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THE DESIGN AND ERECTION OF THE PENNSYLVANIA LIFT BRIDGE No. 458 OVER THE SOUTH BRANCH OF THE CHICAGO RIVER

By W. L. Smith and W. W. Priest, Under the Direction of J. C. Bland, Engineer of Bridges.

Read April 12, 1915.

THE DESIGN.

W. L. Smith.

General Description of New Bridge.

The bridge consists of a double track riveted truss lift span, 272 ft. 10 in. c. to c. of end posts, with two trusses of the Pratt type, having inclined top chords and two towers, one at each end, which are from 30 ft. 2 in. to 30 ft. 8 in. wide by 53 ft. 6 in. long and about 185 ft. high. The structure is skewed at an angle of about 47° 20′ to the center line of the stream.

At each end of the bridge there is a sectional counterweight consisting of two structural steel frames covered with about 315 cubic yards of concrete having the following proportions: one part Portland cement, two parts sand and four parts slag.

The approximate weight of each counterweight is:

Structural steel	93,100 lbs.
Concrete	1,489,100 lbs.
Total	1,582,200 lbs.

Careful estimate showed that the cost of the counterweights would be substantially the same whether slag or broken stone was used, with possibly a slight difference in favor of broken stone.

Rivet plugs were also considered, but were not used because they were difficult to obtain in sufficient quantities and were quite expensive.

The machinery house is situated on the top of the lift span at its center. The floor is 4½-in. plank supported by steel beams. The floor area is about 1,150 sq. ft. The walls are of 1½-in. cinder concrete ("float finish" outside) on metal lath supported by a steel frame. The roof is of tin construction laid on steel trusses and purlins. This house is supplied with a 5-ton overhead traveling crane for placing motors, etc.

The operator's house (floor area about 90 sq. ft.) is made of the same material as the machinery house and is suspended therefrom. Steel stairways connect it with the machinery house and with the floor deck of the span.

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

To render the truss span readily accessible at its extreme height, a ladder has been added to each tower extending from the top of the tower to the top of the lift span when fully raised. These ladders will be extended to provide access to the span at any point of its travel.

Operation.

To permit the passage of vessels, the truss span can be raised to its maximum height from its normal position (a lift of 111 feet) in 45 seconds. The span and its counterweights are suspended by 64, 2¼-in. plow steel ropes over 8, 15-ft. sheaves of structural and cast steel. (16, 2¼-in. ropes are connected to the top chord at each end of each truss, pass over a pair of sheaves and are attached, by means of equalizing devices, to the counterweights.)

The span is operated by two No. 162 Westinghouse railway type interpole 220-volt, 300-H. P. motors which are located in the machinery house. These motors are geared to four cast steel operating drums, each of which carries four 1⅛-in. plow steel operating ropes. These ropes pass over deflection sheaves at the ends of the span—two going up and two going down at each corner—and are fastened to the top and bottom, respectively, of the towers. Either motor alone can operate the driving mechanism. A 50-H. P. gasoline engine will also be installed for emergency service. This engine will lift the span to its maximum height in about ten minutes.

The operator's house contains all operating levers and switches and a mechanical indicator showing position of span.

Limit switches cut off the current when the span has reached its limiting positions, and solenoid brakes are applied automatically. Hand brakes are provided as an additional safeguard. Rail locks at each end of the span are operted by one 3¼-H. P. direct-current No. 2 type K. G. Westinghouse 220-volt motor placed at the foot of each tower.

There is a ball signal on top of the machinery house and there are semaphores at the bottom of each truss and on each end channel pier.

Defects Developed in Operation, Etc.

By far the most serious trouble, and the most expensive as well, has developed in connection with the sheaves. Owing to their size—15 ft. 0 in. pitch diameter—they are of "built-up" construction, i. e., each sheave consists of a center steel casting or sleeve and seven sections of cast steel rim segments, the rim and sleeve castings being connected with a web of built-up riveted steel construction. The detail drawing for the sheaves called for "all contact surfaces between rim segments, between web plates and rim and between web plates and sleeves to be finished to accurate bearing over entire area." As the design of the sheaves was based upon such finish, i. e., the rim segments and center sleeve were not connected to the "built-up" web with sufficient rivets to transmit all the stresses, it was essential that this requirement be met.

The sheaves as manufactured, however, did not meet this requirement. To insure proper action between the rims and webs four splice plates were added to each connection between the web diaphragms and rims, this affording eight additional turned bolts in double shear at each of these points.

To prevent creeping of the built-up portions on the sleeves four alternatives were considered:

(1) To cut out a circular strip from each web and sleeve casting and replace it with a steel ring having a driving fit.

(2) Drill holes for 1-in. pins on line between center castings and hub and drive tight fitting pins into them.

(3 Replace rivets connecting center casting to webs with loose fitting bolts which would permit the webs to move enough to come to a bearing on sleeve casting without shearing the bolts.

(4) To pour opening between webs and sleeves full of sulphur.

It was decided to adopt (1) or (2) and as (2) permitted the work to be done in the field it was adopted. Forty pins 1 in. diameter, 4 in. long, having a computed bearing stress of about 10,000 lbs. per square inch, were added to each sheave as described under (2). After the pins were driven the outside edges of the holes were calked so that they could not work out.

The cost of adding the splice plates was about $390 per sheave or $3,100 for the bridge; the cost of adding the pins was about $3,500 for the bridge.

Operation of the rail locks was difficult at times owing to the failure of the span to seat itself each time in exactly the same position. This was remedied by attaching entering tongues to the end floor beams of the truss span. These entering tongues engage centering guides which are carried by the floor beams at the span ends of the towers.

As originally designed, the operating cables were supported on the center line of the curved top chord by gum wood rollers mounted in steel brackets. These rollers wore rapidly and in some cases failed to turn. The ropes, when slack, abraded each other and the top chords of the trusses.

To overcome this excessive wear on the rollers—a maximum of about ⅝ in.—and to remedy the other conditions above de-

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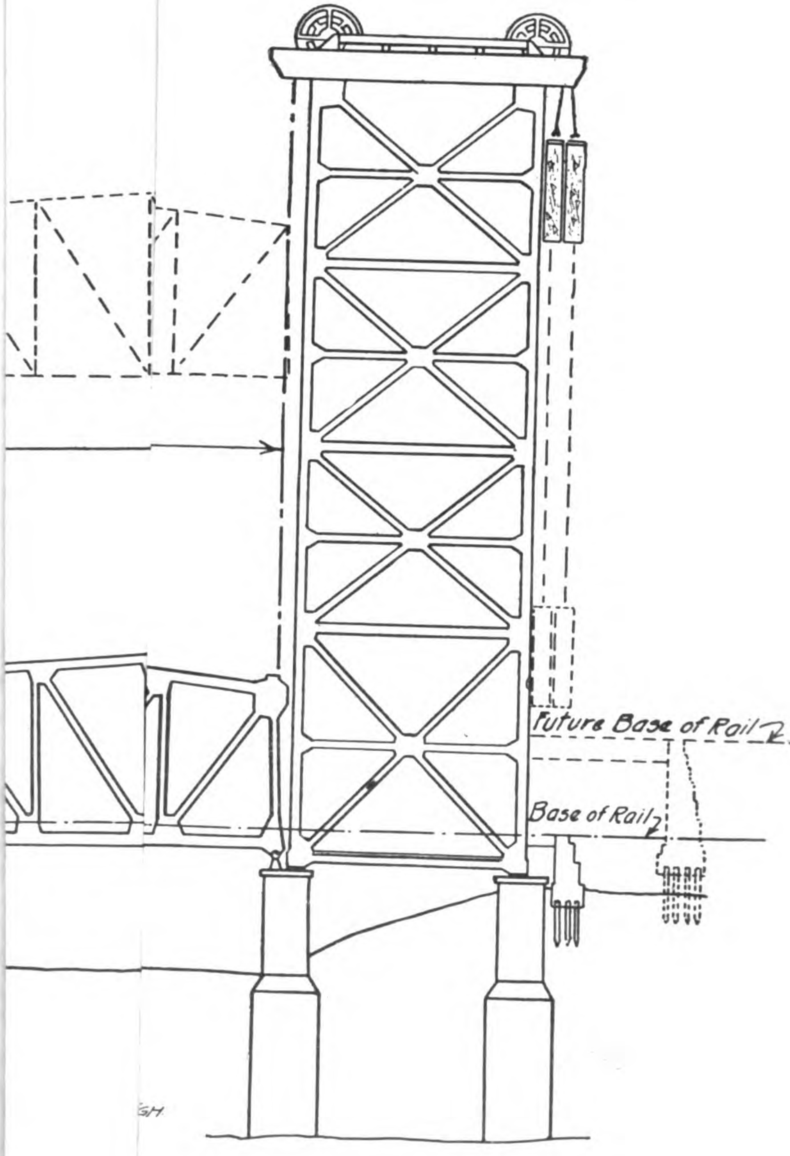
*Smith-Priest—Pennsylvania
Bridge.*



scribed, deflection sheaves were mounted on the upper chords at each end of each truss.

Adjustment of the downhaul ropes is provided near their connection to the tower columns by means of turnbuckles. When the ropes were slack, these turnbuckles tipped from their normal vertical positions and fouled the ends of the lift span. This condition was remedied by bending a strap around the cables just above the turnbuckles and connecting it to the columns.

The counterweight ropes engage forged steel equalizer bars which are connected, in turn, to the counterweights. The details necessitate considerable spreading of the ropes at these points. As designed, when the counterweights approached the extreme upper limit of their travel the change in angle between the ropes was sufficient to cause excessive wear in the equalizer bars from the turning and grinding of the pins in them. The consulting engineers for this bridge have designed a device which it is thought will, when installed, reduce this objectionable feature to a minimum.

Deficiencies in the Design.

In constructing another bridge of this type we would first of all, in view of our experience with the main sheaves of this bridge and of our Bridge No. 443 (another bridge of the same type of construction), insist that enough rivets be furnished or other positive means be employed to carry all stresses, no reliance being placed upon bearing of the component parts, and also provide a device whereby there would be no "unbalanced load" from counterweight ropes. The lift span as designed is supposed to be perfectly balanced when at mid-height only. At all other points of its travel there is an "unbalanced effect" from these ropes.

This unbalanced condition is plainly a maximum amount when the lift span is at the extreme limits of its travel. When beginning to lift the span from its normal position, therefore, besides overcoming frictional resistances from the weights of the moving span, counterweights, etc., we also have to overcome this maximum unbalanced condition. It should be a comparatively easy matter to overcome this objectionable feature.

Provision should be made for cutting off the power when the operating cables break at either end of the span. As designed, there is nothing to prevent the operator from continuing to hoist the span after the operating ropes break at either end and a condition which is apt to result in wedging the span tight between the tower columns. If the span is not properly counterbalanced, it can be seen that the end of the span where the breakage occurs might, under certain conditions, be pulled down.

Reasons for Adoption of This Type Here.

As stated elsewhere in this paper, it is expected that the tracks will be raised some twenty to twenty-five feet at this point in the future. This type of bridge is particularly well adapted to making such a change. The tower bracing has been so arranged that when this track elevation is attained, the present lower transverse struts can be replaced with the present tower floor beams. All that then remains to be done to the superstructure before putting it in service is to place the supporting columns under the corners of the lift span and raise the tower floor system and the approach girder spans to their new positions and rivet them up.

It will be noted that the entire truss span is raised to bring the track to the new grade—not the floor system alone. This means that not only is the expense and delay incidental to disconnecting the floor system and riveting it up again in its position avoided, but with the raising of the entire lift span the underclearance is increased so as to permit the passage of many small boats which would otherwise require operation of the bridge.

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

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

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

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

As rigid under traffic as a fixed span.

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

General Notes.

At both ends of the lift span centering castings on the trusses engage corresponding centering castings on the tower columns. At the fixed end a small clearance is provided; at the other (expansion) end ample clearance is provided for the longitudinal expansion and contraction of the span.

The shoes which bear directly on the masonry are similar at both ends of the span. They consist of massive cellular cast steel blocks with vertical lips (two each) which project above their top surfaces and engage the vertical sides of the truss pedestals. The bearing area on masonry of each shoe is 2 ft. 11 in. $\times$ 5 ft. 4 in. $=$ 2,240 sq. in.

The pedestals bearing upon these shoes are pin connected to the trusses. At the expansion end their bearing surfaces are curved, affording ordinary rocker action for the truss span under expansion and contraction.

An expansion joint is provided in the floor system at the connection of the stringers to the floor beam near the center of the span.

Train thrust frames are provided at about one-quarter span length from each end of the truss span to carry the horizontal loads (from braking trains, etc.) directly into the trusses. The object in using these frames is to prevent lateral bending in the floor beams.

The equalizer bars are of forged steel. Forged steel was considered preferable to rolled steel for this important service inasmuch as it is more thoroughly worked in its manufacture.

Sufficient clearance was provided throughout for the construction of a second bridge of this type at the minimum distance from the present bridge of 35 ft. 6 in. center to center.

Every precaution has been taken to insure safety in the operation of the bridge. One large hood has been provided which covers completely all machinery within the machinery frame. This hood can be readily removed by the overhead hand crane when it is necessary to inspect the machinery, etc. Smaller hoods cover all couplings, revolving set screws, projecting keys, etc., outside the machinery frame. Movable steps have also been provided for passing over the main shafts, which are about three feet above the floor.

The signals indicate "clear" only when the lift span has been raised to its extreme height.

Alternating current is supplied by the Commonwealth Edison Company and is transformed and stored for use in the railroad company's plant. The wire from main feeder line from trolley to the control panel is two 500,000 cir. mil. cables per leg. The size of conduit is 1½ in. The wire from control panel to each motor is two 500,000 cir. mil cables per leg.

The computed torque at the motor shaft is approximately 8,700 ft.-lbs.

Typical Sections.

The top chord members of the lift span are of the usual type of construction consisting of two built-up channels connected with a cover plate. All web and bottom chord members consist of two built-up channels with flanges "turned inside," connected with lacing.

The maximum stress in the top chord is 2,363,000 lbs. The section carrying this stress consists of:

1—Cover plate 38x⅝ in.
2—Web plates 34x¾ in.
4—Web plates 34x½ in.
2—Web plates 24x⅝ in.
2—Web plates 32x$\frac{9}{16}$ in.
2—Top angles 4x4x½ in. (inside).

2—Top angles 4x4x$\frac{5}{8}$ in. (outside), and
2—Bottom angles 6x6x$\frac{5}{8}$ in.
Total sectional area = 239.69 sq. in.
Depth = 2 ft. 10$\frac{1}{4}$ in. b. to b. of angles.

The maximum stress in the bottom chord is 2,277,600 lbs. The section carrying this stress consists of:

2—Plates 36x$\frac{1}{2}$ in.
2—Plates 36x$\frac{7}{16}$ in.
2—Plates 24x$\frac{5}{8}$ in.
2—Plates 34x$\frac{1}{2}$ in.
2—Plates 34x$\frac{5}{8}$ in.
2—Plates 24x$\frac{1}{2}$ in.
4—Angles 6x6x$\frac{5}{8}$ in.
Total sectional area = 226.44 sq in.
Depth = 3 ft. 0$\frac{1}{4}$ in. b. to b. of angles.

The tower columns are of "H" section. The lowest section of the most heavily loaded column carries a maximum computed vertical load with span down (present construction) as follows:

Dead load	1,186,300 lbs.
Wind load	809,400 lbs.
Total direct load.......	1,995,700 lbs.

In addition to the above this column resists a bending moment due to wind of 8,640,000 in.-lbs.

The same column carries a maximum computed vertical load with span down (future construction) as follows:

Dead load	1,238,600 lbs.
Live load	232,300 lbs.
Wind load	913,000 lbs.
Total direct load.......	2,383,900 lbs.

In addition to the above this column resists a bending moment due to wind of 5,500,000 in.-lbs.

It, as well as all other lower column sections, is made up as follows:

8—Angles 6x4x$\frac{5}{8}$ in. (legs of four corner angles "turned inside").
1—Plate 32$\frac{3}{4}$x$\frac{3}{4}$ in.
2—Plate 20x$\frac{5}{8}$ in.
2—Plate 30x$\frac{3}{4}$ in.
2—Plate 30x$\frac{11}{16}$ in.
2—Plate 30x$\frac{9}{16}$ in.

The sectional area = 216.44 sq. in. The length of these sections is about 80 feet; their weight about 40 tons each.

Assumed Loading.

(a) For truss span.

Dead load = 9,740 lbs. per lineal foot of double track truss span.

Live load, p + Q = 5,500 lbs. uniform + 66,000 lbs. concentrated per track. Concentrated load for floor system connections = 99,000 lbs. per track.

Wind load: For upper laterals, 150 lbs. per lineal foot of span; for bottom laterals, 200 lbs. per lineal foot of span—static, 300 lbs. per lineal foot of span—moving.

(b) For towers.

Dead load—from truss span, etc., and own weight.

Live load—same as for truss span.

Wind load: Span down, 30 lbs. per sq. ft.; span up, 15 lbs. per sq. ft.—reduced in each case by Duchemin's formula.

Tower columns are designed to carry at the same time either—

(1) Dead load + live load + wind load, span down (all as above).

(2) Dead load + live load + wind load, span up (all as above) + impact (= 25% weight of span), or

(3) Dead load + live load (all as above) + impact (= 25% weight of span).

The above cases were considered for both present and future constructions.

Specifications.

For design and manufacture of structural steel work, Specifications for Railway Bridges, Pennsylvania Lines West of Pittsburgh, dated April, 1906, except:

(1) Allowable unit stresses increased 15% for all members carrying loads from two tracks at the same time.

(2) Allowable unit stresses in tower columns increased one-third when loaded under cases (1) and (2) of assumed loading for towers.

(3) Allowable unit stresses in tower and traction bracing: Tension, 12,000 lbs. per sq. in.; compression, 12,000 — 44 L/r lbs. per sq. in. In truss laterals, 9,000 lbs. per sq. in.

For machinery and electrical equipment special sepcifications were prepared.

The operating ropes were designed in accordance with the following clause:

"The ratio of the total stress (including bending) to the elastic limit shall not exceed 75%; the ratio of ultimate to direct stress shall not be less than 4.5 in. In explanation of what may seem an excessively high ratio (75%) of total stress to elastic limit, it may be added that the consulting engineers considered this ratio a proper one for the following reasons:

Bridge steel is ordinarily stressed to about one-half its elastic limit, impact included. Bridge steel, however, is less thoroughly worked in its manufacture and is less uniformly reliable than the materials entering into the construction of wire ropes.

Approximate Weight of Structural Steel Work and Machinery.

*Lift span	2,623,000 lbs.
Towers (including approach stringers)	3,136,000 lbs.
Counterweights	196,000 lbs.
Grillages (including grillages for one-half the end bearings of future span)	83,000 lbs.
Machinery and castings	749,000 lbs.
Ropes	154,000 lbs.
Total	6,941,000 lbs.

Substructure.

The substructure consists of eight piers supporting the tower columns and two masonry abutments, the backwalls of the latter being "squared up" to the tracks. The four northerly piers were made large enough to carry the adjoining tower columns of the future span, as well as the north columns of the present span.

These four northerly piers were rectangular in section with rounded ends. Their top dimensions were 11 ft. 6 in. wide x 28 ft 0 in. long (both dimensions measured under coping). The length of each pier was maintained constant throughout its height; its width, however, was increased, in its lower portion, to 15 ft. 0 in.

The four southerly piers, which are designed to carry the south columns of the present span only, are rectangular in section with rounded ends in their upper portions. Their lower portions are of circular section. Their top portions are 11 ft. 0 in. wide x 16 ft. 0 in. long (under coping); in the lower portion their diameter is 16 ft. 0 in.

A thoroughly braced steel shell varying from ¼ in. to ½ in. thick, encasing each pier from top to bottom, was manufactured in sections about 4 ft. 0 in. long.

The lowest section of each shell was provided with a cutting edge. Two vertical passageways were provided throughout each shell to provide access to the working chamber (in the bottom sec-

*Machinery and operating houses, walkways, decks, etc., added make weight of lift span = 3,200,000 lbs. (= total weight lifted—ropes excluded).

tion) and for transporting earth, materials, etc., to and from this working chamber.

The sinking of the shells was effected by the weight of the concrete, with which the successive sections were kept filled, and by excavating within the chambers at the feet of the shells. No compressed air was used, the working chambers being practically sealed from the entrance of water.

Simultaneously with the sinking of each shell the successive joints were stuffed with lamp wicking which was well smeared with tallow and fastened together with bolts spaced about 2 in. c. to c.

With but few exceptions the piers (including the shells) were sunk to bedrock (at elevation from 55 to 60 feet below Chicago city datum). In these exceptions the cutting edges of the shells rested some 6 to 9 ft above the rock. In such cases the material between the cutting edges of the shells and the rock was excavated and the excavations were filled with concrete, making the concrete piers continuous and monolithic to the rock.

The concrete mixture throughout was approximately 1:2½:5.

It took from a minimum of about ten days each to sink the smaller piers to a maximum of about one month each to sink the larger piers.

They sank very close to their intended positions, the maximum variation therefrom, for a larger pier, being about 8 inches.

The abutments were constructed to a depth of —2.67 (2.67 ft. below Chicago city datum) and rest on wood piles spaced from about 2 ft. 6 in. to 3 ft. 0 in. center to center. At the east end of the bridge the by-pass under the tower is dredged out to a depth of about 12 feet. To preclude any possibility of sliding of the east abutment (owing to slipping of intervening soil into the by-pass) reinforced concrete piles about 30 feet long were driven tight together, to act as sheet piling, in a straight line between the most easterly pair of piers. Supporting the upper ends of the piles a reinforced concrete beam 3 ft. 0 in. deep and 4 ft. 0 in. wide was extended between the two piers.

The two pairs of piers bounding the main channel are protected by timber fender construction which presents no unusual features.

Miscellaneous.

Messrs. Waddell & Harrington, Kansas City, Mo., were the consulting engineers for this bridge and prepared the stress sheets, general detail drawings and special specifications for the mechanical and electrical equipment under the general direction of Mr. J. C. Bland, Engineer of Bridges, Pennsylvania Lines West of Pittsburgh. It was completed and placed in service on July 30, 1914, at an approximate total cost of $750,000.

May, 1915

The bridge was fabricated and erected by the Pennsylvania Steel Company of Steelton, Pa.

THE ERECTION.

W. W. Priest.

This bridge was built to take the place of a two track swing span of considerable less length, and its location, with reference to the swing span, was such that the latter could not be swung into any position in which it would clear the new bridge with the latter in position for railroad traffic.

In deciding on the method for erecting the new bridge, two important points had to be kept in mind:

1st: That no interference with navigation would be permitted.

2nd: As it was necessary to maintain the railroad traffic on the old swing span up to the time it was transferred to the new bridge, the latter would have to be erected so as to permit the operation of the old span and also so the transfer of the railroad traffic to the new bridge could be made in the shortest possible time, bearing in mind that the old span would have to be removed before the new span could be put in position for traffic.

Two methods to take care of these conditions were considered, one of which entailed the erection of the lift span on falsework parallel with and close to the dock line at some point on the river and afterward floating it into its permanent position on barges.

The other method contemplated erecting the lift span in its permanent position on falsework high enough to clear river navigation.

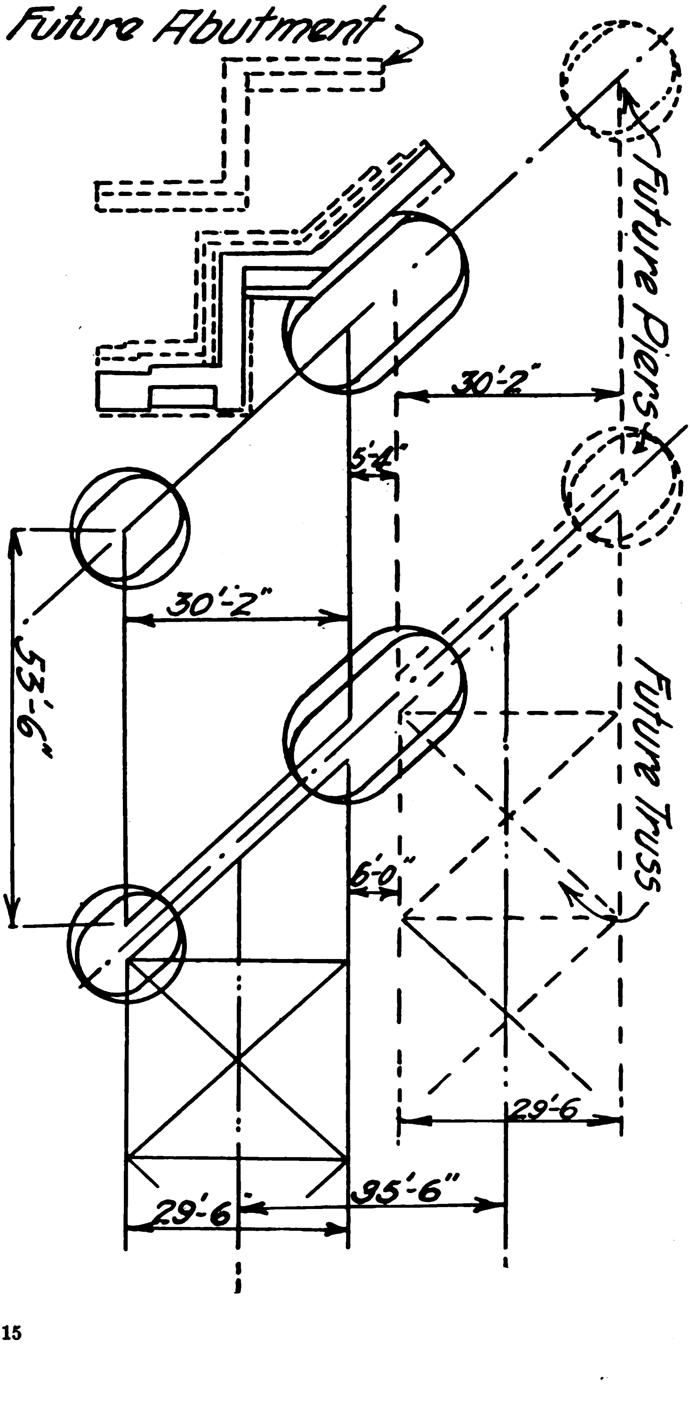
The latter was adopted because it seemed to offer less chance of delay to navigation and a better opportunity to quickly transfer the railroad traffic from the old to the new bridge.

The south end of the old span in its normal position interfered with the erection of the south tower, and clearance for the erection of this tower was obtained by extending the old abutments and changing the line of the old span so it would clear the new superstructure at south end.

The columns in towers were erected in three sections, the bottom section being 80 ft. 11¼ in. long, the middle section 46 ft. 11¼ in. long, and the top section 57 ft. 2⅝ in. long, making a total height of 185 ft. 1⅛ in. from top of masonry to top of columns. The total height of the towers from masonry to center line of sheaves is 195 feet.

On top of these columns, plate girders 75 ft. 2 in. long were placed, upon which the sheaves for counterweight cables were mounted. These girders weigh 40,000 lbs. each and the sheaves 59,760 lbs. each.

The erection of the south tower was commenced on Septem-



PIER AND ABUTMENT PLAN — NORTH END
BRIDGE No. 458 PENNA. LINES W. OF PITSBGH. Smith-Priest Paper.

ber 4, 1913, the bottom sections of columns, each of which weighs 81,000 lbs., were set in place by a derrick car. (See Fig. 1).

After setting the bottom sections of the columns the tower bracing for these sections was bolted in place and an "A" frame derrick with two booms, especially designed for completing the erection of the tower, was set up on the part of the tower erected.

The erection of the north tower was started in the same way a little later. In order to erect these towers within a reasonable time it was necessary to provide a derrick that would handle a load

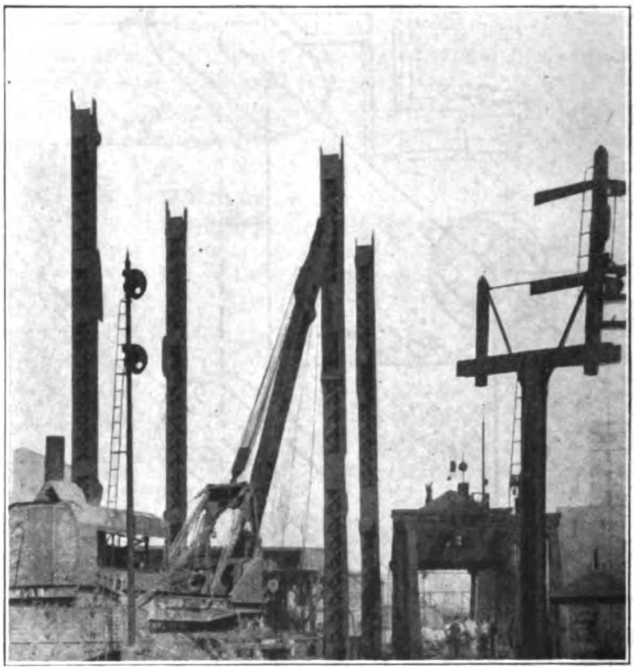
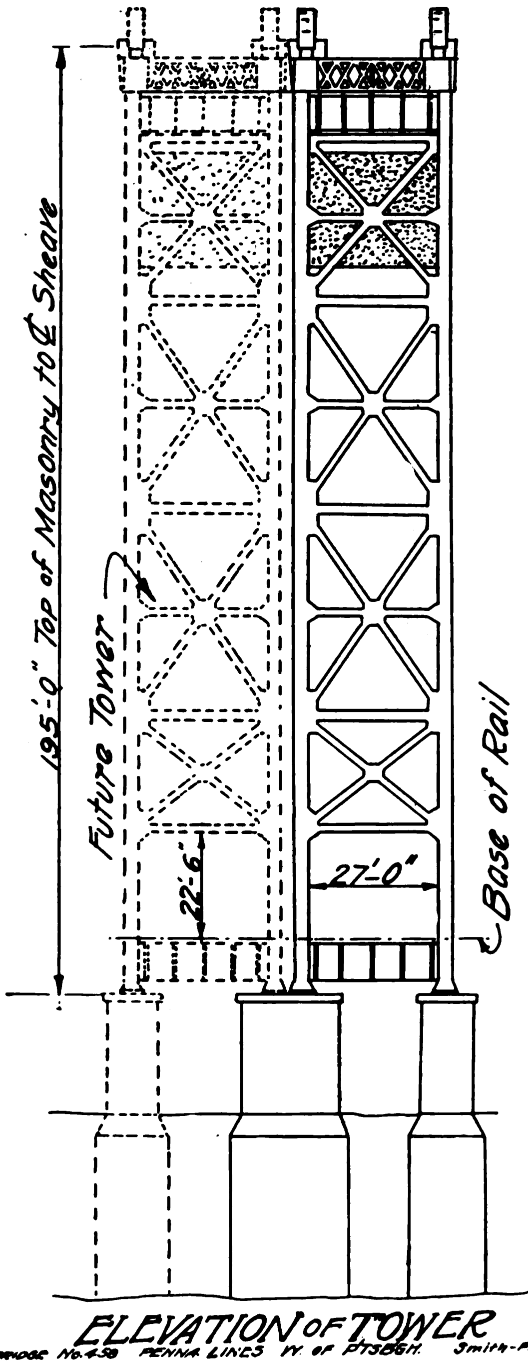


Fig. 1. Three Column Sections Erected—South Tower Facing South Derrick Car Placing Fourth Section.

of about 30 tons at a 44 ft. radius, and it had to be of such design as to permit its being shifted from point to point easily and quickly. The derrick designed for this work consisted of a timber "A" frame and two booms set up on a combination wood and steel beam which was secured to vertical timbers bolted to two sides of the tower, these timbers being extended as the erection of tower progressed. Fig. 2 shows the derrick in first position for proceeding with the erection of the tower.

Each end of the beam supporting the "A" frame was provided with links to which hoisting tackle was hitched to move the derrick up or down as might be required.



ELEVATION OF TOWER
BRIDGE No. 4.58 PENNA. LINES W. OF PTSBGH. Smith-Priest Paper.

When hoisting material with one boom the other boom was swung as near as possible in line with it and anchored to the material previously erected so as to react against the loaded boom.

This derrick was light, easily and quickly moved, and proved very satisfactory in erecting the towers. After the derrick was erected in first position it had to be raised twice during the erection of the tower. Fig. 3 shows the derrick in its final position in the tower, from which position a portion of the falsework for supporting the lift span was erected and also the end panels of the lift span. (See Figs. 3 and 4.)

In order to clear the river navigation, the falsework for lift span was built in a fan shape and could only extend from the towers to the third panel point from each end of the span, leaving a gap of about 108 ft. under which no falsework could be placed.

The falsework for the lift span consisted of three main legs under each end of each truss, arranged as are the sticks in a fan,

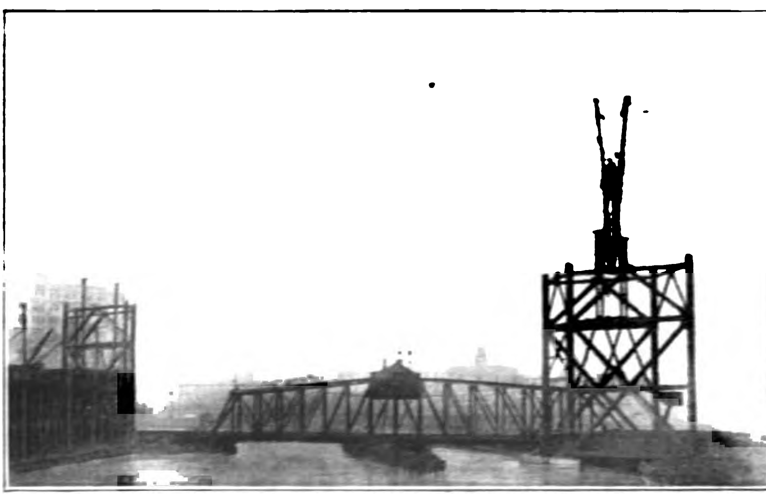


Fig. 2. "A" Frame in First Position in Tower.

the top of the inner one being 31 ft. 8 in. from the center line of inner column in towers, the center one 58 ft. 9 in. and the outer one 85 ft. 11 in. from the same point, the lower end of all the legs being set close together on a concrete foundation built on the masonry at the foot of the inner columns in the towers.

The horizontal thrust developed in the falsework legs by the load they had to carry was taken care of by securing their upper ends to the towers by means of eye bars and plates, and of course they were all thoroughly braced at properly spaced intermediate points.

The inner and center legs of the fan consisted of four 10 in. by

12 in. timbers bolted together, and the outer one of four 10 in. by 12 in. and six 12 in. by 12 in. timbers bolted together, making a sectional area of 1,344 sq. in. in this leg.

The outer leg was 146 ft., 10⅜ in. long on its center line and was built to sustain a load of 360 tons applied at its upper end.

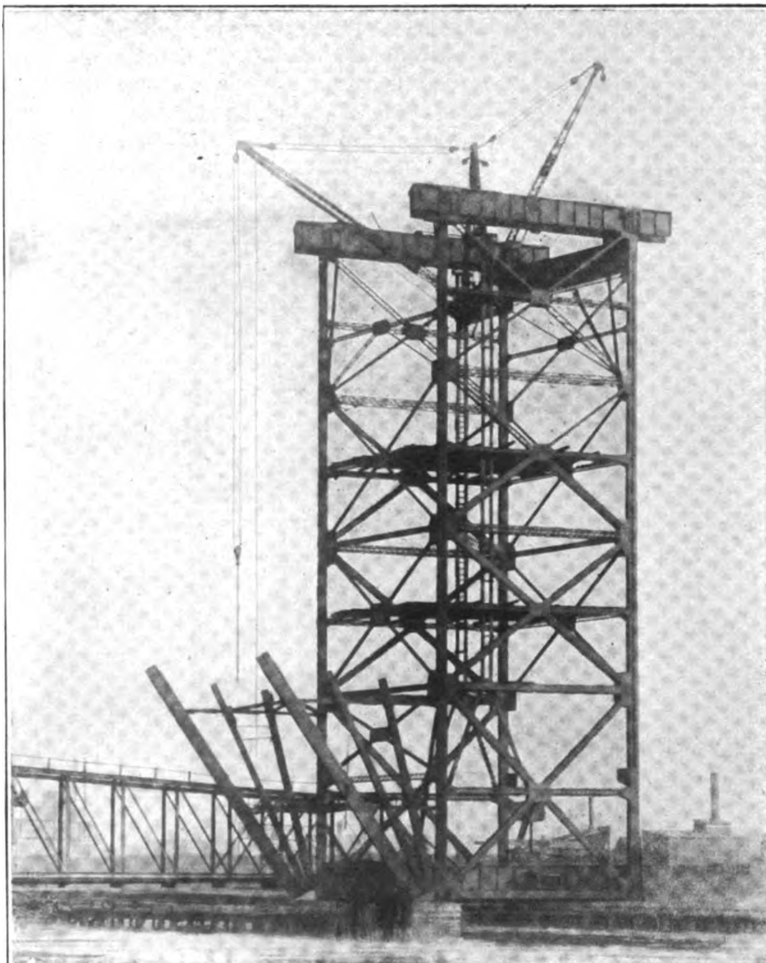


Fig. 3. Showing "A" Frame in Final Position in Tower.

The method of erecting the lift span was as follows:

With the "A" derricks in their final position for erecting the towers, the inner and center legs of the fan falsework were completely erected and properly braced, and the outer leg partially erected. The end panels of one of the trusses was then assembled on the falsework, together with the floor system and the end posts, end sections of bottom chord and end diagonals of the other trusses.

When this was done the "A" derricks were transferred from

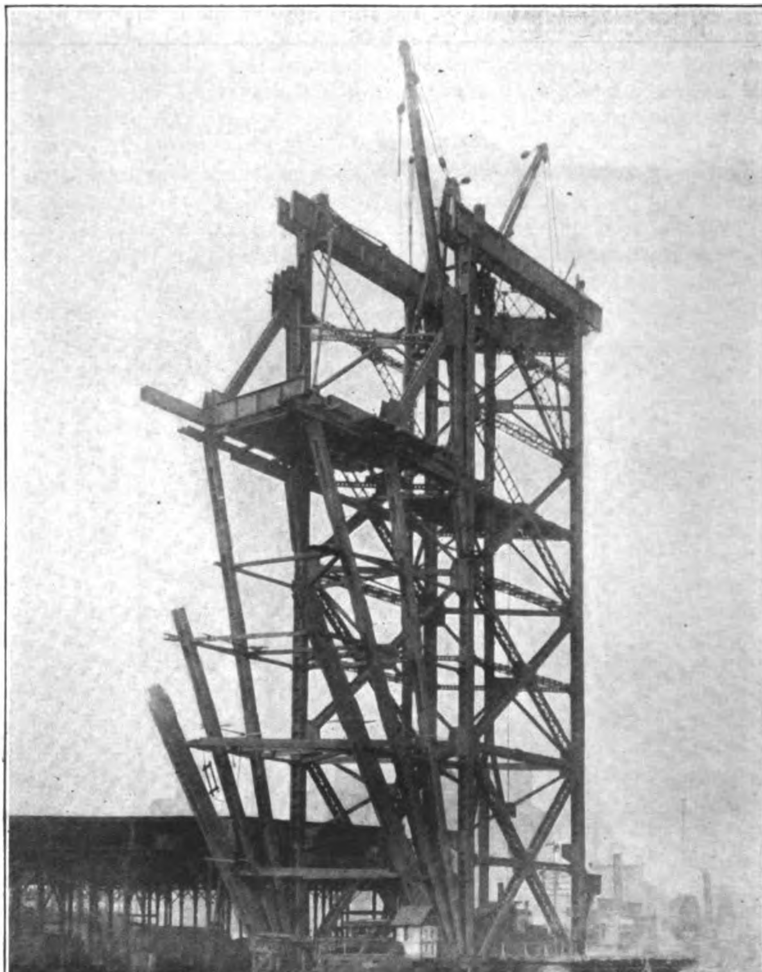


Fig. 4. Showing End Panels of Lift Span and Part of Fan False Work Erected.

their position in the towers to a position over the first intermediate post of one truss and the end post of the other truss as shown in Fig. 5.

The fan falsework was then completed and the second panels of one truss and the first panels of the other truss, together with the floor system and lateral bracing, was completely assembled on the falsework, after which the "A" derricks were moved forward one panel and another panel was completely assembled.

The "A" derricks were again moved forward one panel which brought them to their final position for the erection of the span, also to the extreme point of the falsework's support and to the gap under which no falsework could be placed. (See Fig. 6.)

This gap of 108 ft. between the ends of falsework support was cut down to 73½ ft. by the projecting ends of the bottom chord sections already in place, and was closed by connecting the bottom chords with a center section 73 ft. 5$\frac{1}{16}$ in. long which was

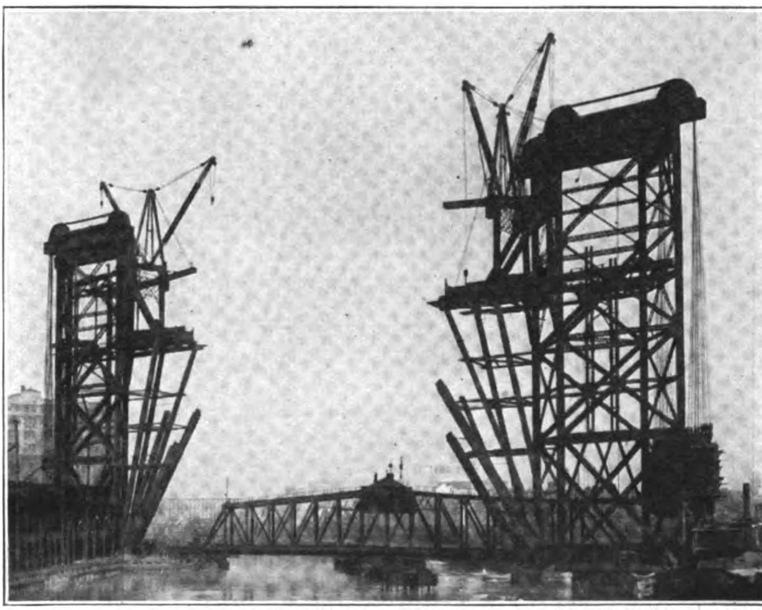


Fig. 5. Showing "A" Frame in First Position on Lift Span.

swung into position by the two "A" derricks as shown in Fig. 7. This center section of bottom chord weighs about 36 tons.

The erection of the trusses was then completed and the remaining four panels of floor system, lateral and sway system put in place. The span was erected at a clear height of 130 ft. above low water. Fig. 8 shows the trusses completely assembled, except the top chord and diagonal in one panel.

During the erection of the span the counterweight frames were erected and the concrete forming the counterweights poured. These counterweights for each end, when completed, weighed about 800 tons, and were supported by steel brackets attached to foot of columns until the lift span was ready for trial operation. As soon as the erection of the lift span was completed the machinery house was erected, the machinery installed, and the floor deck and tracks put in place. The falsework under the lift span was not removed until the span was loaded with practically all the weight it would carry when completed ready for traffic.

The bridge was completed ready for operation, except some little work on the floor deck, on July 13, 1914, and the span was operated from its maximum height to a point down as far as it

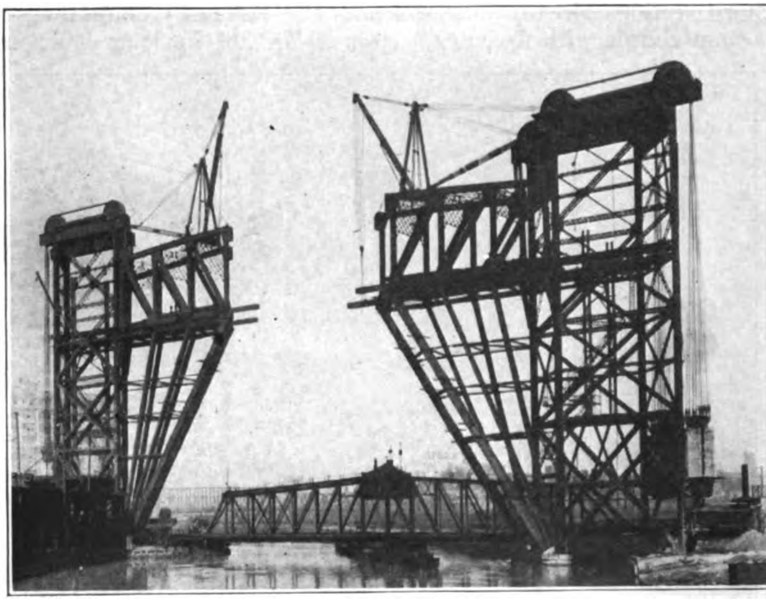


Fig. 6. Showing "A" Frame in Final Position on Lift Span.

weight was provided by filling some of the pockets left in the counterweights for this purpose, and the structure was then ready for traffic as soon as the old swing span could be moved out of the way so that the lift span could be lowered onto its bearings.

There were three different schemes considered for removing the old swing span, as follows:

1st. To swing it around parallel to the river channel over barges partly loaded with water, and then lift it clear of the pivot could be lowered without fouling the old swing span. This operation was for the purpose of determining if the span was properly balanced by the counterweights, and it showed that each of them required an additional weight of about 25 tons. This additional pier by pumping the water from the barges, after which it could

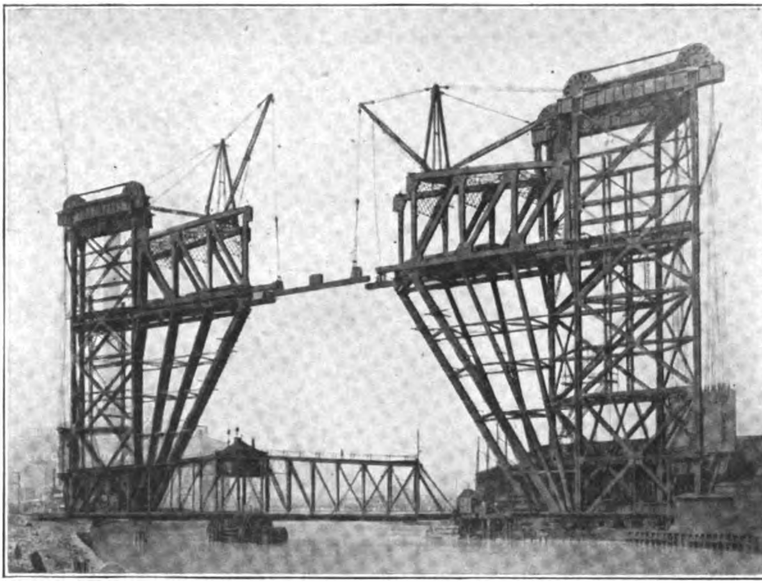


Fig. 7. Center Section of Bottom Chord of Lift Span Being Swung into Position.

Fig. 8. Showing Trusses of Lift Span Completely Assembled, Except Chord and Diagonals in One Panel.

be towed to any point selected and dismantled. This scheme was the best one considered, as it eliminated any interference with navigation and promised the least delay between the time of abandoning traffic on the old span and its resumption on the new, but it was abandoned on account of its cost, the lowest tender for furnishing tugs, barges, etc., for carrying it out being about $8,000.

2nd: A scheme to pivot the span on the south end of west truss, float it around on barges parallel to the south dock line and land it on falsework built close to the south side of the river, where it could be dismantled and loaded on cars, was also abandoned on account of its cost.

3rd: To float two scows partially loaded with water under the

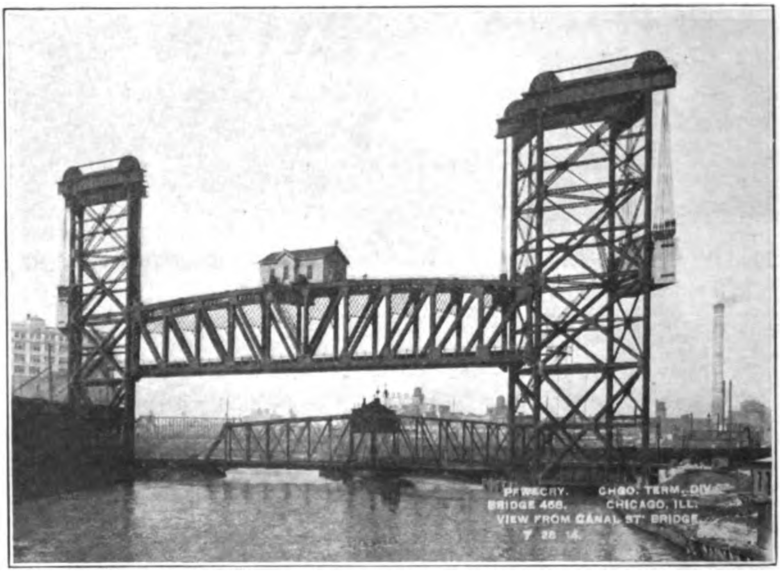


Fig. 9. Showing New and Old Bridges the Day Before Old Bridge Was Removed.

north end of the span, block up on the scows under each panel point of this end of the span, then cut the span in two at a point near the north side of the pivot pier with acetylene flame and float the north end out of the way. When this was done the end of the span resting on the pivot pier was to be jacked east about 4 feet so as to clear the new span and left in this position until dismantled.

The Erection Department of the Pennsylvania Steel Company estimated the time necessary to do the work as outlined under scheme 3 at five hours. This scheme was adopted and preparations were made to carry it out on July 29, 1914. Fig. 10 shows north

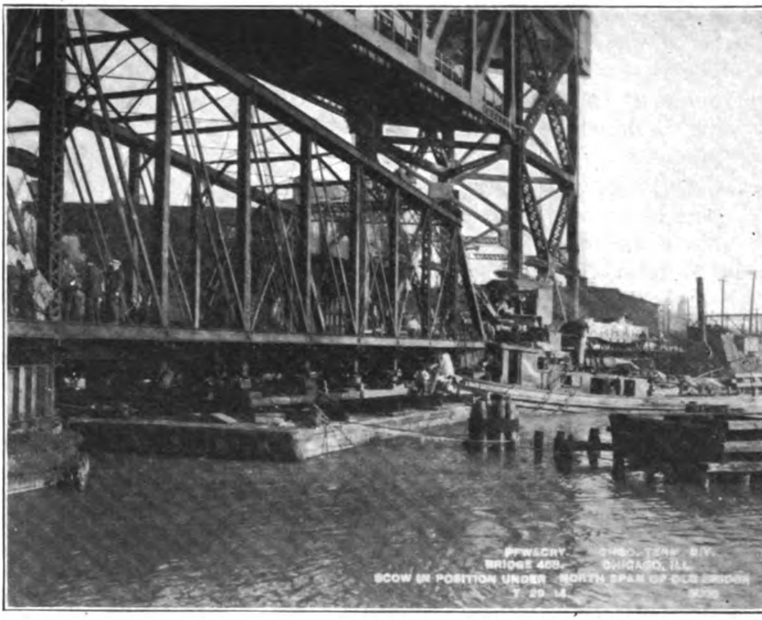


Fig. 10. Scows in Position Under North Span of Old Bridge.

Fig. 11. South Span of Old Bridge, North Span Removed.