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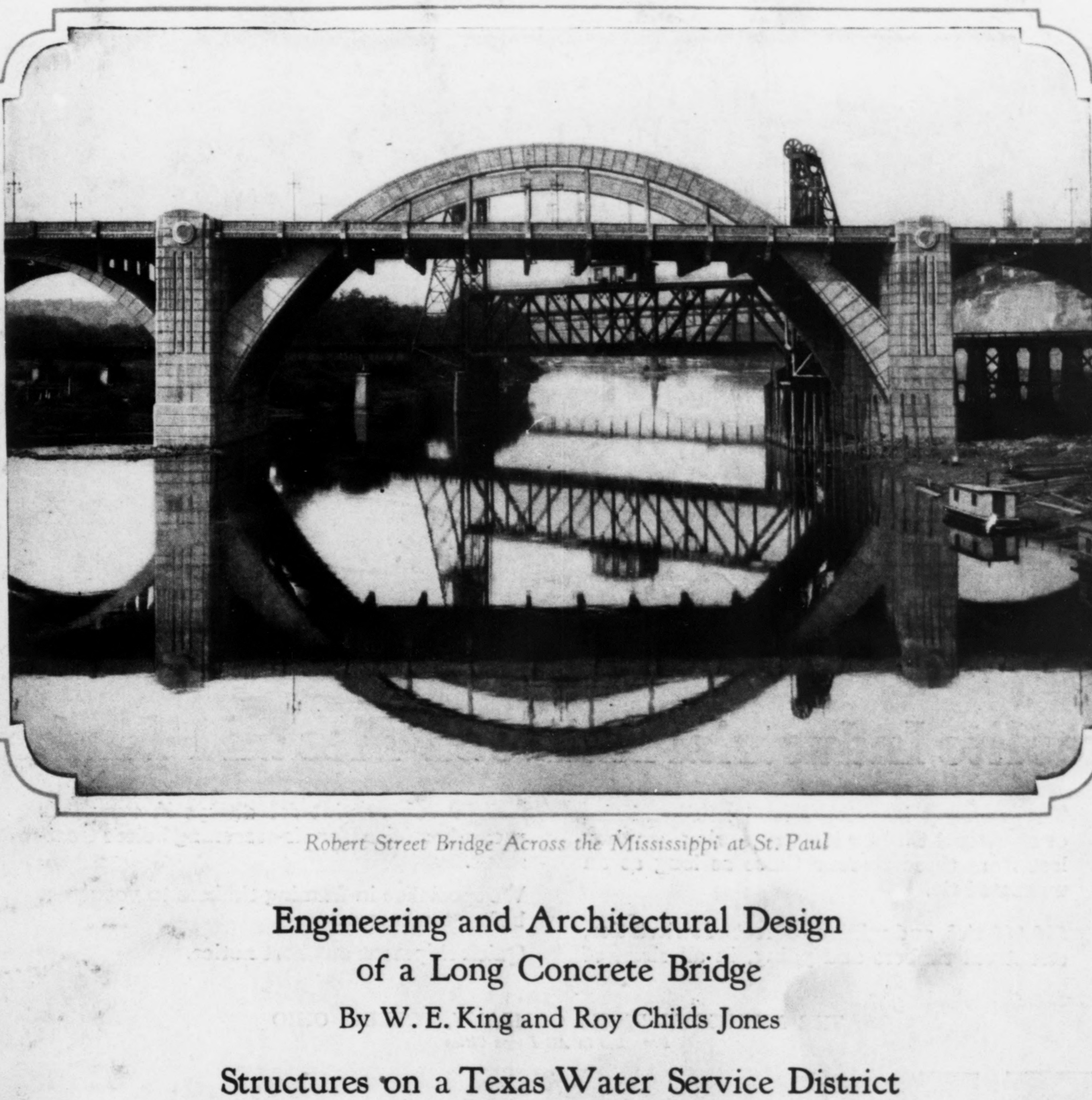

Robert Street Bridge Across the Mississippi at St. Paul

Engineering and Architectural Design of a Long Concrete Bridge

By W. E. King and Roy Childs Jones

Structures on a Texas Water Service District

By J. L. Lochridge

Straightening the Chicago River Involves Many Problems

Concrete Drydock Completed in Twelve Months

Engineering and Architectural Design of a Long Concrete Bridge

Robert St. Bridge at St. Paul Has 264-Ft. Two-Rib Arch Channel Span Flanked by Five-Rib and Barrel Arch Spans—Structural Steel in Main Spans

BY W. E. KING AND ROY CHILDS JONES
Of Toltz, King & Day, Inc., Engineers and Architects, St. Paul, Minn.

A CONSPICUOUS structure at St. Paul, Minn., is the new Robert St. bridge across the Mississippi River, which was opened for traffic Aug. 6, 1926. It is a reinforced-concrete arch structure replacing a steel bridge built in 1881, the replacement being made to secure a wider roadway. As the old bridge had a minimum width of 32 ft. between curbs, with two sidewalks 8½ ft. wide, congestion resulted largely because vehicles could not pass the street cars. The new bridge has a 56-ft. clear roadway with two 10-ft. sidewalks, the bridge being made the full width of the street. Interest in this bridge centers in the reasons for the adoption of the parts which make up the whole, and in the architectural treatment. The construction methods will be dealt with in a separate article.

* * *

Engineering Features of the Bridge

BY W. E. KING

ANY study of the engineering features of this bridge must take into consideration the very congested local conditions, the location of nearly every pier being predetermined by the clearances required by existing structures and railroad property. From the general elevation, Fig. 2, it will be noted that the bridge crosses successively over Second St., the freight shed and tracks of the C., St. P., M. & O. Ry., the tracks of the St. Paul Union Depot, which handles the entire passenger traffic of the city, the main line of the Chicago Great Western R.R., and the river channel of the Mississippi as defined by the War Department, the south end of the bridge then terminating in a busy manufacturing district. The elevation of the roadway was determined by the grade of Robert St. at the two ends of the bridge and by the limitation that the grade on the bridge proper must not exceed 3.1 per cent, which was the grade of the old bridge. These conditions, combined with the clearances required by the railroads and the government clearance of 47 ft. above ordinary high water, practically determined the location of the roadway.

The total length of the bridge including approaches is approximately 1,900 ft. Starting at the north end, there are a reinforced-concrete trestle with three spans of varying length totaling 89 ft.; a skew steel deck-girder span of about 53 ft. across Second St.; three flat barrel arches with a combined length of about 291 ft. followed by a two-rib arch span of 264 ft., c. to c. of piers, and four five-rib arch spans totaling 514 ft.; then a 311-ft. concrete trestle on concrete piles, with eight spans; and finally 260 ft. of earth fill between buildings and concrete retaining walls.

Foundation conditions were determined by borings made during the winter of 1923, some twenty holes being driven about 10 ft. into a fairly level layer of Shakopee limestone found at El. —81 or about 66 ft. below the river bottom. The material above the limestone consists of indiscriminate layers of gravel, coarse sand and fine sand, with occasional layers of clay and pockets of fine sand (possibly quicksand). On the north shore is encountered a layer of sandstone overlain with "tumble rock" from the cliffs above and cemented together with clay into a hardpan.

Foundations and Piers—Two types of foundations were considered: Piles and a caisson type which would extend the concrete foundations to the limestone ledge. The latter would have provided the unyielding foundation so necessary for an arch bridge, but the expense would have been much greater than for the wood-pile design which was adopted. In order to secure rigidity, the piles were jetted and driven to a practical refusal. In every case they were driven below any possible river scour and were cut off at or below low-water line. At the north end, where piledriving was impossible, the foundations were set upon hardpan. For the trestle and retaining wall at the south end of the bridge, where there is no danger of scour, short concrete piles were cast in place, with cores driven to refusal. These foundations were stopped at El. +6, the only consideration being that they would extend below frost line.

One of the interesting problems lay in the location of piers 1, 2, 3 and 4. It will be noted that the government river channel, 160 ft. wide, passes under the bridge at a skew to the south. The location of this channel is determined by that of the lift span of the Chicago Great Western R.R. This line crosses under the bridge on a skew to the north and it was necessary to locate a pier in the angle between these clearances if an arch span was to be used.

Pier 2 had to be no wider above ground than the pier of the old steel bridge. Pier 1 likewise was limited as to width by the clearance lines of existing railroad tracks. A glance at the design of these piers, Fig. 3, will show that they are much more slender than would ordinarily be used, and are in fact vertical reinforced-concrete cantilever extensions of the wide footings below ground. South of pier 4 there were no limitations as to pier locations, so that span length was determined by economic considerations, taking into consideration also the architectural appearance of the bridge as a whole.

Each pier footing consists of a heavily reinforced concrete mattress resting on piles spaced 30 in. c. to c. in each direction. Above this, the piers are of mass concrete, reinforced for temperature stresses and for eccentric loading. Where piers are extended for architectural treatment, the concrete has been cored out for economy, the exterior walls being approximately 2 ft. thick. Above the haunches of the arches the piers are hollow and are utilized for manholes for the public utilities.

Through-Arch Span—The main point of interest is the 264-ft. through-arch span, 244 ft. in the clear between piers. This design was necessary because of the government requirements of 62-ft. headroom above low-water for a skewed channel width of 160 ft. or 180 ft. along the bridge. With the roadway elevation as fixed, a through span was necessary if an arch span was to be used. This through-arch span is shown in Fig. 4.

Since the station of the north springing line was fixed, as was also the point where the intrados passed the clearance line of the channel, the variations open to the designer in trying to adapt the neutral axis to the dead-load equilibrium polygon were restricted. While for the final design a curve was found to give a line of resistance which remained within the middle third, the deviations of the polygon from the neutral axis were a maximum at the crown and at the quarter points. Instead of the usual compound curves resembling a basket handle, with the long radius at the crown and the shorter radii at the haunches, the radius is 122.16 ft. at the crown and 191.60 ft. at the haunch.

The two main ribs, each 6 ft. wide and 8 ft. deep at the crown, were spaced 64 ft. 8 in. c. to c., so as to clear the roadway 8 in. at each curb. The sidewalks were then detoured outside of the ribs in a way which adds to the architectural interest of the roadway. This is shown in Fig. 5. Considerable thought was given to the lateral support of these arch ribs and also to the expansion and contraction relative to the roadway.

A structural steel frame is used for each arch rib, as shown in Figs. 4 and 7. The steel arches are designed to be erected either as cantilevers supported from temporary towers or on falsework. They are designed to carry the weight of the steel structure, including the steel floor system, and also the dead-load of the concrete arch proper. The dead-load of the concrete roadway and the live-loads are carried by the composite concrete and structural steel arch.

The heavy box section of the top and bottom chords of the steel arch permitted the building in of the steel posts and hangers between the two side members of the chords in such a way as to give lateral support to the arch rib from the floor girders. The arch ribs have heavy steel cross bracing below the roadway. These steel arches were erected on timber falsework, the crown closing sections being drilled in the field. Steel floor girders 22 ft. apart were used to give a maximum clearance over the river channel. The use of structural steel hangers provided a simple method of delivering and distributing the floor girder reactions to the arch ribs. A covering of gunite on wire mesh was applied to the girders and bracing in order to protect the steel with a minimum of dead weight.

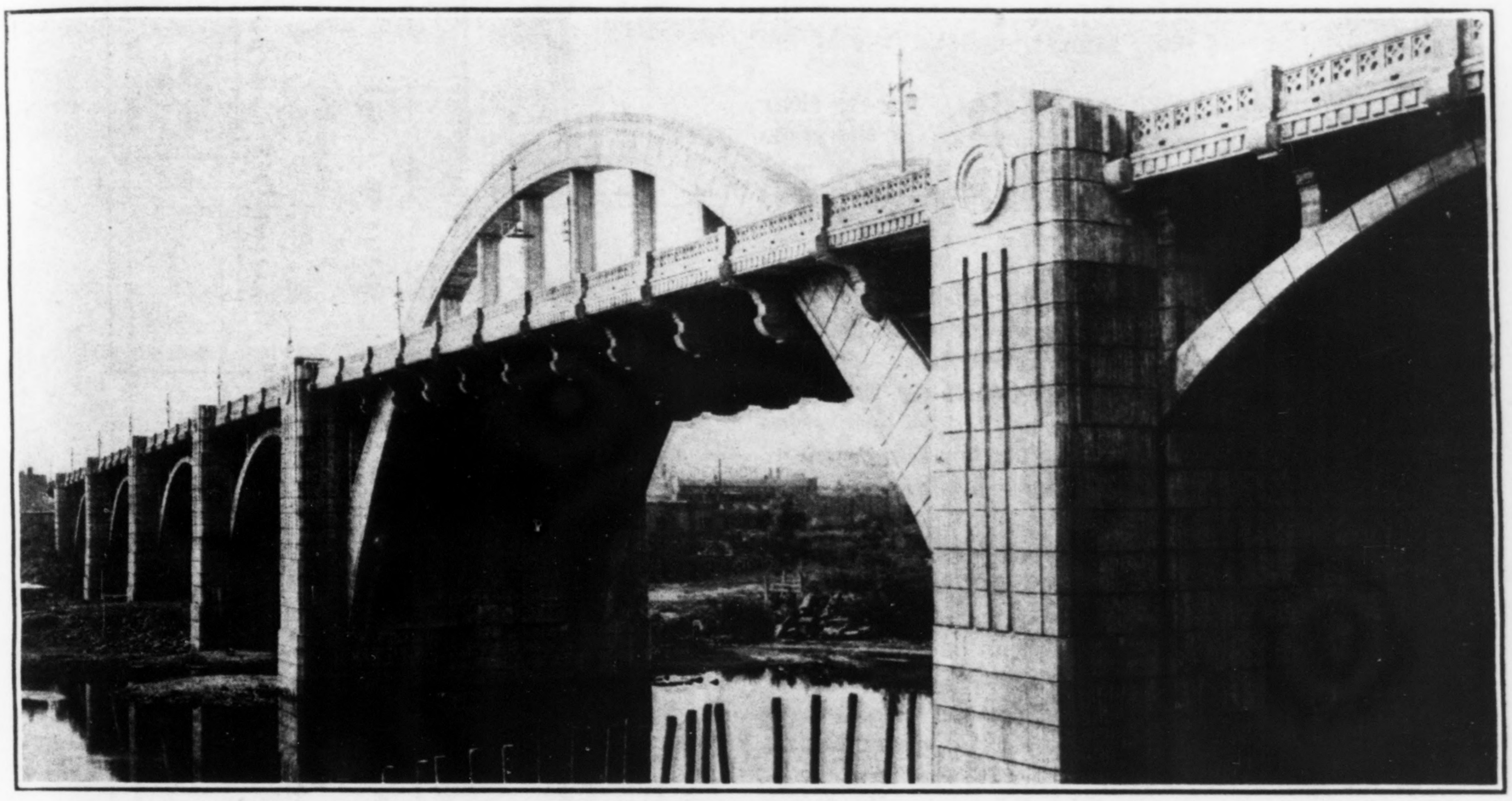

FIG. 1—LOOKING ALONG THE COMPLETED ROBERT ST. BRIDGE, ST. PAUL, MINN.

Special care was used in pouring the sections of the arch ribs so as to transfer the load to the concrete without disrupting the bond between the concrete and the structural steel. The concrete was poured in sections so as to equalize the load on the arch, with provisions for keys between the sections.

Barrel and Ribbed Arches—The design of the three barrel arches has no special interest except in the application of an arch design to very trying conditions. Thus, arch 2, a 70-ft. span, crosses the four main tracks leading west from the union depot, which tracks carry traffic for nine railway systems. With the center of the north track at the minimum allowable distance of 8 ft. from the south face of pier 1, clearances for the intrados of arch 2 were figured as closely as for the portal of a through steel truss span.

Clearances for the intrados of arch 3 were determined from the requirements of the main line of the Chicago Great Western R.R. which runs under the arch at an angle of 55 deg. with the center line of the bridge. This arch is a 98-ft. span with the springing line 50 ft. above low water and 35 ft. above the springing line of arch 4. Arch 1 is a skew arch, 96 ft. long at the center line, with its springing line 6 ft. below that of arch 2 and

1 ft. below that of arch 3. The three arches are designed to make up a unit, with the north abutment and pier 3 acting as its abutments.

In piers 1 and 2 the shafts have been designed to resist the moments resulting from the excess of the thrusts of the longer spans 1 and 3 over arch 2. The thickness of the piers below track clearance lines was increased from 8 ft. to 14 ft. and the footings, 32 ft. wide with piles spaced 30 in. c. to c., were offset 24 in. from the center line of the pier. The moment of the thrust of barrel arch 3, with its springing line high above that of the ribbed through arch, exceeded that of the latter. The footing, 42 ft. in width, was offset 18 in. from the center line of the pier to reduce the maximum load on the piles along the south face of the footing.

The five-rib arches, 5, 6, 7 and 8, are 112-ft. clear spans set at varying heights to accommodate the slope of the roadway. They are designed and located to neutralize the thrust of the through arch at pier 4. One of these spans is shown in Fig. 6.

Girder Spans—The steel span of the north approach crossing Second St. consists of twelve 48-in. plate girders, 46 ft. long, protected with gunite. Steel girders were necessary to give the required headroom for trucks. The 33-ft. concrete girders forming the eight spans of the south approach were flanked on both sides by shops of the American Hoist & Derrick Co. Much of the area under this portion of the bridge has been bricked in to be used for shop purposes, the girder and beam type of span being well adapted for this use.

Expansion Joints—The provision for expansion and contraction, together with the waterproofing of such joints, is regarded by the designers as one of the very important things to be provided for. An inspection of a large number of concrete bridges showed that in nearly every case failure or deterioration was taking place at the expansion joints. Wherever concrete was arranged to slide on concrete the joints usually failed by the failure of the concrete to slide and thus cracking elsewhere. Many joints failed because of water being trapped and freezing. Some joints filled with dirt which, caked with bitumen, appeared to be nearly as hard and inelastic as the concrete itself. In general, no satisfactory joint was found and the joint used on the Robert St. bridge, Fig. 3, while having some advantages, is regarded only as a step in the right direction.

The location of expansion joints is shown in Fig. 2. In general there are no sliding surfaces. The joints are placed at frequent intervals and any movement is taken up by bending in the supporting columns. This results, as a rule, in a double set of supporting columns. These columns (or one column and the adjacent pier) are separated by a 1-in. layer of elastite. The expansion joints extend completely through the roadway, sidewalks and railings in every case.

For waterproofing the roadway joints a crimped lead plate of special composition was placed across the joint, the ends being brazed together to make a tight joint across the roadway. This plate is held in place by the bolts which extend through the anchors in the concrete below and also the 3x3-in. angle above. It extends back into the fabric waterproofing so as to make a tight job. The steel angle is used as a stop for the brick paving, which rests on a ¾-in. sand cushion, the joint in the paving being closed with asphalt. The concrete deck slab was waterproofed with three layers

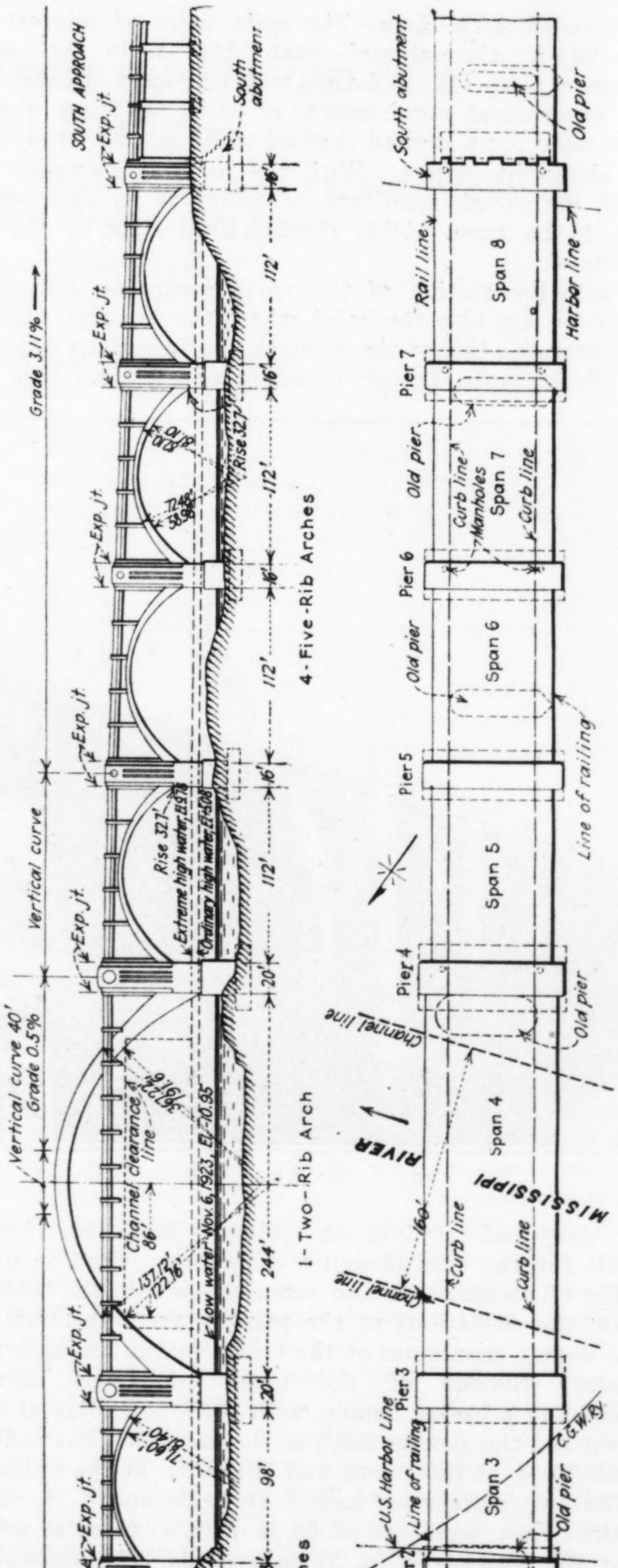

FIG. 2—ELEVATION AND PLAN OF ROBERT ST. BRIDGE OVER MISSISSIPPI RIVER

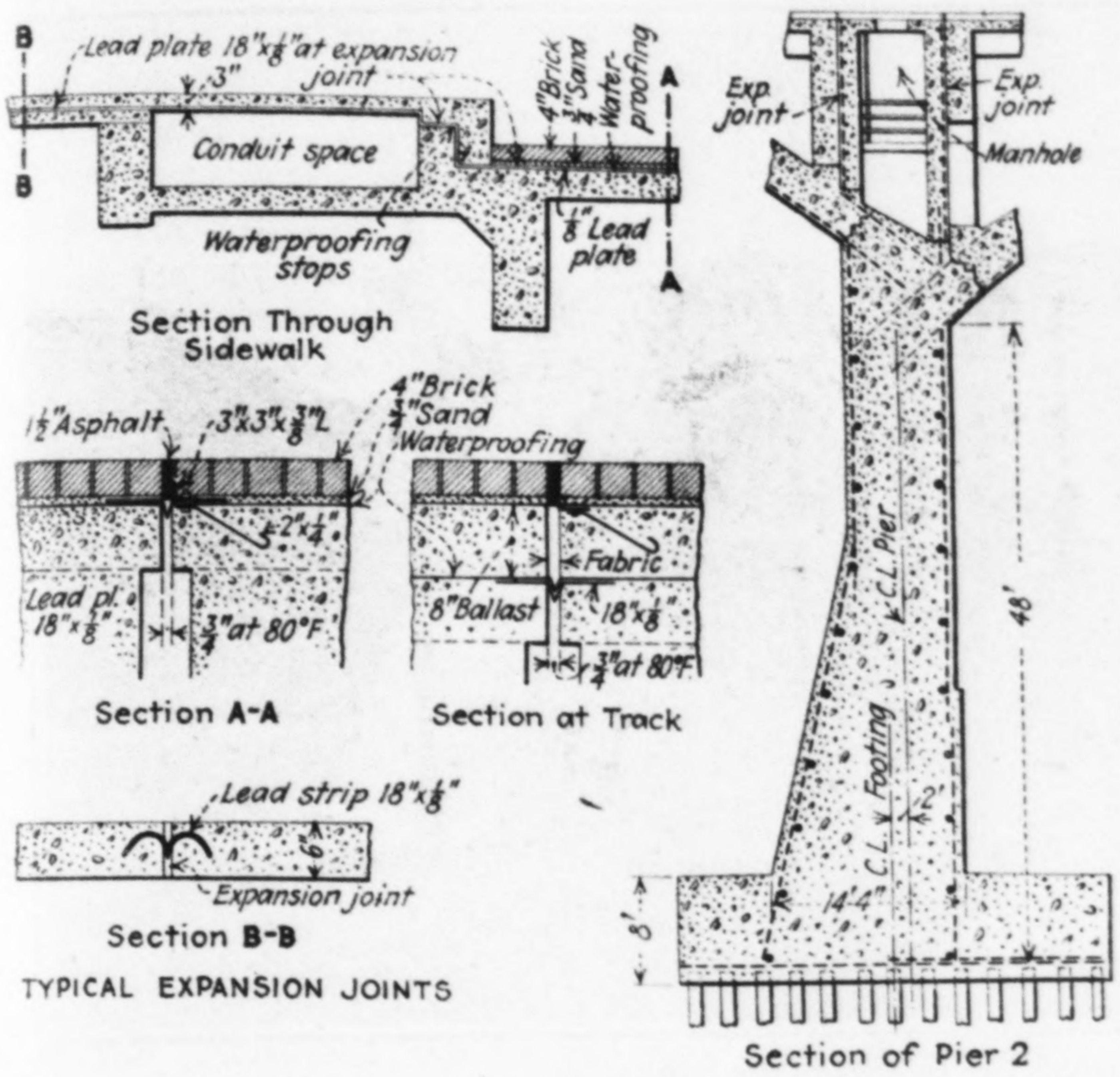

FIG. 3—EXPANSION JOINTS AND PIER SECTION

of cotton fabric impregnated with asphalt and laid with four coats of asphalt.

Paving and Conduits—A 4-in. brick pavement on 1-in. sand cushion provided the wearing surface of the roadway, except that granite blocks were used between the car tracks. This paving was laid to a 5-in. crown, with 10-in. curbs protected by a steel angle. The tracks, with steel ties, were placed by the street car company and concreted in place in the trough provided in the deck. Electric conduits of two power companies and the telephone and telegraph companies were provided for in two troughs under the sidewalks, suitable manholes being provided at the piers. Space for two gas mains was also provided.

Lighting—With a series system arranged in two circuits, lighting intensities on the roadway will vary from 1 to 2 foot-candles with the minimum of glare. An attempt has been made to harmonize the appearance of the combination light and trolley poles with the remainder of the bridge, as shown in Fig. 8. Standard tubular steel trolley poles spaced approximately 125 ft. apart have double pendant lantern units suspended from special cast-iron brackets at 17 ft. above the roadway. Simple base castings and brackets are the only decorations applied to these poles.

Each lighting unit consists of a 400-cp. lamp enclosed in a lantern with rippled glass sides and top equipped with prismatic dome refractor designed to throw the light on the roadway. Lamps are operated upon a 6.6-ampere primary circuit with transformers at the base of each pole.

* * *

Architectural Features

By Roy Childs Jones

THE Robert St. bridge is unique in that its designers included in their own permanent organization both architects and engineers. This fact made it possible to take full account of the mingled utilitarian and esthetic problems present in a structure destined to serve as a civic monument. Instead of architects being called in to beautify with applied ornament a predetermined structure, architects and engineers combined during the initial stages of the design in an effort to select and

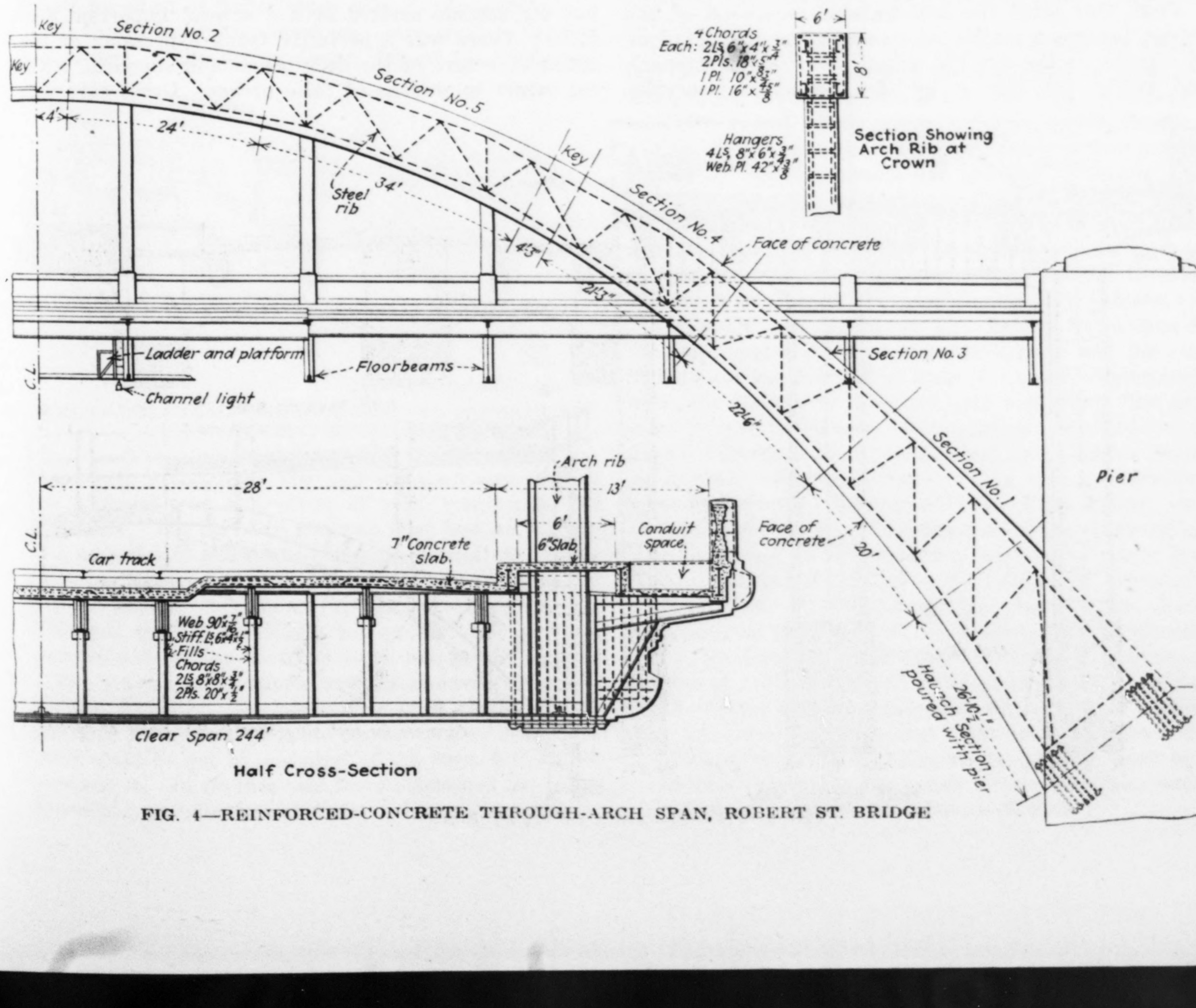

FIG. 4—REINFORCED-CONCRETE THROUGH-ARCH SPAN, ROBERT ST. BRIDGE

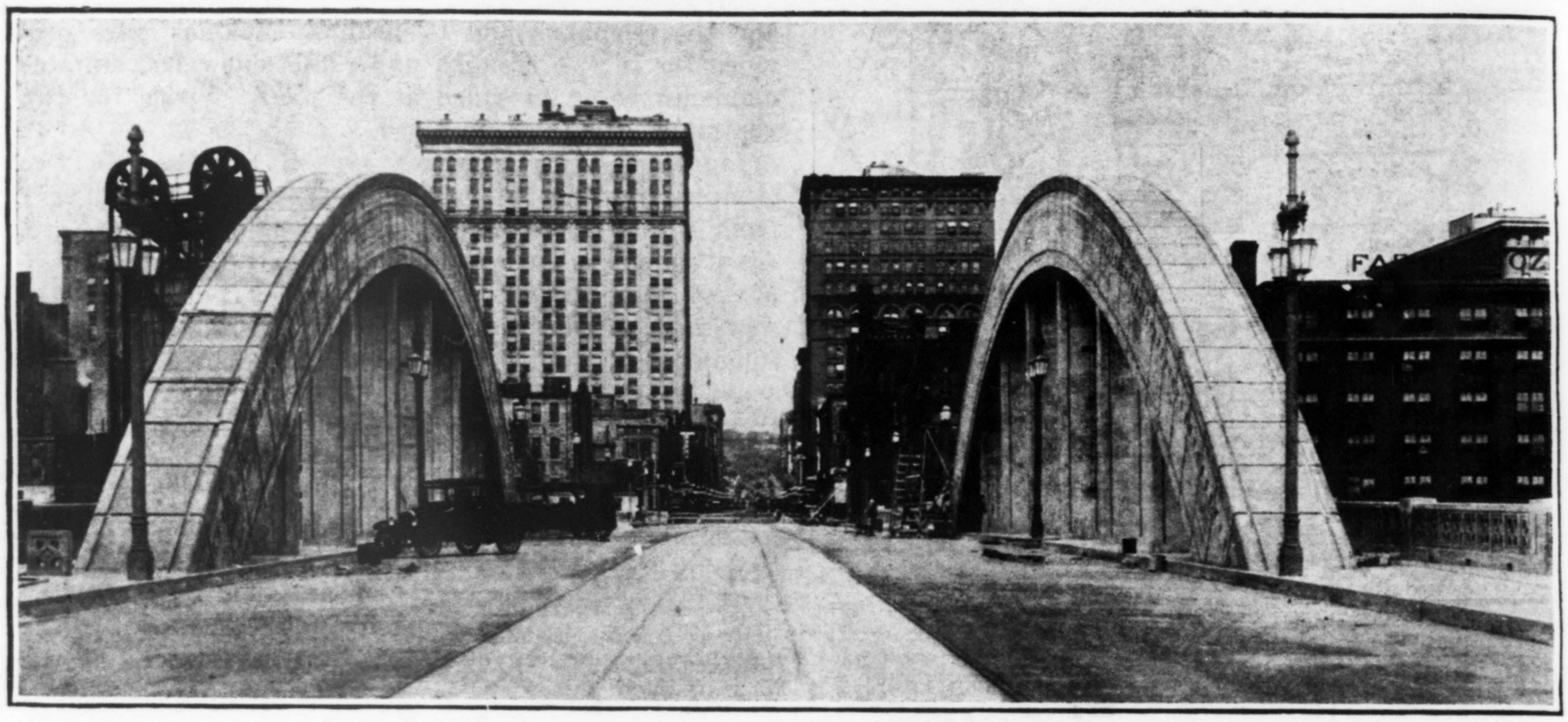

FIG. 5—DECK OF THROUGH-ARCH SPAN OF ROBERT ST. BRIDGE

control the structural features so as to secure for the bridge an inherent beauty of form and proportion.

With the complicated requirements of street grades and of railroad and channel clearances, any simple and regular composition of arches and piers was out of the question. The first effort was to minimize this difficulty by throwing all possible emphasis on the center span, and subordinating the approach spans with their discordantly varying shapes. To carry out this intention, the approach spans were made as short as possible, and their piers were kept comparatively simple.

From this point the architectural treatment of the bridge became a matter of so managing the detail as to express to the eye the actual qualities of strength and utility put into it by the engineer. Generally speaking, this was accomplished, not by adding ornament, but by controlling the shapes and proportions and relations of the structural members. Fortunately there is an inherent beauty in the mathematically determined curve of an arch that only needs letting alone to be apparent. In this case nothing was done to the arches except to emphasize their size and spring by certain lines of shadow, and by seeing to it that the curves swept unobstructedly and dominatingly from haunch to crown.

Treatment of the piers and deck was more difficult, but the scheme arrived at had several important features. There was a perfectly frank expression of the actual structure of the deck beams and supports, without resort to screens of false arches. Great care was

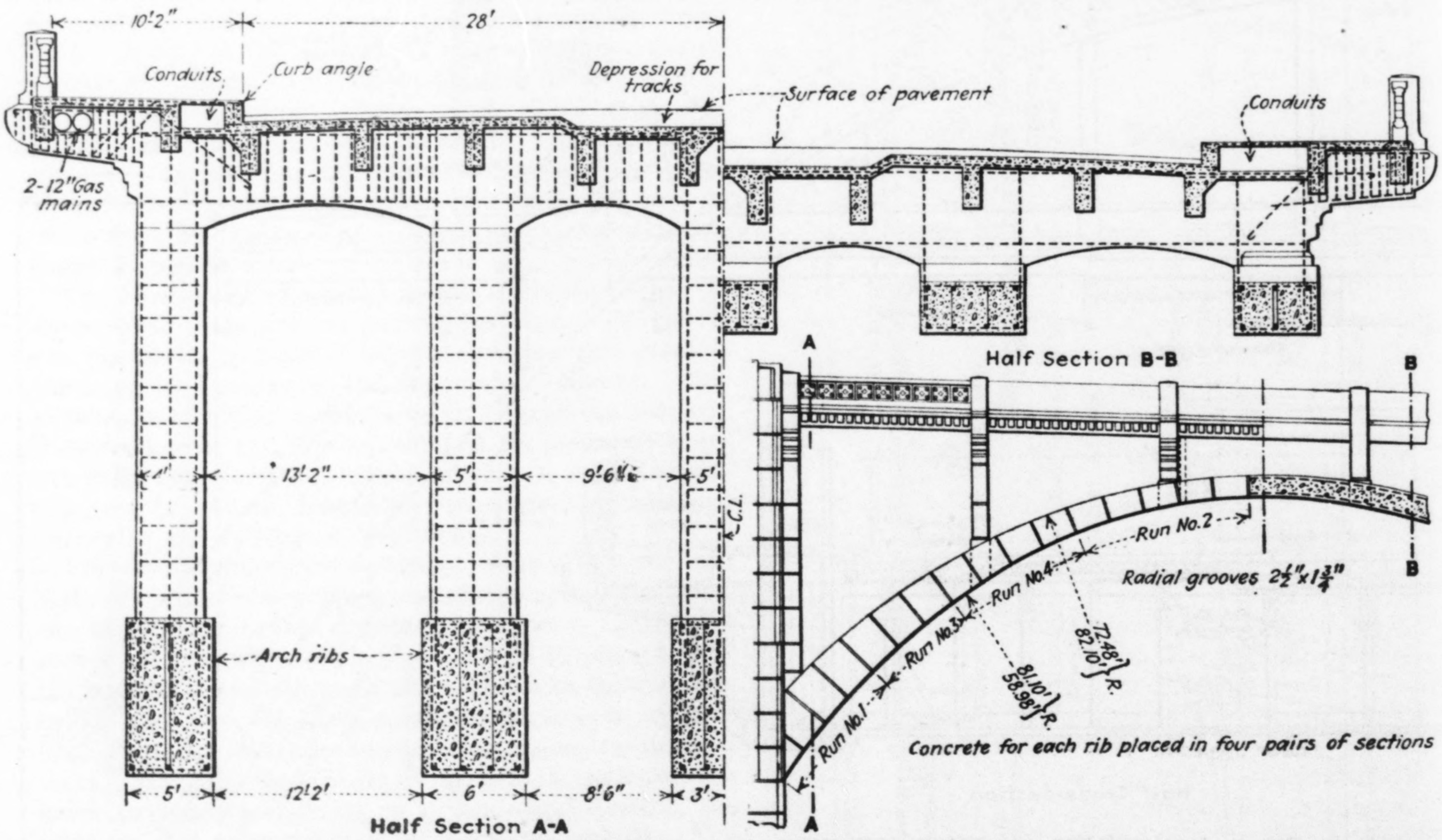

FIG. 6—FIVE-RIB ARCH OF 112-FT. SPAN

taken to preserve the continuity of the actual line of the deck and to emphasize and enrich it by appropriate sectioning.

Hand-Railing—As a part of the deck treatment, the rail naturally came in for a large share of attention, since the appearance of a bridge from the point of view of the people who use it is almost entirely a matter of the rail. Furthermore, it is seen from close up, and must have a refinement of scale and a degree of finish not easy to obtain in concrete. In addition, it must not only have great rigidity to resist the possible impact of heavy modern trucks, but sufficient flexibility to expand and contract without unsightly cracks.

The problem was met here by a scheme of precast perforated panels anchored between poured, heavily reinforced members at top and bottom, and between posts from side to side. This design is shown in Fig. 8. Concealed expansion joints were provided by slots in the posts which received the ends of the rail sections. This satisfied the engineer's requirements as to strength, while to the architect the interesting feature of the treatment lay in the fact that it promises a decorative quality growing unmistakably out of the material, and of being susceptible to such a degree of finish as to avoid the crudeness so often seen.

Decorative Treatment—As a contrast to the sweeping horizontality of the deck, the piers have had their vertical quality emphasized. Strong vertical lines of shadow were created by a control of the pier section. Grooves were run up and down the faces of the piers, their depth decreasing toward the bottom so as to create an effect of diminishing shadow to enhance their interest. The pier corners were beveled, with successively increasing set-backs from bottom to top to create an effect of batter when seen in perspective.

All this detailed treatment was necessarily a matter not merely of the bridge design, but of the bridge as

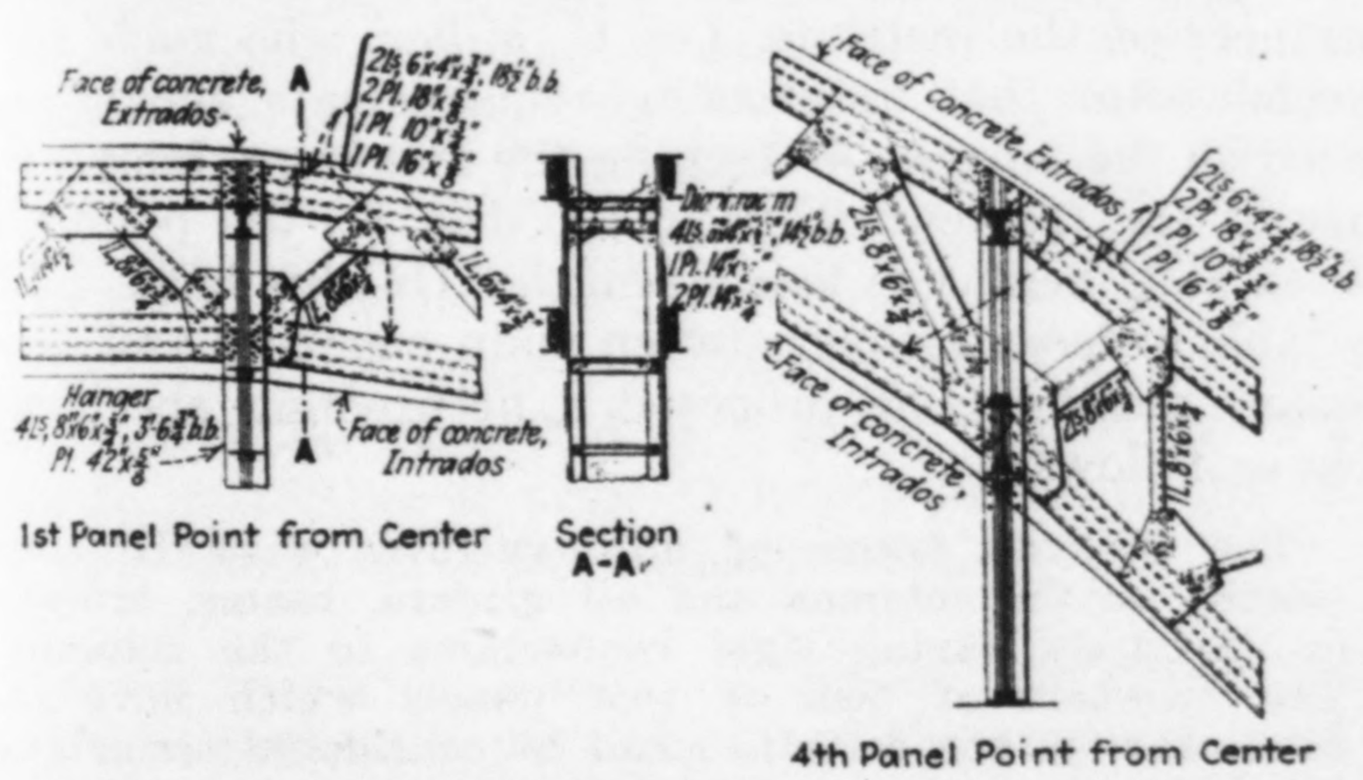

FIG. 7—DETAILS OF STRUCTURAL STEEL FRAMING OF ARCH REINFORCEMENT

a concrete structure. The appropriate expression of the material was a problem of vital concern to the architects. The fact that concrete is at best an intractable material to work with; that its natural decorative value as to color and texture is practically nil; that the cost of any of the known surface treatments to get color and texture artificially was prohibitive; all these were considerations that had to be taken into account.

The scheme of treatment arrived at involved several definite features. No color other than what could be got with the natural concrete and no texture other than what could be got by a control of the form boards was resorted to. In no case was there attempted any effect depending on unbroken surfaces or lines. It seemed to the designers that the tendencies of forms to spring, and of concrete to vary, inevitably doomed such attempts to failure in realization. Continuity of surface and line was considered as the sum of a series of

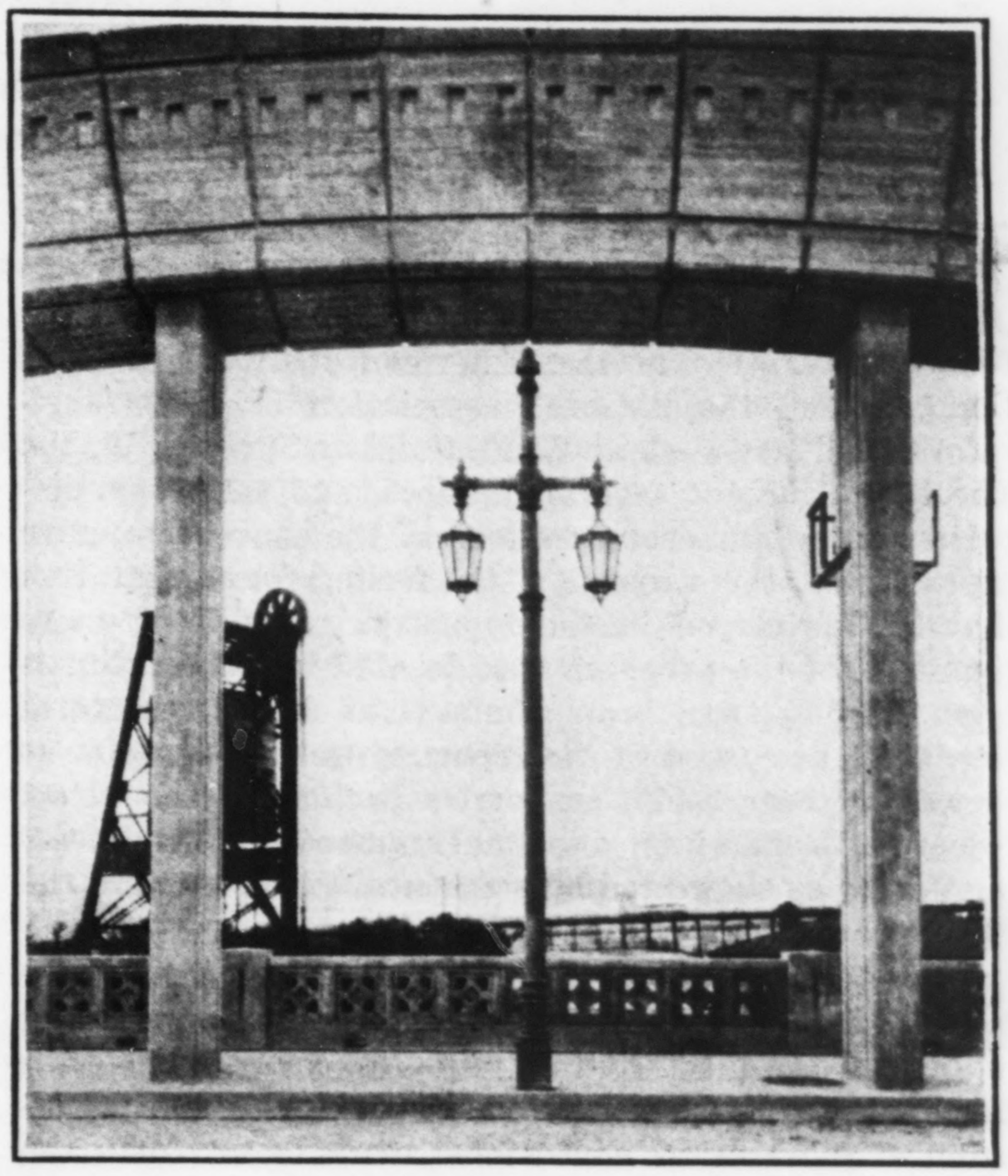

FIG. 8—DECORATIVE TREATMENT OF ROBERT ST. BRIDGE

definitely bounded segments rather than as literal continuity.

The working out of this principle involved the breaking up of all surfaces with lines of light and shade. The natural divisions between periodic runs of concrete were recognized by horizontal grooves, whose location was carefully controlled. Further modeling was accomplished by vertical breaks and grooves, by bevels, and by wedge-shaped indentations. The idea was to make, out of the natural patches of lighter and darker toned material, patterns definitely bounded by strong lines of shadow; and to effect an emphasized interest in light and shade in place of the unattainable color interest.

Engineers and Builders—The Robert St. bridge is the joint undertaking of Ramsey County and the city of St. Paul, represented by Paul N. Coates, county engineer, and George M. Shepard, city engineer. The contractor was the Fegles Construction Co., Ltd., with James Patterson as superintendent and Charles Moore as assistant superintendent. Plans and specifications were prepared by the firm of Toltz, King & Day, Inc., St. Paul, Minn., which also supervised the construction. The personnel of this company was as follows: Max Toltz, mechanical engineer; W. E. King, structural engineer; B. W. Day, architect; Roy Childs Jones, architectural designer; P. E. Stevens, office engineer; W. A. Thomas, electrical engineer; John F. Greene, in charge of arch design and resident engineer. The cost of the bridge and approaches complete is about $1,750,-000, including paving, street lighting and engineering.

[*This article will be followed in an early issue by another describing the construction operations and the lessons learned therefrom.*—EDITOR.]

Some Lessons Learned in Building Long Concrete Bridge

Pile Foundations—Proportioning and Handling Concrete—Slump Tests—Erecting Steel Ribs for 264-Ft. Span—Arch Deflection Tests

BY JOHN F. GREENE

Resident Engineer, Toltz, King & Day, Inc., Engineers and Architects, St. Paul, Minn.

CONSTRUCTION work on the new Robert St. concrete arch bridge over the Mississippi River at St. Paul, Minn., presented a number of special features. The design of this bridge was described in *Engineering News-Record* of Oct. 21, p. 732. Among the construction features involved were: The removal of a 5-ft. stone sewer; the diversion of the sewage to clear the foundations of the bridge, and the construction of a new reinforced-concrete sewer; the removal of the old steel bridge, containing approximately 2,200 tons of steel; the removal of the old stone piers, including the pile foundations; the construction of cofferdams for seven pier foundations; the driving of 3,700 permanent wood piles from 40 to 54 ft. in length; the driving of 800 cast-in-place concrete piles; and the placing of 50,000 cu.yd. of concrete, 1,100 tons of reinforcing steel and 1,000 tons of structural steel. Two of the five-rib 112-ft. arch spans are shown in Fig. 1, while Fig. 2 shows one of the structural steel ribs for the 244-ft. arch.

Removal of Old Bridge Piers—Because three of the piers of the new bridge were so located as to conflict with piers of the old bridge, it was necessary to remove the latter completely. This condition involved enlarging the cofferdams to enclose both the old and the new foundations; the removal by blasting of the masonry of the old piers; the removal of a pad consisting of three layers of 8-in. timbers resting on piles spaced 36 in. c. to c., and the pulling of the piles. These piles, entered on the construction records of the old bridge as from 24 to 30 ft. long, actually ran from 7 to 20 ft. in length, with an average of less than 15 ft. The 8-in. timbers, placed in 1885, were in good condition and were used for arch falsework.

The cofferdams for the piers, approximately 100 x 50 ft., were constructed of 35-ft. steel sheetpiling driven through sand with a double-acting steam hammer to a penetration of not less than 12 ft. below the bottom of the finished excavation. This piledriving was carried out in accordance with a well conceived plan which gave excellent results. All the sheetpiles were set in place and driven only a few feet before making the closure. They were then driven in stages of approximately 5 ft., the hammer swinging in light leads from a derrick boom. The piles were carefully plumbed at regular intervals. Obstructions encountered in driving, which included piles and timbers of the cofferdams for the piers of the old bridge, were removed after pumps had been installed and the water lowered. Before the installation of the pumps, the excavation was carried on with a 1-yd. clamshell bucket. The bracing for the cofferdams was framed and packed in tiers at the water surface and was allowed to settle as the water was pumped out. It consisted of 10-ft. bays with four tiers of 12 x 12-in. timbers.

Piledriving—Piles for piers 3 and 4, at the ends of the main span, were 54 ft. long; those for the remaining piers were 40 ft. long. They were of Norway pine, fresh cut, and inspected in the woods of northern Minnesota at the time of cutting. Driving the 54-ft. piles through sand was a difficult operation which involved considerable skill in the use of a high-pressure jet in order to secure the alignment of piles 30 in. c. to c., without interference. They were driven with a steam hammer having a 5,000-lb. ram and a stroke of 36 in. In those piers in which the piles were not driven to rock, a penetration of 1 in. in ten blows was adopted as a stopping goal, giving a bearing of 150,000 lb. per pile according to the *Engineering News* formula.

The pine piles were peculiarly suitable for such driving because of their shape. A Norway pine grows with a very slight taper from tip to butt; many of the piles 54 ft. long were 10 in. at the tip and only 14 in. at the butt. Cypress and cedar piles with a heavy swell from tip to butt were driven to a full stop at 25 ft., even with the jet. After trying to drive such piles in the first pier, the results were so unsatisfactory as to warrant the exclusive use of Norway piles.

Approximately 800 concrete cast-in-place piles were used in the foundations for eight concrete bents and for a high concrete retaining wall on the east side of the south approach. While the plans called for 30-ft. piles, the average penetration did not exceed 24 ft. The type of pile used, having a heavy taper like that of a cedar pile, appeared to bring up very quickly when driven in sand. A steam hammer was used for driving the steel core; frequently it was necessary to drive the steel core more than 3 ft. at a penetration of 1 in. in ten blows in order to get a 20-ft. pile. A core with a larger tip and a smaller butt would have been more satisfactory.

Construction Plant—The main concrete plant, containing a 1-yd. mixer, was located on the south bank of the river, under the deck of the old steel bridge. Sand and gravel were delivered in bottom-dump cars over a trestle which passed under the south arch. These materials were dumped into the boot of an elevator which raised and dumped it on a belt conveyor for delivery to the bins. Cement was hauled in over the deck of the old bridge in trucks, and was dumped into a chute which ran through the material bins to the measuring hopper.

Concrete was conveyed from the mixer in four-car trains of 1-yd. side-dump cars drawn by a gasoline locomotive over a timber trestle east of and parallel to the center line of the bridge. A "marine tower" with a hoist on a deck 15 ft. above the track, ran along this trestle. The concrete for the superstructure was dumped into a hopper at the foot of the tower and elevated to the level of the deck, thence to be conveyed to place by buggies. Concrete for the pier substructures was dumped into a 2-yd. bottom-dump bucket handled by a derrick. Concrete for columns and pier walls was placed through chutes composed of 8-in. pipes in 3-ft. lengths fastened together by chains.

Concrete Proportions—The specifications called for six bags of cement per yard of concrete; crushed trap rock for the substructure; crushed limestone for the superstructure; 1½ minutes per batch in the mixer, and a tentative 1:2:4 mix. Preliminary tests were made to determine the effect of over-sanding. It was found that over-sanding to the extent of 1 of cement to 2.6 of sand to 3.4 of crushed rock resulted in an average compressive strength at 28 days of 3,800 lb. per square inch. Hence, over-sanding was permitted when necessary. For consistency, a slump of 2 in. was specified for mass work and 3 in. for the remainder.

It was the intent of both the engineers and the contractor to obtain the strongest possible concrete for the illustration. Run a 7-in. slab on a grade of 3 per cent with a 2-in. slump and a water content of less than 4.5 gal. of water per sack, measured at the mixer. The resulting concrete, after straight-edging and working, will be soupy.

It is the judgment of the writer that samples for testing should be taken only in the hole and from the wettest concrete in the hole, not from the mixer or from buggies. This is the only way to allow for the results of an accumulation of a slight excess of water from each batch or the accumulation from an occasional sloppy batch which has passed the mixer man and which the inspector has hesitated to waste.

Any specification which allows the contractor to use

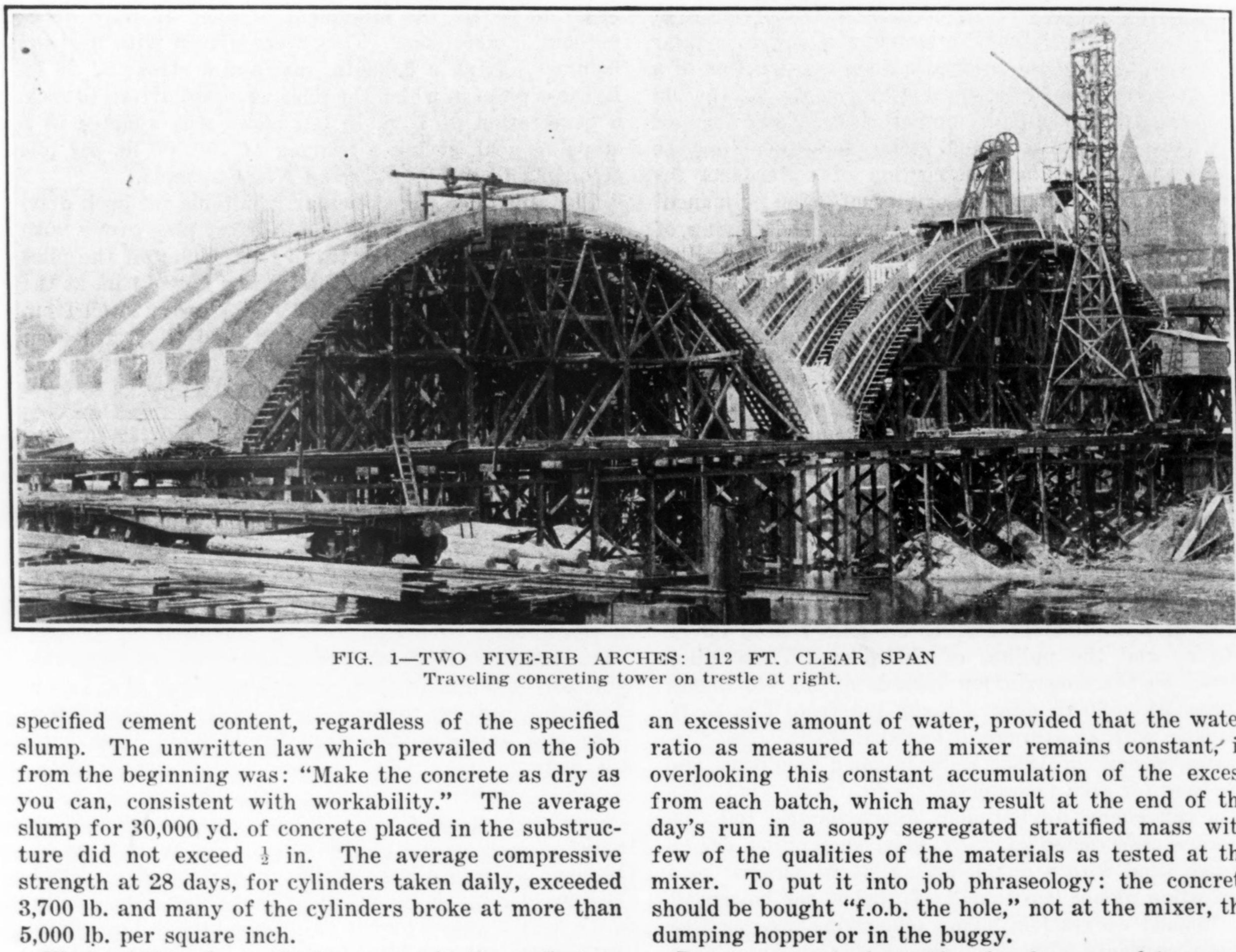

FIG. 1—TWO FIVE-RIB ARCHES: 112 FT. CLEAR SPAN
Traveling concreting tower on trestle at right.

specified cement content, regardless of the specified slump. The unwritten law which prevailed on the job from the beginning was: "Make the concrete as dry as you can, consistent with workability." The average slump for 30,000 yd. of concrete placed in the substructure did not exceed ½ in. The average compressive strength at 28 days, for cylinders taken daily, exceeded 3,700 lb. and many of the cylinders broke at more than 5,000 lb. per square inch.

Work of this class calls for a conscientious effort on the part of the contracting organization and the most careful supervision on the part of the inspection staff, but the results are worthy of the effort and speak for themselves. The work has been done so well that pier after pier stands, after the removal of forms, without patches or rock pockets.

Water Content of Concrete—It may be appropriate to consider a phenomenon with which many construction men are familiar. Assume a column or a wall, large enough for a man to work in. Place concrete in this wall of a consistency to give a 1-in. slump at the mixer or as sampled out of a buggy. Run 7 ft. in depth of this concrete in three hours with men spreading the concrete. At the end of the run there will be loose water on the top of the concrete and the material tested in the hole may show a slump of 4 in. Take another an excessive amount of water, provided that the water ratio as measured at the mixer remains constant, is overlooking this constant accumulation of the excess from each batch, which may result at the end of the day's run in a soupy segregated stratified mass with few of the qualities of the materials as tested at the mixer. To put it into job phraseology: the concrete should be bought "f.o.b. the hole," not at the mixer, the dumping hopper or in the buggy.

Proper concrete inspection involves complete supervision of every yard of concrete placed, by a man trained to know when the consistency is right for the proper compacting of the concrete, who will see that the concrete is properly worked to form a dense mass, and is spaded to give smooth surfaces free from rock pockets. This man, with the job slogan in mind, "dry it up whenever you can," may vary the consistency many times in a day. When running the main arch ribs, enclosing heavy structural steel members, consistencies were varied frequently. And since the inspector in the hole is naturally inclined to take a little more water than he needs, in order to reduce the labor of spading, it is incumbent upon the inspector at the mixer to keep the men in the hole asking for more water. There should be a constant pressure at all times to keep the consistency close to the limit of workability, if the aim is to get the greatest value for money expended.

Through-Arch Main Span—The structural steel ribs provided in the through arch of 244-ft. clear span to transfer the floor load from the 90-in. steel floorbeams to the concrete ribs, were fabricated with the ends of the chord members milled to give full bearing at each joint. When the steel had been erected, it was found that many of the joints did not have full bearing, the openings varying from 1/100 to ⅛ in. It was decided to full rivet all joints, holes being drilled in the field, and plates and rivets added to develop the chord sections. One of these ribs is shown in place in Fig. 2.

The forms for the concrete ribs were suspended from the structural steel. Each rib was divided into ten sections and three keys, one key at the crown and one near each third point. This is shown by the drawings in the previous article already noted. It was required that an interval long enough to develop 1,000 lb. per sq.in. in compression should elapse between successive days of concreting, in order to lessen the probability of destroying the bond between the concrete and the structural steel. The average compressive strength of the cylinders at four days was 1,560 lb. and the maximum at five days was 2,216 lb. The keys were concreted six days after the last section had been poured.

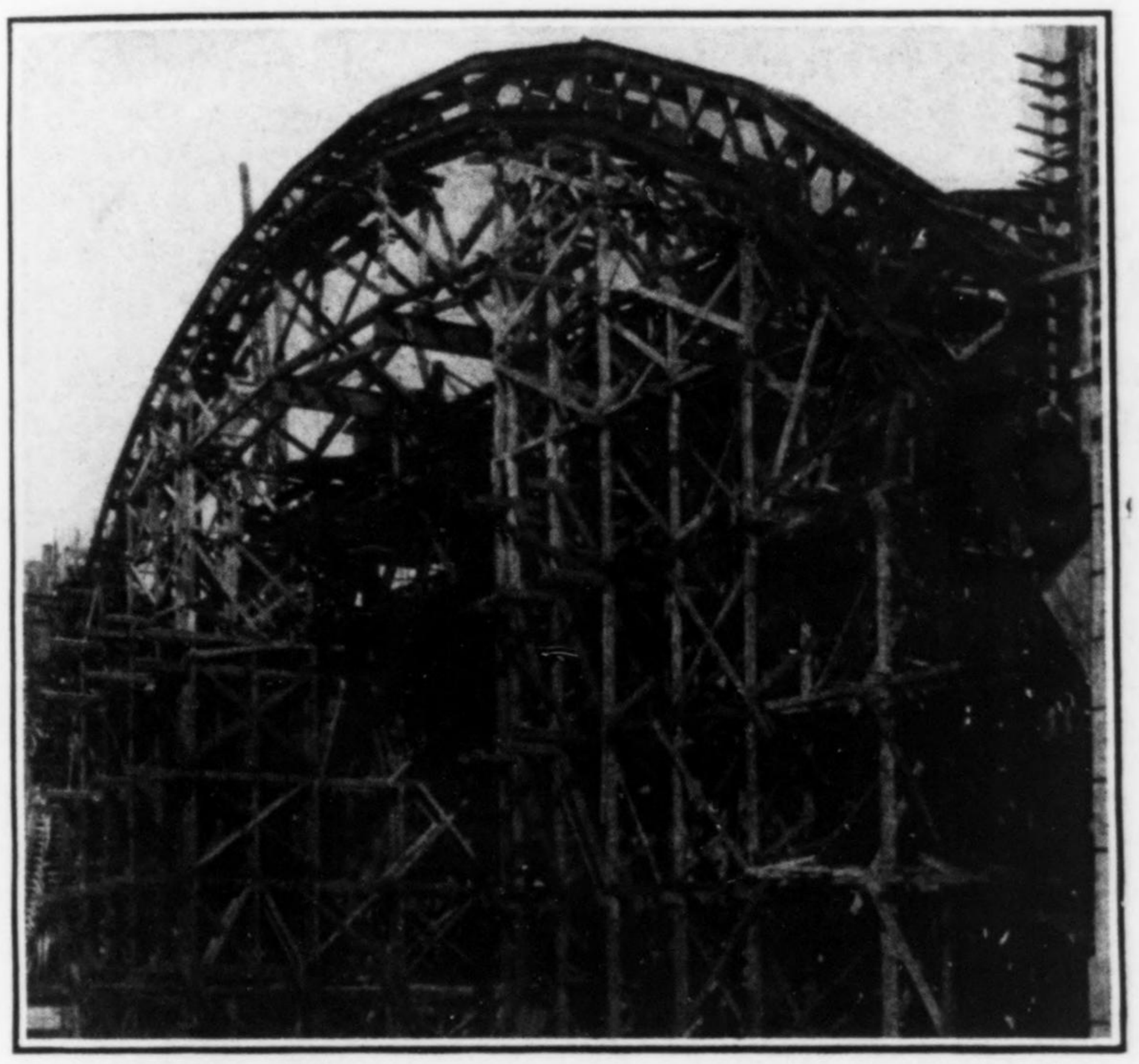

FIG. 2—STEEL RIB ERECTED FOR 264-FT. CONCRETE ARCH

Arch Deflections—An excellent opportunity offered to measure vertical movements of the main arch resulting from: (1) Rib shortening under load, (2) rib shortening due to setting of the concrete, and (3) changes due to rise and fall of temperature. The first reading was taken upon the completion of the riveting of the structural steel March 8, with the temperature of the air at 14 deg. F., and thereafter at intervals of a week until the final reading July 10.

Computations for deflection were based upon the following formulas:

$$\Delta h = \frac{h}{l}\,\Delta l + i_h \Delta l$$

$\frac{h}{l}\,\Delta l$ = Deflection due to rib shortening from temperature.

$i_h \Delta l$ = Deflection due to temperature moments.

i_h = Influence ordinate for crown thrust for a unit load at the crown.

h = rise at intrados.

b = clear span.

On May 18, just after the keys had been concreted, when the steel ribs were carrying the full load, with no deflection due to the setting of the concrete, the following conditions prevailed:

Temperature change from 14 deg. to 66 deg.	+52
Computed rise due to temperature	+0.091
Computed deflection due to load	−0.033
Computed net rise	+0.058
Measured rise; average of two ribs	+0.061

On July 10, upon completion, and two months after the keys had been concreted, conditions were as follows:

Temperature change from 14 deg. to 70 deg.	+56 deg.
Computed rise due to temperature	+0.098
Computed deflection due to load	−0.047
Computed net rise	+0.051
Measured rise	+0.023
Net settlement during two months	−0.028

Computed deflections were based upon an assumed modulus of elasticity of 4,000,000 for the concrete. This assumption was based upon curves shown on page 39, Bulletin No. 5 of the Lewis Institute, for concrete with a compressive strength of 4,000 lb. It is interesting to compare these measured deflections with those found by Considère and Freyssinet for long-span concrete ribs constructed with semi-hinges. Assuming a compressive stress of 350 lb. per square inch:

		Per Cent
(1) Shortening due to load—per unit of length	0.00009	13.6
(2) Shortening due to 40 deg. drop in temp.	0.00028	42.5
(3) Shortening due to setting of concrete at three months (Considère)	0.00029	43.9
Totals	0.00066	100.0

The shortening due to the setting of the concrete at 60 days, on the Robert St. bridge, concreted in the manner already described, and with a structural steel frame carrying the load during the setting of the concrete, was 0.00009, approximately one-third that found by Considère for concrete ribs constructed with falsework centering in the ordinary manner. The unit shortening to compare with Considère's finding, assuming the compressive stress at 350 lb., was as follows:

		Per Cent
(1) Shortening due to load	0.00009	19.6
(2) Shortening due to 40 deg. drop in temp.	0.00028	60.8
(3) Shortening due to setting at 60 days	0.00009	19.6
Totals	0.00046	100.0

Assuming the modulus of elasticity at 2,500,000, the tabulation would be as follows:

	Robert St. Bridge		— Considère —	
		Per Cent		Per Cent
(1) Shortening: load	0.00014	27.4	0.00014	19.7
(2) Shortening: 40 deg.	0.00028	55.0	0.00028	39.4
(3) Setting	0.00009	17.6	0.00029	40.9
Totals	0.00051	100.0	0.00071	100.0

Apparently the structural steel frame and the method employed in concreting the arch ribs contributed to a marked reduction in the amount of the settlement of the arch after the completion of concreting.

Concrete Railing—The parapet or hand rail was constructed by the dry process to give the surface a texture free from air and water holes. Blocks approximately 24 in. square and 7 in. thick, of a somewhat intricate design, were prepared in aluminum molds. With two sets of molds, a crew made forty blocks in nine hours.

This railing was constructed as follows: Forms for a somewhat massive post were erected containing two galvanized inserts to receive the railing on both sides. In concreting a post, a light steel form, 12 in. deep and 4 in. smaller than the post, was placed inside of the outer wooden forms, a shell of dry mixed concrete was pounded in, the core was filled with ordinary wet concrete and the steel form was raised a foot. One man would complete six posts in a day.

In the next operation, the running of the base, the men chose to complete the section with the dry mix, scoop out the core and refill with normal wet concrete. The intermediate precast panels were set by stone masons with ¼-in. joints, grouted and pointed. The top section of the rail was prepared in the same manner as the base. At both ends of each bay a ¼-in. strip of elastite was inserted between the post and the panel to allow for expansion.

The success of the dry-process method of making an ornamental rail involves the application of water to the finished rail many times each day for a period of two weeks. While compressive tests of the dry concrete at 28 days showed only 1,300 lb. per sq.in. tests were made to ascertain whether cleavage planes would result between the dry and the wet portions. Sections of the rail were removed and it was found that there were no cleavage planes and that the combination was a structural unit. The completed handrail, just as it emerged from the forms, with no patching, rubbing or grinding, is of a texture which compares very favorably with sandstone. Models for the twelve medallions which crown the outer faces of the piers were prepared by the Brioschi-Minuti Co., St. Paul.

Plans and specifications for the Robert St. bridge were prepared by Toltz, King & Day, Inc., St. Paul, Minn., who also supervised the construction: Max Toltz, mechanical engineer; W. E. King, structural engineer; B. W. Day, architect; Roy Childs Jones, architectural design; P. E. Stevens, office engineer; W. A. Thomas, electrical engineer; and John F. Greene in charge of arch design and resident engineer. The bridge was built by the Fegles Construction Co., Ltd., Minneapolis, Minn., with James Patterson as superintendent and Charles Moore as assistant superintendent.

Reinforcing a Steel Viaduct: Northern Pacific Ry.

Towers Strengthened by Additional Steel on Columns and Bracing—New Girder Spans—Precast Concrete Slabs for Ballasted Track—Methods of Handling Work Under Traffic

TO PROVIDE for the heavier locomotives and higher speeds of trains now operated on the main line of the Northern Pacific Ry., extensive reinforcement has been applied to the Granite Gulch viaduct near Granite, Idaho, and about 50 miles east of Spokane, Wash. During recent years, the heavier types of passenger and freight engines have been prohibited from crossing this viaduct, while all engines operating over it have been restricted to a slow speed. The reinforcement eliminates speed restrictions and permits the operation of the heavy engines used on adjacent parts of the line. The decision to reinforce this structure was based on the economy, as determined by estimated costs, over any other method of eliminating the restrictions as to engines and speeds. Comparative costs also determined the method of reinforcement, which was shown to be more economical than any other methods considered. A view of part of the completed structure is given in Fig. 1.

Original Structure—The Granite Gulch viaduct is a single-track structure, 1,169 ft. long, with a maximum height of 110 ft. from base of rail to surface of water. It is on a tangent and on a grade varying from level to 0.38 per cent. There are two girder spans of 31 ft. out to out, on each approach; twelve tower girder spans of 31 ft. and eleven fish-belly girder spans of 61 ft. between the towers. These twelve towers range from 35 ft. 9 in. to 98 ft. 9 in. in height. The approach girder spans are supported on granite masonry abutments, rocker bents and the end tower bents. These two rocker bents are 22 ft. and 28 ft. in height. Fig. 2 shows the general design.

The viaduct, as built in 1893 in connection with a change of line, was designed for a loading of two 116-ton, consolidation 2:8:0 engines, followed by a train load of 3,000 lb. per foot, equivalent to Cooper's E-33 loading. This was the standard for the railway company's bridge design from 1891 to 1897. All metal in the original viaduct is wrought iron, except that the rivets are of rivet steel.

Masonry—Pedestal piers of granite masonry, supporting the tower columns, are built on solid rock, except that the piers for two towers in the lake are on piles 8 to 63 ft. in length, driven through mud to solid rock. Piers founded on rock and on piling average 12 and 16 ft. in height, respectively. Most of these pier foundations were of sufficient size to provide for the additional bearing area of the bases of column reinforcement. For the three towers at each end, however,

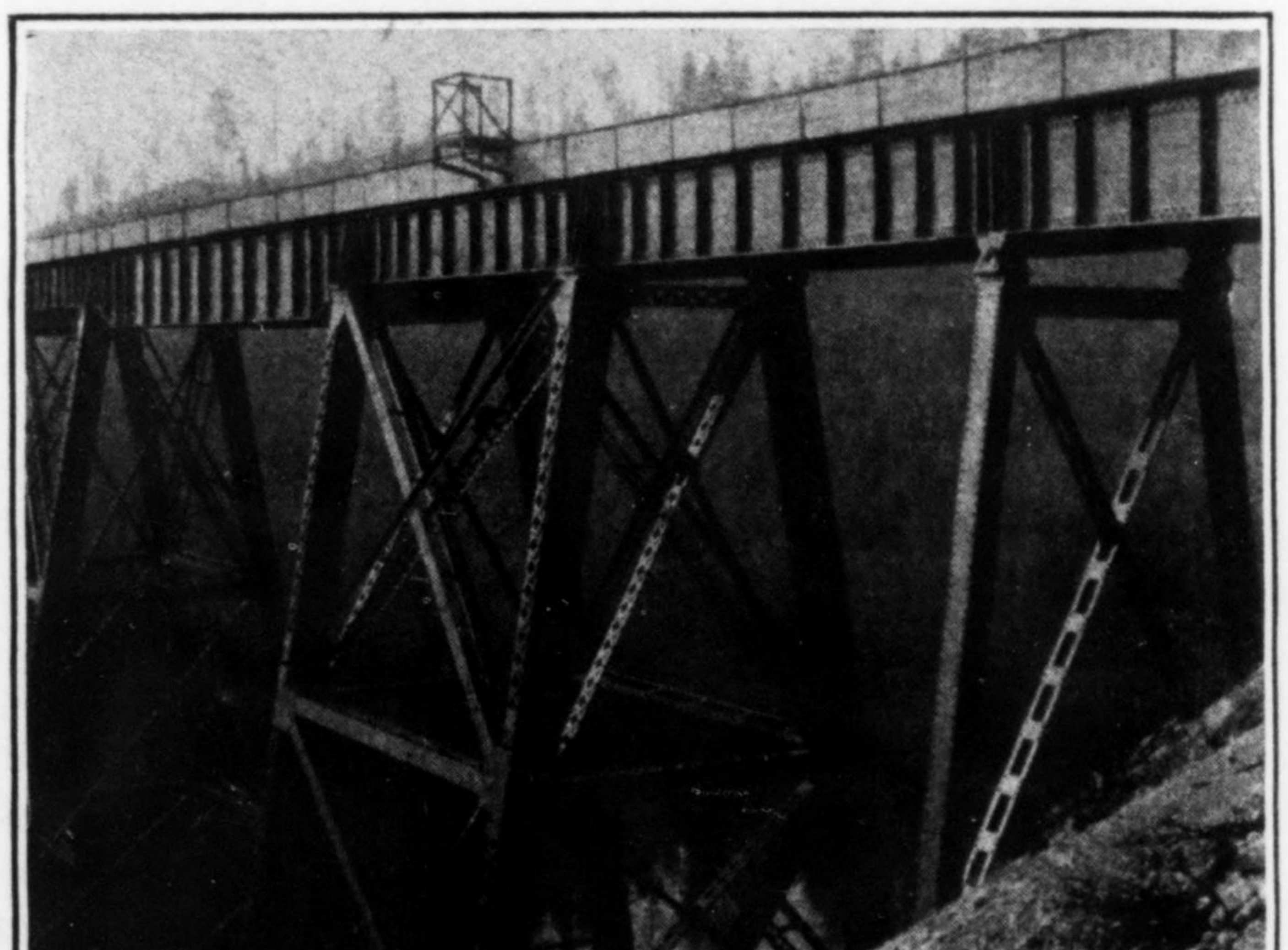

FIG. 1—GRANITE GULCH VIADUCT: NORTHERN PACIFIC RY.

Note reinforced columns and bracing. New rocker bent at left. Concrete slab deck.