, to be withdrawn before the concrete fully set up, but this proved unsatisfactory as a little settlement of the forms caused uneven lines. Later a method of cantilever support, as suggested by the engineers was used with good success. This method of building forms was followed throughout the entire superstructure and worked out very well.

In placing the reinforcing steel, only method was usually possible whereby the bars could be threaded in the stirrups.

This evolved into a rule of thumb method of placing which the workmen easily learned and was followed throughout the structure. The chairs used were those manufactured by the Universal company and were placed often enough to support the steel to the exact gage called for in the plans. As soon as the superintendent learned that no slipshod work would be permitted, the steel was well placed.

Before pouring any concrete, the forms were thoroughly washed with the special object of having all construction joints entirely clean and all pockets free of shavings, debris etc. When everything was ready the approval was given and the work allowed to proceed. In placing the concrete, the plan followed was to work uniformly across the transverse section, in order to evenly distribute the weight and settlement. The deck was screeded to guides set by instrument and made to conform to the roadway grade and crown. These screed strips were set with a construction camber also and when checked later were found to conform closely to the desired grade. The deck was floated to grade after screeding several times.

The surface consisted of two rubbings with carburundum stones and water, the first given as soon as the forms were stripped and the next as soon as possible thereafter. All form marks were removed in the initial rubbing, while the second, rubbing with a finer stone, filled the small irregularities and gave a smooth, white surface.

The first section of superstructure was poured December 23, 1928, and was heated to 70 degrees for 14 days. The protection used was a large circus tent which proved quite unsuitable because in handling over form work, it was easily snagged and torn. The heat was furnished by salamanders

with tubs of water furnishing the moisture. This was the last section poured until April 1929, because of the severe cold weather and the cost of the required heating. During the summer the remaining section of the viaduct was poured. This being in hot weather, water curing was sufficient. The viaduct section was completed nearly to schedule.

The handrail and lamp posts were constructed as close after the completion of the viaduct as possible. The forms for the handrail and pilasters were lined with Presdwood, a commercial product manufactured by the Masonite Co. which was entirely suited to this type of work. The spindles were made in cast iron forms as were the lamp posts, the forms being furnished by the city. The spindles were cast as separate units and were a little difficult to make. The concrete, in setting would shrink and tear away from the base. This was overcome by pouring slowly and using a form vibrator, operated by electricity. The same difficulty was encountered in the lamp posts and was overcome in the same way, The pilasters were cast first, then the base with the rods protruding over which was set the spindles which were concreted in the coping pour. The concrete used was a 1:1:1 mix, the sand was well graded and 1/4" peastone used as coarse aggregate. This sand grading has much to do with the surface finish, for if properly graded, a very smooth, dense surface is exposed that needs little finishing.

The surface of the deck was floated to grade and a 1/2" sand cushion was placed under the brick. In April 1930, the City of Lansing paved the roadway using Metropolitan Paving brick of the highest grade. Three lines of white brick were used to divide the road into four traffic lanes which proved quite suitable. The brick were washed in with hot asphalt which makes a very durable paving.

In January 1930, the construction of the arch ribs began after the completion of the river foundations. The arches were all formed and in each set the arches were poured in pairs, the two outside and then the two inside. The centering remained in place for 21 days. Wooden wedges were used to bring the forms to the true arch curve. The arches were poured from the south towards the north span. The concrete was deposited simultaneously at each springing line and worked symmetrically to the crown. In no case was it necessary to load the crown to prevent springing.

After the arches were poured, the formwork for the rest of the superstructure was set completely and the columns poured and stripped of forms for examination. The cinder concrete was poured over the crown section where it would be impossible to remove forms. This made an ideal concrete material, being light and easy to place.

At the end of 21 days, the centering was struck and the deck concrete poured in three pours over each arch, the two end sections being cast simultaneously. Construction joints were placed at the third points and the concrete separated by copper plates which was to permit elastic movement in the arch. The arch superstructure was completed late in March 1930.

In order to protect the concrete, a huge building was erected over the entire area to be poured and unit steam heaters were installed about every 50'. These were fed from the boilers on the cranes and they kept the temperature to the

required point for the required time, which was usually 70 degrees for 14 days. All aggregates were sufficiently heated so that the concrete could be deposited at a temperature between 60 to 75 degrees. This protection was required on all concrete cast during freezing weather.

The completed viaduct was accepted by the City of Lansing June 30, 1930.

[redacted]