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THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS*

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With Discussion by Messrs. Carl Gayler and S. W. Bowen.

Synopsis.

The object of this paper is to bring out a discussion of the type of reinforced concrete arch in which the reinforcement is composed of structural shapes and is designed to support the weight of the forms of plastic concrete in the arch ring on ribs.

The paper describes the design and construction of four reinforced concrete viaducts, in two of which arch spans of the above mentioned type are used. These spans range in length from 125 to 200 ft., in the clear. Three-hinged, ribbed arches are used, having hemispherical, ball and socket, cast-steel hinges. No falsework was used except such comparatively light timbering as was needed to erect the structural steel reinforcement of two of the spans.

It is concluded that, for high structures, and for those over streams subject to sudden and great variation of water level, this method is cheaper and safer than to use falsework supported from the bed of the stream.

* Presented at the meeting of October 21st, 1914.

The tables give unit and total costs, as well as the cost per linear foot, per square foot of horizontal and vertical projection, and per cubic foot of volume, for each viaduct.

Introduction.

The City of Fort Worth, in Tarrant County, Tex., is traversed by the Trinity River and its branches, the Clear and West Forks. As will be seen by the map, Fig. 1, this stream is very crooked, and cuts the town up badly, necessitating a number of bridges. The city proper lies on the high ground, around which the stream flows. The growth of the residence districts toward the east and west, and the great increase in traffic between the city proper and North Fort Worth, in which the packing houses are located, has recently made necessary the construction of a new viaduct and the reconstruction of three old ones.

The principal feature of this paper is the description of the design and construction of the three-hinged, ribbed-arch spans. In the ribs and rib braces these spans have structural steel reinforcement which is designed to support the weight of the forms and plastic concrete of the ribs and braces during construction. By this means falsework was dispensed with, except such light timbering as was used in erecting some of the structural reinforcement.

These viaducts were built with the proceeds of the sale of bonds issued by the county. The total amount of funds available for construction purposes was about $656 000, exclusive of the accrued interest on the funds in bank.

The firm of Brenneke and Fay, Consulting Engineers, of St. Louis, was selected by the Commissioners' Court to prepare plans and specifications for, and to supervise the construction of, the four viaducts.

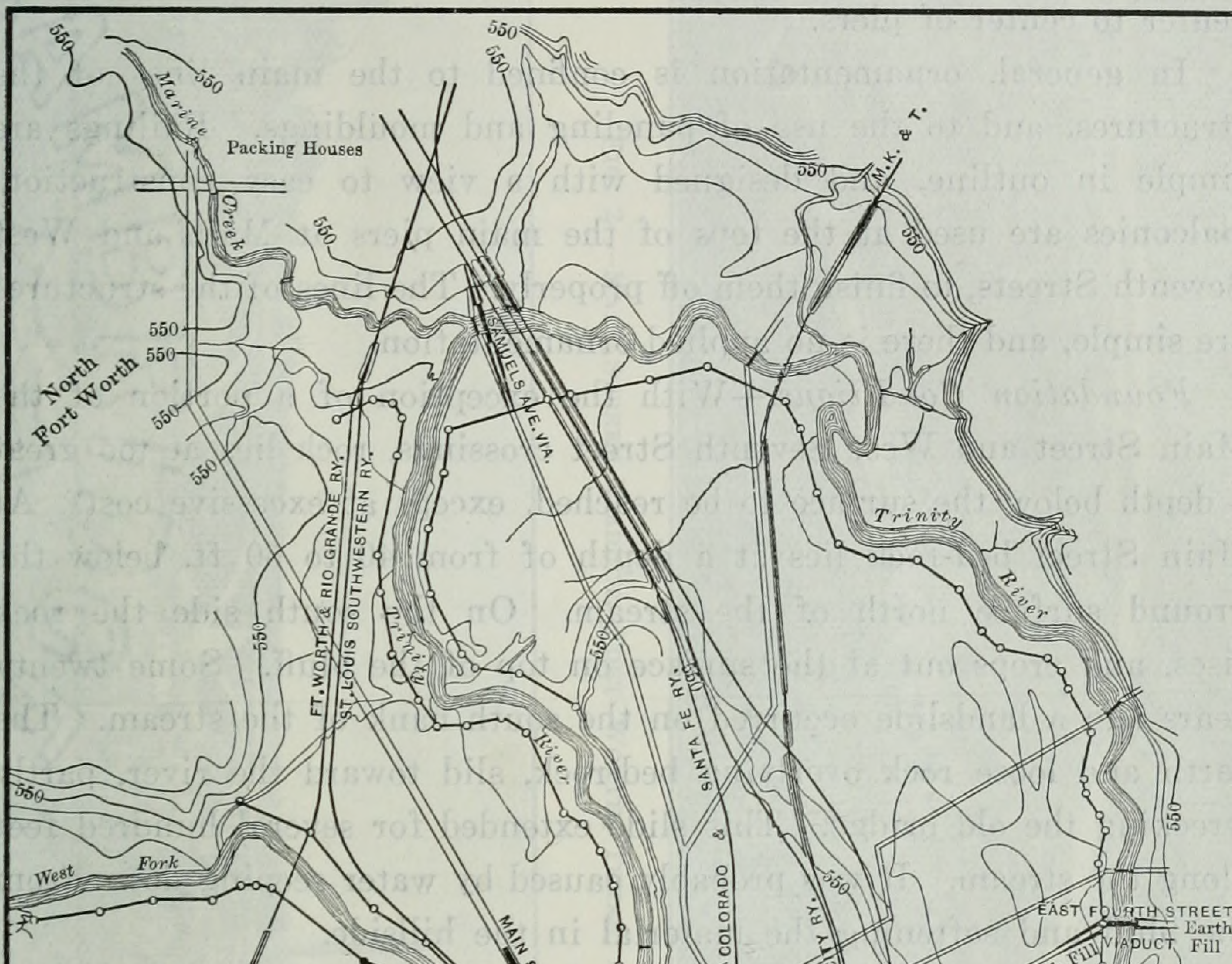
The bridges which the viaducts were to replace were old, and too light and narrow to take care of the increasing traffic. The general feeling among the residents of the county was that the new structures should be planned for the future, and be built in such a manner, and of such materials, that they would require the minimum amount of maintenance, and last indefinitely. Therefore, reinforced concrete was selected as the material best suited to the conditions, and was used for all parts of the structures. Creosoted wood block paving was recommended by the engineers as being light, smooth, and durable, but owing to the high cost of such paving at Fort Worth, vitrified brick was finally selected.

Design.

General Description.—The structures are known, respectively, as the Main Street, West Seventh Street, Samuels Avenue, and East Fourth Street Viaducts. They are mentioned in the order of their cost, the Main Street Viaduct being the largest of the four. The general map, Fig. 1, shows the location of the crossings, and Plate XVIII shows the four structures drawn to the same scale, so that their relative size can readily be seen.

The widths of roadways and sidewalks are as follows: Main Street, 54-ft. roadway and two 8-ft. sidewalks; West Seventh Street, 43-ft. roadway and two 6-ft. sidewalks; Samuels Avenue and East Fourth Street, 30-ft. roadway and two 5-ft. sidewalks. In each case provision is made for two electric interurban car tracks in the roadway. These tracks are placed in the center of the roadway for the Main and West Seventh Street Viaducts, and at the sides for the two smaller structures. The Main Street Viaduct is the most important of the four, and is on the main artery of travel between the city proper and North Fort Worth. Therefore, it is made wider than the others. The width of roadway is such that four wagons and two street cars can pass abreast. The width of the roadway of the West Seventh Street Viaduct is the same as the width, between curbs, of the street leading up to the crossing. This is ample to accommodate two wagons and two street cars abreast. The two smaller viaducts, being short, and of less importance, as to the volume of traffic to be carried, than the first two named, have roadways of sufficient width to allow only one wagon and two street cars to pass abreast.

From out to out of concrete construction, the length of the Main Street Viaduct is about 1 752 ft., made up of one 225-ft. arch span over the stream, two 175-ft. and one 150-ft. arch spans, one 68 ft. 9-in., two 62 ft. 6-in., seven 50-ft., and two 25-ft. girder spans. The remainder is made up of earth fills enclosed by retaining walls of the semi-gravity type. The West Seventh Street Viaduct is about 1 041 ft. long, consisting of one 137 ft. 6-in. arch span over the stream, and seven 50-ft. girder spans; the remainder consists of retaining



TOPOGRAPHICAL MAP
OF PART OF
FORT WORTH
IN THE VICINITY OF TRINITY
RIVER AND BRANCHES
SHOWING LOCATION OF LEVEE
AND VIADUCTS
SCALE OF FEET
0 500 1000 2000
N
Packing Houses
Marine Creek
North Fort Worth
550
FT. WORTH & RIO GRANDE RY.
ST. LOUIS SOUTHWESTERN RY.
SAMUELS AVE. VIA.
M.K. & T.
Trinity River
SANTA FE RY.
COLORADO & GULF
FORT WORTH & DENVER CITY RY.
MAIN STREET VIADUCT
EAST FOURTH STREET VIADUCT
Earth Fill
C. R.I. & G. RY.
Fort Worth Proper
WEST-SEVENTH STREET VIADUCT
Waterworks
West Fork
Clear Fork

FIG. 1

walls. The other two structures each have nine 50-ft. girder spans, and are 450 ft. long, exclusive of the approaches, which consist of earth fills without retaining walls. All the spans are measured from center to center of piers.

In general, ornamentation is confined to the main lines of the structures, and to the use of paneling and mouldings. Railings are simple in outline, and designed with a view to easy construction. Balconies are used at the tops of the main piers at Main and West Seventh Streets, to finish them off properly. The lines of the structures are simple, and there is no applied ornamentation.

Foundation Conditions.—With the exception of a portion of the Main Street and West Seventh Street crossings, rock lies at too great a depth below the surface to be reached, except at excessive cost. At Main Street bed-rock lies at a depth of from 40 to 50 ft. below the ground surface north of the stream. On the south side the rock rises, and crops out at the surface on top of the bluff. Some twenty years ago a landslide occurred on the south bank of the stream. The earth and loose rock overlying bed-rock, slid toward the river, partly wrecking the old bridge. This slide extended for several hundred feet along the stream. It was probably caused by water seeping down from the bluff and softening the material in the hillside.

The material on the south side of the river consists of loam and clay interspersed with loose rock and boulders brought down by the slide. This material is underlaid with ledges of rock, varying in thickness from 6 in. up, with thin layers of hard clay or shale between. The layers are horizontal, and increase in thickness with the depth. The material on the north bank, to a depth of 10 or 12 ft., consists of a black, sticky, clay-like substance, which has the property of remaining very hard when moist and not exposed to the air, but which disintegrates when exposed to dry air for a few days. Under this surface material there is a very dense yellow clay, underlaid with sand. This sand increases in coarseness with the depth, and merges into gravel near bed-rock. The surface material was tested at a depth of 8 ft., and sustained a load of 3.3 tons per sq. ft. without appreciable settlement. The material north of the stream at Main Street is fairly typical of that at the other crossings.

River Conditions.—The Trinity River, though normally a small stream, hardly larger than a creek, is subject to sudden freshets, and

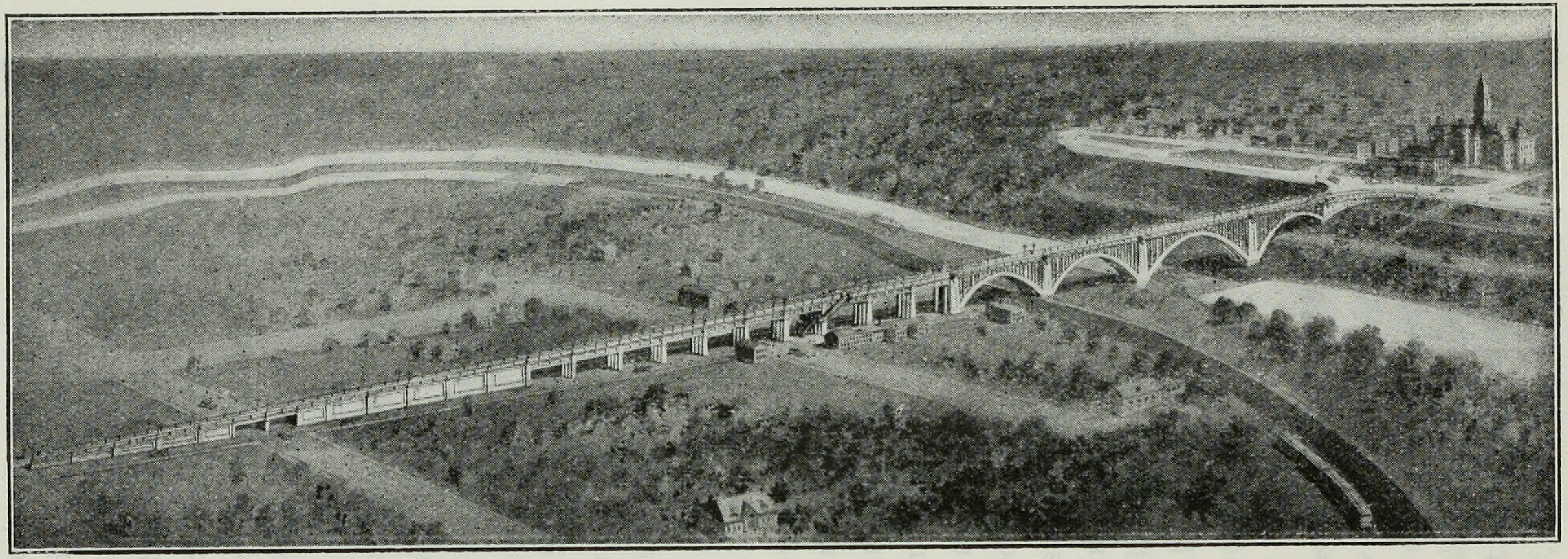
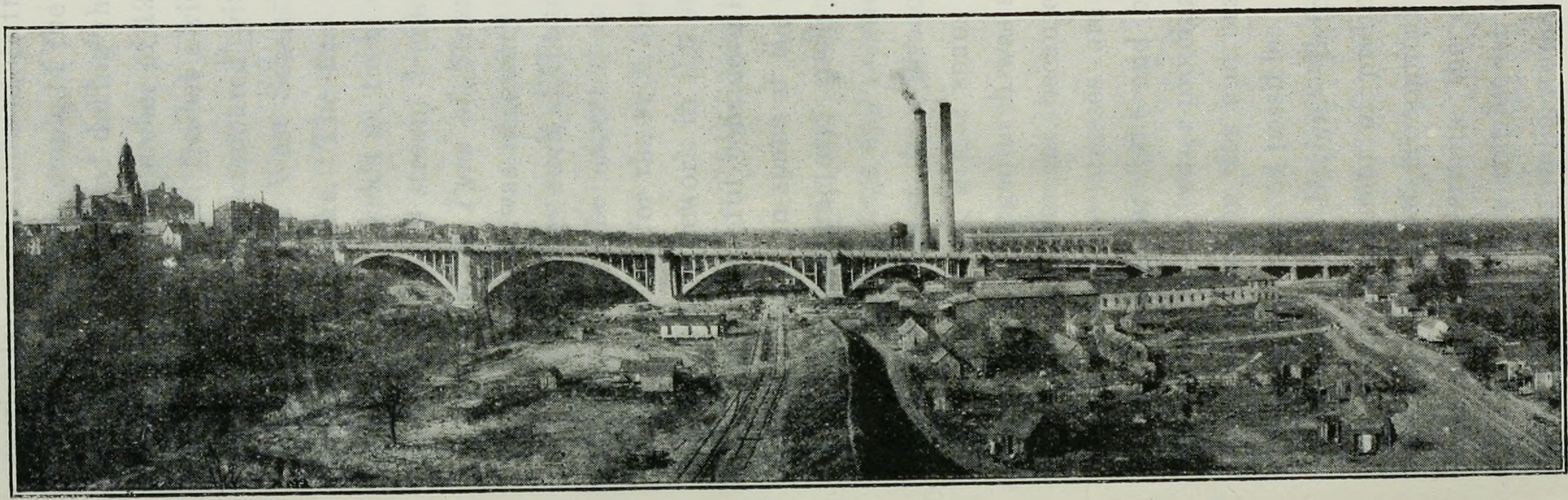


FIG. 2.—MAIN STREET VIADUCT, FORT WORTH, TEXAS, FROM AN ARTIST'S PERSPECTIVE.



carries large quantities of drift. The banks are steep, and the run-off is rapid. As an example of the rapidity of the rises, may be mentioned a flood that occurred during the summer of 1912, before work started, and another in September of 1913, while work was under way. During the first mentioned freshet a rise of 23 ft. occurred in 24 hours. This rise was gradual compared with the last mentioned flood, where the rise, measured at West Seventh Street, amounted to a total of 16 ft. in about 1½ hours. The first 14 ft. of this rise took place in about 1 hour. Such floods as these are not at all unusual, and make work in the bed of the stream decidedly hazardous.

Selection of Type of Structure.—Because of the conditions mentioned above, it was necessary to adopt a type of structure that would not be injured by such slight unequal settlement as, under the circumstances, might be expected to occur. It was also thought advisable to use, at least for the arch spans, a method of construction that would not require falsework in the stream.

After a careful consideration of various types, it was decided to use, for the main spans of the two larger viaducts, three-hinged, ribbed arches, with structural steel reinforcement designed to support the weight of the forms and plastic concrete of the ribs and braces during construction. For the approach spans and for the river spans of the smaller viaducts, girder spans were adopted.

The three-hinged arch was selected because it would not be strained by unequal settlement, because the stresses are statically determinate, and temperature stresses are eliminated. Ribbed construction was adopted as being light and best adapted to the use of hinges, and also because no water-proofing would be required. Structural reinforcement for the ribs and braces was used in order to dispense with falsework, as far as possible.

Preliminary Design.—Before beginning the actual designing, a considerable amount of preliminary work was done in order to determine the most economical type of structure for various heights, and also to determine the economical span lengths for each type. Three types were considered, as follows: earth fills enclosed by semi-gravity retaining walls, girder spans, and arch spans.

In this preliminary work sufficient designing and laying out was done to enable the various quantities to be determined with sufficient accuracy for the purpose in view. These quantities were multiplied by estimated unit prices, and the cost per linear foot of structure, of each type, for various heights and span lengths, was obtained. These results were then plotted, and the economical span lengths were determined, as well as the points at which it was theoretically economical to change from one type of construction to another. The information obtained from the plots was followed as closely as circumstances would permit. In some cases, however, the physical conditions encountered determined both the span lengths and the type of structure. For example, at Main Street, arch spans were used over the levee, and between the south abutment and Pier No. 1. Girder spans would have been cheaper at these points, if the height only were considered. It was not permissible to place piers in the levee, and a long span had to be used. It was not thought advisable to divide the span between the South Abutment and Pier No. 1, on account of the possibility of a recurrence of the slides mentioned previously; in other words, the fewer piers on the south bank of the stream at this point, the better. Consequently, arch spans were used for these portions of the crossing.

Girder spans would have been cheaper than the arch at West Seventh Street, but, as this structure is near one of the city parks, and in a high-class residence district, an arch was used over the stream, for esthetic reasons.

Girder spans were used throughout for the Samuels Avenue and East Fourth Street crossings, for the sake of economy, and to afford the maximum clear opening with minimum height. This required the use of falsework in the stream. This was undesirable, but it was considered that as in this case the girder spans were considerably cheaper than self-supporting arches, and that as these crossings were situated so that the force of the current at flood stage was much less than at the other crossings, girder spans would answer the purpose.

The selection of the three-hinged ribbed arch with forms supported from the reinforcement, was governed principally by the character of the foundations, and by the erratic behavior of the stream. However, at Main Street, estimates were made of the cost of this construction as compared with that of a three-hinged ribbed arch supported on falsework in the usual manner. In making this comparison, the cost of the falsework and the rib reinforcement required for this scheme was compared with that of the rib reinforcement of the arches

PLATE XVIII.
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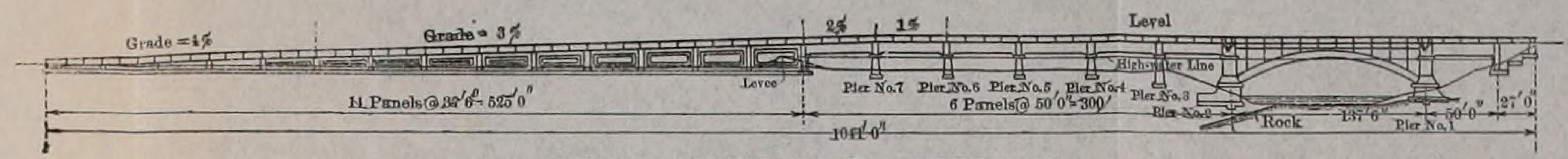
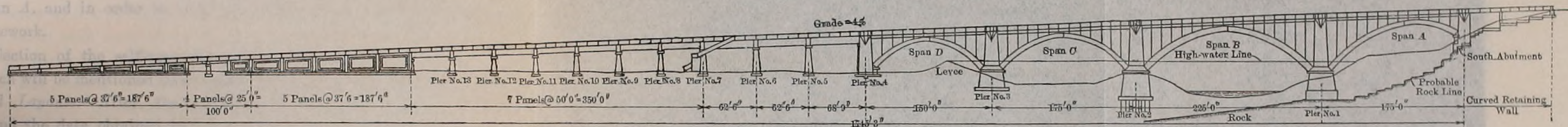
GENERAL ELEVATIONS AND SECTIONS FORT WORTH VIADUCTS FORT WORTH, TEXAS

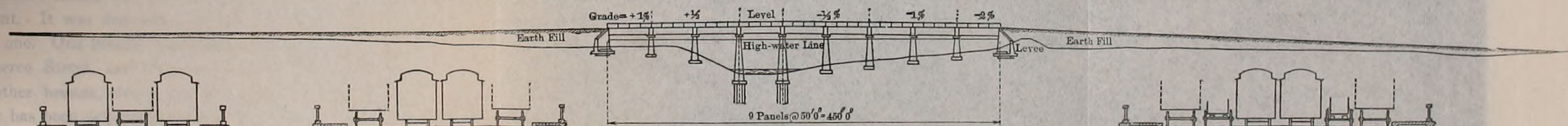


ELEVATION OF WEST SEVENTH STREET VIADUCT.

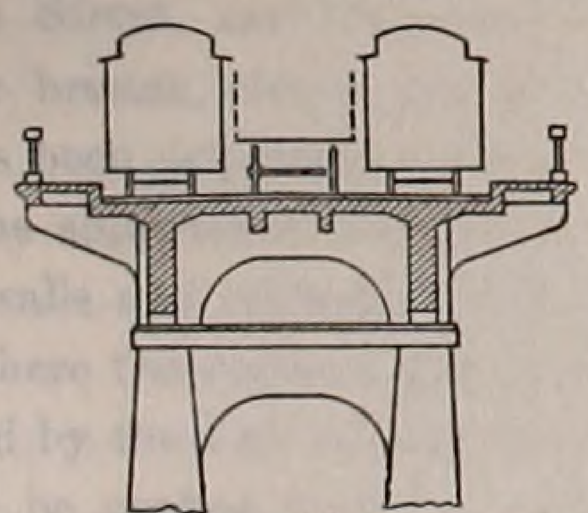


ELEVATION OF MAIN STREET VIADUCT.

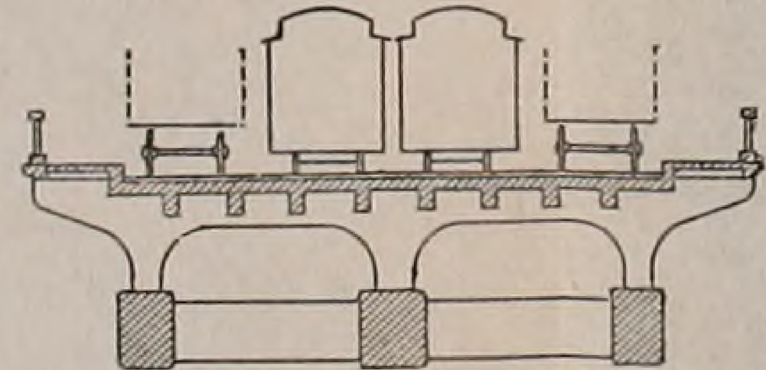
ELEVATION OF SAMUELS AVENUE VIADUCT.



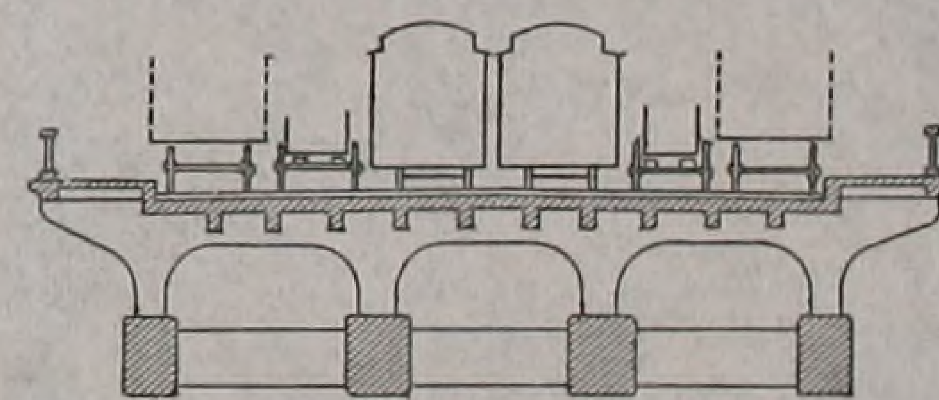
ELEVATION OF EAST FOURTH STREET VIADUCT.



CROSS-SECTION THROUGH GIRDERS
SAMUELS AVE. AND EAST FOURTH ST. VIADUCTS.



CROSS-SECTION THROUGH ARCH
WEST SEVENTH STREET VIADUCT.



CROSS-SECTION THROUGH ARCH
MAIN STREET VIADUCT.

as built. The hinges, and such portions of the rib forms as were common to both schemes, were not included in the estimate.

It was found that the structure on falsework would have cost slightly less than as built. This was caused by the fact that Spans *A, C,* and *D* are too low for economy with the self-supporting type of construction. Span *B,* however, is high enough to make this type economical. It was decided to use the self-supporting type for all spans, because of the danger from floods, the unstable character of the soil under Span *A,* and in order to avoid the troubles due to settlement of the falsework.

The selection of the self-supporting type was amply justified by events which will be mentioned later, under the head of "Construction".

General Layouts.—After the preliminary designing had been finished, and the data obtained had been arranged for use, the work of preparing the general plans was started. The Main Street structure is typical of the work, and will be used as an example throughout the remainder of this paper.

The general plan of this crossing, Plate XIX, shows its location with reference to the principal streets and public buildings. It will be seen that a direct approach to the south end of the structure was prevented by the Court House and Jail buildings, which block Main Street at this point. It was desirable, therefore, to make the south approach a double one. One branch, carrying north-bound traffic, connects with Commerce Street, and the south-bound traffic is accommodated by the other branch, which connects with Houston Street. Sufficient property has been acquired to allow the street corners to be rounded off, so that the approaches may follow easy curves.

Curved retaining walls and railings are being constructed south of the south abutment, where the viaduct proper ends. It is expected that the block now occupied by the Jail and other buildings, will be cleared by the City, and will be parked to form a pleasing approach to the crossing.

The general elevation was laid out to fit the conditions, keeping in mind the data obtained from the preliminary designs. It will be seen by referring to this general elevation, Plate XX, that a symmetrical structure was not possible, owing to the contour of the ground.

Structural Features.—Before going into the matters of loading, unit stresses, and methods of design, it may be well to give a description of some of the features of the Main Street Viaduct, in order to show the character of the construction, and to make clearer some of the problems involved in the design.

As shown by the plans, the deck consists of slabs carried on longitudinal stringers, which connect to floor-beams. These floor-beams are in turn carried by the four main longitudinal girders of the girder spans, or by the spandrel posts, which rest on the four ribs of the arch spans. The sidewalks are carried on cantilever extensions of the floor-beams outside of the outer girders or ribs. It will be noticed that, for the girder spans, such stringers as would come close to the girders are omitted, and their place is supplied by the cantilevered top flanges of the main girders. Those cantilevered top flanges also provide the necessary compressive area for the girders.

Two ribs or main girders would have been more desirable in some respects than four, as this arrangement would have done away with the continuous girder action of the floor-beams, and made the reactions on the girders and ribs statically determinate. The heavy concentrated loading and the width of the roadway, however, would have made the floor system extremely heavy. Therefore the four-rib arrangement was adopted for this crossing. At West Seventh Street, three ribs were used, and two lines of main girders for the other two structures.

The same length of floor system panels was adopted for all the structures. This made the design of the stringers practically uniform throughout. The length used was 12 ft. 6 in. from center to center of floor-beams, which gave stringers of reasonable size and brought the loads on the girders and arch ribs at reasonably short intervals. The floor-beams are of uniform thickness throughout, which thickness, 17 in., is the same as that of the spandrel posts. The posts are purposely made thinnest in the direction of the axis of the structure, so that the bending caused by the changes in the length of the deck due to temperature, will not over-stress them.

Expansion joints are provided in the girder spans at each pier, and in the deck of the arch spans, at the crown, and at each pier. The main girders slide on top of the small piers, and the stringers slide in recesses or pockets in the floor-beams. All expansion joints are packed with tar-paper and asphalt to form a closed but compressible joint, and all main girder joints are masked by pilasters. Planed steel, bed-

PLATE XIX.
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and bearing-plates are used under the expansion end of each girder and stringer. These plates have anchorage angles riveted to them, and are further anchored with bent anchor-bolts passing through holes in these angles. The sliding members, and those on which the sliding members rest, are also reinforced at these bearing points with loops of steel, to provide for the shearing stresses which result from the friction of the plates on each other. In order to reduce this friction as much as practicable, the bed- and bearing-plates are planed in the direction of movement, and the sliding surfaces in contact are heavily coated with tallow. U-shaped copper plates, filled with asphalt, are used in the roadway slabs to close the opening. Similar plates placed in the reversed position to those in the roadway slabs, and with the asphalt filling omitted, are used in the sidewalk slabs and curbs. These copper plates are used at each expansion joint in the structure.

The arch ribs are braced together at the foot of each spandrel post by transverse braces, which are of the same width as the spandrel posts and floor-beams, and as deep as the ribs will allow. These braces are placed in vertical planes.

The arch ribs are of uniform thickness, but vary in depth, decreasing regularly from the haunch toward the crown. This is contrary to the theoretical shape of a three-hinged rib, which should be deepest at about the quarter point of the span, and diminish in depth toward the haunch and crown. This departure from the theoretical shape was made for the sake of appearance only. As the hinges are concealed in the finished rib, it was considered that the ribs would look best if made to follow the lines of a hingeless rib.

Each hinge consists of two steel castings, having ball and socket joints. At the haunches one of these castings is built into the pier, and the other is bolted to the structural rib reinforcement, as are also the castings forming the crown hinge.

All the piers for the arch spans are of considerable width in order to resist overturning forces and to give the appearance of massiveness required by the long spans. They are hollowed out as much as is consistent with strength and economy of forms, in order to reduce their weight and to save material.

The small piers supporting the girder spans consist of four square shafts, battered on the four faces. These shafts rest on a common base and are connected at the top by a cap and diaphragm of T-section.

The retaining walls are of the semi-gravity type. The toe of the section is reinforced for bending stresses, and the reinforcement extends high enough up the back of the wall to relieve the concrete of undue tensile stresses. Expansion joints are provided at intervals of 37 ft. 6 in., and pilasters are used at these points to mask the joints. The walls have a small coping and are paneled to relieve the appearance of flatness.

The South Abutment is founded on rock, as is also Pier No. 1, which is on the south bank of the stream. Pier No. 2, on the north bank, is supported on a spread foundation, with a portion of the load carried on timber piles. All other piers and walls are carried on spread footings, resting on the strata previously referred to, at depths ranging from 4 to 18 ft. below the finished ground surface.

Railings, and the stairways leading down to the street level at North Second Street, are of reinforced concrete, cast in place, except the railing panels and hand-rails, which were cast in moulds and erected.

The stairways, with the exception of the steps leading to the lower landing, are supported entirely from the piers and girders by structural steel cantilever beams encased in concrete. This method was adopted, instead of supporting partly on the piers and girders and partly on independent posts and foundations, in order to avoid the danger of damage due to unequal settlement. The lower steps were built on independent foundations, with a sliding joint between the steps and the remainder of the stairway, to take care of unequal settlement.

Provision is made for carrying electric wires and small pipes under the sidewalks, with ducts leading into the railing posts and up to the lighting fixtures.

In order to reduce the dead weight of the deck as much as possible, the usual ballasted track construction was dispensed with. Anchor-bolts were set into the slab over the stringers, and the rails were secured to these bolts with cast-steel clips. Steel bearing-plates were provided under the rails at each pair of bolts, which occurred at intervals of 27 in. The rails were lined and brought to proper level with eccentric washers and shim plates. A fiber pad, ½ in. thick, was placed between the rail and the bearing-plate, as a shock absorber. This method of securing the rails is indicated on the plans.

The reinforcement of the arch ribs and braces consists of medium-steel structural shapes, supplemented by reinforcing bars. Structural

PLATE XX.
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shapes are also used for the main supports of the stairways. All other reinforcement consists of square twisted bars, of high elastic limit. These were substituted for the round, mechanical-bond bars called for on the plans, and were made equivalent in cross-section to the round bars which they replaced. All bars, except some of the large sizes, were rolled from the heads of rails, and were hot-twisted. Cold-twisted bars, from medium-steel billet stock, were used for the large sizes, on account of the difficulty of getting rail heads of sufficient volume for this purpose. Tests of the two classes of material showed that they were practically equal in quality.

As shown on the plans, the rib reinforcement consists of curved lattice girders, to the ends of which are bolted the cast-steel hinges. The chords of these girders are made up of four angles arranged in the shape of a cross. All web members are made up of two angles. Three of these steel girders are used in each inner, and two in each outer, rib of each span. The individual steel girders are laced together in the plane of the top and bottom chords, and are also connected by transverse frames, in vertical planes, under each post. The transverse braces connecting the four ribs are reinforced by lattice girders which connect to the rib reinforcement. No reaming or painting is called for. The hinge castings are bolted to the steel ribs, and all other connections, both shop and field, are riveted.

At each upper panel point of the steel ribs, clip angles were provided, to which were secured the transverse timber beams, from which the rib forms were supported. The method of supporting these forms is indicated on the plan showing the arch rib reinforcement.

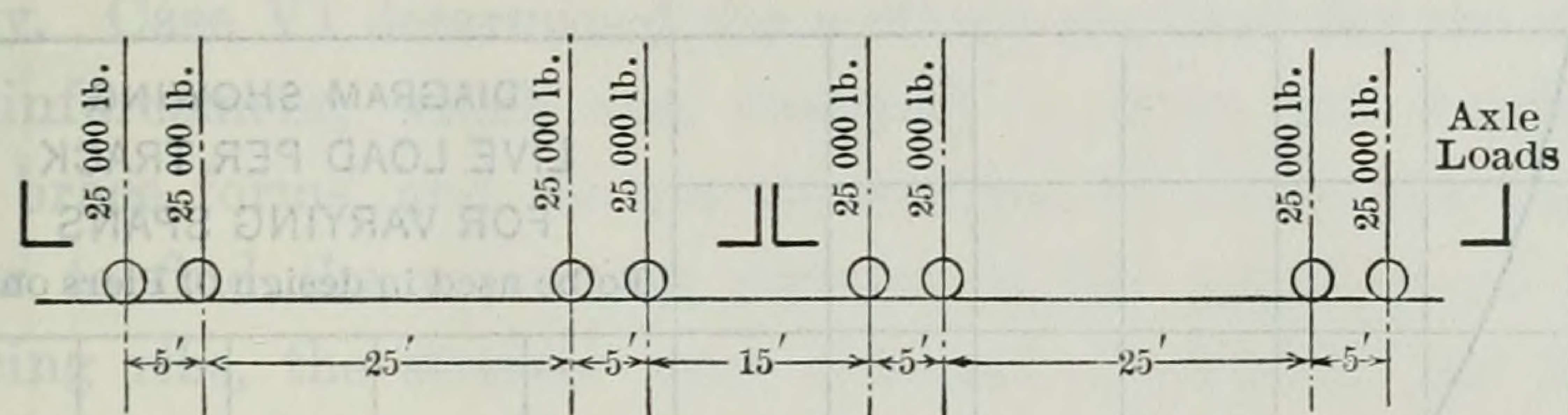
A system of lateral rods, for use during erection, was provided in the plane of each chord of the ribs. These rods were used to line up the steelwork accurately, and to hold it in line while the ribs were being concreted. The rods were not removed until after the completion of the work. The transverse braces, mentioned previously, are also reinforced in each face with continuous bars, the function of which is to resist bending in the braces due to transverse forces.

In addition to the steel reinforcement required to resist the calculated stresses in the members, a certain quantity of steel is used to prevent the formation of shrinkage, settlement, and temperature cracks. This steel is used in the exterior faces of thin walls, in a longitudinal direction in the slabs, and to reinforce the corners where two walls join. All braces, ribs, and posts are wrapped with heavy wire closely spaced, to which the longitudinal steel is secured.

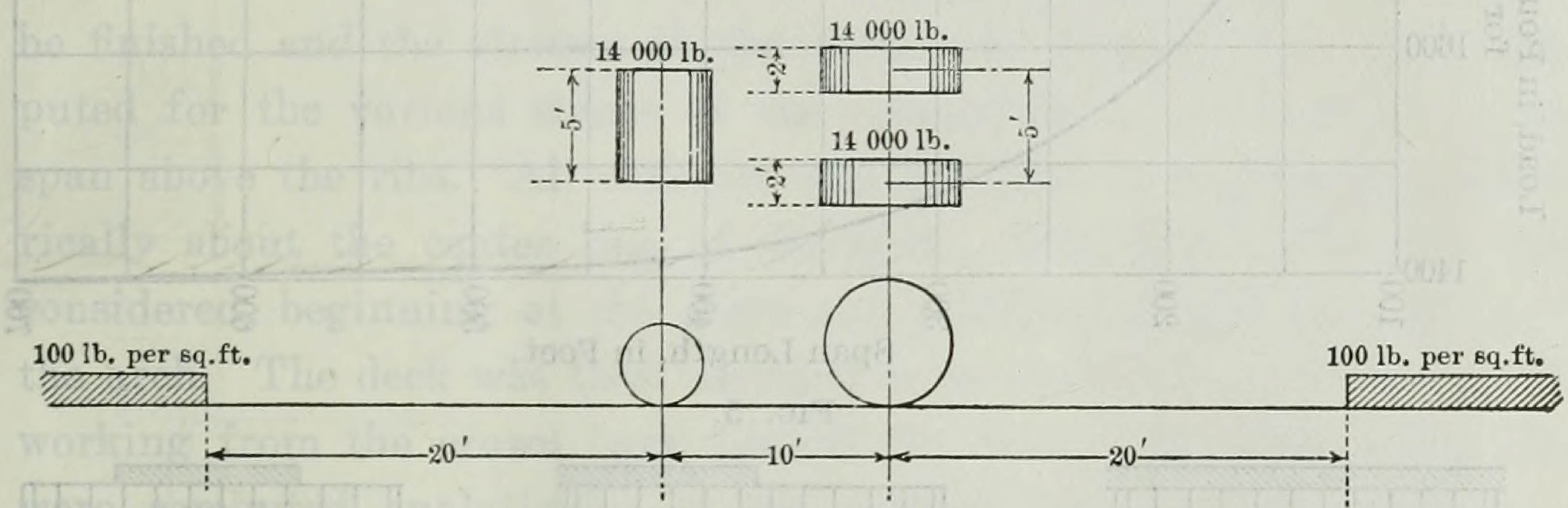
The concrete used in the construction of the viaducts is of three classes: No. 1 concrete, used for floor systems, girders, arch ribs, braces, spandrel posts, railings, stairs, and rib seats of large piers, composed of 1 part Portland cement, 2 parts sand, and 4 parts broken stone; No. 2 concrete, used for retaining walls and abutments, girder piers, and large arch piers (except rib seats), composed of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts broken stone; No. 3 concrete, used only for paving foundations, composed of 1 part Portland cement, 3 parts sand, and 6 parts broken stone. A local sand of good quality was used, and the broken stone was a hard limestone of the same quality as that from the quarries near Jacksboro, Tex. The maximum sizes of stone for the various classes of concrete are as follows: for No. 1 concrete, used for railing, $\frac{3}{4}$-in., all other No. 1 concrete, 1-in.; for No. 2 and No. 3 concrete, 2-in. The stone was graded below these maximum sizes, and the dust was screened out. A very dense concrete was obtained.

Loading.—As all the viaducts are in or near the city, and are likely to carry the same class of traffic eventually, the same live loading was used throughout. The loading data are shown by Figs. 4 to 7, inclusive. The 50-ton electric cars and the 21-ton road roller shown thereon, were used for the members of the floor system, together with a uniform load of 100 lb. per sq. ft. on the sidewalks and remaining roadway surface. For maximum stresses in the floor-beams, the sidewalks were assumed to be unloaded. The arch ribs and the main longitudinal girders were designed for the foregoing car loading, together with 100 lb. per sq. ft. on the sidewalks and the remainder of the roadway. The road roller was also considered in connection with the car loading, with the foregoing uniform live load on the sidewalks and the remainder of the roadway. A load of 200 lb. per lin. ft. of each sidewalk was also used, in order to provide for the weight of pipes and conduits that might be carried in the spaces provided for them under the sidewalks. Pier footings were designed for a uniform load per linear foot of each car track, as given on Fig. 5, together with a uniform load of 100 lb. per sq. ft. on the sidewalks and the remaining roadway surface.

Impact was allowed for according to the following formula: $I = \frac{100\,S}{L + 300}$, where I is the impact to be added to the live-load stress, S is the calculated maximum live-load stress, and L is the loaded length, in feet, that produces the maximum stress in the member.



Two 50-Ton Electric Cars as above on each Track
Area covered by 4 cars as above 20′x 100′



One 21-Ton Road Roller as above on any part of Roadway
Width covered by Roller 10′.

The car and roller given above, together with 100 lb. per sq. ft. on sidewalks and remainder of roadway, to be used in design of floor system, main longitudinal girders, and arch ribs. For Piers and footings, use 100 lb. per sq. ft. of sidewalks and Roadway and uniform track loading as per diagram, except Case II in design of arch Piers, where wheel loads are to be used to give maximum thrusts.

IMPACT

On sidewalks	none
Road Roller	25%
Other roadway loads and track loads	$I = S\,\frac{100}{L+300}$

where I = Impact to be added to the Live Load Stress.
S = Calculated maximum Live Load Stress.
L = Loaded Length, in feet, which produces maximum stress in the member.
No impact to be used in design of Piers and footings.

Fig. 4.

This impact allowance is one-third of that required by the original formula, submitted by the American Railway Engineering and Maintenance of Way Association, for impact on steel railway bridges, and was arbitrarily assumed. This formula was used only for car and uniform roadway loads. Stresses produced by the road roller, which is a slowly moving load, were increased by 25% in all cases. No addition was made for impact to the stresses produced by the uniform sidewalk loading, as it was considered that when this sidewalk load was a maximum, that is, due to a closely packed crowd of people, it neces-

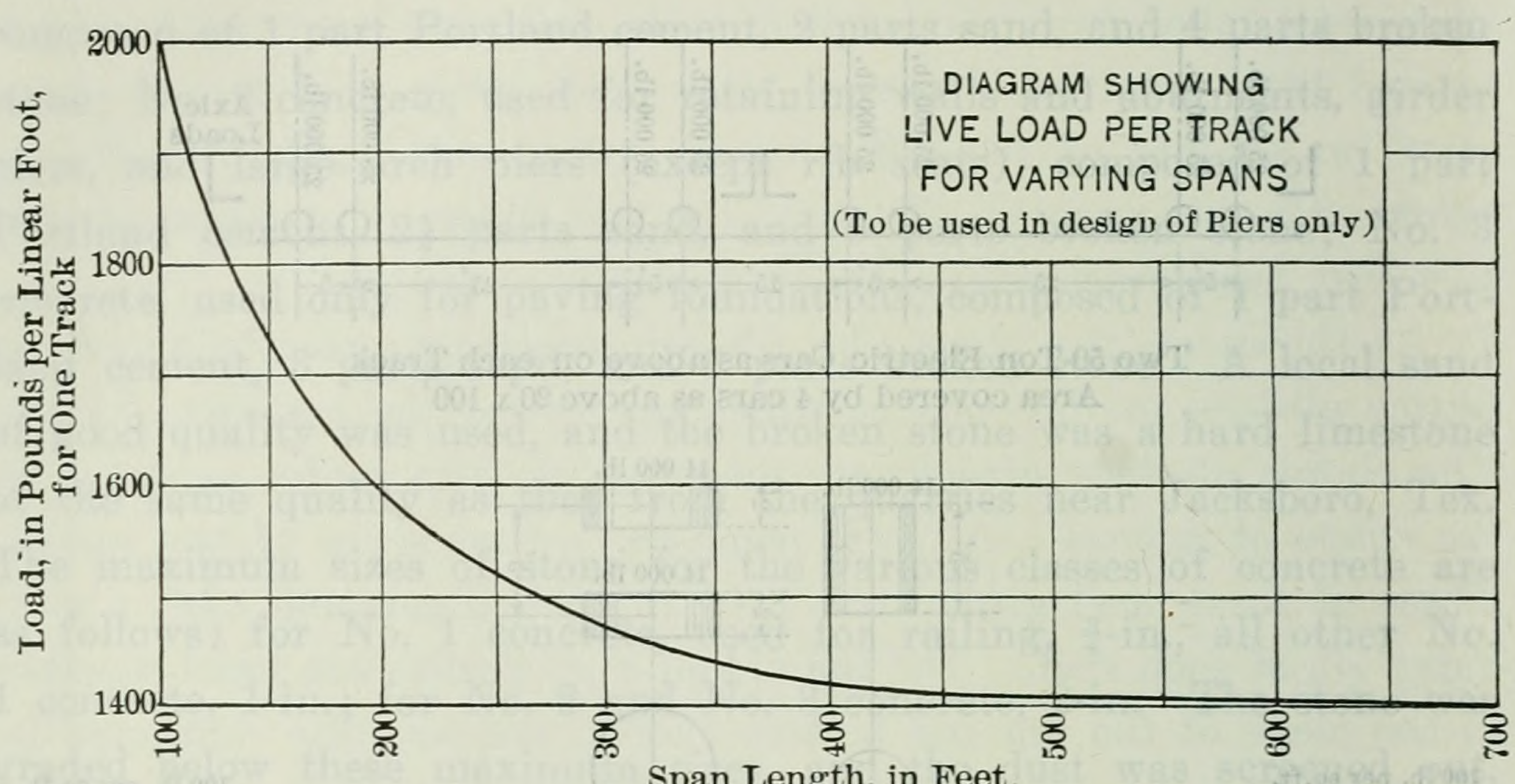


Fig. 5.

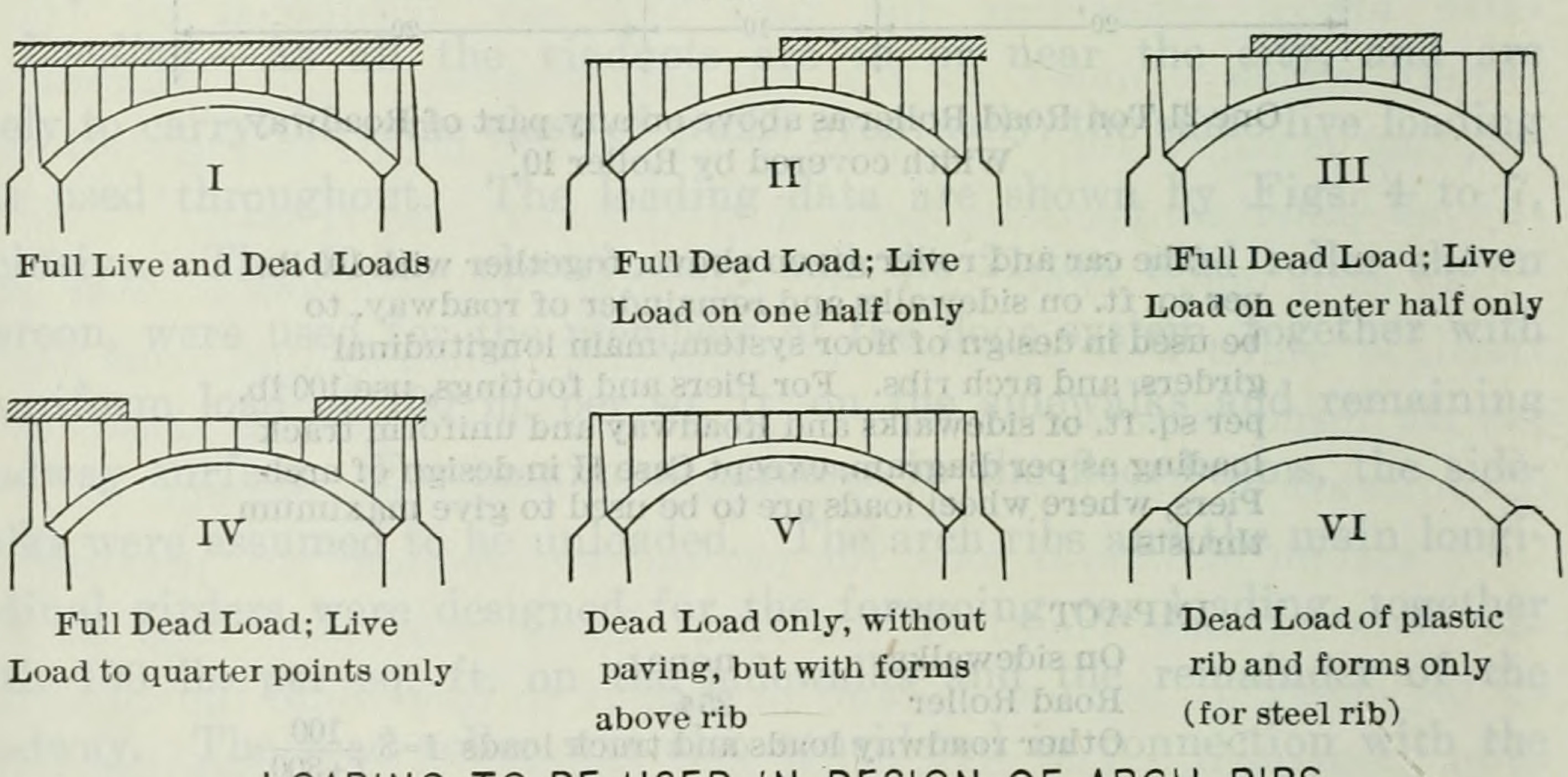


Fig. 6.

sarily would be nearly quiescent. The live loads for the piers were not increased for impact, as it was believed that the impact would be absorbed before reaching the footings.

The various conditions of loading for which the arch ribs were designed, are indicated on Fig. 6. Cases I to IV require no explanation, except to say that they served to produce maximum stresses in the ribs of all spans except those of the unsymmetrical arch, Span *A*. On account of this lack of symmetry, these cases were elaborated on, in order to be sure that the maximum stresses had been found. Cases V and VI are construction conditions, and were elaborated on considerably. Case VI determined the sections required for the structural steel reinforcement, which was designed to carry the weight of the rib and brace forms, and the plastic concrete in these ribs and braces. In order to find the maximum stresses in the members of the steel reinforcing ribs, the stresses were obtained graphically for the forms filled with concrete from the haunch, successively to the foot of each post. In Case V the forms of the posts and deck were assumed to be finished and the stresses in the reinforced concrete ribs were computed for the various stages of the concreting of the portion of the span above the ribs. All concrete was assumed to be placed symmetrically about the center line of the span. The post loads were first considered, beginning at the piers and working toward the crown of the arch. The deck was then assumed to be concreted, panel by panel, working from the crown back toward the piers. Stresses in the ribs were computed analytically, at the critical points, for each step as outlined above. By this method of procedure in concreting, the stresses in the ribs were kept within the limits set for working conditions in the finished structure. The arch piers were designed for the conditions indicated on Fig. 7, which needs no explanation. The piers for the girder spans were designed only for full vertical load.

In addition to the above mentioned vertical loads, the structures were designed to resist transverse forces due to wind and water pressure. The arch ribs and their braces, in particular, were designed with great care to resist these transverse forces. In general, in computing these transverse forces, the water line was assumed to be at the top of the levee. Below this line the pressure due to the current was taken at 200 lb. per sq. ft. of vertical projection of each arch rib and spandrel post. Above this line the wind pressure was assumed at 30 lb. per sq. ft. of vertical projection of the structure, up to a line 10 ft. above the top of the roadway. The force assumed for the current corresponds, according to the formula used, to a velocity of about 8 or 9 miles per hour, which is probably not far from the actual velocity at flood stage.

At the West Seventh Street crossing, where the rise of the arch is small, and the high-water line comes well up on the arch rib, and where considerable drift is carried by the stream, more severe conditions

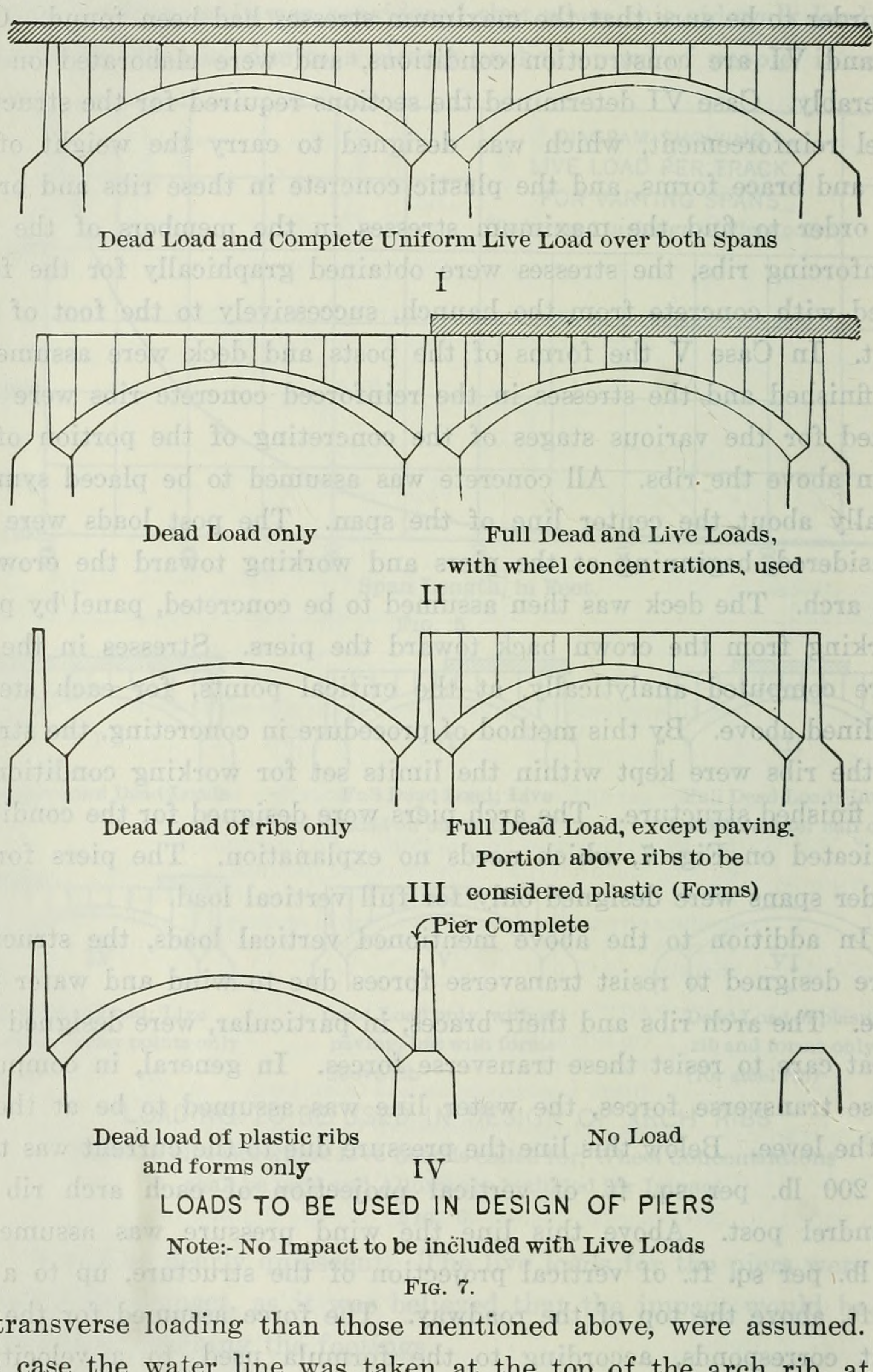


FIG. 7.

of transverse loading than those mentioned above, were assumed. In this case the water line was taken at the top of the arch rib, at the crown, in order to allow for the possibility of drift collecting against the span.

Specifications.—With certain modifications, in regard to unit stresses, "Watson's Specifications for Reinforced Concrete Bridges" were used.

The following is a list of the unit stresses used for the various parts of the structure:

	Pounds per square inch.
Tension in steel bars used for flange reinforcement	20 000
Tension in steel bars used for web reinforcement	15 000
Compression in concrete in cross-bending, as in beams	750
Compression in concrete, combined direct and bending, as in arch ribs and posts	500
Compression in concrete of arch ribs for stresses due to combined vertical and transverse loads	650
Bond between concrete and steel, for deformed bars	120
Shear in concrete	60
Or, a maximum on the section, including shear taken by steel, of	150
Bearing of hinge castings on concrete specially reinforced	800
Compression in structural rib reinforcement due to weight of forms and plastic concrete in arch ribs and braces, $16\,000-70\frac{l}{r}$, or a maximum of	12 500
Additional compression in structural rib reinforcement, under working conditions, and compression in steel of other compression members = 15 times the corresponding compressive stress in concrete. Possible maximum total compression in structural steel rib reinforcement.	20 000
Tension on net section of structural steel reinforcement	16 000
Bearing, tension and compression in cast-steel hinges	20 000
(Bearing area of the hinges taken as the projected area of the hemispherical joint.)	
Bearing on rivets	24 000
Shear in cast steel	10 000
Shear on rivets	12 000
	Pounds per square foot.
Bearing on soil at depth of 6 ft. below finished surface	5 000
For each 1 ft. increase in depth, more than 6 ft., add	100
	Pounds.
Load on each timber pile	20 000

Methods of Design.—There is nothing unusual in the design of the structures, with the exception of the methods of designing the arch ribs and braces. Briefly, sidewalk slabs and main girders were designed as simple beams; and continuous members, such as floor-beams, stringers, and roadway slabs, were designed for 0.8 of the maximum positive moments, considered as simple beams. In all cases, where members were continuous, steel was provided over the supports to take the negative moments at these points. The quantity of this steel was two-thirds of that required to take the maximum positive moments in the members. The shears and reactions for continuous members, such as floor-beams and stringers, were increased by from 10 to 25% (depending on the number of spans) more than what these functions would have been for simple spans.

In order to make clear the methods used in the design of the arch ribs, braces, and hinges, a detailed account of this part of the work will be given.

It will be seen that the structural steel arch rib reinforcement takes stress in two independent steps: First, as the ribs are filled with concrete, the members take certain initial stresses, which, as the concrete sets, remain in the steel. This initial stress in the steel was not allowed to exceed a maximum of 12 500 lb. per sq. in. in compression, as given in the list of unit stresses. Second, after the ribs are finished and the construction of the posts and deck is under way, and later, when the structure is carrying live load, stresses are set up in the ribs, which are carried jointly by the steel and concrete in the usual manner. The assumed ratio of the moduli of elasticity of steel and concrete was taken as 15. Therefore, the maximum possible compressive stress in the structural steel reinforcement of the ribs would be 12 500 lb. plus 15 times 500 lb., or 20 000 lb. per sq. in. This is based on the assumption that the ribs and braces are completely filled with concrete in so short a time that it does not begin to set before all the load is in place, and also that the stress in the concrete at the center of gravity of the steel chord is 500 lb. per sq. in. As a matter of fact, the ribs were concreted in sections, and the braces were not concreted until the concrete in the ribs had set for several days. Consequently, the initial compression in the structural reinforcement was much less than that mentioned above. The center of gravity of the steel chords is some distance in from the face of the concrete, so that the concrete

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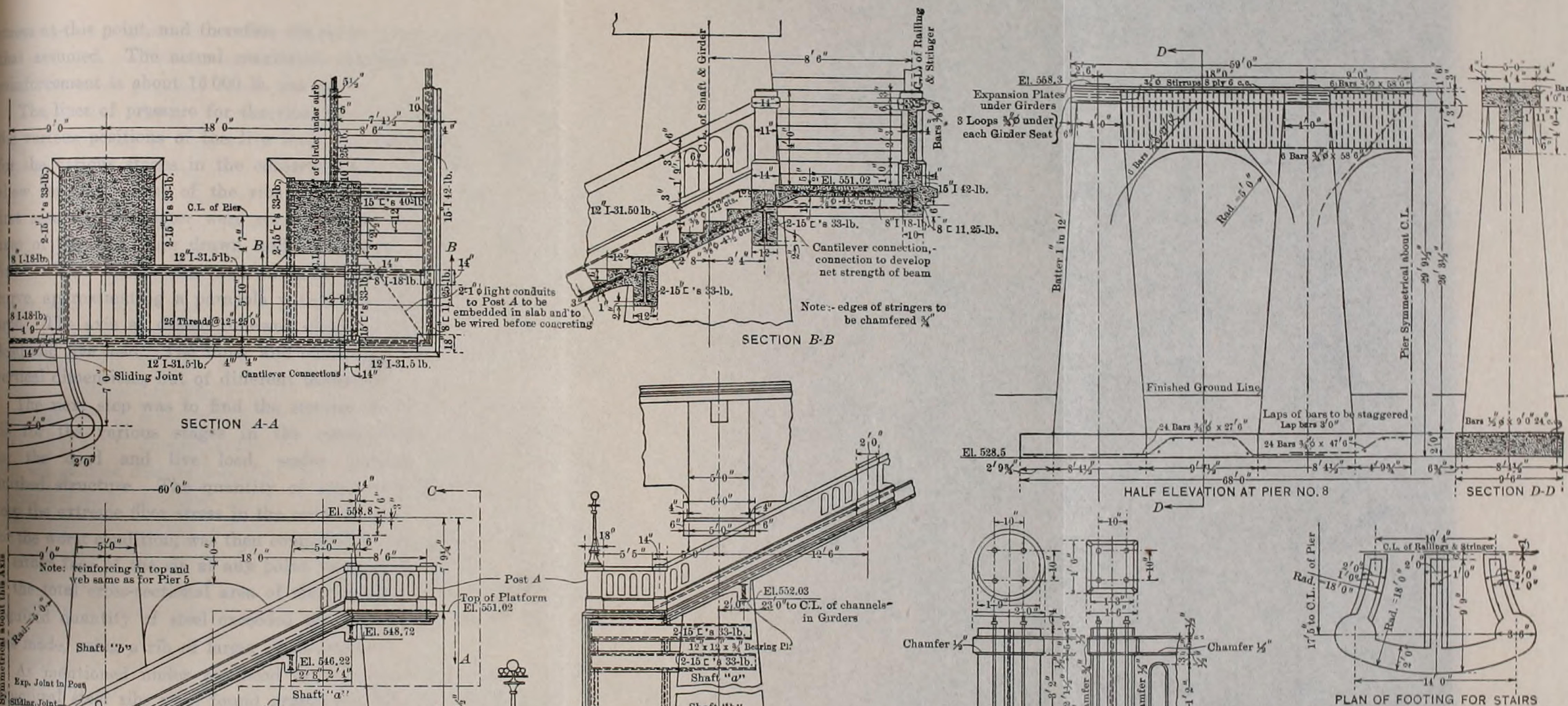

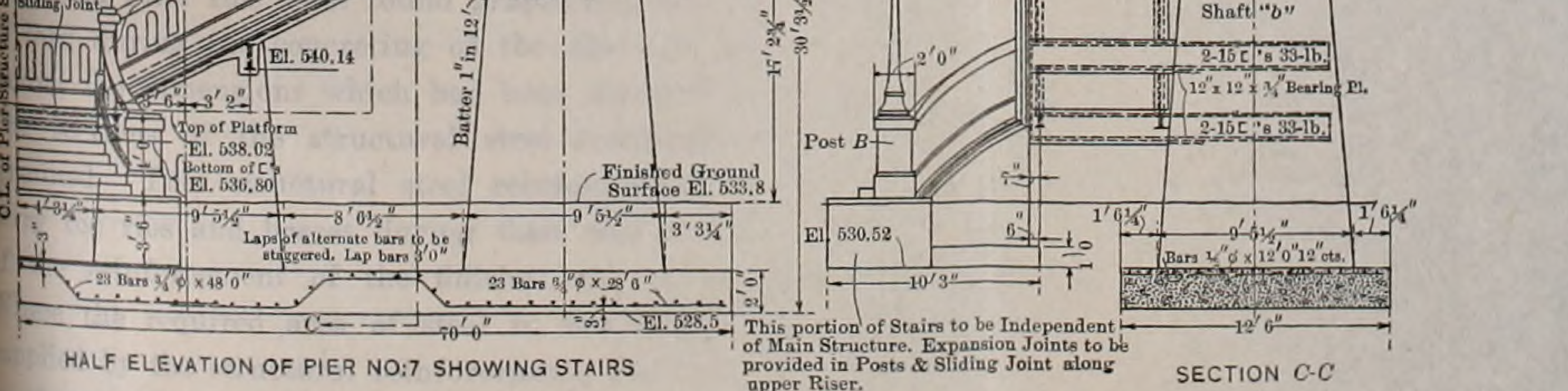

DETAILS OF PIERS 7 AND 8
MAIN STREET VIADUCT
FORT WORTH, TEXAS

GENERAL NOTES

Reinforcing bars to be securely wired together at all laps.

All vertical edges of piers to be chamfered thus;

All other edges & square corners of piers " "

All reinforcing bars to have some form of Mechanical bond; to be of sectional Area equivalent to that shown, and to have elastic limit not less than 50000 lb. per sq. in.

Open-Hearth Structural Steel to be used.

Use ⅞" rivets where possible. Allowable stresses on rivets to be 12000 lb. per sq. in. for shear and 24000 lb. per sq. in. for bearing

Materials and workmanship to be in accordance with Manufacturers Standard Specifications

MIX OF CONCRETE

Stairs and Railing	Piers
1-part Portland cement	1-part Portland cement
2- " sand	2½- " sand
4- " broken stone	5- " broken stone

All by volume

stress at this point, and therefore the stress in the steel, is lower than that assumed. The actual maximum compression in the structural reinforcement is about 16 000 lb. per sq. in.

The lines of pressure for the ribs were worked out graphically for the various positions of the live load, combined with dead load, and for the various stages in the construction of the deck. In order to allow for the weight of the ribs themselves, their dimensions were first assumed, and later were revised where necessary. The neutral axes of the ribs were drawn midway between the extreme pressure lines. This made the neutral axis of the rib a constructive curve, approximating a parabola with its vertex at the crown of the arch. The axis as drawn followed closely the pressure line for full dead and live load. The inner and outer ribs were made of the same vertical dimensions, but of different thickness.

The next step was to find the stresses in the reinforced concrete rib for the various stages in the construction of the deck, and for the dead and live load, under working conditions in the finished structure. The quantity of steel required at each point to keep the extreme fiber stress in the concrete down to 500 lb. per sq. in. for the worst condition, was then computed. The required area of steel in either face of the rib at any point was not allowed to exceed 2.5% of the total cross-sectional area of the rib. Where, on first trial, the required quantity of steel exceeded the above maximum, a re-design was made, using a rib of larger cross-section.

As mentioned under the head of "Loading", the stresses in the structural steel ribs were found graphically for the various positions of load during the concreting of the ribs and braces, using the rib having the dimensions which had been determined as just described. The sections of the structural steel reinforcement were then determined. This structural steel reinforcement, though designed to carry the ribs and braces during their construction, also forms part of the reinforcement of the finished rib, as previously mentioned. Where the required area of steel in the finished rib exceeded that supplied by the structural reinforcement, the deficiency was made up by the addition of reinforcing bars. These bars do not run the full length of the ribs, but stop off where they are not needed. It will be noticed that these bars take little, if any, initial stress, such as is taken by the structural reinforcement. Additional bars are placed in the sides of the ribs, running full length. These bars help to resist transverse bending stresses, and to prevent surface cracks from appearing.

The transverse forces of wind and water pressure, mentioned previously, were assumed to be resisted wholly by the ribs and braces acting as a series of portals, in the same manner as the columns and girders of an office building act to resist wind pressure. The deck was not given credit for any resisting power, because of the presence of the expansion joints over the haunch and crown of the span.

Points of inflection were assumed in the braces, midway between each pair of ribs, and in the ribs, midway between the braces. The ribs being hinged at the haunches and crown, were computed as being free to move at these points. Bars were provided at the haunches, however, anchoring the ribs to the piers, and at the crown, anchoring the two halves of the rib together. These bars were placed in the plane of the neutral axis of the rib, and tended to resist transverse, but not vertical movement, of the ribs. The stresses in the ribs and braces were computed at each section, for the moments, shears, and direct thrusts in the members. The maximum stresses at each point in the ribs, due to transverse forces, were combined with the maximum stresses due to the vertical loads, and the ribs were proportioned accordingly.

The design of the hinges naturally received considerable attention. At first, a hinge was studied which consisted of two castings, having semi-cylindrical bearings, and a pin. This type has a number of advantages, but was finally abandoned on account of the difficulty of setting the hinges on the piers with sufficient accuracy. Unless the axes of the pins are placed exactly at right angles to the vertical plane through the longitudinal axis of the rib, eccentric stresses will result in the rib, which may be too large for safety. It was finally decided to use hinges composed of two ribbed steel castings, having a hemispherical ball and socket joint, as shown on the plans. This joint was machined, and, by allowing movement in all directions, made great accuracy in setting the castings unnecessary. This joint also made it possible to adjust the structural rib reinforcement properly in vertical planes, with the temporary adjustable lateral rods provided for that purpose. This type of hinge reduces to a minimum the possibility of eccentric stresses in the ribs.

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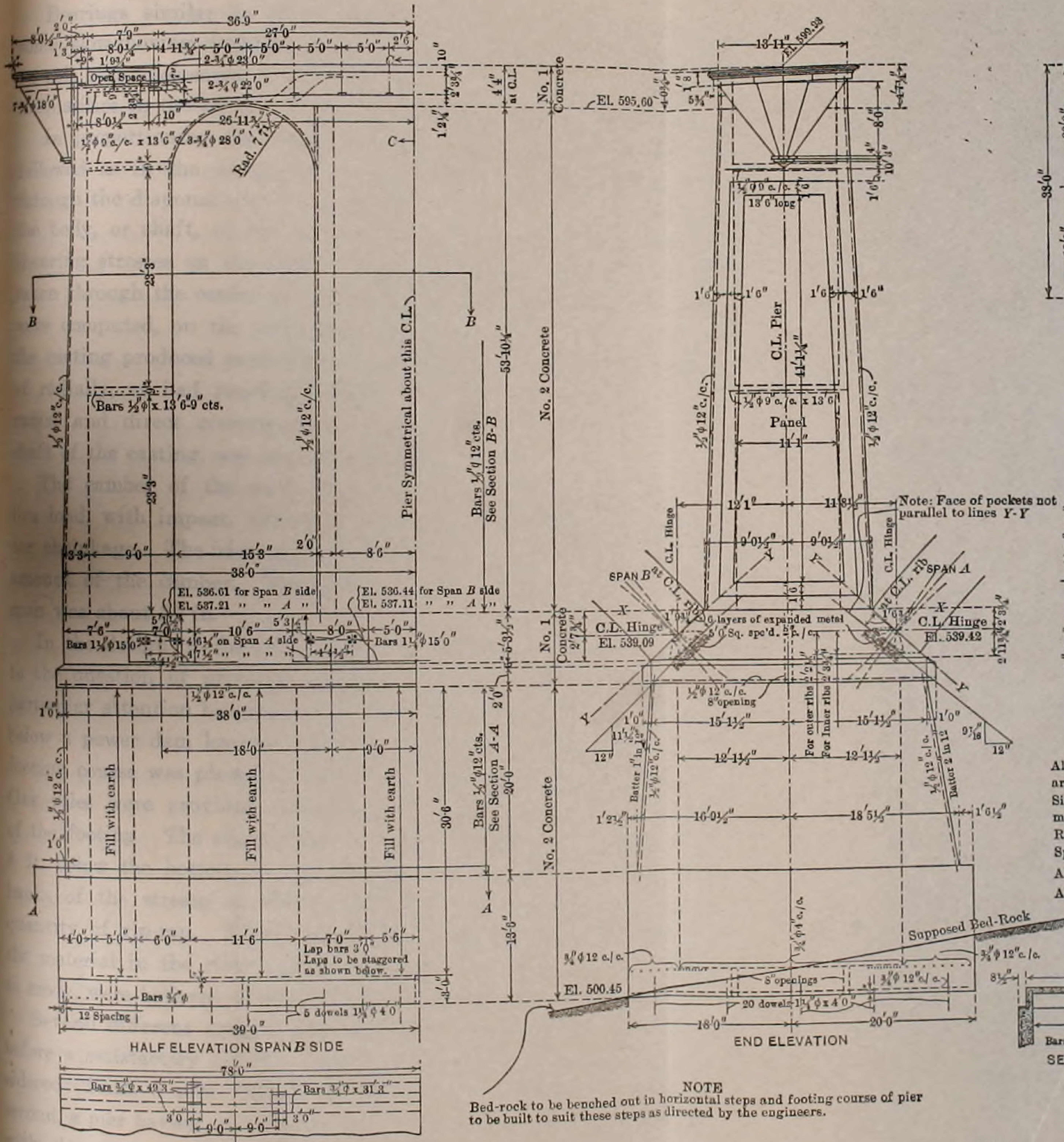
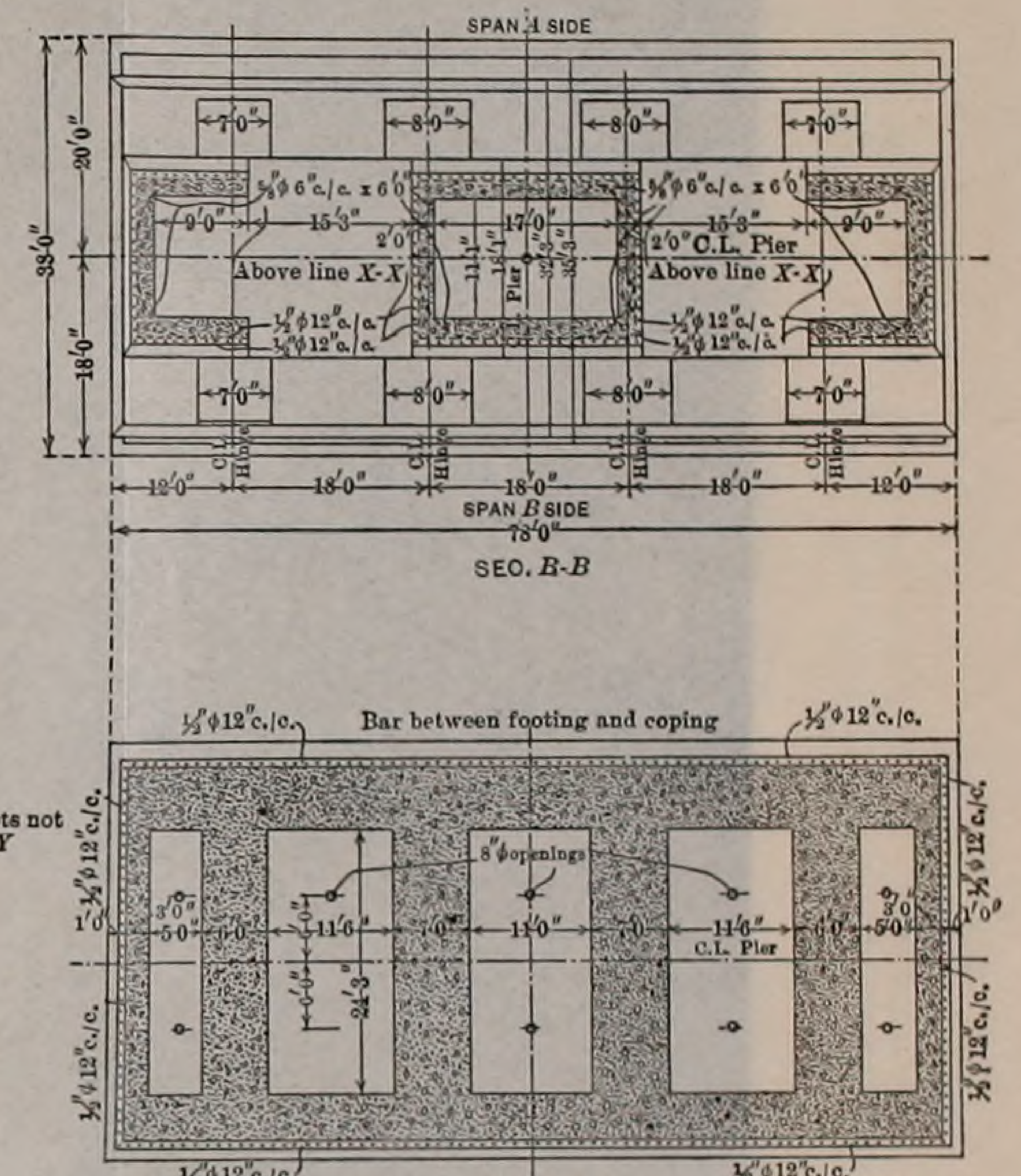


HALF ELEVATION SPAN *B* SIDE

PLAN SHOWING ARRANGEMENT OF REINFORCING BARS IN FOOTING

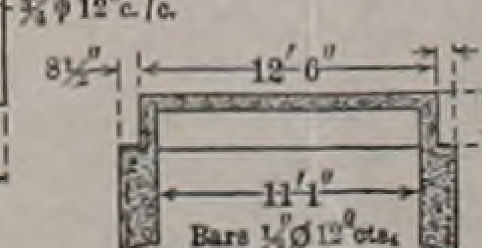
END ELEVATION

NOTE
Bed-rock to be benched out in horizontal steps and footing course of pier to be built to suit these steps as directed by the engineers.



SEC. *B-B*

SEC. *A-A*



SEC. *C-C*

GENERAL NOTES

All reinforcing bars to have some form of mechanical bond and to be of sectional area equivalent to that shown and to have elastic limit not less than 50000 lb. per sq. in.
Six layers of expanded metal 5′0″ square with 3″ mesh of No. 10 standard gauge metal placed back of each hinge.
Recessed portion of panels to be bush-hammer dressed.
Span *A* side of pier same as span *B* side unless otherwise shown.
All vertical edges of pier to be chamfered thus ¾″
All vertical edges & square corners of copings to be chamfered thus ¾″

Mix for No. 1 Concrete	Mix for No. 2 Concrete
1-Part Portland Cement	1-Part Portland Cement
2 Parts Sand	2½-Parts Sand
4-Parts Broken Stone	5-Parts Broken Stone
All by Volume	All by Volume

DETAILS OF PIER NO. 1
MAIN STREET VIADUCT
FORT WORTH, TEXAS

Bearings similar to these hinges had previously been used for some slow-moving multiple trucks, carrying extremely heavy loads. In this case, movement in all directions was necessary to take up the inequalities in the track. These bearings gave perfect satisfaction.

In proportioning the hinge castings, much the same method was followed as in the design of a column base. A section was taken through the diagonal rib of the hinge, at the junction of the rib with the body, or shaft, of the casting, and the tensile, compressive, and shearing stresses on the section were computed. A section was also taken through the center of the hinge, and the stresses on this section were computed, on the assumption that the reaction on the base of the casting produced moments at right angles to this section, instead of radially, as had previously been assumed. The resultant of the radial and direct compressive stresses at the circumference of the shaft of the casting, was also investigated.

The camber of the arch spans was computed for full dead and live load, with impact, combined with temperature and an allowance for shrinkage. The rise of the structural ribs was increased by the amount of the camber. The total calculated camber for the longest span was about 3½ in.

In the design of the main piers, considerable attention was given to the question of protection against scour. Pier No. 2 was given particular attention because of the fact that it is only a short distance below a power dam having a fall of about 15 ft. The bottom of the footing course was placed at about 5 ft. below the bed of the stream. Oak piles were provided, extending about 15 ft. below the bottom of the footing. The sheet-piling for the coffer-dam was driven about 8 ft. below the bottom of the footing, and was left in place. The banks of the stream at this point were also protected by a large quantity of rip-rap. These precautions, together with the fact that the material in the river bed at this point is a stiff clay, not likely to erode, were believed to afford ample protection against scour.

Several different designs and estimates were made for Pier No. 2 before a satisfactory solution was reached. The following were considered: First, a pier similar to Pier No. 1, founded on bed-rock; second, a pier having its base at about the level of the bed of the river, with the vertical loads carried on eight cylinder piers, two under each line of arch ribs; third, a pier resting on piles driven to rock;

and fourth, the pier as built, having moderately spread footings, but with a portion of the load carried on piles which do not extend to rock.

The first scheme was abandoned because of the cost, which would have made that of the structure exceed the appropriation. The second scheme was considered inadvisable because of the difficulty of taking care of the unbalanced horizontal thrust. As the vertical load would have been carried down the cylinders to rock, the friction between the pier base and the material under it could not safely be counted on to resist the horizontal thrust. There then remained, to resist this thrust, only the pressure of the earth against the body of the pier and against the cylinders. Computations indicated that this resisting pressure would be insufficient, unless the body of the pier was extended to a considerably greater depth. This would have made the cost too great. The third scheme was rejected for practically the same reasons as those given above for the second scheme, and also because, after investigation, it was considered doubtful whether piles could be driven to rock through the mixture of sand and gravel overlying it. The fourth scheme proved to be the cheapest of those considered, and was believed to be the most feasible. Therefore it was adopted, with the provisions mentioned above to protect the pier from scour.

The small piers were designed for full vertical load only, using the unit pressures mentioned previously.

Rankine's theory of earth pressure was used in the design of the retaining walls. The weight of the earth fill was taken at 100 lb. per cu. ft. A superposed load of 100 lb. per sq. ft. was assumed, to take care of the roadway and sidewalk loads. For convenience in computing the pressures, the unit weight of the fill was considered to be increased by the ratio, $\frac{d'}{d}$, where d is the actual depth of fill, and d' is the actual depth plus the depth of the imaginary fill required to produce the same unit vertical pressure as the superposed load. In the foregoing case, $d' = d + 1$ ft. For the higher sections, this increase is negligible, but, for the lower ones, it is worth taking into account.

In concluding this portion of this paper, a few words in regard to the advantages of the three-hinged, ribbed arch may be in order.

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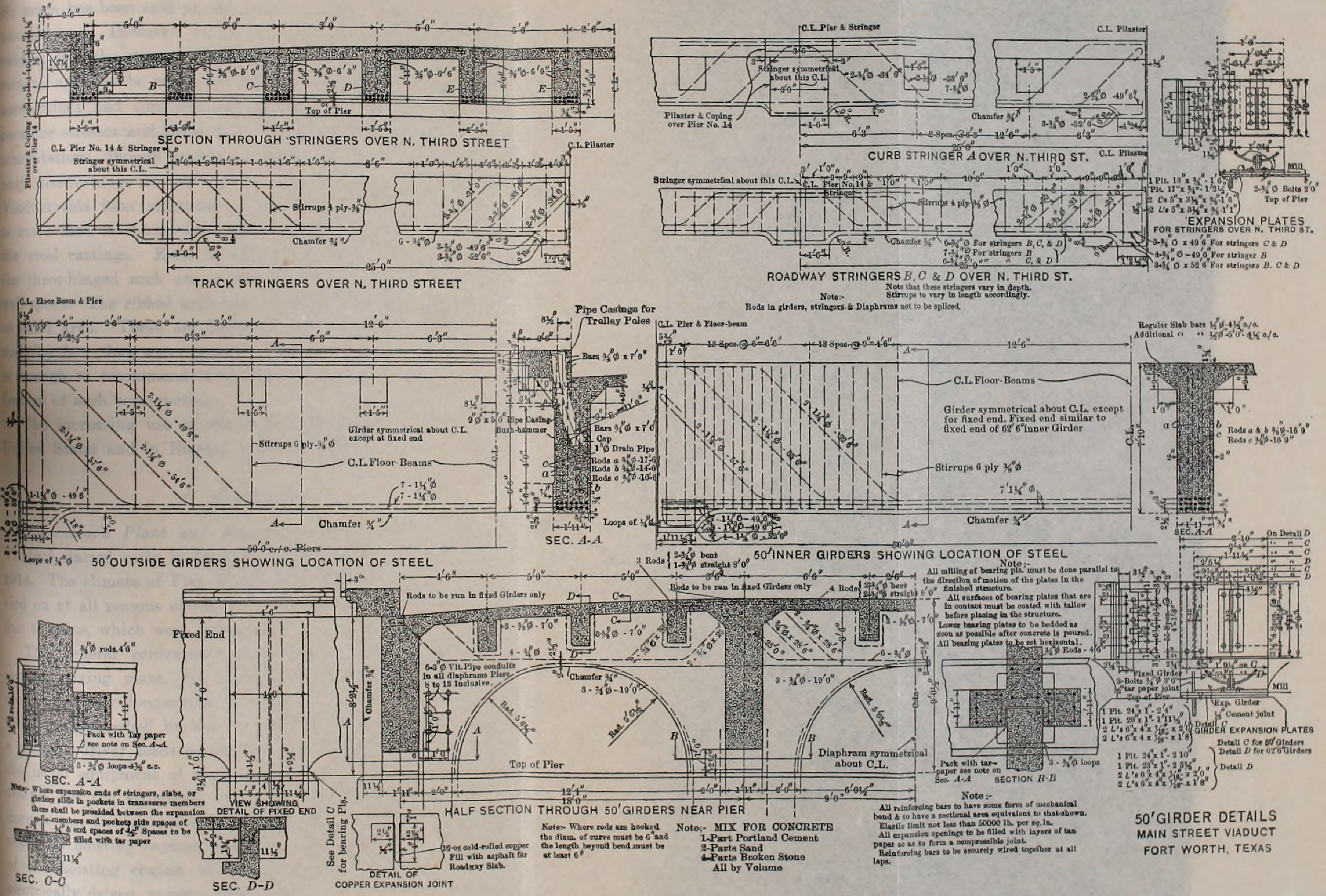

50' GIRDER DETAILS
MAIN STREET VIADUCT
FORT WORTH, TEXAS

So much has been said on this subject, that little that is new can be brought out. However, to go over the ground again briefly, it is evident that the stresses in an arch of this type are statically determinate, and can be computed with as much accuracy as the loading assumptions and computations of the dead weight warrant. Temperature stresses and those due to settlement are eliminated, and the computations, though somewhat long, are simple. Practically, the only disadvantage is the cost of the hinges. In the Main Street Viaduct this item amounted to about $3\frac{1}{2}\%$ of the total cost. This is more than it should have been, owing to the high price paid for the steel castings. Even at this price, when all the advantages of the three-hinged arch are considered, the money paid for hinges is well spent. The ribbed arch has two great advantages over the barrel, or solid arch. These are lightness, with consequent saving in cost, and the fact that water-proofing is not required. The latter, alone, is enough to recommend it to those who have had trouble with this feature of arch construction.

The increasing use of the three-hinged ribbed arch, both in the United States and in Europe, is evidence of its many advantages.

Construction.

Contractor's Plant and Equipment.—Work on the Main Street Viaduct started early in December, 1912, and was finished in March, 1914. The climate of Fort Worth is such that concreting can be carried on at all seasons of the year. The only interruptions were those due to rains, which were infrequent.

The general contractor's plant and equipment consisted of a central mixing plant, two distributing towers, with hoists, chutes, etc., an electric locomotive and cars for transporting concrete, derricks and clam-shell buckets for handling excavation and materials, a pair of driver leads suspended from the boom of a derrick, and the usual equipment of smaller tools and appliances. The sub-contractor for the structural steelwork had a steel stiff-leg derrick, a derrick car, two gin-poles, a two-bent timber gantry traveler, and the necessary hoisting engines and minor appliances. With the exception of one hoisting engine, all the general contractor's equipment was electrically driven, current being obtained from the local power company and transformed at the site. The sub-contractor's equipment was operated by steam.

While the pier excavations were under way, the mixing plant was being built near Pier No. 3 on the down-stream side of the structure. This consisted of an elevated hopper divided into two parts, one for rock and one for sand, a measuring box for sand and one for stone, and a mixer capable of handling a batch of a little more than 1 cu. yd. of concrete. Behind the mixing plant was a stiff-leg bull-wheel derrick for unloading materials and for supplying the hoppers of the mixing plant. Materials were brought to the site on cars, on a spur-track which crossed the line of the viaduct alongside of the mixing plant.

Concrete was discharged from the mixer into steel hopper cars drawn by an electric locomotive running on a narrow-gauge track. The concrete was carried in these cars to the two distributing towers, where it was hoisted and conveyed through the chutes to its destination. The larger of these towers was stationary, and was about 200 ft. high. This tower served the portion of the viaduct between Pier No. 4 and the South Abutment, except where concrete was discharged directly into the footings of Pier No. 3 from the mixer. The other and smaller tower was built so that it could be moved when required. It served the portion of the structure between Pier No. 4 and the North Abutment. For the retaining walls, concrete was carried in the hopper cars to the north end of the walls, where it was switched back on a trestle midway between the walls, and having the track parallel to and about 6 ft. above the tops of the walls. From this trestle the concrete was passed through chutes into the walls.

Construction Methods.—Steel forms were used where there was sufficient duplication to make them economical. They were especially advantageous on the girders and floor system, which had been laid out with the view of making the work as simple, and giving as much repetition, as possible. For instance, all floor-beams were made alike, so far as the dimensions of the concrete were concerned; all panel lengths, and consequently all stringers of each kind, were alike. The wall forms were made up of squares and rectangles of such size that they could be handled readily by two or three men. These forms were stiffened by angles, and were bolted together by bolts passing through the flange angles. They were braced by timber verticals and

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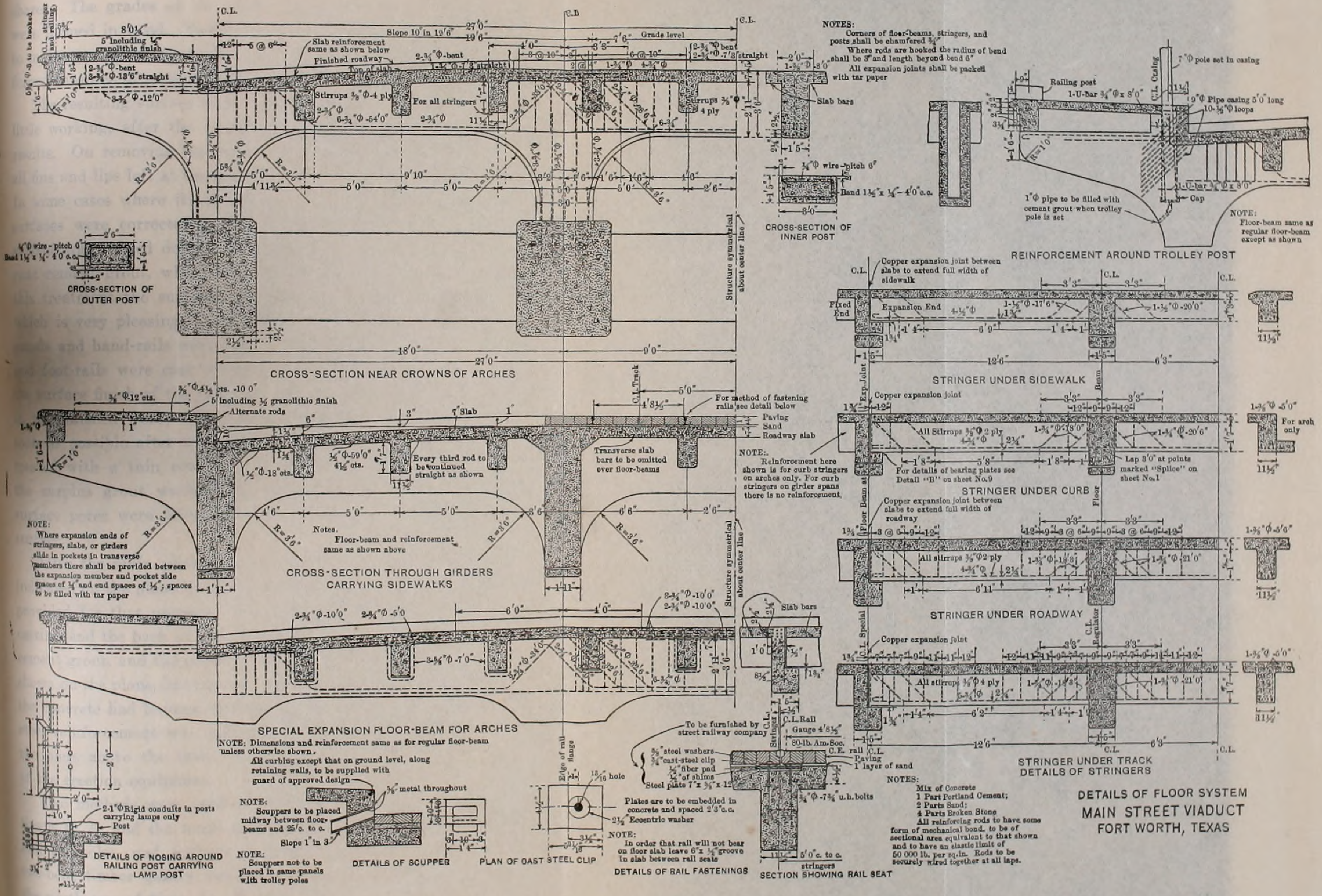


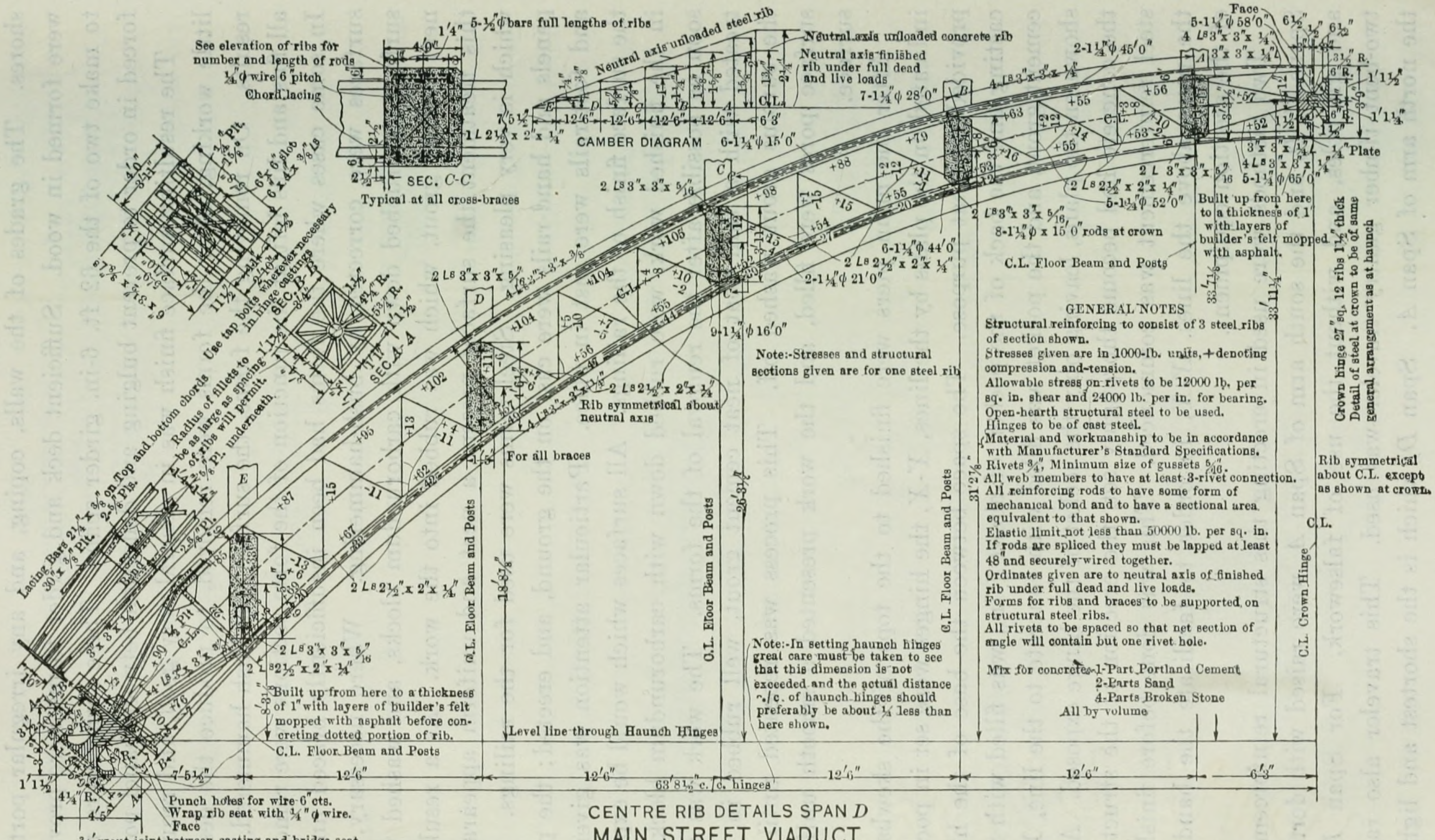
DETAILS OF FLOOR SYSTEM
MAIN STREET VIADUCT
FORT WORTH, TEXAS

shores. The grades of the walls, coping, and any irregular portions were formed in wood. Sufficient deck and girder forms were provided to make two of the 62 ft. 6-in. girder spans. These forms were reinforced in order to prevent bulging and deflection between panel points.

The resulting surface finish was in general very good, and required little working, after the forms were removed, to produce the desired results. On removing the forms, the surface was bush-hammered, and all fins and lips left at the junction of the form sections were removed. In some cases where the forms had been insufficiently braced untrue surfaces were corrected by bush-hammering. Where necessary, the surface was rubbed down with carborundum blocks, and washed with neat cement grout, which was rubbed into the work. As a result of this treatment the surfaces present a neat and uniform appearance, which is very pleasing. Steel forms were used for the railings. The panels and hand-rails were cast on the ground, and erected; the posts and foot-rails were cast in place. Particular attention was given to the surface finish of the railings. All surfaces which would be exposed in the finished work, were rubbed down with carborundum blocks as soon as possible after the removal of the forms. The work was then treated with a thin coat of neat cement grout, well rubbed in, and the surplus grout washed off. This process was repeated until the surface pores were filled, and the work presented a smooth uniform surface.

After the main piers were finished to the tops of the skewbacks, indicated on the plans by the lines, *X-X*, the hinges were set in pockets provided for that purpose. The space between the base of the hinge casting and the back of the pocket, of about $\frac{3}{4}$ in., was filled with neat cement grout, and the pocket was filled with concrete to the line, *Y-Y*, shown on the plans, leaving only the socket of the hinge exposed. After the concrete had become thoroughly set, the erection of the structural steel reinforcement was commenced. This was done before finishing the piers above the lines, *X-X*, in order to facilitate the handling of the erection equipment.

Two methods were used in erecting this structural reinforcement. Spans *C, D,* and the south arm of Span *A,* were raised with derricks and gin-poles, and without the use of falsework. For Span *B,* a two-bent timber gantry traveler was used. This traveler also raised the north arm of Span *A*. Span *D,* which is the shortest and lightest

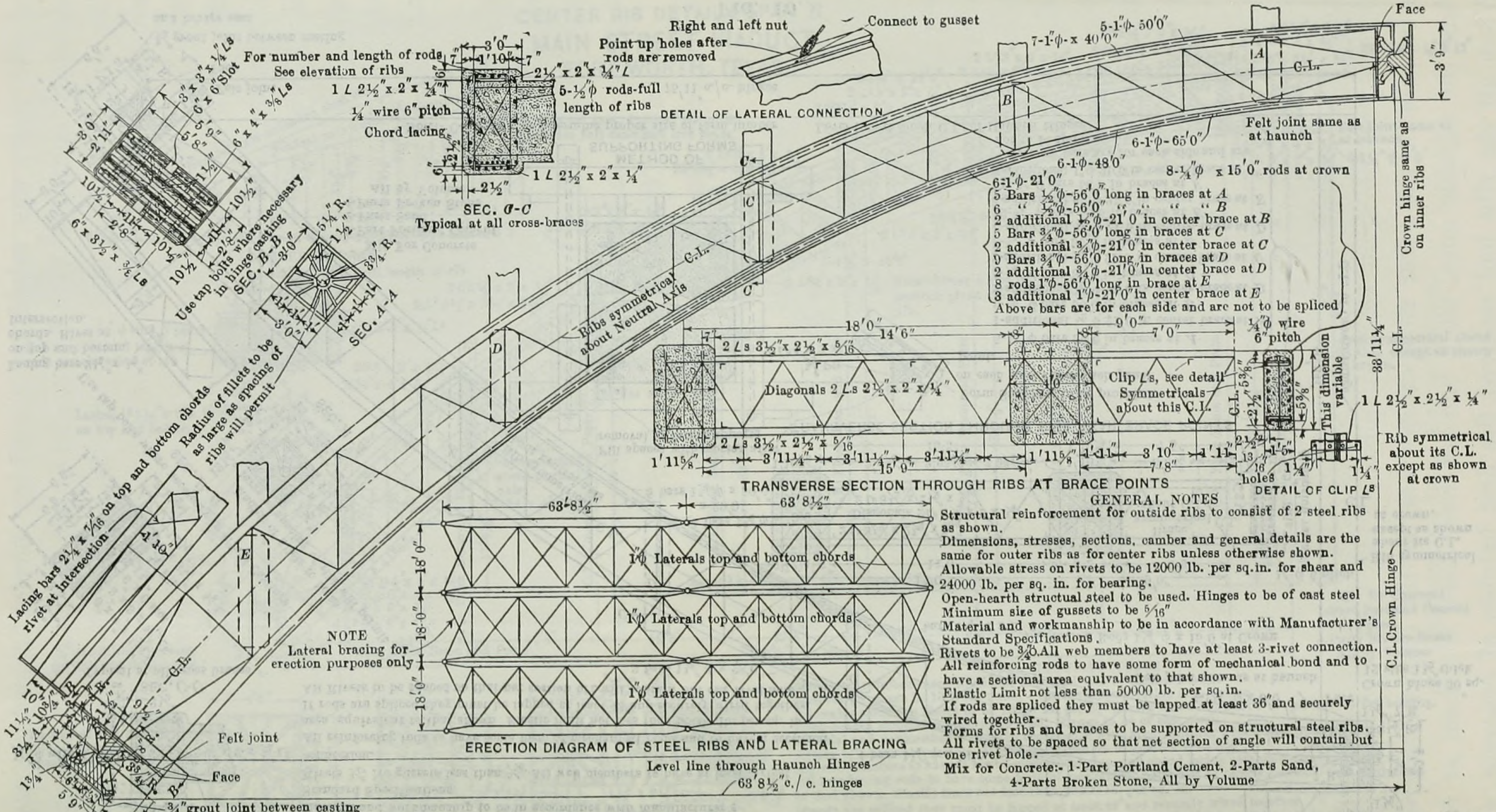


CENTRE RIB DETAILS SPAN *D*

MAIN STREET VIADUCT

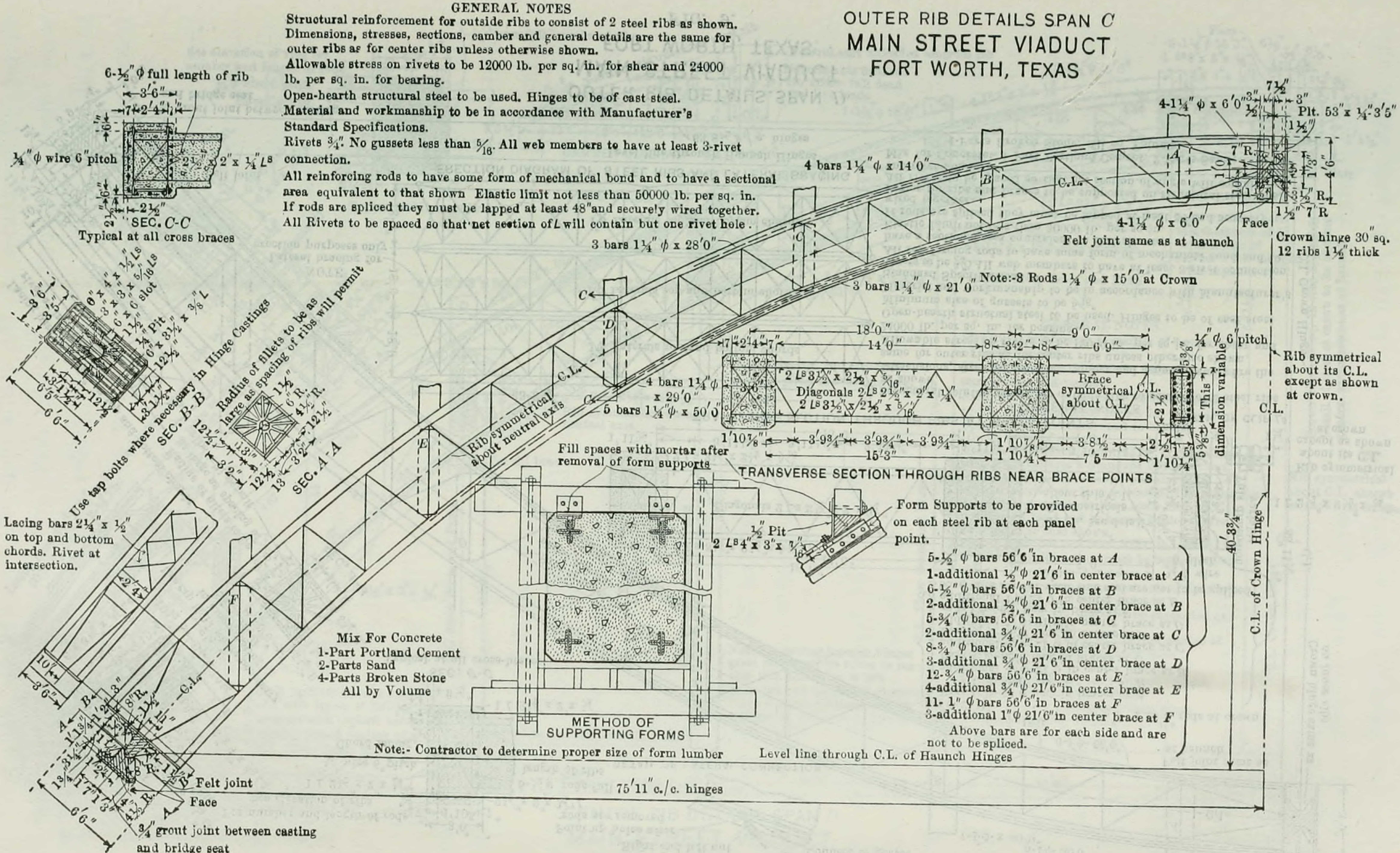
FORT WORTH, TEXAS

FIG. 8.



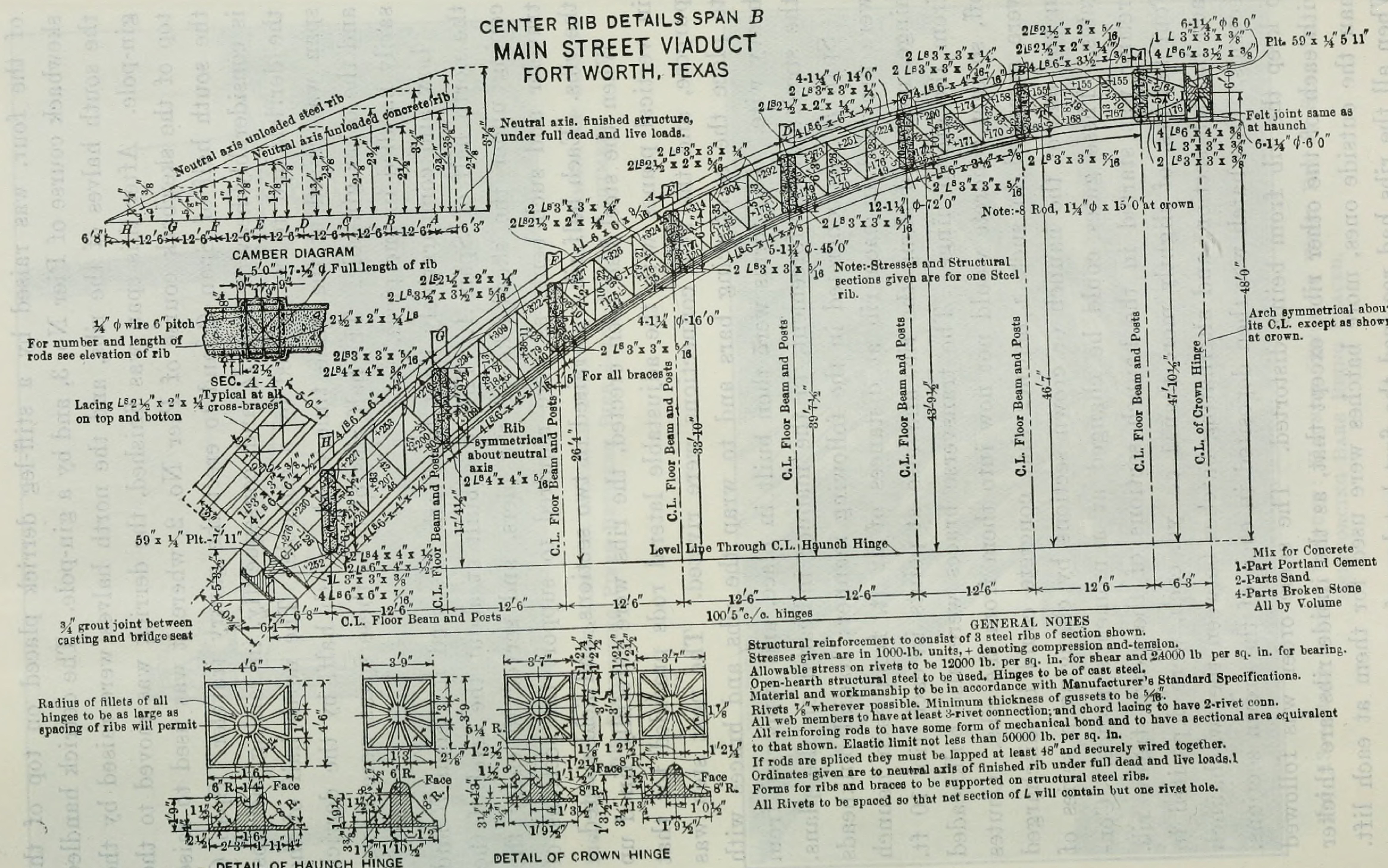
OUTER RIB DETAILS SPAN *D*
MAIN STREET VIADUCT
FORT WORTH, TEXAS.

FIG. 9.



OUTER RIB DETAILS SPAN C
MAIN STREET VIADUCT
FORT WORTH, TEXAS
GENERAL NOTES
Structural reinforcement for outside ribs to consist of 2 steel ribs as shown.
Dimensions, stresses, sections, camber and general details are the same for outer ribs as for center ribs unless otherwise shown.
Allowable stress on rivets to be 12000 lb. per sq. in. for shear and 24000 lb. per sq. in. for bearing.
Open-hearth structural steel to be used. Hinges to be of cast steel.
Material and workmanship to be in accordance with Manufacturer's Standard Specifications.
Rivets 3/4". No gussets less than 5/16. All web members to have at least 3-rivet connection.
All reinforcing rods to have some form of mechanical bond and to have a sectional area equivalent to that shown Elastic limit not less than 50000 lb. per sq. in.
If rods are spliced they must be lapped at least 48" and securely wired together.
All Rivets to be spaced so that net section of L will contain but one rivet hole.
6-1/2" ϕ full length of rib
1/4" ϕ wire 6" pitch
SEC. C-C
Typical at all cross braces
SEC. B-B
SEC. A-A
Use tap bolts where necessary in Hinge Castings
Radius of fillets to be as large as spacing of ribs will permit
Lacing bars 2 1/4" x 1/2" on top and bottom chords. Rivet at intersection.
Rib symmetrical about neutral axis
4 bars 1 1/4" ϕ x 14'0"
3 bars 1 1/4" ϕ x 28'0"
3 bars 1 1/4" ϕ x 21'0"
4 bars 1 1/4" ϕ x 29'0"
5 bars 1 1/4" ϕ x 50'0"
4-1 1/4" ϕ x 6'0"
Plt. 53" x 1/4" 3'5"
Face
Felt joint same as at haunch
Crown hinge 30" sq. 12 ribs 1 1/2" thick
Note:-8 Rods 1 1/4" ϕ x 15'0" at Crown
Rib symmetrical about its C.L. except as shown at crown.
Brace symmetrical about C.L.
This dimension variable
TRANSVERSE SECTION THROUGH RIBS NEAR BRACE POINTS
Fill spaces with mortar after removal of form supports
Form Supports to be provided on each steel rib at each panel point.
5-1/2" ϕ bars 56'6" in braces at A
1-additional 1/2" ϕ 21'6" in center brace at A
6-1/2" ϕ bars 56'6" in braces at B
2-additional 1/2" ϕ 21'6" in center brace at B
5-3/4" ϕ bars 56'6" in braces at C
2-additional 3/4" ϕ 21'6" in center brace at C
8-3/4" ϕ bars 56'6" in braces at D
3-additional 3/4" ϕ 21'6" in center brace at D
12-3/4" ϕ bars 56'6" in braces at E
4-additional 3/4" ϕ 21'6" in center brace at E
11-1" ϕ bars 56'6" in braces at F
3-additional 1" ϕ 21'6" in center brace at F
Above bars are for each side and are not to be spliced.
Mix For Concrete
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume
METHOD OF SUPPORTING FORMS
Note:- Contractor to determine proper size of form lumber
Level line through C.L. of Haunch Hinges
C.L. of Crown Hinge
Felt joint
Face
75'11" c./c. hinges
3/4" grout joint between casting and bridge seat

FIG. 10.



CENTER RIB DETAILS SPAN B
MAIN STREET VIADUCT
FORT WORTH, TEXAS
Neutral axis unloaded steel rib
Neutral axis unloaded concrete rib
Neutral axis, finished structure, under full dead and live loads.
CAMBER DIAGRAM
Full length of rib
1/4" ϕ wire 6" pitch
For number and length of rods see elevation of rib
SEC. A-A
Lacing L^s 2 1/2" x 2" x 1/4 on top and botton
Typical at all cross-braces
Rib symmetrical about neutral axis
For all braces
59" x 1/4" Plt.-7'11"
3/4" grout joint between casting and bridge seat
C.L. Floor Beam and Posts
Level Line Through C.L. Haunch Hinge
100'5" c./c. hinges
Note:-Stresses and Structural sections given are for one Steel rib.
Note:-8 Rod, 1 1/4" ϕ x 15'0" at crown
Felt joint same as at haunch
Arch symmetrical about its C.L. except as shown at crown.
C.L. of Crown Hinge
Plt. 59" x 1/4" 5'11"
Mix for Concrete
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume
Radius of fillets of all hinges to be as large as spacing of ribs will permit
DETAIL OF HAUNCH HINGE
DETAIL OF CROWN HINGE
GENERAL NOTES
Structural reinforcement to consist of 3 steel ribs of section shown.
Stresses given are in 1000-lb. units, + denoting compression and - tension.
Allowable stress on rivets to be 12000 lb. per sq. in. for shear and 24000 lb per sq. in. for bearing.
Open-hearth structural steel to be used. Hinges to be of cast steel.
Material and workmanship to be in accordance with Manufacturer's Standard Specifications.
Rivets 7/8" wherever possible. Minimum thickness of gussets to be 5/16".
All web members to have at least 3-rivet connection; and chord lacing to have 2-rivet conn.
All reinforcing rods to have some form of mechanical bond and to have a sectional area equivalent to that shown. Elastic limit not less than 50000 lb. per sq. in.
If rods are spliced they must be lapped at least 48" and securely wired together.
Ordinates given are to neutral axis of finished rib under full dead and live loads.
Forms for ribs and braces to be supported on structural steel ribs.
All Rivets to be spaced so that net section of L will contain but one rivet hole.

FIG. 11.

of the four, was raised by a stiff-leg derrick placed on top of the skewback course of Pier No. 3, and by a gin-pole. The derrick handled the south halves of the ribs, and the north halves were raised by the gin-pole. After this span was finished, the derrick was moved to the top of the skewback course of Pier No. 2, where it was used to raise the south half of Span *C*, and to erect the gantry traveler. Span *C* is considerably heavier than *D*, and, consequently, in raising the ribs, the derrick had to be assisted by a gin-pole. The north half of the span was raised at the same time as the south half, by the derrick and the other gin-pole. The half ribs of both these spans were necessarily handled in one piece.

Low and comparatively light falsework was used for Span *B* and the north half of Span *A*. This was built at about the level of the coping under the skewbacks of the piers, and served to carry the traveler and such bents as were required to support the sections of the ribs. Each half rib was raised in two sections, and field-riveted.

When the steel had been erected, the ribs were carefully lined up in vertical planes with the adjustable lateral rods provided for that purpose, and the field connections were riveted. The next step was to place the reinforcing bars and to wrap the ribs and braces with heavy wire. The forms were then built in place, and supported from the structural reinforcement, in the manner indicated on the plans.

Span *D* was concreted in the following manner: Radial bulkheads were provided in each rib at distances of 22 ft. from each haunch hinge, measured along the rib, and also at points in each rib, 10 ft. from the crown hinge. The transverse braces were also bulkheaded off, so that concrete would not flow into them from the ribs. Chutes were arranged in such a manner that concrete could be discharged in any one of the haunch or crown sections by changing a series of gates. These gates could be changed at a moment's notice. Concreting was started in the haunch sections of the down-stream rib. Four batches, of slightly more than 1 cu. yd. each, were deposited in each haunch section, and alternating batches between the two haunch sections. One batch was then deposited in each of the crown sections, to keep the rib from being distorted. The same order was followed with each of the other ribs, except that, as the inside ribs are thicker than the outside ones, more batches were used for them at each lift. When all the ribs had received the first batches of concrete, as noted,

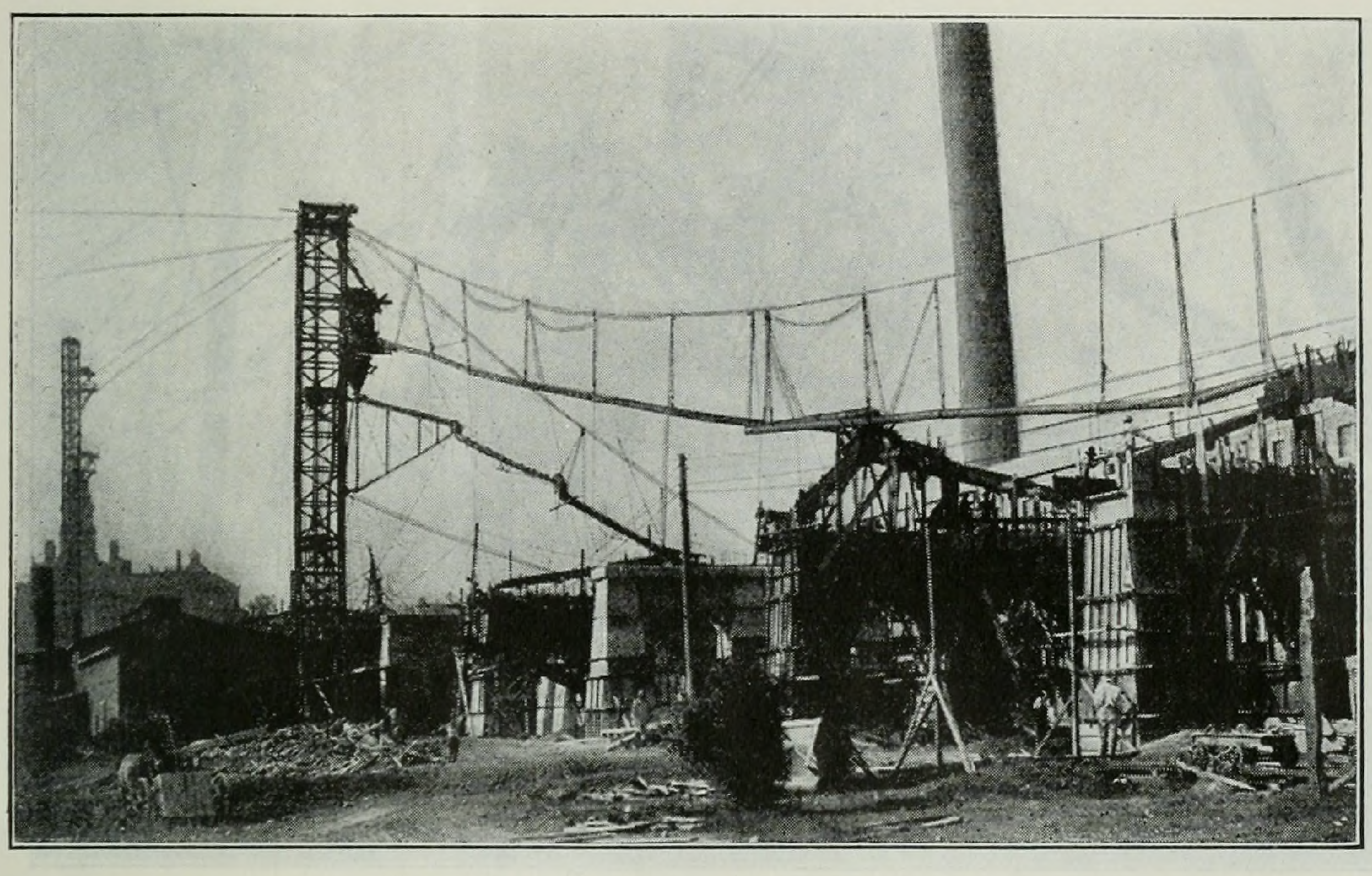


Fig. 12.—Main Street Viaduct: Movable Tower and Arrangement of Chutes for Concreting Piers Between Pier 4 and North Abutment.

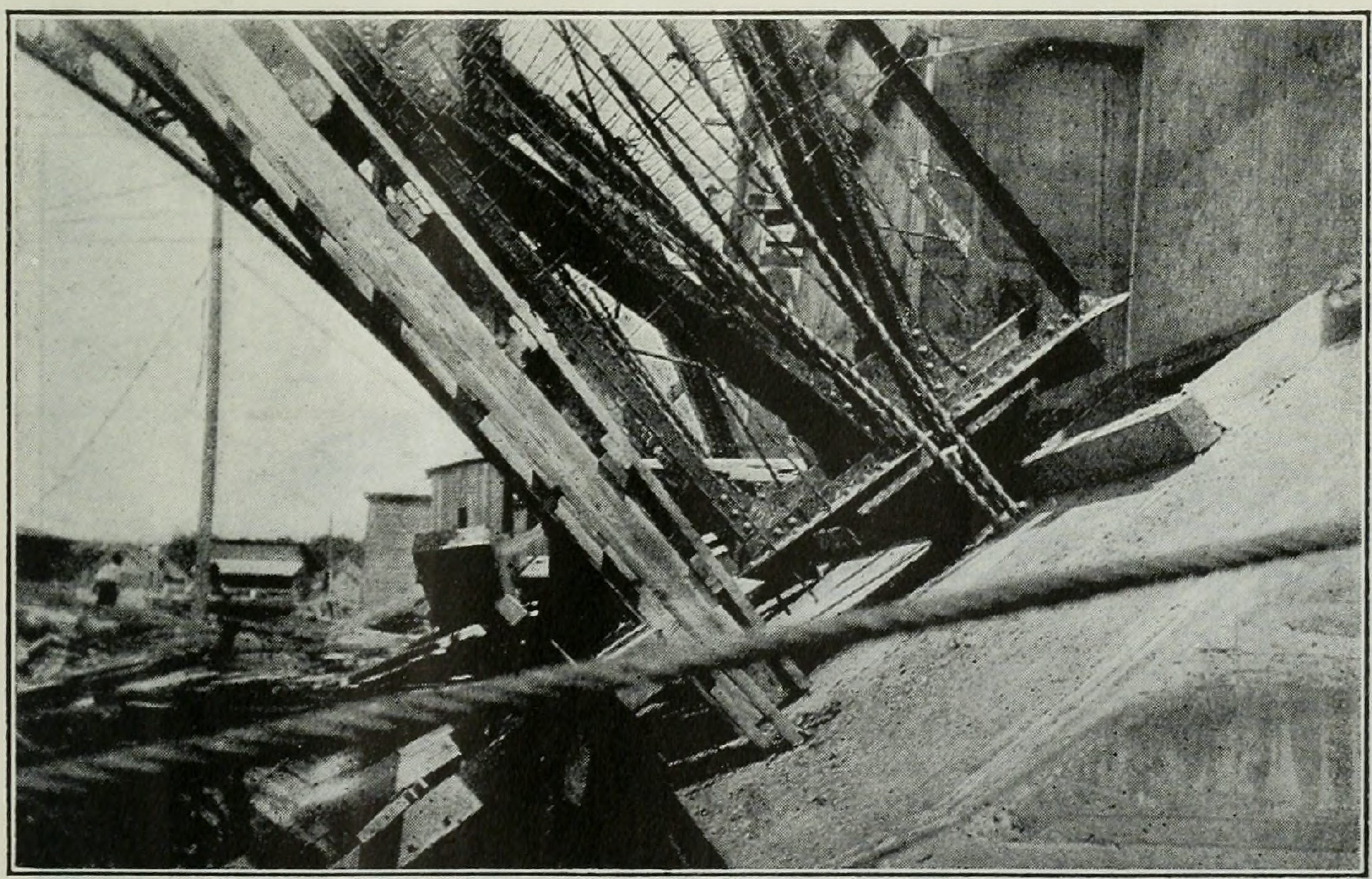


Fig. 13.—Main Street Viaduct: Haunch Hinge of West Rib of Span D. Reinforcing Bars Being Placed.

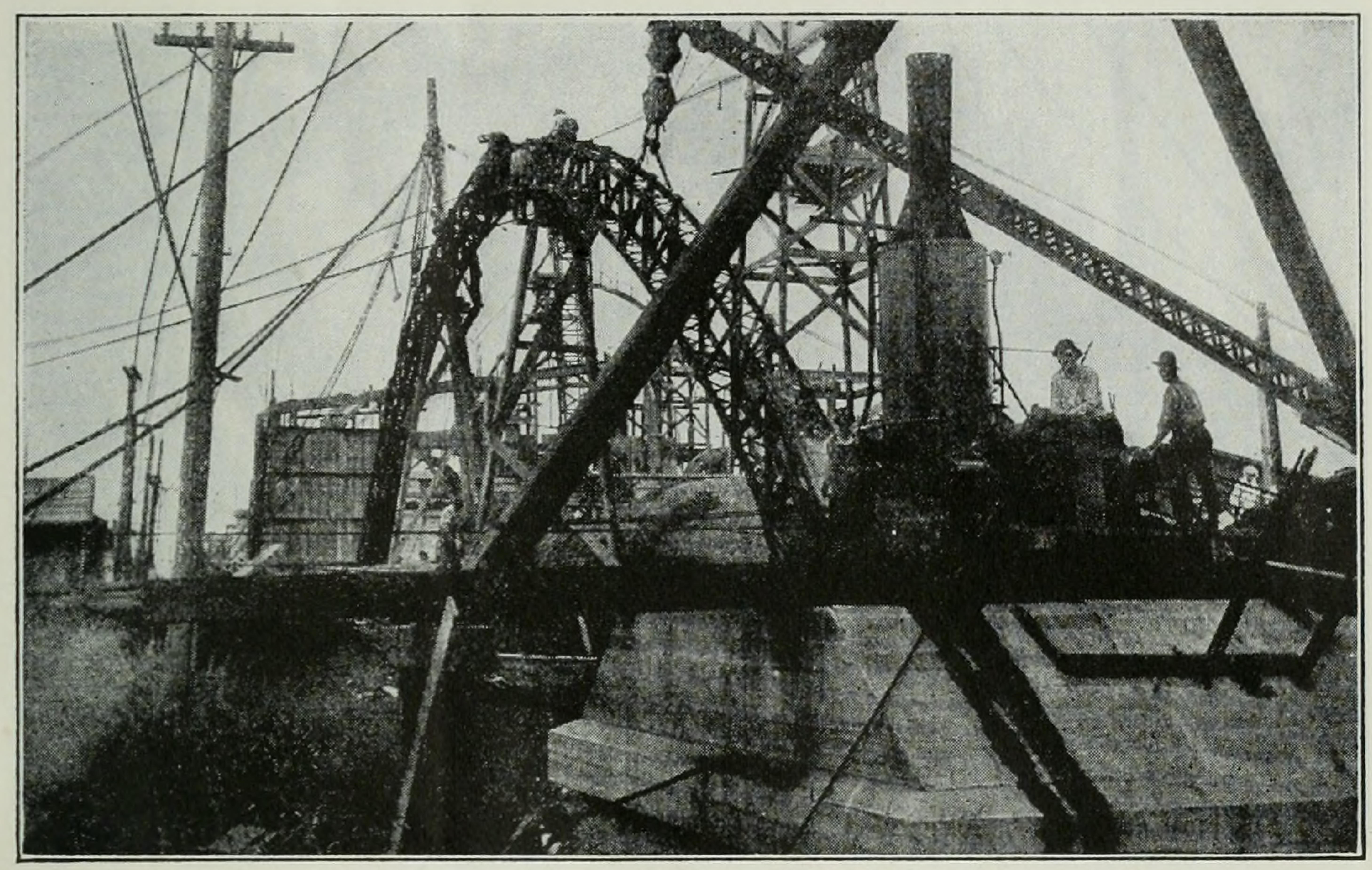


Fig. 14.—Main Street Viaduct: Raising of First Structural Rib. West Rib of Span D Raised with Derrick and Gin-Pole.

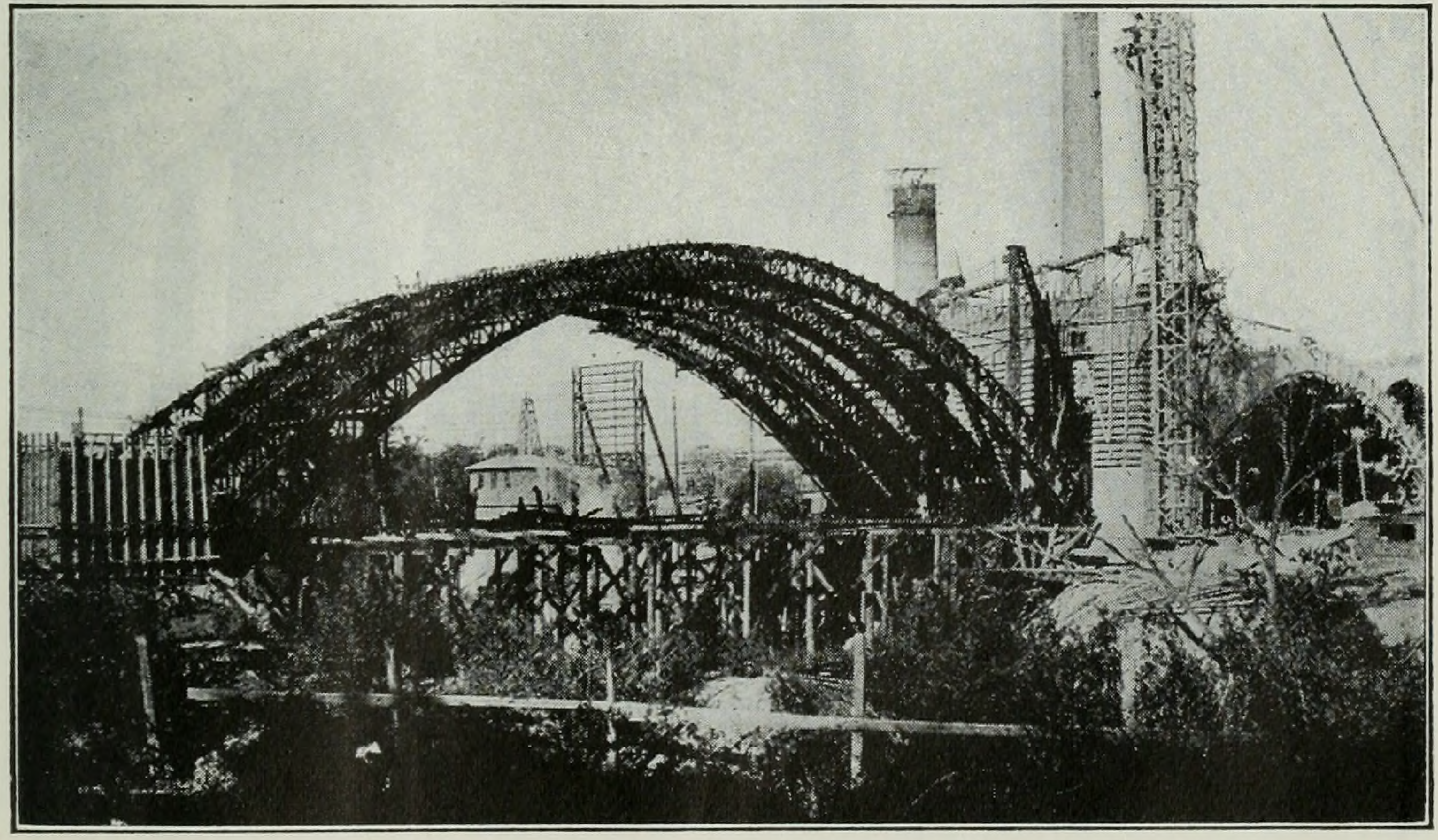


Fig. 15.—Main Street Viaduct, Span B: Steel Arches Erected.

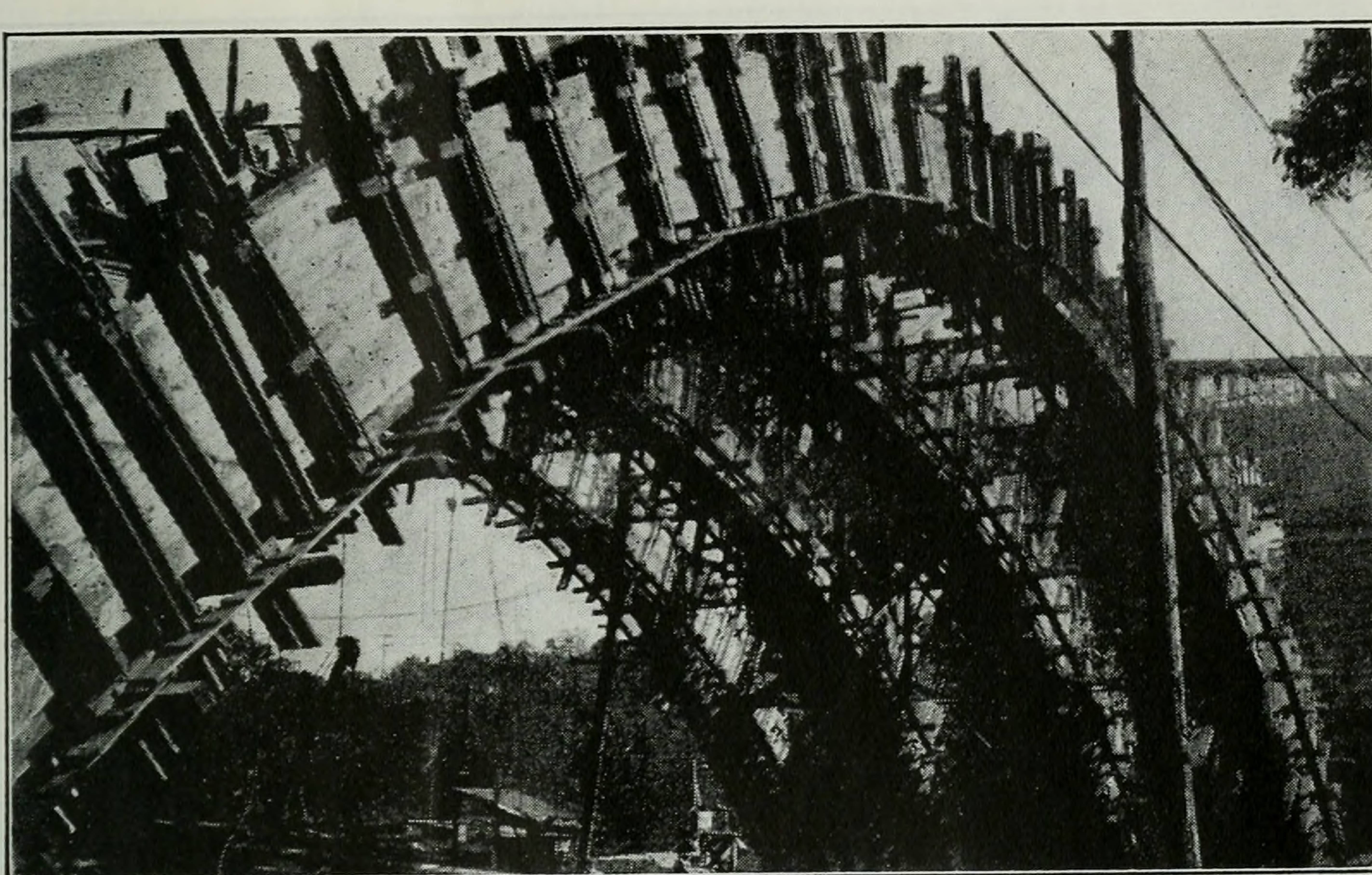


FIG. 16.—MAIN STREET VIADUCT, SPAN C: SHOWING RIBS COMPLETELY FORMED.

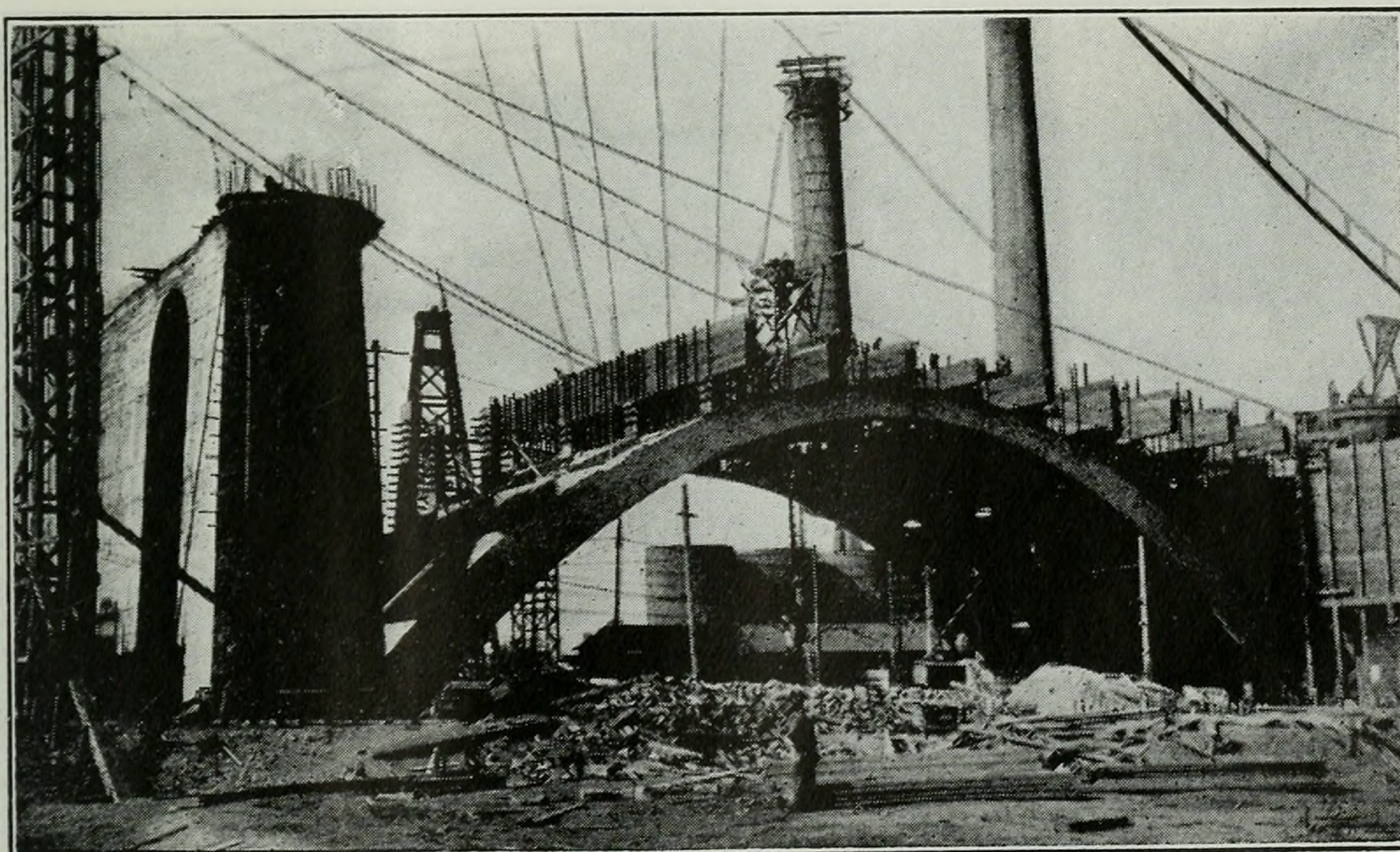


FIG. 17.—MAIN STREET VIADUCT: FORMING DECK OF SPAN C.

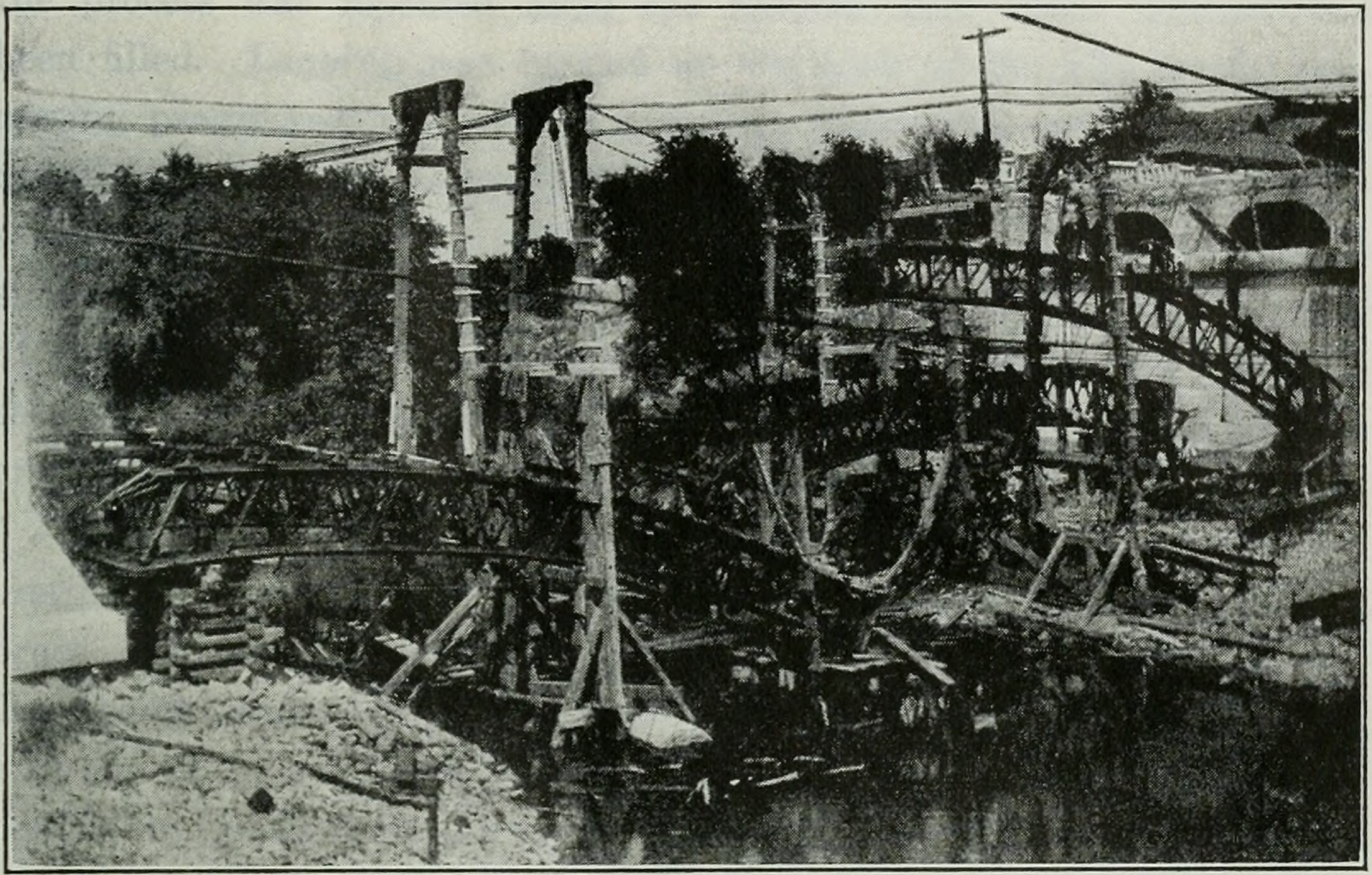


Fig. 18.—West Seventh Street Viaduct: Raising East Half of South Rib of Arch Before the High Water.

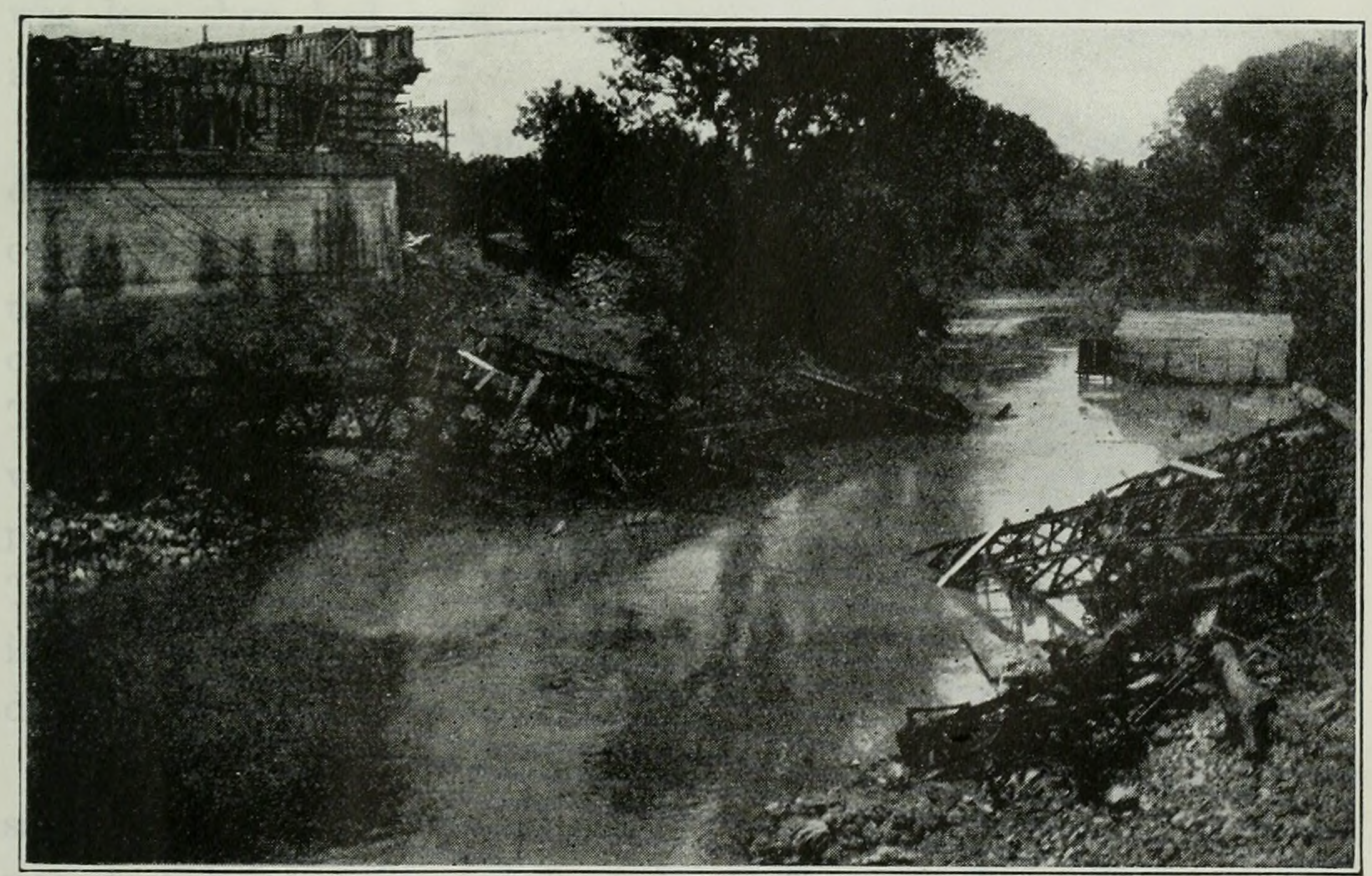


Fig. 19.—Arch Ribs After the Water Went Down, West Seventh Street Viaduct.

the process was repeated until the haunch and crown sections had been filled. Lagging was carried up the back of the ribs as the concreting progressed, to hold the concrete, which was necessarily mixed quite wet, in order to work well around the reinforcement. After setting for about 4 days, the bulkheads in the ribs were removed, the surface of the concrete was roughened up, and the closure sections of the ribs were concreted in about the same manner as the haunch sections. The transverse braces were not concreted until the rib concrete had set for several days.

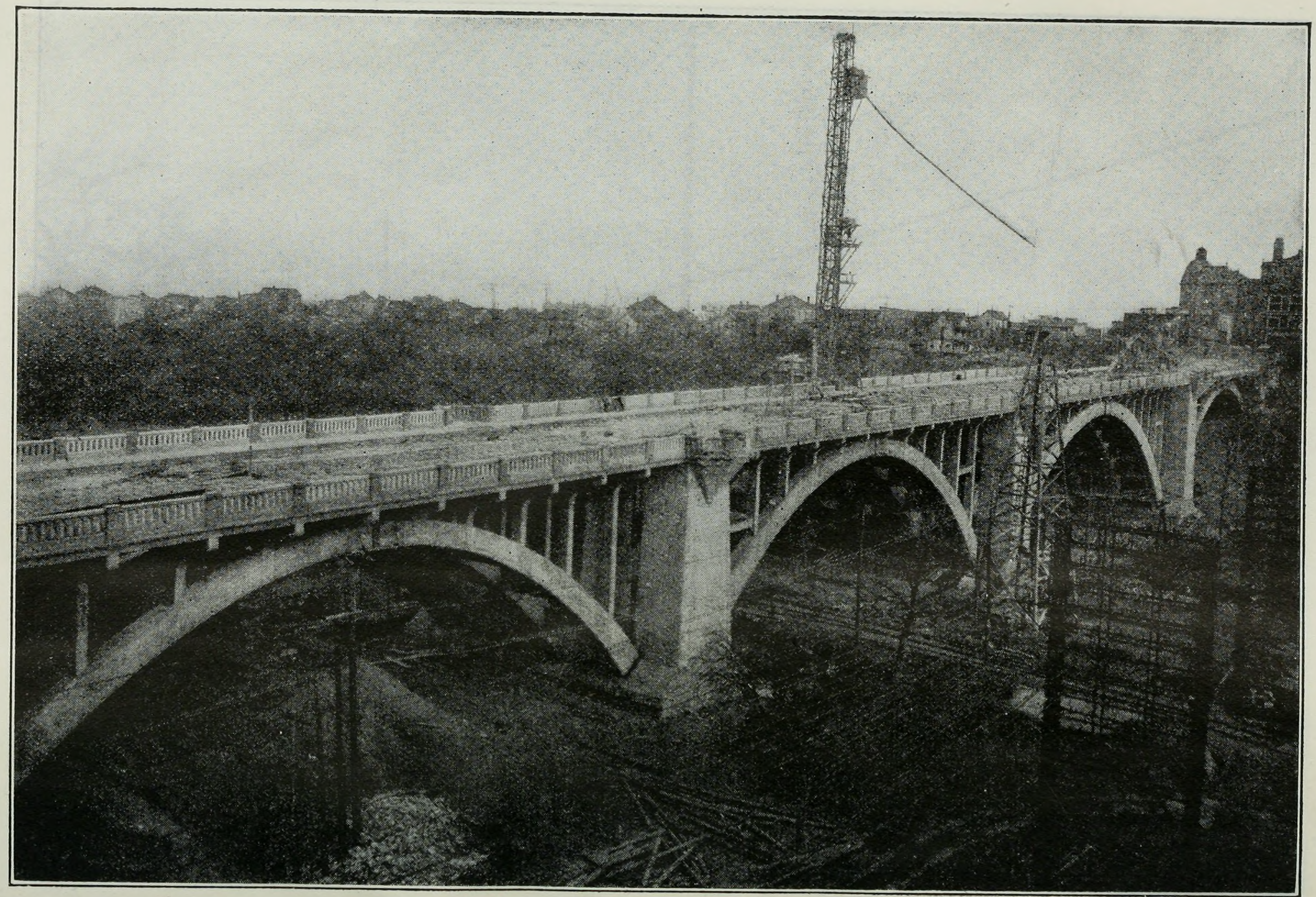
In order to keep track of the distortion of the ribs while concreting was in progress, sash weights were suspended from the steel brace reinforcement close to each rib, at four points along the rib. One weight was placed at the brace on each side of the crown hinge, and one at each quarter point of the span. These weights were suspended by small wires, and carried scales which were wired to the weights. A stake was driven in the ground alongside of each weight, and a nail in the stake acted as a pointer on the scale. The initial reading was taken at each point, and another set of readings was made after each round of concreting. At first a slight settlement of the crown points was observed, followed by a slight rise. The distortion was less than had been anticipated, amounting to a maximum of about ¼ in.

The concrete in the ribs and braces was allowed to set for several days before the work of building the forms for the posts and deck of the span was begun. This portion of the work was concreted in the manner indicated under the head of "Loading". The posts were concreted, beginning at the haunches and working toward the crown. Then the deck was concreted from the crown back toward the haunches, with the exception of the middle section, over the crown, which was placed last, in order to provide for the expansion joint at the crown. The piers, in the meantime, had been finished to their full height, in order to insure stability and provide for the connection of the deck to the piers.

The concreting of the other spans was handled in practically the same manner as that just described. When the concreting of a section was started, it was carried through continuously, by working day and night if necessary, so that the work was made as nearly monolithic as possible.

The plant and equipment on the other viaducts was similar to that used at Main Street, but, of course, on a smaller scale. All four jobs were under construction at the same time, and were finished on or before the last of March, 1914.

Difficulties Encountered.—There was some difficulty, at Main Street, in the early stages of the work, in sinking the foundations of the main piers. Pier No. 1 gave the most trouble. The unstable character of the soil on the south side of the stream was responsible for this. Extra heavy timber sheet-piling and a large number of heavy shores of timber and steel were used. In order to give the material on the hillside south of the pier the least possible chance to slide, the excavation was made in four sections. The first section excavated was the down-stream quarter of the pier. This excavation was about 20 ft. in length, measured up and down stream. After rock was reached, the surface was carefully cleaned off and several drill holes were put down to test the rock. The surface of the rock was found to be very rough, with a slight drop in elevation away from the stream. The drill holes showed that the layers of rock were very thick and had only minute seams between them. There had been some fear that the pressure of the bank behind the pier might cause the pier to slide on the rock, before sufficient weight could be added to prevent it. With this in mind, preparations were made to cut a dowel in the rock so as to bond the pier and the rock together. The fear of sliding was dispelled when the extremely rough character of the rock was seen, and the dowel scheme was abandoned. The pier section was concreted up to the top of the 13 ft. 6-in. footing course, and the next section, which was the third quarter from the down-stream end, was excavated and concreted to the same height as the first. The up-stream quarter was next finished, and lastly the second quarter from the down-stream end. Of course, all longitudinal reinforcing bars in the portion of the pier below the top of the footing course, had to be spliced, on account of the method of construction adopted. Care was taken to bond the sections of the footing course together by vertical re-entrant joints extending the full height of the 13 ft. 6-in. course. Above the top of this course the pier was built as a monolith in the usual manner. In concreting the footing sections it was not thought advisable to remove all the shores as the concrete was carried up. Consequently, some of the steel members from the



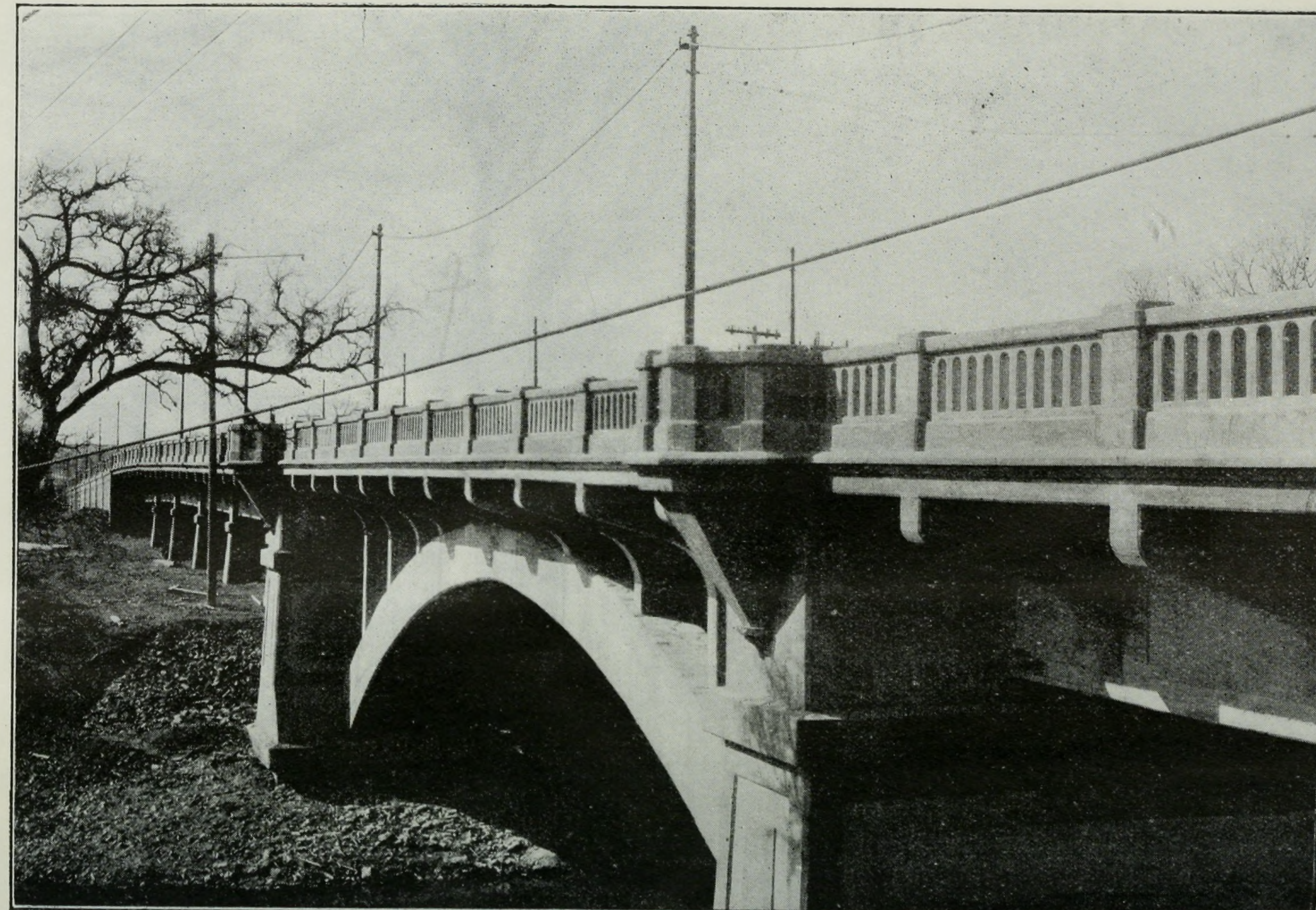


FIG. 21.—WEST SEVENTH STREET VIADUCT.

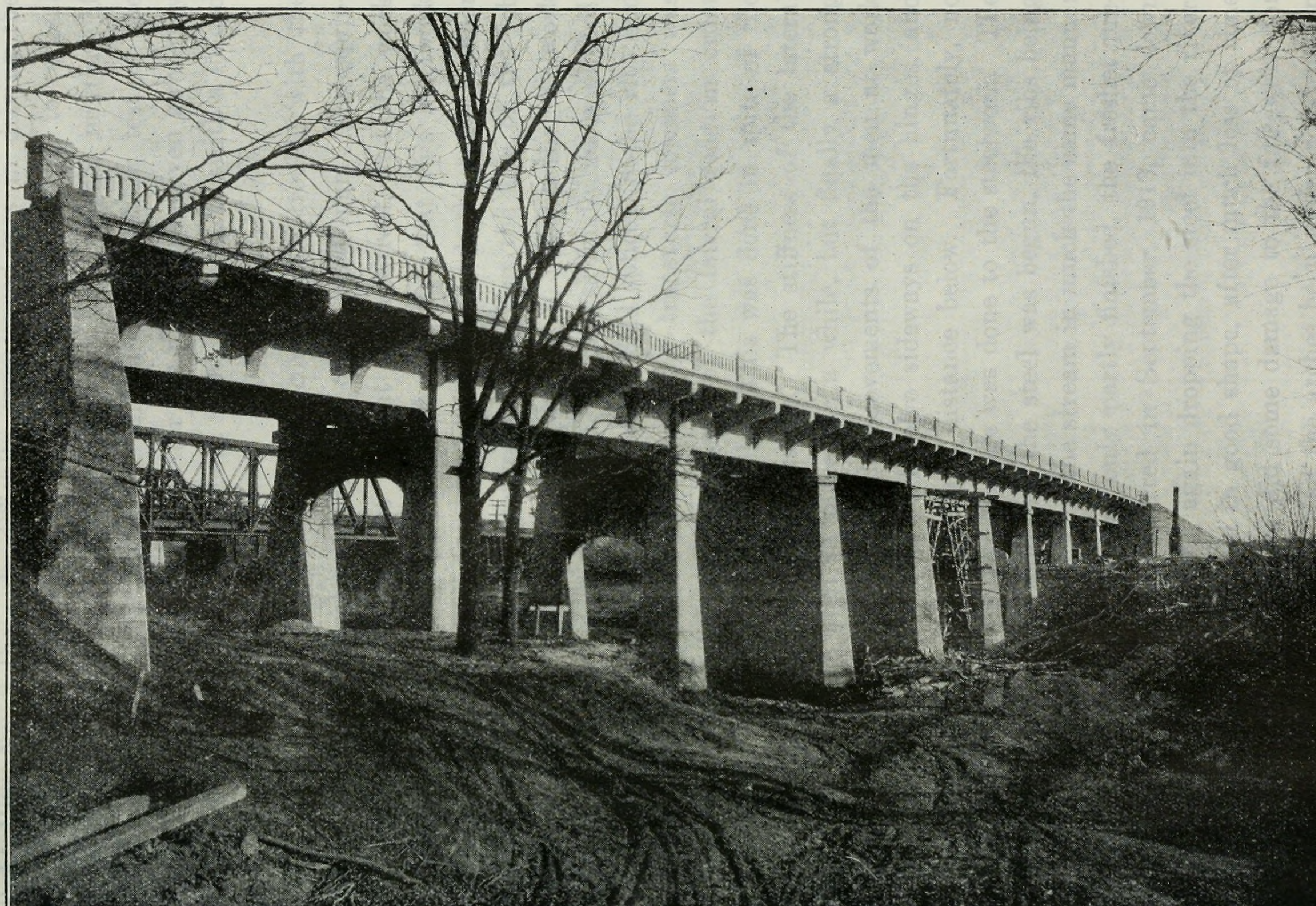


FIG. 22.—SAMUELS AVENUE VIADUCT

old bridge, which had been used for shoring, were built into the pier and left.

At Pier No. 2 the only difficulty experienced was in driving the oak piles. These were about 20 ft. long, and were driven to refusal by using a drop-hammer. Driving through the compact mixture of sand and gravel at this point was difficult. A bearing test of the material in the bottom of the pit after the piles were driven gave very little settlement under a load of about 8 000 lb. per sq. ft., with a loaded area of about 4 sq. ft.

At West Seventh Street there was considerable trouble with the erection of the structural reinforcement of the arch span. The equipment was not as good as that used at Main Street, and the work progressed slowly. The engineers had expected that this steelwork would be handled by derricks on the piers in much the same manner as at Main Street. The sub-contractor felt that the size of the job did not warrant this equipment, and decided to crib up the steelwork on falsework in the bed of the stream. This was finally accomplished, and the falsework was removed, after the field splices in the ribs had been riveted. While the riveting of the transverse members was under way, the erector removed some of the lateral rods in order to get at the rivets with less trouble. This was done in spite of the protest of the engineers' representative. The stiffness of the lateral connections held the arch in place for a while, but finally a strong wind, aided to some extent by the movements of the men at work on the span, caused the ribs to revolve sideways on the hinges, and then drop into the stream a short distance below. Fortunately, no one was killed, and but slight damage was done to the steelwork. The work of re-erecting and repairing the steel was begun, the ribs being supported on timber falsework in the stream in much the same manner as before. When the work was only partly finished, the freshet previously mentioned, which occurred in September, 1913, came down and washed out the falsework, again dropping the steel into the river. The work was finally replaced in good shape, after much loss of time and money. This freshet also did some damage to the falsework for the traveler at Main Street. Had the arches been under construction on falsework in the usual manner, much more serious damage would undoubtedly have resulted.

Figs. 12 to 22 show the work at various stages, and are self-explanatory.

Cost of Viaducts.—Tables 1 to 5 give the quantities for each of the viaducts, and the unit prices bid by the successful contractor. They also give the cost per linear foot of each structure, the cost per square foot of horizontal and of vertical projection, and the cost per cubic foot. The cost per square foot of vertical projection is based on the area of projection of each structure between the finished

TABLE 1.—Cost of Main Street Viaduct.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading........................	4 720 cu. yd.	$0.35	$1 652.00	0.43
2	Foundation excavation........	15 227 " "	1.00	15 227.00	3.96
3	Rock excavation...............	444 " "	2.00	888.00	0.23
4	(*a*) Concrete No. 1 (1:2:4).....	10 611 " "	10.25	108 762.75	28.16
	(*b*) Concrete No. 2 (1:2½:5)...	14 880 " "	6.85	101 928.00	26.40
	(*c*) Concrete No. 3 (1:3:6)......	438 " "	6.25	2 737.50	0.71
5	Railings........................	3 875 lin. ft.	2.00	7 750.00	2.01
6	Structural steelwork...........	1 537 400 lb.	0.05	76 870.00	19.90
7	Steel reinforcing bars.........	1 375 150 "	0.035	48 130.25	12.46
8	Steel castings..................	205 460 "	0.07	14 382.20	3.72
9	Iron castings...................	11 173 "	0.04	446.92	0.11
10	Anchor-bolts and T. P. casings.	21 311 "	0.06	1 278.66	0.33
11	Steel dowels....................	98	3.00	294.00	0.08
13	Rip-rap.........................	836 cu. yd.	1.50	1 254.00	0.32
14	Manholes.......................	2	50.00	100.00	0.03
15	Removing old bridge..........	....	....	2 500.00	0.65
17	Timber piles....................	194	10.00	1 940.00	0.50
	Total			$386 141.28	100.00

TABLE 2.—Cost of West Seventh Street Viaduct.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading........................	5 424 cu. yd.	$0.25	$1 356.00	1.22
2	Foundation excavation........	5 570 " "	0.61	3 397.70	3.07
3	Rock excavation...............	132 " "	3.00	396.00	0.36
4	(*a*) Concrete No. 1 (1:2:4).....	2 890.4 " "	11.82	34 164.53	30.90
	(*b*) Concrete No. 2 (1:2½:5)...	5 138 " "	7.25	37 250.50	33.69
	(*c*) Concrete No. 3 (1:3:6).....	440 " "	6.15	2 706.00	2.45
5	Railings........................	2 120 lin. ft.	2.25	4 770.00	4.31
6	Structural steelwork..........	196 975 lb.	0.0475	9 356.31	8.46
7	Steel reinforcing bars.........	389 089 "	0.029	11 283.58	10.21
8	Steel castings..................	45 628 "	0.058	2 646.42	2.39
9	Iron castings...................	4 470 "	0.03	134.10	0.12
10	Anchor-bolts and T. P. casings.	7 063 "	0.047	331.96	0.30
11	Steel dowels....................	30	1.50	45.00	0.04
12	Sidewalks on earth fill........	5 240 sq. ft.	0.21	1 100.40	1.00
13	Rip-rap.........................	363 cu. yd.	2.00	726.00	0.66
14	Manholes.......................	4	38.00	152.00	0.14
15	Removing old bridge..........	....	....	750.00	0.68
	Total			$110 566.50	100.00

TABLE 3.—COST OF SAMUELS AVENUE VIADUCT.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	1 111 cu. yd.	$0.30	$333.30	0.58
2	Foundation excavation	2 739.6 " "	1.00	2 739.60	4.78
3	(a) Concrete No. 1 (1 : 2 : 4)	1 237.2 " "	12.00	14 846.40	25.91
	(b) Concrete No. 2 (1 : 2½ : 5)	3 413.6 " "	7.50	25 602.00	44.68
4	Railings	906 lin. ft.	2.00	1 812.00	3.16
5	Structural steelwork	12 739 lb.	0.05	636.95	1.11
6	Steel reinforcing bars	247 755 "	0.03	7 432.65	12.97
7	Iron castings	3 420 "	0.04	136.80	0.24
8	Anchor-bolts and T. P. casings.	6 388 "	0.06	383.28	0.67
9	Rip-rap	690.25 cu. yd.	2.00	1 380.50	2.41
10	Timber piles	200	10.00	2 000.00	3.49
	Total			$57 303.48	100.00

TABLE 4.—COST OF EAST FOURTH STREET VIADUCT.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	4 107 cu. yd.	$0.40	$1 642.80	3.11
2	Foundation excavation	2 832 " "	1.00	2 832.00	5.37
3	(a) Concrete No. 1 (1 : 2 : 4)	1 236 " "	12.00	14 832.00	28.11
	(b) Concrete No. 2 (1 : 2½ : 5)	2 734.5 " "	7.50	20 508.75	38.87
4	Railings	905 lin. ft.	2.00	1 810.00	3.43
5	Structural steelwork	12 739 lb.	0.05	636.95	1.20
6	Steel reinforcing bars	238 061 "	0.03	7 141.83	13.54
7	Iron castings	3 420 "	0.04	136.80	0.26
8	Anchor-bolts and T. P. casings.	6 388 "	0.06	383.28	0.73
9	Rip-rap	907 cu. yd.	2.00	1 814.00	3.44
10	Timber piles	82	10.00	820.00	1.56
11	Removing existing piers	....	......	200.00	0.38
	Total			$52 758.41	100.00

TABLE 5.—SUMMARY OF COST OF FOUR VIADUCTS.

	Main Street.	West Seventh Street.	Samuels Avenue.	East Fourth Street.
Cost per linear foot of structure	$244.60	$122.45	$141.29	$130.59
Cost per square foot of horizontal projection	3.66	2.10	3.28	3.02
Cost per square foot of vertical projection	6.34	6.93	3.53	3.60
Cost per cubic foot of volume between finished ground line and crown of roadway	0.091	0.119	0.082	0.083
Total estimated cost, including paving, lighting, and engineers' fees	$428 882.00	$127 472.00	$63 584.00	$58 766.00

ground line and the top of the roadway. For the horizontal projection, the extreme width over the copings or stringer mouldings was taken. In computing the cost per cubic foot, the volume included between vertical planes through the copings, and between the finished ground line and the top of the roadway, was taken. The total, on which the above unit costs are based, is the cost of the structure based on the contractor's unit prices and the quantities as given in the tables, to which has been added the cost of the paving, lighting, and engineers' fees.

Personnel.—The work was designed and its execution supervised by Messrs. Brenneke and Fay, Consulting Engineers, of St. Louis, Mo. The writer, as Principal Assistant Engineer for that firm, was responsible for the design of the viaducts, and exercised executive supervision over the work. L. H. Faidley, Jun. Am. Soc. C. E., had charge of the detailed work of preparing the plans, and William Holden, Jun. Am. Soc. C. E., was in direct charge as Resident Engineer.

The contract for the Main Street Viaduct was awarded to the Hannan-Hickey Brothers Construction Company, of St. Louis. The Tarrant Construction Company, of Fort Worth, had the contract for the West Seventh Street Viaduct, and the other two structures were awarded to the McKensie-Williams Construction Company, of Webb City, Mo. The Virginia Bridge and Iron Company, of Roanoke, Va., were the sub-contractors for the structural steelwork of the Main and West Seventh Street structures, and erected the steelwork at Main Street. Austin Brothers, of Dallas, Tex., were the sub-contractors for the erection of the steelwork at West Seventh Street.

In conclusion, the writer wishes to express his thanks to Mr. Faidley, and to Mr. Holden for valuable assistance in the preparation of this paper.

DISCUSSION

Carl Gayler,* M. Am. Soc. C. E. (by letter).—It is not often that important structures such as these concrete viaducts are described as fully and clearly as in this paper. Publications of this kind are of great value to the Profession. Mr. Gayler.

Reinforced concrete viaducts of the general layout and appearance of the four Fort Worth viaducts are not uncommon, but the construction of arches by using steel centers embedded in the concrete of the arches and forming an integral part of them is of recent date.

At first glance, comparison with the Melan arch suggests itself; there are, however, the following radical differences between the two systems:

1. The steel ribs of the Melan arch are not designed to be self-sustaining over the full length of the span under the weight of the concrete of the arch proper.

2. The usual assumption for the computation of Melan arches—that the concrete resists the direct thrust and the steel ribs take up the bending moment—is, considering the distance between the steel ribs, open to criticism. On the other hand, the suppositions under which the arches of the Fort Worth viaducts were calculated are definite and logical, and may be summarized as follows: The steel ribs alone carry, in the first stage of the work, their own weight, that of the concrete as it is being poured into the forms around them, and that of the lateral braces. The stresses thus produced in the steelwork can be accurately ascertained; the concrete is merely dead weight, and the question of the difference of the moduli of elasticity of the two materials is not involved. The final stresses in the arch ribs resulting from the additional weight of the superstructure, and from the live loads, are provided for as in other reinforced concrete arches, the steelwork of the ribs forming part of the steel reinforcement, as fully explained by Mr. Bowen.

3. The distribution of the metal through the body of the concrete in the ribs of the Fort Worth arches is far superior to that in any Melan arch ever built.

After careful consideration of the problem with which the consulting engineers had to deal, that is, to build, safely and economically, the arches of the viaducts under the difficult conditions of a limited clearance, the probability of sudden and extraordinary floods, and an unstable river bed, the writer is convinced that the plan adopted was the one logical solution of the problem. Under analogous conditions, it will undoubtedly be repeated.

The conclusion, however, which Mr. Bowen has reached, that "for high structures, and for those over streams subject to sudden and

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Mr. Gayler. great variation of water level, this method is cheaper and safer than to use falsework supported from the bed of the stream", needs modification, inasmuch as, in the case of high structures, this method has to compete, not so much with ordinary falsework, as with steel centers placed below the concrete.

Where the height of the arch above the high-water line admits of the use of "free" centers, that is, centers under the concrete, the latter method, in most cases, will be found to be the more economical.

In the first place, the height of the steel centers is then independent of the thickness of the concrete arches or arch ribs, and great saving in the weight of the steel, especially in long spans, can be obtained by choosing an economical height for the steel trusses or arches.

It is also feasible to get along with a set of centers for one (longitudinal) half of the arch only, and to utilize this set again for the other half. On a number of concrete viaducts great saving in the cost of erection has been obtained in thus using the falsework or centering over again, as for instance, on the great viaduct built a few years ago in the Duchy of Luxemburg, on the Walnut Lane Bridge, at Philadelphia, Pa., the Rock River Bridge, at Cleveland, Ohio, and the Kings Highway Viaduct, at St. Louis, Mo. Even the scrap value of the free centers is an item worth taking into account.

The saving in steel gained by utilizing the steel ribs as reinforcement of the arch will thus in most cases be greatly overbalanced if free centers are used. The arches of numerous concrete viaducts have been proportioned so that bending moments were practically eliminated and, of course, in all such cases no strengthening of the arch by the embedded steel ribs would be claimed.

Another advantage of free centers is that they are applicable to the circular as well as to the ribbed arch. The method used on the Fort Worth arches, except for short spans, is probably limited to the ribbed arch, on account of its greater available thickness.

An excellent illustration of the advantages of free centers over embedded centers, in the case of a high structure, is found in the successful erection of the Rock River Bridge, at Cleveland, Ohio, by three-hinged steel centers, designed by Wilbur J. Watson, M. Am. Soc. C. E.*

Mr. Bowen states that the object of his paper is "to bring out a discussion of the type of reinforced concrete arch in which the reinforcement is composed of structural shapes and is designed to support the weight of the forms of plastic concrete in the arch ring on ribs", but a few remarks on other features of the viaducts may be allowed.

In looking over the foundations of the piers, as shown on Plates XVIII, XIX, and XX, an impression was left on the writer's mind that the old conflict between the sincere endeavor of the engineer

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* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 1.

to obtain the best results and the barrier set up by a limited appropriation has not been decided fully in favor of the engineer, as far as the question of foundations of the piers of the Fort Worth bridges is concerned. This is written with considerable diffidence, because a thorough acquaintance with the nature of the material in the river bed alone could justify criticism. The following points, therefore, are brought out merely as suggestions, and will be touched on as briefly as possible. Mr. Gayler.

The footings of a number of the piers in the bed of the river, that is, inside of the levee protections, for instance, Pier 3 of the Main Street Viaduct, the piers of the Seventh Street Viaduct with the exception of Nos. 1 and 2, and some of the piers of the East Fourth Street Viaduct, are not carried down as deep as the river bed. For their protection against scouring, rip-rapping is relied on, which may, or may not, be properly renewed in years to come.

The base of Pier 2 of the West Seventh Street Viaduct, judging by the small-scale elevation on Plate XVIII, seems so provokingly close to rock that it may not be unreasonable to ask why this chance to obtain an unyielding foundation on both sides of a rather flat concrete arch was not utilized.

Even the problem of a solid foundation for Pier 2 of the Main Street Viaduct, carefully as it seems to have been studied by the consulting engineers, leaves some doubt in the mind whether rock might not have been reached at a reasonable price. According to Plates XVIII and XX, the 15-ft. oak piles at the north side of the pier reach down to within a few feet of it.

As to the design of the bearings of the track rails, adopted for reasons of economy, it is not likely to be repeated on other viaducts. The life of the fiber pad, ½ in. thick, which is to act as a shock absorber, will not be a long one, and the hammering of the wheels under the electric railway traffic will then be a serious matter for the slabs and girders.

In speaking of the advantages of the three-hinged, ribbed arch, it was an oversight on Mr. Bowen's part to claim that water-proofing is, in every case, a necessity for the circular arch; but, aside from this, every word he has said is true, and should be welcomed by every structural engineer. Concrete arches should either be built with hinges or efficiently reinforced against temperature stresses. Too long, for the credit of the Profession, have concrete arches been built in the United States with disregard for temperature stresses or under assumptions of changes of temperature which, on their face, are not true.

Changes of temperature in the body of concrete arches produced by long-continued heat or cold can be ascertained, either by judgment confirmed by experience, or by actual tests on existing structures.

Mr. Gayler. Whenever excessive stresses result from such changes, hinges should be used.

As the writer expressed it in a previous publication: This (that is, the assumption of inadequate changes of temperature in order to obtain convenient stresses in the arch) is the only instance on record in the history of engineering where an attempt is made to adjust the laws of Nature to the works of man, instead of *vice versa*.

It is true that in former years the three-hinged steel arch had gradually to give way to the two-hinged or to the fixed arch, on account of its lack of rigidity under moving loads, but with the heavy concrete arch this defect is, to a great extent, overcome, and should not be a serious objection to the use of the center hinge.

Mr. Bowen. S. W. BOWEN,* M. AM. SOC. C. E. (by letter).—Referring to the use of "free" centers mentioned by Mr. Gayler, these were considered for the Fort Worth viaducts, but were not adopted for the following reasons: At West Seventh Street, there was not sufficient clearance between the high-water line and the underside of the arch rib to permit of their use. At Main Street, all the spans were of different lengths, which, at most, would have permitted the use of the centers only twice, once for each half of the structure. The ribs required considerable permanent reinforcement, so that in this case it was more economical to place the centers within the ribs, and use them both as centers and reinforcement.

"Free" centers have been used to great advantage in a number of large viaducts where there were several spans of the same length, and where little or no reinforcement was required in the arch, for example, for the "Tunkhannock", and other viaducts, built by the Delaware, Lackawanna and Western Railroad Company. The first use made of these centers in the United States was for the Rocky River Bridge, at Cleveland, Ohio, mentioned by Mr. Gayler.

Pier 2, at West Seventh Street, is founded on a very dense mass of gravel and boulders. It was originally intended to place it on bed-rock, but, after an examination of the foundation, it was decided to stop at the gravel formation. The other piers were carried down to such a depth that, considering the ample bank protection, it seems highly improbable that there will be any trouble from scour. The reason for stopping the piles under Pier 2 at Main Street short of bed-rock is explained in the paper.

The arrangement for fastening the track rails did not prove to be as satisfactory as had been expected, principally on account of the difficulty of setting the anchor-bolts. This scheme was used in order to reduce the dead load on the structure.

In conclusion, the writer wishes to express his thanks to Mr. Gayler for his valuable discussion.

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