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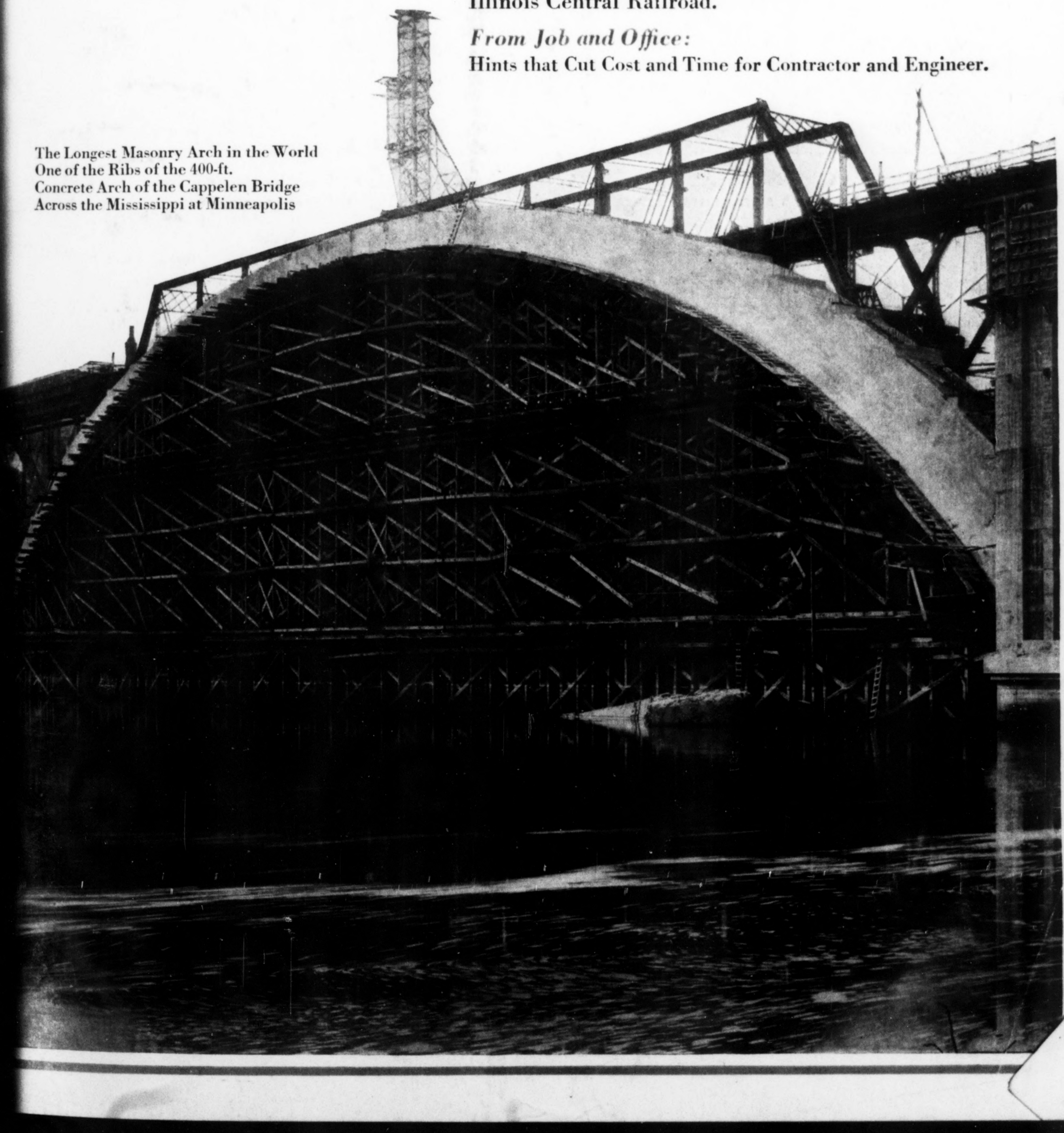
Report of the Annual Convention of the American Roadbuilders' Association.

Design of the Cappelen Bridge at Minneapolis the World's Longest Masonry Arch.

Rail Reclamation Practice on the Illinois Central Railroad.

From Job and Office:
Hints that Cut Cost and Time for Contractor and Engineer.



The Longest Masonry Arch in the World
One of the Ribs of the 400-ft.
Concrete Arch of the Cappelen Bridge
Across the Mississippi at Minneapolis

Design of 400-Ft. Concrete Arch of the Cappelen Memorial Bridge

Largest Concrete Arch Ever Built Will Be Completed This Year—Span Required to Provide Navigation Waterway and Clear Old Bridge—Thrust Partly Balanced by 199-Ft. Side Spans

WITH the largest concrete span in the world, a 400-ft. arch containing 2,300 cu.yd. of concrete in each of its two ribs, the Mississippi River bridge at Franklin Ave., Minneapolis, recently renamed the Cappelen Memorial Bridge in honor of the engineer who planned it and began its construction, is one of the most important and interesting works of structural engineering now in progress. Aside from its magnitude, it is remarkable for being built by the city as force-account work; all Minneapolis construction has been done in this way for years.

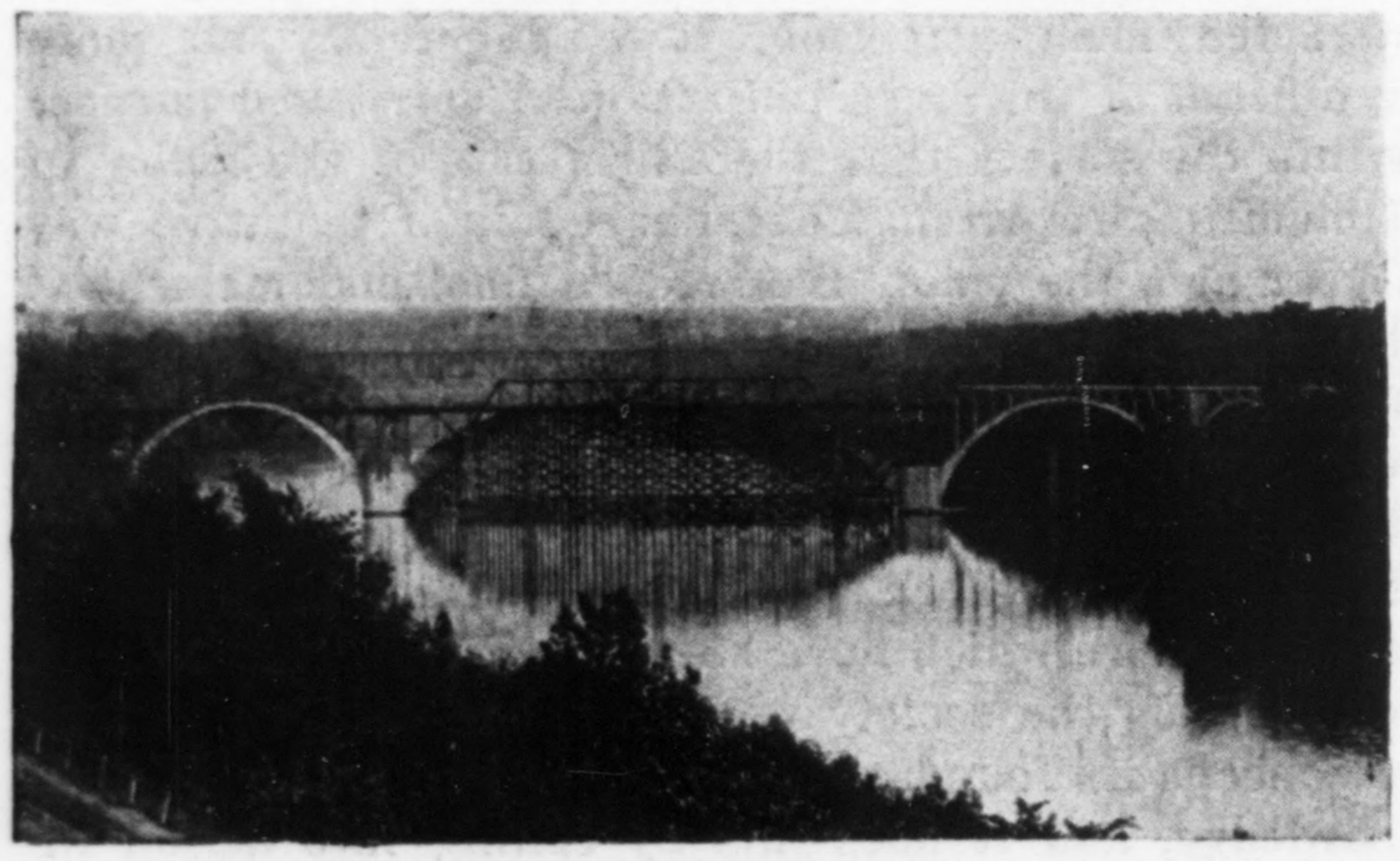


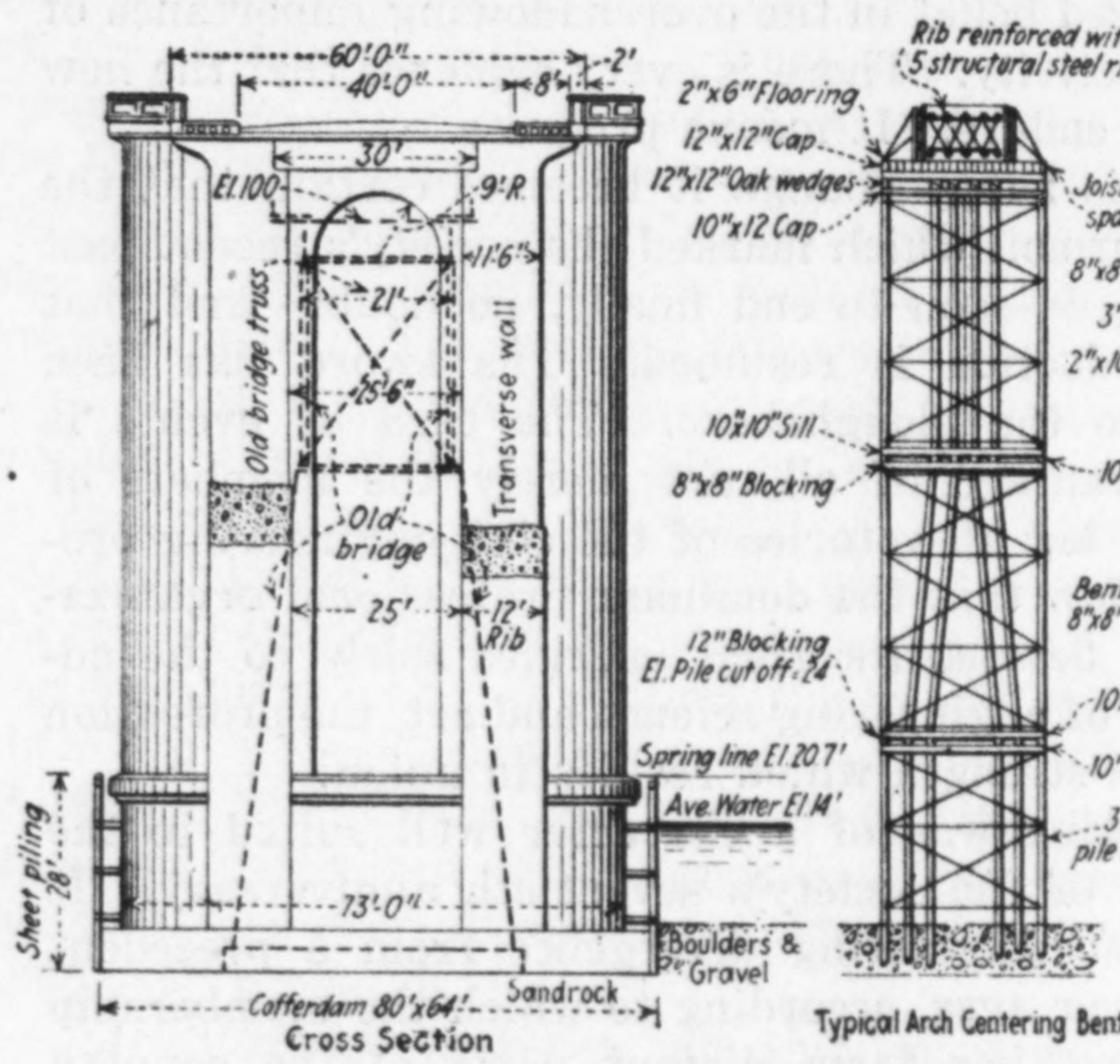
FIG. 1—CAPPELEN BRIDGE OVER MISSISSIPPI RIVER
Centering of 400-ft. arch under downstream rib

Though it constitutes a formidable engineering problem, the bridge is strikingly simple both in its design and in the construction methods being applied to it. Under N. W. Elsberg, superintendent of construction for two years and now city engineer, the work has progressed to completion of the piers and ribs, and concreting of the floor is now in progress. The coming season will see the completion of the structure.

Preliminary data, with an outline sketch, were given in these pages Feb. 12, 1920, p. 335. The leading considerations and data of the design, made available by Mr. Elsberg and K. Oustad, bridge engineer for the city, are given in the present article.

Governing Conditions at the Site—Located in the southwestern residential district of Minneapolis, Franklin Ave. crosses the Mississippi as one of the main connecting links between two well settled sections. The old structure here is a five-span steel bridge about thirty years old, with a roadway only 18 ft. wide; its 302-ft. main span is of through arrangement, while the four 174-ft. side spans are deck. It has long been inadequate and its replacement has been contemplated for a number of years. After completion of the South Third Ave. bridge—ten 200-ft. concrete arch spans—seven years ago, F. W. Cappelen, the city engineer, at once put the Franklin Ave. crossing problem under study. The new concrete arch bridge is the outcome of this study.

Permanence and monumental appearance were desired in the new bridge, the latter quality because of park and residential surroundings and the scenic value of the site. This pointed to a masonry or concrete arch structure. With regard to navigation requirements, however, a span of at least 300 ft. would be necessary. Nevertheless, Mr. Cappelen developed a project for a concrete arch crossing to replace the old bridge, and it was subsequently approved by the city government and



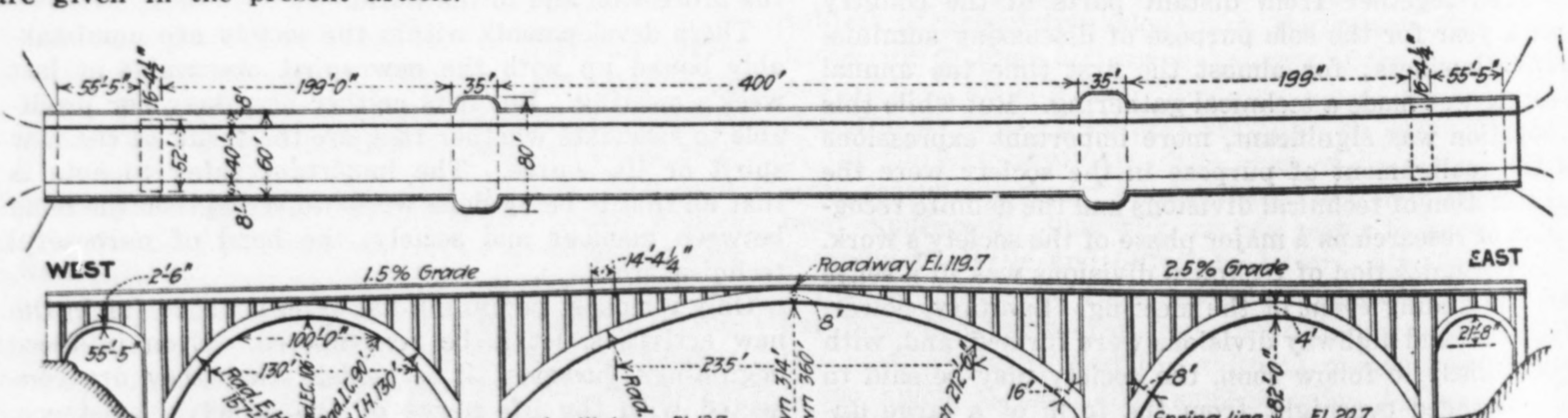


FIG. 2—FRANKLIN AVE. CONCRETE ARCH BRIDGE—THREE-STORY CENTERING FOR MAIN ARCH RIB

the necessary funds voted by the city council. Construction began in the latter part of 1919.

General Conditions—Franklin Ave. crosses the river near the head of the Mississippi River gorge lying between Minneapolis and St. Paul. The river is about 900 ft. wide here, or about 1,100 ft. between the edges of the bluffs. The banks are about 100 ft. high, of limestone, while the river bed is in St. Peter sandstone, thinly covered with drift and alluvium. Sound rock lies 15 to 30 ft. below water. Thus, a good foundation is available for any type of structure. River navigation is not at present important, but is expected to become so in future with the aid of ponding by the government dam several miles downstream, completed a few years ago. A wide channel with substantial clearance height was therefore demanded by the War Department (minimum width 300 ft., clearance height 50 ft.).

as now being carried out, and as shown in the drawing and views herewith.

The detailed design, largely worked out by Mr. Oustad, is unusually simple in its architectural features, making use of almost no ornament; for example, the ends of the spandrel columns are simple square faces, without any molding. The result is a demonstration of what can be achieved with plain details provided the structural proportioning is good.

For simplicity in construction, a slab-and-floorbeam deck (without stringers) was adopted. Since street-car loadings were to be provided for, a reasonable limitation of slab thickness restricted the column spacing to about 15 ft. longitudinally of the bridge. The adopted spacing in the main span (14 ft. 4½ in.) was maintained also in the side spans, with suitable detail adjustment of span length.

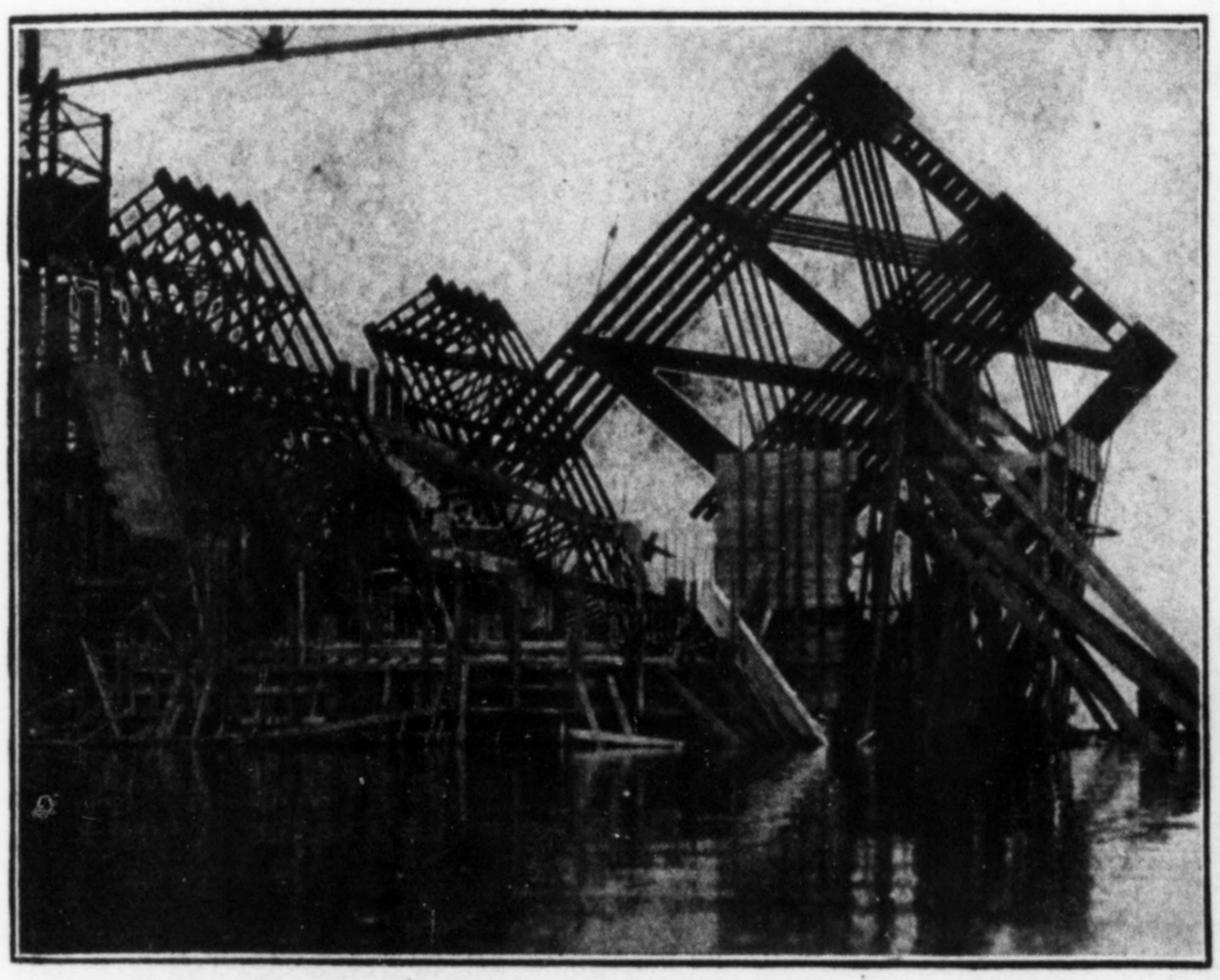
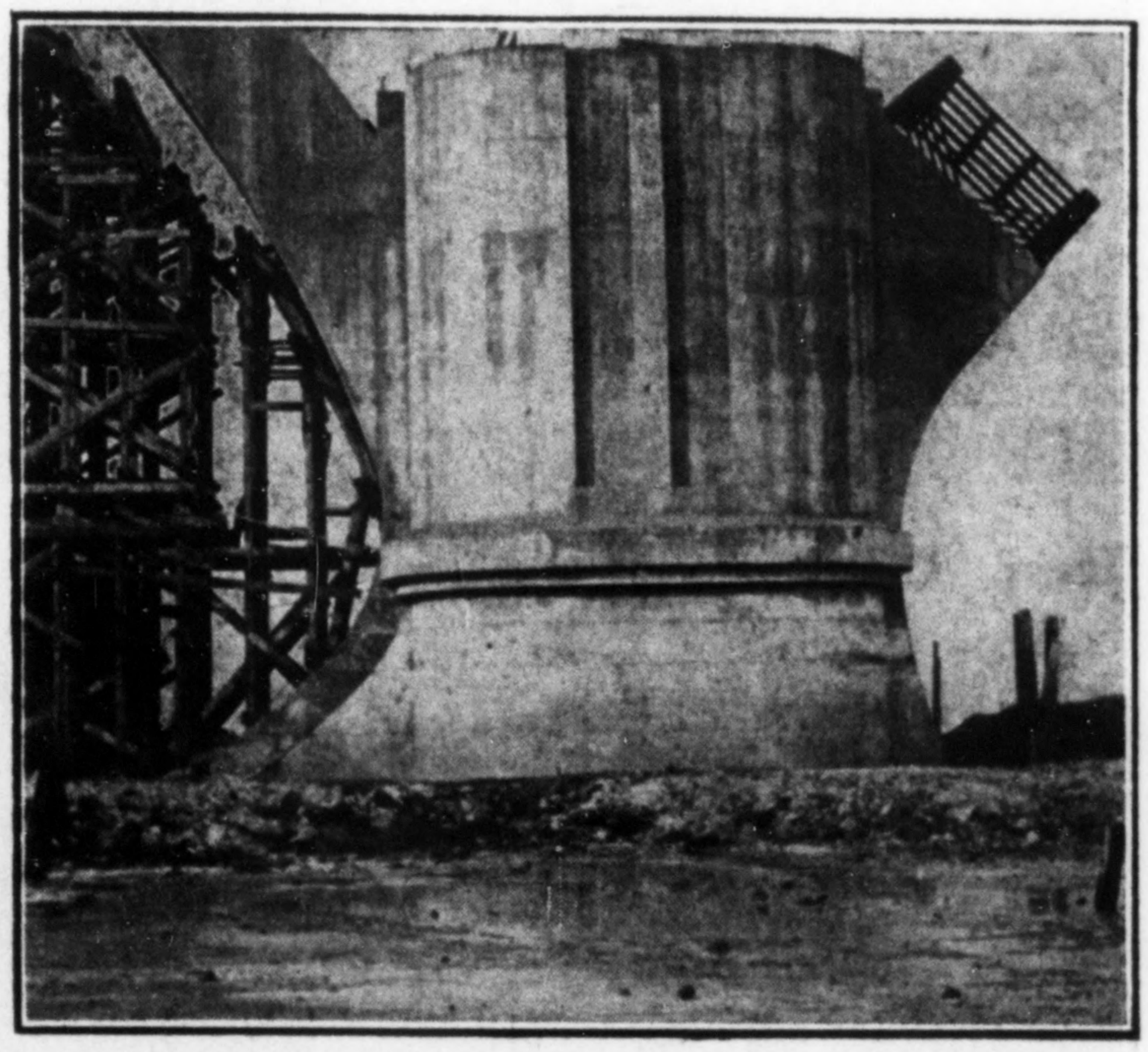



FIG. 3—RIB REINFORCEMENT AND COMPLETED PIER WITH RIB SECTIONS

It was desired to interfere as little as possible with traffic across the old bridge during construction, and to maintain at least a pedestrian crossing throughout the work. Further, the possibility of using the old bridge for transportation of material during construction was an important consideration. It seemed necessary, therefore, to clear the old structure by building outside of it, and to make the main span long enough to pass beyond the old piers.

General Design—Applying these datum points to a preliminary sketch layout, a central arch span of about 400 ft. resulted. The height of the banks called for a floor elevation some 10 ft. higher than that of the old bridge, the latter being below bank level. With a moderate grade up to the center of the new structure, then, a rise of about 90 ft. was obtainable in the central span.

In order to transmit to the sides of the gorge the larger part of the thrust of this span, it was desirable to make the side spans as long as possible. Keeping the level of their outer springing lines at the bottom of the gorge limited their span to about 200 ft., however. Under these conditions the thrust on the main piers could not be balanced, unless the side arches had been made of shallow rise by depressing the crown far below roadway level, which was not considered esthetically desirable. But a pleasing proportioning of spans was obtained, and Mr. Cappelen's skill and good taste in design soon developed this layout into the general plan

The deck width was governed by the width of the approach streets as defining the character of present and future traffic. The streets have generally a 40-ft. roadway, and this figure was adopted for the bridge. As there are other bridges reasonably near by, the nearest a mile downstream and three-fourths of a mile upstream, no very intense concentration of traffic at this crossing is looked for, and it is believed that the 40-ft. width will always be adequate. The total deck width of 60 ft. obtained after two 8-ft. sidewalks were added agreed well with the structural requirements of the arches as to width. To clear the old bridge, the arch ribs had to be spaced 25 ft. apart in the clear; a rib width of 12 ft. was considered the minimum permissible, and this brought the over-all width of the arch structure to 49 ft., leaving 5½ ft. of bracketing for the sidewalk on either side.

Arches—In laying down the general design, Mr. Cappelen had made it an initial condition that the springing line of the arches should be above high water. There is a possibility of 5 or 6 ft. of flood rise above pool level, and accordingly the pier belt course which defines the elevation of spring was placed at about this height above low water. The arches themselves are approximately parabolic; more exactly, they are curves fitted closely to the mean pressure line and then faired out by drawing three-centered intrados and extrados curves. The crown thickness of the 400-ft. arch is 8 ft. and the skewback thickness 16 ft. The corresponding thicknesses of the 199-ft. arches are 4 ft. and 8 ft.

With these curves and dimensions, the arches do not experience any extreme-fiber tension under any conditions. The reinforcing steel could not be proportioned for definite stresses, therefore, but it was designed by judgment, to give a good interbonding of all parts of the rib. It represents about 0.8 per cent of the rib section. (See the pressure lines under half-live load, Fig. 6.)

Structural-steel reinforcement was adopted, for the sake of simplicity in erection and certainty of true positioning. Each rib contains five longitudinal frames or trusses, tied together by frequent cross-frames. The chord angles are 6 x 6 x ½ in. in the large arch and 4 x 4 x ⅜ in. in the small. The frames enter the pier concrete only far enough for bond, and do not extend to the base course (Fig. 5).

Piers—It was expected that a good foundation would be found about 25 ft. below pool level. In actual construction, the west main pier was founded at 26 ft. below pool level, and the east main pier at 20 ft. The rock encountered is a massive sandstone, moderately soft, and somewhat fissured; the strata dip slightly from both banks toward the middle of the river. There being a net outward thrust on the main piers due to the excess thrust of the large arch, in order to keep the resultant pressure line as steep as possible the pier was made solid to a level well above the springing line; thence upward it carries two hollow shafts rising to roadway level. The thrust is transferred into the foundation rock chiefly by friction on the base, though there is also lateral bearing of the concrete against the rock sides of the foundation excavation, which was cut 2 to 3 ft. into the rock to get good bearing material. The base is horizontal, and no artificial keying or doweling was thought necessary to supplement the natural roughness of the rock surface obtained.

On account of the inclination of the thrust, the pier had to be made unsymmetrical in order to keep the resultant near the center of the foundation. The resulting pier design is quite unusual. There is bottom reinforcement to take care of footing tension, and skin reinforcement all around for protection against shrinkage effects. The footing pressure is about 4½ tons per sq.ft.

Construction conditions made it necessary to concrete the base course in separate sections. Care was taken to key and dowel the sections together, and a second base course extending over the whole foundation area was concreted on top of the lower or seal course, for further integration of the pier concrete.

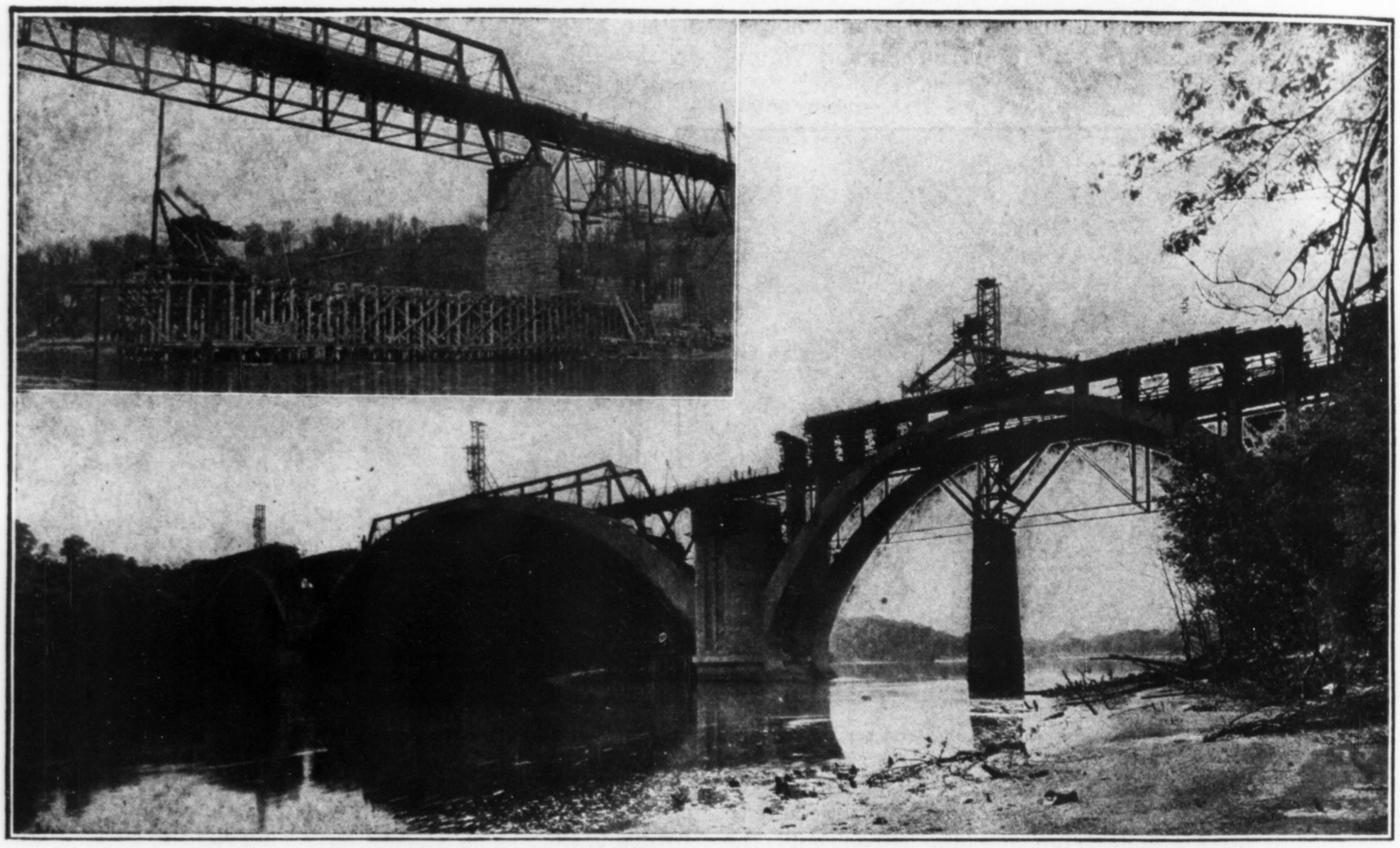


FIG. 4—THREE SPANS CROSS THE 868-FT. WIDTH BETWEEN THE MISSISSIPPI GORGE BLUFFS

The old steel bridge, shown in the insert view, was narrow enough to permit of building the ribs of the new arch structure alongside and at the same time keeping the old crossing in partial service.

The end piers are simple, bearing on the rock practically at the springing-line level.

Deck Design—Originally it was planned to place precast floor-beams in notches formed in the tops of the spandrel columns. Most of them are being cast in place, however, while for convenience some are being precast.

These beams, having a net span of 25 ft. and carrying a floor area of nearly 360 sq.ft., are 24 x 54 in. in section. The slab, cast over them as a separate operation, is 12 in. thick over the roadway and 9 in. over the sidewalks and will carry a pavement of wood block on sand or sand-cement fill on the roadway, and a sidewalk pavement of 24-in. square concrete tile on cinder filling.

In the original design a rather elaborate baluster handrail was planned. Precast handrail of this type that was placed on the South Third Ave. bridge has not proved wholly satisfactory, however, and it is thought preferable to cast the rail in place. A much plainer rail, moreover, is considered quite as satisfactory in appearance as one containing ornate spindles.

Floor expansion is provided in general at alternate floorbeams, or about 29 ft. apart. The expansion end of the slab is here supported on a brass plate laid on half of the floorbeam. The curb is cast in place, separate from the floor slab. The sidewalk coping or fascia course, which also forms the face of the balustrade, is similarly cast in place.

Loading and Stress Analysis—A highway loading of maximum type was assumed in proportioning the structure. The arches were analyzed by the elastic theory as fixed-end arches, the radial springing-line joints being considered fixed. Partial loading and temperature effects were considered.

For the two street-car tracks, 40-ton four-axle cars were assumed, using the axle loads direct for the floor slab and beams, and an equivalent uniform load of 1,200 lb. per lineal foot of track for the main arches. In designing the floorbeams, one axle on each track was placed directly over the floorbeam. For designing the slab (spanning from floorbeam to floorbeam), a four-wheel truck was placed in maximum moment position (center of span at one-quarter wheel base). The load of one track was assumed to be borne by an 8-ft. width of slab, this being the width under a 6-ft. tie plus 1 ft. on either side. As the ties will be bedded in cinder concrete, an effective distribution of the rail loads along their length is expected to be obtained.

For the roadway and sidewalks, a load of 100 lb. per square foot for slab and beams was assumed outside the car tracks, and for the arch ribs a load of 80 lb. The

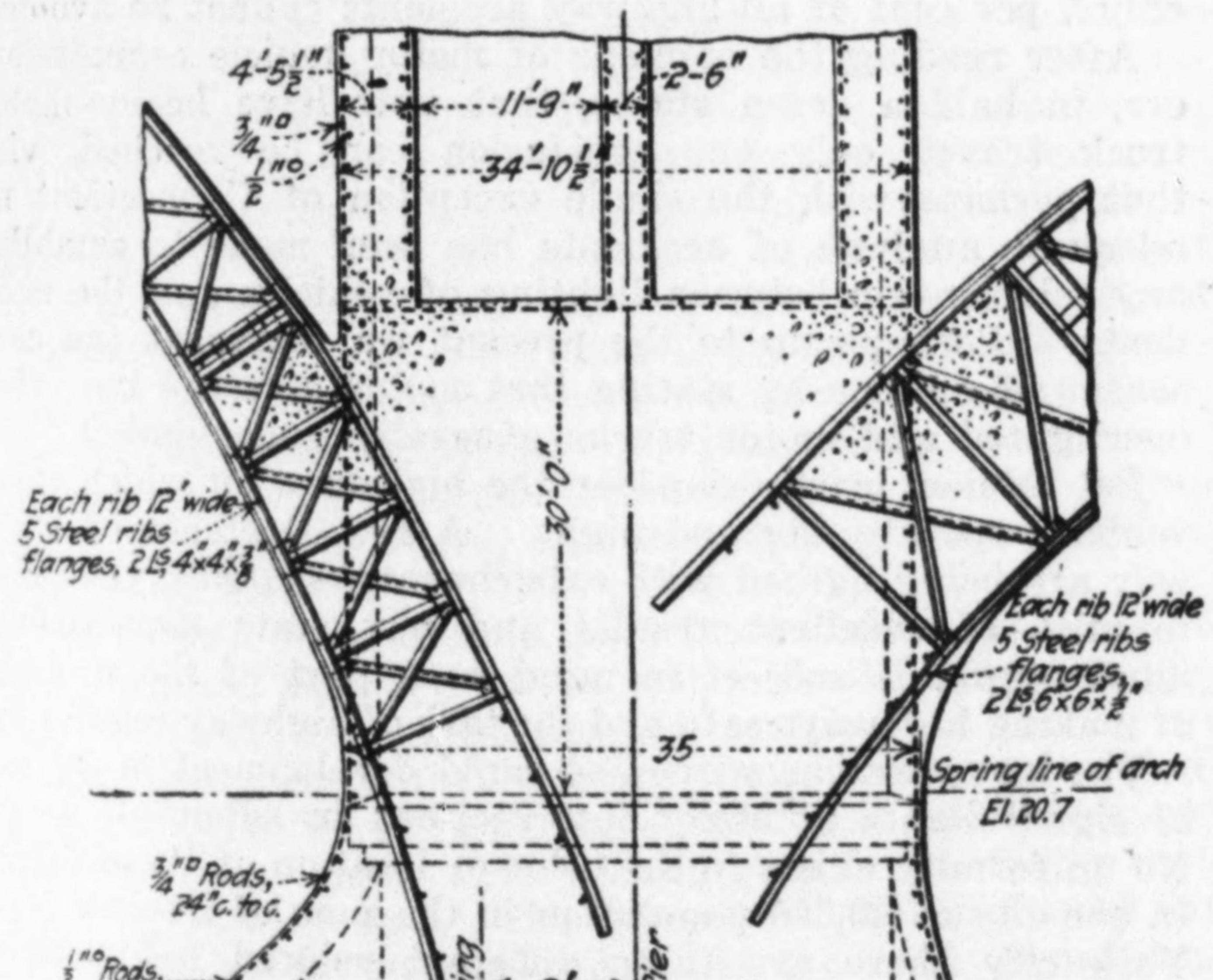


FIG. 5—MAIN PIER AND RIB HAUNCHES

width preempted by the street-car loading was taken as 20 ft., leaving a 10-ft. roadway-loading strip on either side. No impact was added to the roadway loading, but 25 per cent impact was added to the car weight in designing the slab and beams. The compression in the 400-ft. span is 563 lb. per sq.in. at the crown and 408 lb.

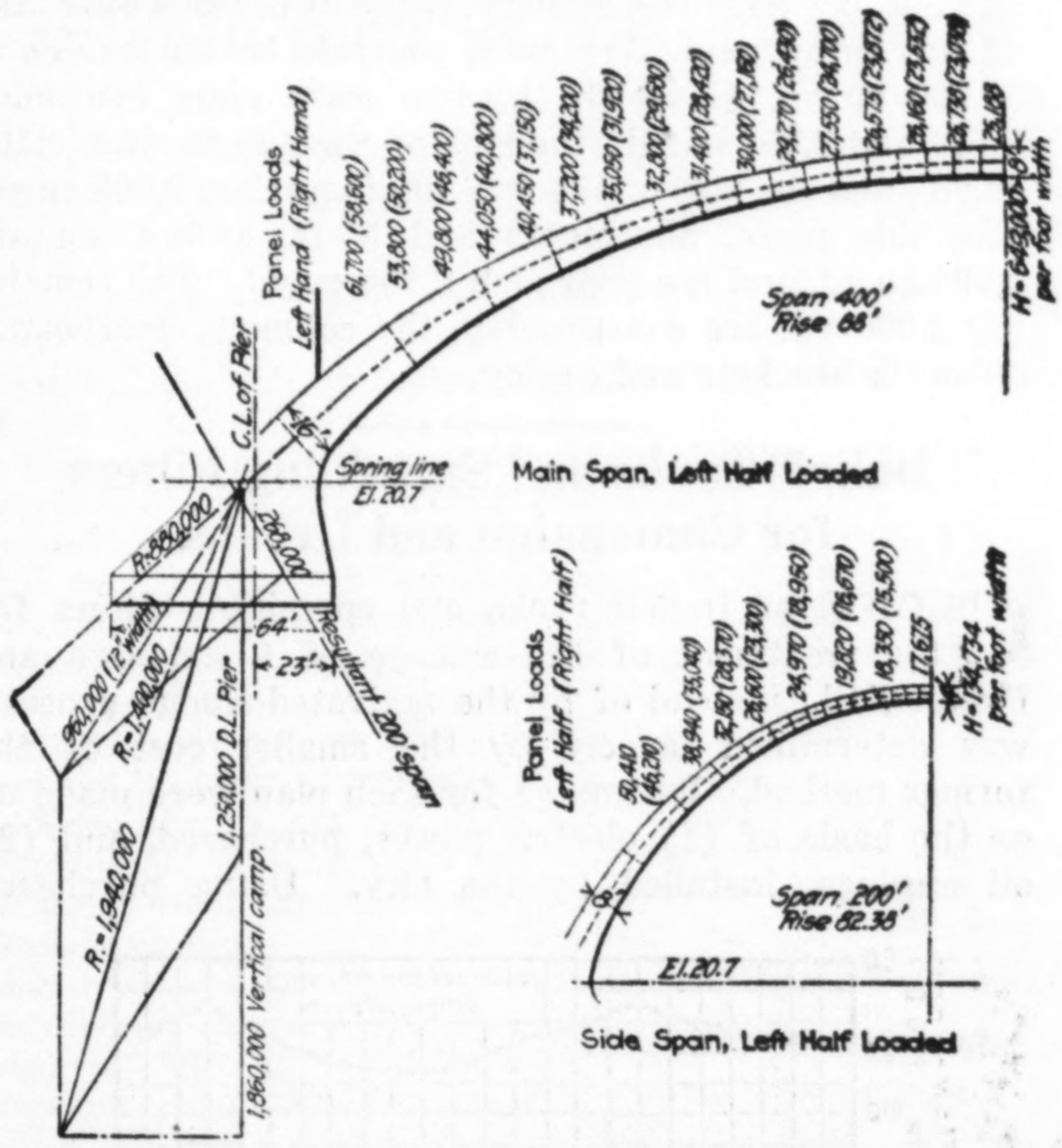


FIG. 6—PRESSURE LINES FOR LEFT-HALF LOADING

at the spring line; a temperature rise of 40 deg. would add 16 lb. per sq.in., making the maximum compression 579 lb.

The arch stresses nowhere exceed 600 lb. per square inch (compression). The slab and beams were designed in accord with the recommendations of the Joint Committee on Reinforced Concrete (650 lb. compression in the concrete, 16,000 lb. tension in the steel, and straight-line stress variation).

Construction—Construction was carried out with an interesting type of centering for the large arch, as indicated in Fig. 2, arranged for successive concreting of the two ribs. Workmanship of high quality was obtained, and the work progressed smoothly according to the schedule laid down in advance, except for the delay caused by a fire in the east portion of the main span falsework last winter, as noted in *Engineering News-Record* Feb. 9, 1922, p. 257. In spite of this the downstream (south) rib of the main arch was concreted in the last week of April and the other rib by fall. The sidewalk slabs were placed before the old spans were removed; the roadway floorbeams and floor are now being constructed after removal of the interfering parts of the old bridge.

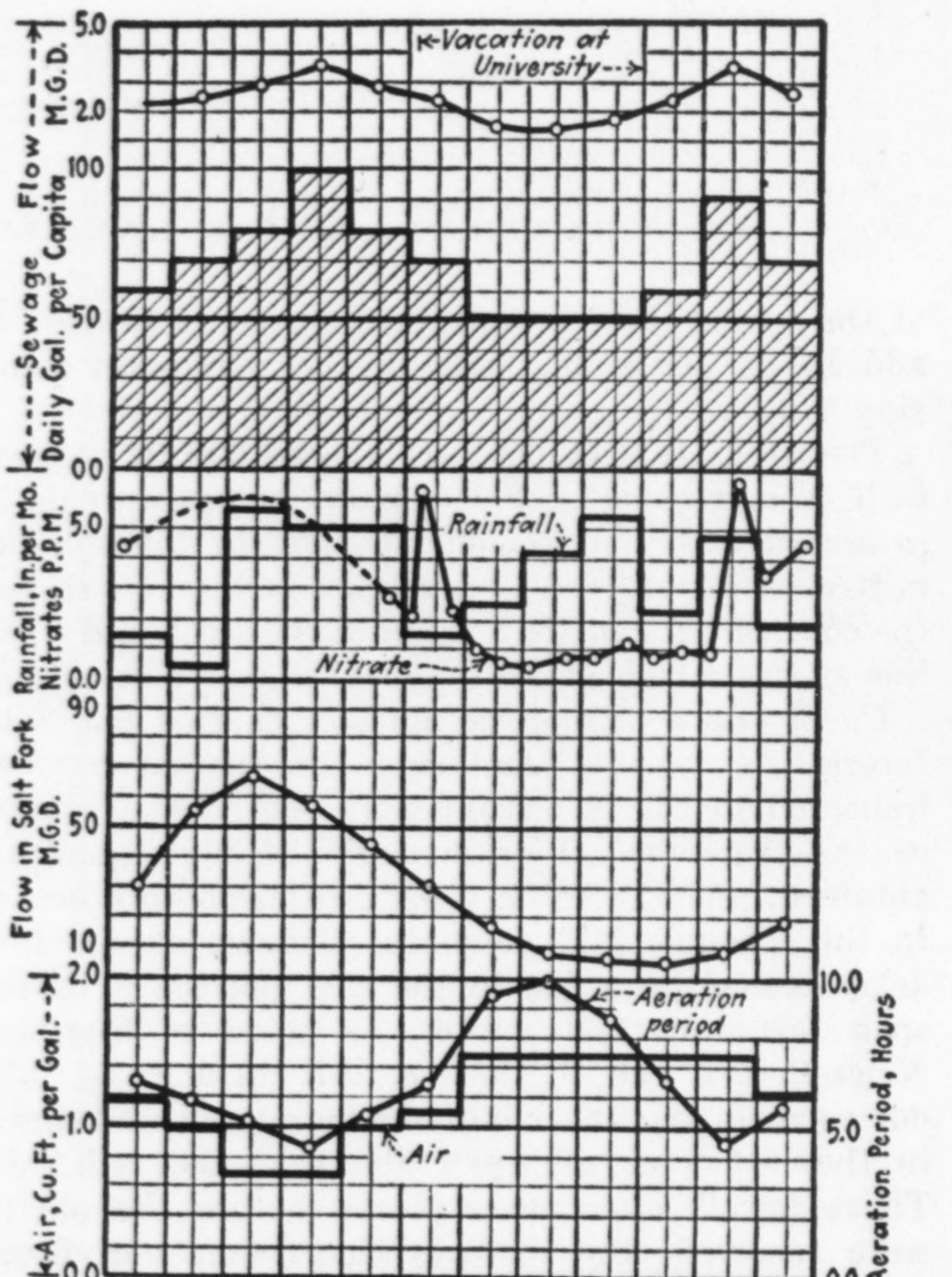
The ribs of the 400-ft. arch settled about 3 in. while they were being concreted, due to compression of the centering. When the centers were struck, about 6 weeks later, very little further settlement took place.

After the general design of the bridge was worked out by the late F. W. Cappelen, city engineer, the details were designed by K. Oustad, city bridge engineer. Until Mr. Cappelen's death the construction work was in charge of N. W. Elsberg. Since the latter became city engineer, F. T. Paul has been in charge of construction. J. E. Lundstrom is general foreman and Wm. McCaulley master mechanic.

Costs of materials and labor have risen since the work was begun, but it is expected that the original estimate of $900,000 for the total cost will not be exceeded. At the beginning common labor and skilled labor were paid 50c and 80c. per hour respectively, while at present 62½c. and $1 are being paid. The total concrete in the bridge is 30,000 cu.yd., of which the two main piers consumed 5,750 and 7,250 cu.yd., the ribs of the 400-ft. arch 4,100 cu.yd., and the ribs of the two 199-ft. arches 2,900 cu.yd. The side piers, abutments and 55-ft. arches contain 4,900 cu.yd. and the floor slab 2,100 cu.yd. The remaining 3,000 yd. are consumed in the columns, floorbeams, sidewalk brackets and coping.

Imhoff Tanks and Sprinkling Filters for Champaign and Urbana

CHOICE of Imhoff tanks and sprinkling filters for the treatment of the sewage of Champaign and Urbana, Ill., instead of by the activated-sludge process, was determined largely by the smaller cost of the former method. Estimates for each plan were made up on the basis of (1) electric power, purchased, and (2) oil engines, installed by the city. Using purchased



FACTORS AFFECTING AERATION PERIODS OF PROPOSED CHAMPAIGN-URBANA SEWAGE-WORKS

Based on estimated quantities in 1925, with reduction in air requirements in spring due to sewage dilution at that time. High nitrates also reduce air required in accordance with experiments of Dr. Edward Bartow.

power, tanks and filters were estimated to cost $400,000, with an annual operation cost of $16,800, making a total annual charge of $46,800. With oil-engine power, the first cost would be $410,000, the annual operation cost $14,200, and the total annual charge $44,950. For the activated-sludge plant, purchased power would require an outlay of $300,000, a yearly operating cost of $44,000 and a total charge of $65,500. If oil-engine power were used the first cost of an activated-sludge plant would be $345,000, the operating expense $33,800 and the total annual charge $69,600. It was these figures that led Pearse, Greeley & Hansen, consulting engineers, to decide on tanks and filters rather than activation, since either method would be satisfactory. The electrical power was figured to cost an average of 2.9c. per kw.-hr. Few, if any, activated-sludge plants in this country, the engineers mentioned state, have been built where the cost was more than 1c. per kilowatt-hour.

In the accompanying chart are shown factors affecting aeration estimated for the 1925 capacity of 2.5 m.g.d. and it was on the required air per gallon that the final figures given above were based.

The sewage-works will be built by the Urbana and Champaign Sanitary District and paid for by a $500,000 bond issue. In the election campaign for these bonds well prepared booklets were distributed to help win the support of citizens and taxpayers.

Lighting Highways and Motor Vehicles

By DAVID BEECROFT
Vice-President, North Atlantic Division, National Highway Traffic Association

Extract from a paper presented at joint meeting of the North Central Division of the National Highway Traffic Association and the Michigan State Good Roads Association.

CONNECTICUT has gone further than any other state in a study of accidents and for two years has investigated every accident occurring on the highways of the state and has classified these accidents showing the number for which the blame is due to motor cars, motor cycles, trolley cars, horse vehicles, railroads, pedestrians, etc. This analysis for 1921 shows that approximately 91 per cent of all accidents on the highways can be eliminated by highway improvements, better highway lighting, improved signals, better lighting of motor vehicles, improved regulation of all vehicles and general education of those using the highway. The Commission of Motor Vehicles believes that only 7 per cent of all highway accidents cannot be avoided.

After reading the opinions of motor vehicle commissioners, in half a dozen states, that now have heavy motor truck travel, only one conclusion can be reached, viz: that perhaps with the single exception of Connecticut no adequate analysis of accidents has been made to establish any relationship between lighting of vehicles and the accidents. Perhaps up to the present we represent the consensus of opinion by stating that motor cars are too often overlighted and motor trucks generally underlighted.

Let us now briefly consider the highways on which these vehicles operate day and night. A few stretches of highway are being lighted with experimental systems, that give promise of excellent results and our state departments must keep this subject in mind as a part of the problem of making highways safe and the task of highway regulation.

The las' year has witnessed rapid development in the use of signal lights at street intersections in suburban areas. No uniformity exists in the color or location of these lights. In one city of 40,000 population in the zone of Greater New York city there are three different colored lights on a leading highway in the distance of two miles. All three are blinking types of intermittent lights, one showing a green light in both directions; another a green in one direction and white in another; and another red in one direction and green in another.

There is no field for intermittent lights of this character on the highway because of the tax they impose on the mental energy of the driver. A steady green light is preferable.

In a night trip of 20 miles north from New York City, and in other directions as well, the driver is confronted with a veritable chaos of lighting schemes, each of which demands a mental problem at a time when the driver is approaching a danger zone and when most of his mental alertness is needed in watching other users of the highway and controlling his vehicle. All highway signal lights of this character should be standardized not only as to color, but as to height and location.