Reinforced-Concrete Viaduct Between Dallas and Oak Cliff, Texas.

One of the longest reinforced-concrete arch viaducts ever built is now under construction across a shallow valley between Dallas, Tex., and a suburb known as Oak Cliff. We have received from Mr. V. H. Cochrane, M. Am. Soc. C. E., a member of the firm of Hedrick & Cochrane, Kansas City, Mo., which is in charge of the construction of the bridge, an extensive account of the work. From this account we have prepared the following article.

General Description.

The Trinity River, ordinarily a small stream not more than 200 ft. wide, runs in a shallow valley between the city of Dallas and its suburb of Oak Cliff. At flood stages, however, the river rises sometimes as much as 50 ft. and spreads across the whole width of its mile and a half valley, completely shutting off the suburb from the main city. Some time ago it was decided that to avoid such interruptions in traffic it would be necessary to build a bridge or viaduct across the river and its valley, and accordingly the County of Dallas about two years ago voted a bond issue of $600,000 with which to build the structure. These bonds were sold and, after a sufficient amount was reserved to cover the cost of the right-of-way and the Oak Cliff approach, a fund of $563,000 remained available for the construction of the bridge. The county is undertaking the work alone without the aid of either the city of Dallas or the city of Oak Cliff. Although at one time the railroads whose tracks were being crossed were expected to contribute something towards the cost of construction, up to this time no such contributions have been received.

In November, 1909, the County Engineer, Mr. J. F. Witt, advertised for competitive plans to be submitted on or before Jan. 1, 1910. This advertisement contained the following chief requirements:

(1) The structure must be of reinforced concrete, of either arch or trestle construction, and must provide a roadway for vehicular traffic, and two sidewalks. Provision must be made for a double track electric railway to be added at some future time.

(2) The clear width between hand rails must be not less than 50 ft., and should be more if a wider structure can be built at a cost not to exceed the amount available.

(3) Conduit spaces easy of access must be provided, having a cross-sectional area of not less than 20 sq. ft.

(4) The live loads to be provided for are as follows:

On each electric railway track, two 100,000-lb. cars.

On the roadway, 100 lbs. per sq. ft., or a 15-ton road roller having a maximum axle concentration of 10 tons.

On the sidewalks, 80 lbs. per sq. ft.

(5) The design should be more than a mere picture or side elevation; sufficient details should be given to enable the relative merits of the design to be determined. Each designer shall furnish a complete analysis of the principal span or spans, and a complete list of working stresses; also sample specifications, an itemized estimate of cost, and a summary of his professional experience.

The County Court appointed an Advisory Board, consisting of Prof. T. U. Taylor, M. Am. Soc. C. E., Professor of Civil Engineering, University of Texas; Mr. Otto H. Lang, M. Am. Soc. C. E., and Mr. N. Werenskiold, to examine the various designs submitted and to report upon them to the court. Out of the 15 designs submitted, the Board recommended the adoption of the arch design submitted by Mr. Ira G. Hedrick, M. Am. Soc. C. E., Kansas City, Mo., with Mr. M. R. Ash, M. Am. Soc. C. E., as associate engineer. With two exceptions Mr. Hedrick's preliminary design was substantially the same as that adopted for construction. These two exceptions were the final adoption of piled footings instead of spread reinforced-concrete footings and the widening of the roadway of the bridge from 40 ft., with two 5-ft. sidewalks, to 44 ft., with two 4½-ft. sidewalks. The first of these changes was required by the order of the Advisory Board, which was not sufficiently satisfied with the compact jointed clay soil which was discovered by borings to guarantee the 2-ton per sq. ft. reinforced-concrete footings originally designed. The second change was made because it was found that the contract price was enough less than the estimated cost to enable the widening.

Fig. 1 is an elevation of the structure. Beginning at the Dallas end, it comprises 4[illegible] ft. of reinforced-concrete trestle construction; 27 reinforced-concrete arches, each 71-ft. 6-in. span; a 98-ft. concrete covered steel plate girder over the river (with 90 ft. clear opening); 24 reinforced-concrete arches of 71-ft. 6-in. span, and 181 ft. of reinforced-concrete trestle—a total length of 5,106 ft. At each end there is a decided angle in the line of the roadway, in the trestle approach portion. It will be noticed that at ordinary river height all but the middle span will be over dry land. At high water, however, the water level will reach to the trestles. The large girder in the middle was required because the U. S. War Department demanded a 90-ft. horizontal clearance and a 60-ft. vertical clearance throughout the entire width of 90 ft. at the river. In order to keep the grade of the viaduct as low as possible, it was therefore necessary to have a girder, and the engineers thought it better that this girder be of steel encased in concrete than of the heavier reinforced-concrete type.

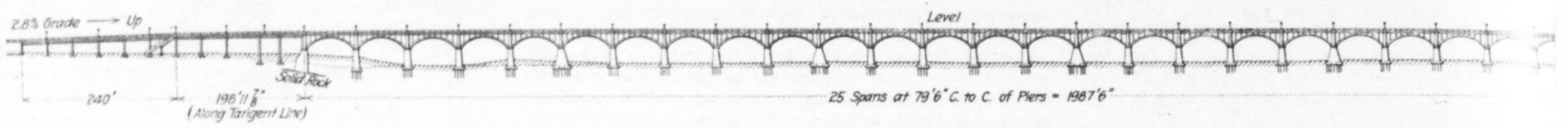


FIG. 1. GENERAL ELEVATION AND PART PLAN OF

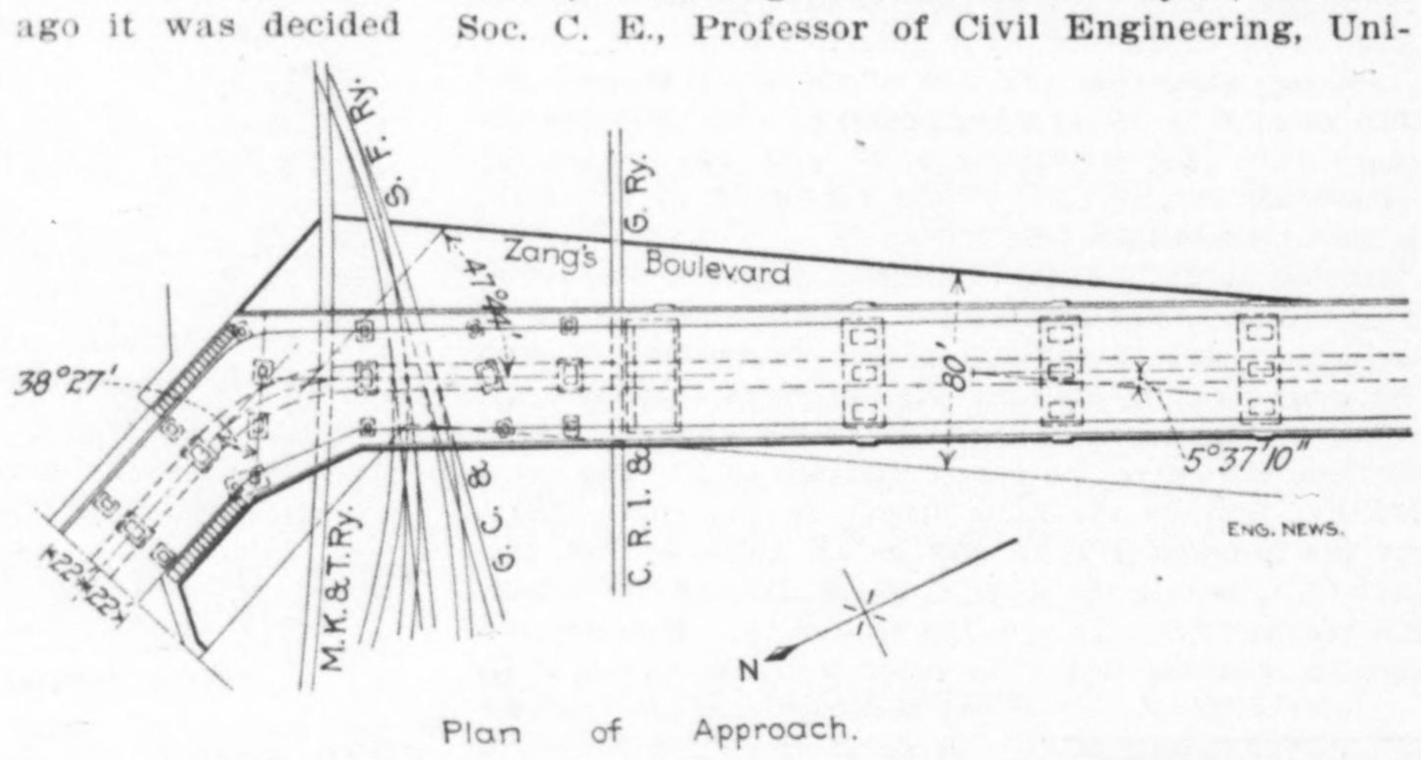


FIG. 2. PLAN OF DALLAS END OF VIADUCT.

Foundation.

The specifications and drawings call for concrete piles for all piers except the two river piers, where long timber piles are used. For the concrete piles the following tests were prescribed:

The safe load assumed for each concrete pile is 30 tons and the contractor will be required to test four piles. Each of the test piles shall first be loaded with 30 tons for 48 hours and must show no settlement. Each pile shall then be loaded with 60 tons and shall show not more than ¼-in. settlement after the load has been applied for 48 hours. Should the test piles fail to sustain the load specified, the contractor must increase the size and length of piling or increase their number, without extra charge, so that in the opinion of the engineer the piling used will carry safely a load equivalent to 30 tons for each pile shown on the plans.

It was soon found that this provision was beyond the possibility of the piles, so it was decided the working load on the piles be reduced from 30 tons to 20 tons each and that the bases of the piers be increased in size. The contractor agreed to stand the extra expense involved, and accordingly the piers were redesigned for the following working loads:

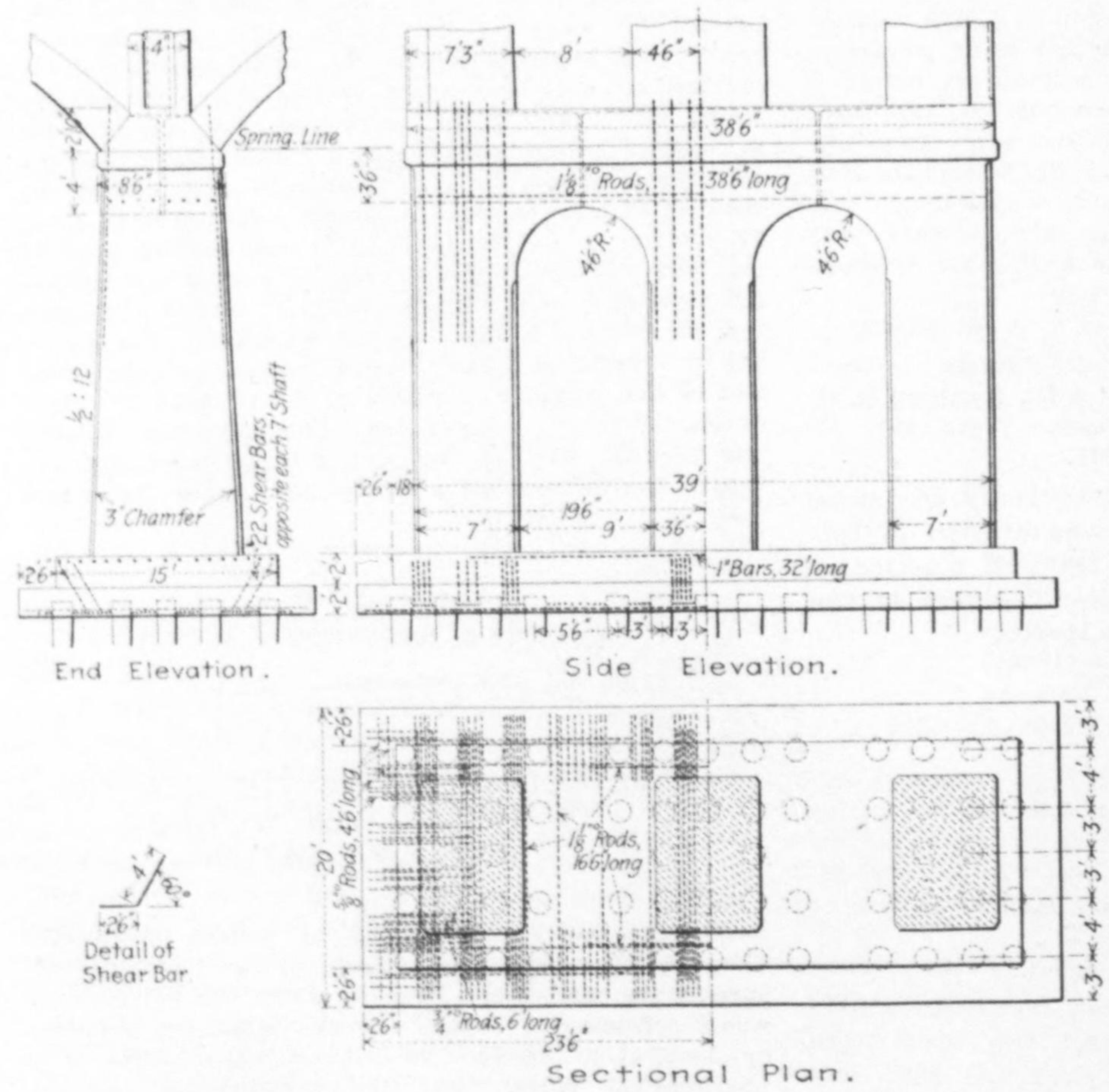
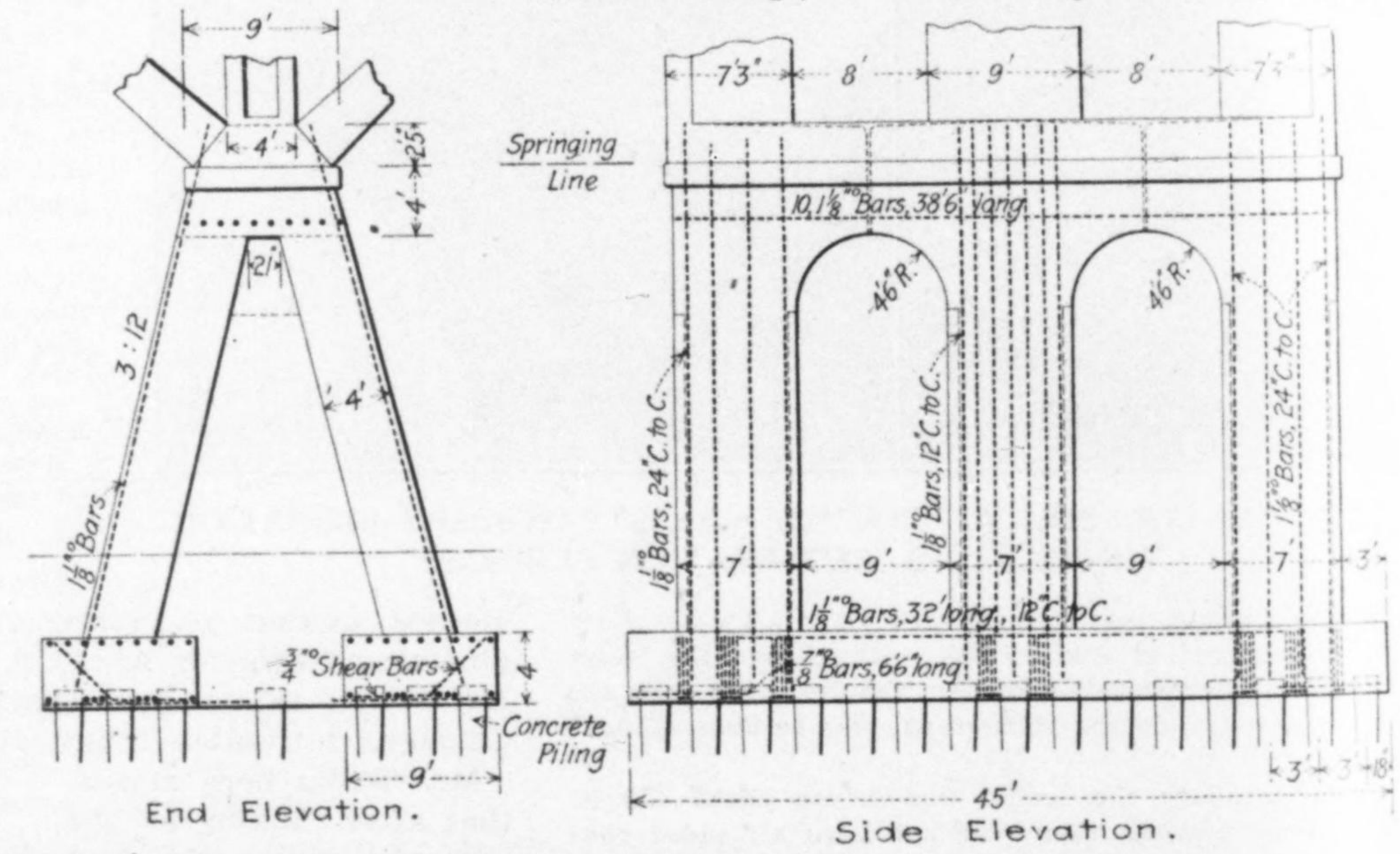


FIG. 3. TYPICAL ARCH PIER ON DALLAS-OAK CLIFF VIADUCT.



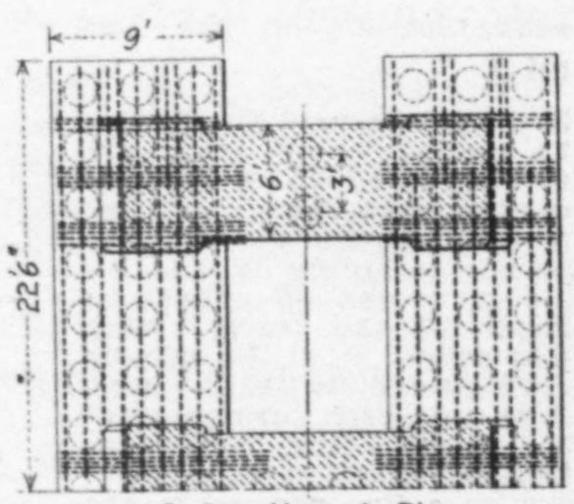


FIG. 4. ABUTMENT PIER.

	Unbalanced live load condition.
Extreme load per pile, tons................	20
Extreme soil pressure, tons, per sq. ft....	1.6

Several tests of the soil were made and fairly uniform results were arrived at. This soil is a hard, compact black or yellow clay underlaid by sand or sand and gravel in places. As the clay is very dry, owing to the long drought which has prevailed at Dallas, the bearing capacity of the soil was tested by pouring water into the pits and allowing it to stand. According to most of the tests, it appeared that under a constant load, settlement continues for a few hours and then practically ceases. When the load was increased to ten tons about, water was added to the pit and was kept standing 3 ft. deep. On the whole, the settlement was proportional to the load, varying in one typical case from 1/16-in. under six tons to ¼-in. under 10 tons per sq. ft. A maximum load of 20 tons produced a permanent settlement of about ⅜-in. This test was made with a 3½ × 3½-in. timber bearing on the soil, but the same results practically were given with a 12 × 12-in. timber.

The concrete piles were made in place by pouring a 1:2:4 concrete into holes prepared by driving a core into the ground and then withdrawing it. The average penetration was about 14 ft. The following table gives the results of two groups of typical tests on concrete piles in place. One is a hammer test and the other a load test.

Pile No.	Penetration, ft.	Blows per inch, last 6 ins.	Settlement in inches under a load of 10 tons.	20 tons.	22½ tons.	25 tons.	30 tons.
52......	13	40	0	0	....	....	0
55......	11.5	54	0	0	....	....	⅜
82......	13.5	40	0	0	...	....	⅜
85......	14	28	0	0	....	....	0
39......	14.8	42	0	9/16	11/16	1 1/16	1¾
42......	14	48	0	9/16	11/16	1⅛	1 11/16
60......	14	60	0	⅜	⅜	⅝	1 7/16

Weight of hammer, 7,000 lbs.; stroke, 36 ins.
Weight of ram, 3,000 lbs.
Weight of former, 4,700 lbs.; Diam. at top, 14 ins. at bottom, 10 ins.

Some of these piles were exposed after being tested, and it was found that several were cracked in one or more places. It appeared that these cracks were caused by the vibration of the ground while the adjacent piles were being driven, and were made while the concrete was green. In a few cases the cracks had opened ½-in. showing that the soil had risen and had lifted the upper portion of the pile. In some cases the cross-sectional area of the pile had been reduced owing to the pressure of the surrounding soil caused by driving adjacent piles. It was found to be impossible to tamp the concrete into the soil so as to increase the diameter of the piles. The subcontractor who was driving the piles was not willing to undertake to remedy these defects by changing the methods of driving and forming the piles. It was decided that it would be safe to assume that the piles would carry an average load of 20 tons each so the subcontractor agreed to increase the size of the bases of the piers at his own expense so as to make good the deficiency in the carrying capacity.

Piers.

There are several types of piers in the bridge. The standard arch pier is shown in Fig. 3. Every fifth pier in the arch span (Fig. 4) is made sufficiently large to resist the deadload thrust of one adjacent span in case the arch on the other side of the pier should fail. The two piers on either side of the river span are of special design to carry the horizontal thrust of the arch spans in bending (Fig. 5). The tension side of each pier is reinforced with corrugated bars. These piers will be founded on 168 timber piles. The abutment pier at the Dallas end of the arch span was carried down to bed rock. The other abutment pier was founded on piles, one-half of which are driven on a slope of 4½ ins. to 1 ft. to care for the horizontal thrust.

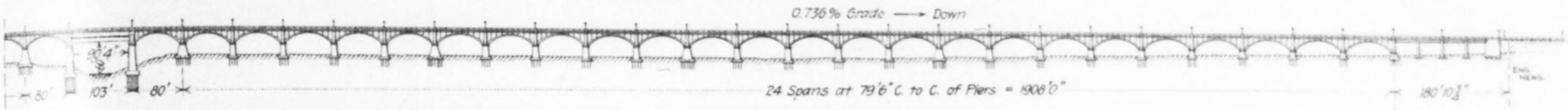
Both the ordinary piers and the so-called abutment piers are pierced longitudinally by two openings 9 ft. wide spanned at the top by arches. The distance from the springing line to the top of these openings is 4 ft.; the reinforcing bars are placed just above them.

Trestle.

The trestle is a reinforced-concrete slab resting on longitudinal girders, which in turn rest on cross walls supported by three columns. The details are shown in Fig. 7. The only extraordinary feature of this design is the rocker bearing provided for the girders shown in Fig. 7. On one side of the cross girder the longitudinal girders are rigidly connected to it by means of a reinforced-concrete bracket. On the other side each longitudinal girder rests on a socket formed by two bent copper plates extending the full width of the girder, the lower plate resting on the cross girder and the upper being fastened to the longitudinal girder. The longitudinal girders on either side are connected by means of reinforcing bars just above the socket, but a cleavage plane is left between them extending through the floor slab so that the girder which rests in the socket is more or less free to rotate. The girders are thus discontinuous and are designed as simple beams. At suitable intervals expansion joints are introduced. At these joints the lower copper plate is replaced by two cast-iron plates, the lower being imbedded in the cross girder and the upper sliding on it, as also shown in Fig. 7.

Arches.

Each arch is 71 ft. 6 ins. clear span and 79 ft. 6 ins. c. to c. of piers with a rise of 17 ft. It consists of a solid arch span, reinforced, as shown in Fig. 6, in upper and lower planes and carrying a flat slab floor by means of vertical cross walls pierced by openings so as to make really three columns carrying the transverse girder. The roadway is so arranged as to allow for a future railroad track, but at present only a macadam roadway has been placed. Expansion joints are provided in the floor slab over the second column from each pier, and the sections are separated from each other by a strip of asbestos board. It will be noted that the roadway slab, the curb and the sidewalk slabs are in one integral piece, but that the sidewalks are supported from a cantilevered bracket.



THE DALLAS-OAK CLIFF CONCRETE VIADUCT.

Concrete-Encased Girder.

The span over the river consists of four steel girders 98 ft. between supports, connected by steel cross girders, the whole to be encased, as shown in Fig. 8, in concrete. The steel girders carry in direct compression the horizontal thrust of the adjacent arch spans. This thrust is carried into the bottom flanges of the girders by means of the special cast-iron shoe shown in Fig. 8. The upright portion of the shoe bears against the concrete of the pile on one side and against the cast-steel wedge on the other, the wedge being driven between the cast-steel shoe and the end of the bottom flange angles. In all other details the girders are of the usual type of construction.

Material.

After a number of tests by Prof. T. U. Taylor as to the relative strength of concrete made with Dallas gravel and Jacksboro stone, the only crushed stone available in the vicinity, it was decided that the gravel concrete was quite as strong as broken-stone concrete for 1:2:4 mixtures, though for leaner mixtures the broken stone showed up better. Consequently, a sand and gravel concrete of 1:2:4 mixture was prescribed, with the proportions to be modified if the engineer so decided. The specifications for the grades of concrete are as follows:

For the arch rings, girders, cantilevers and spandrel walls (above a plane through bottom of cantilevers), and for girders, bents, floor slabs and arch piers (above a plane 3 ft. 6 ins. below the springing line of arches), and

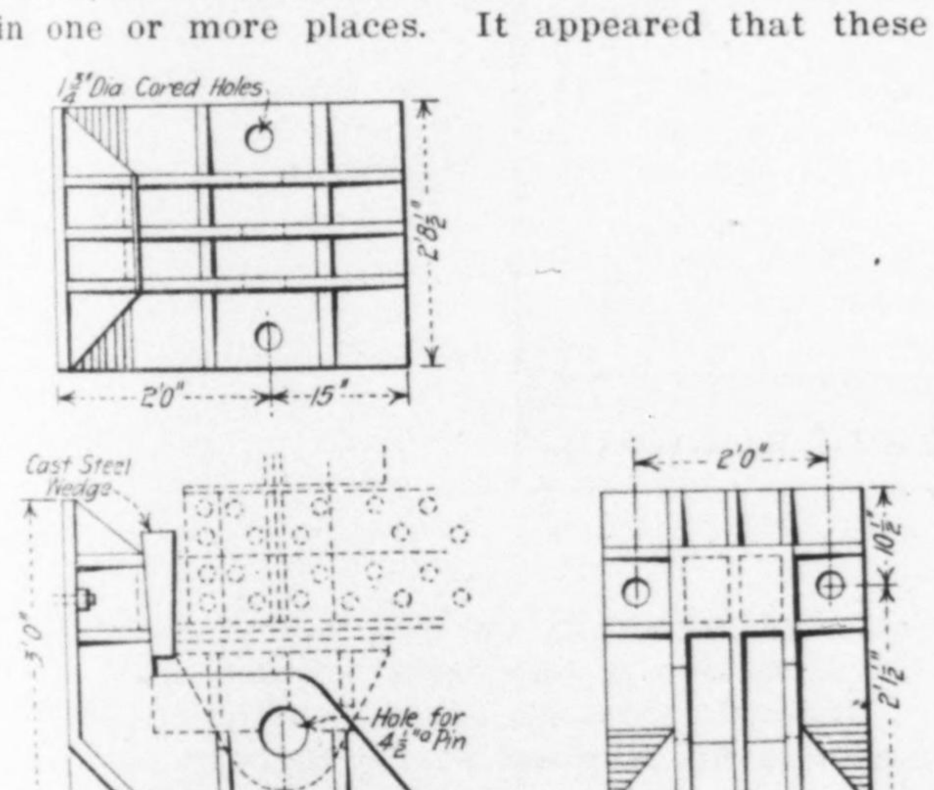
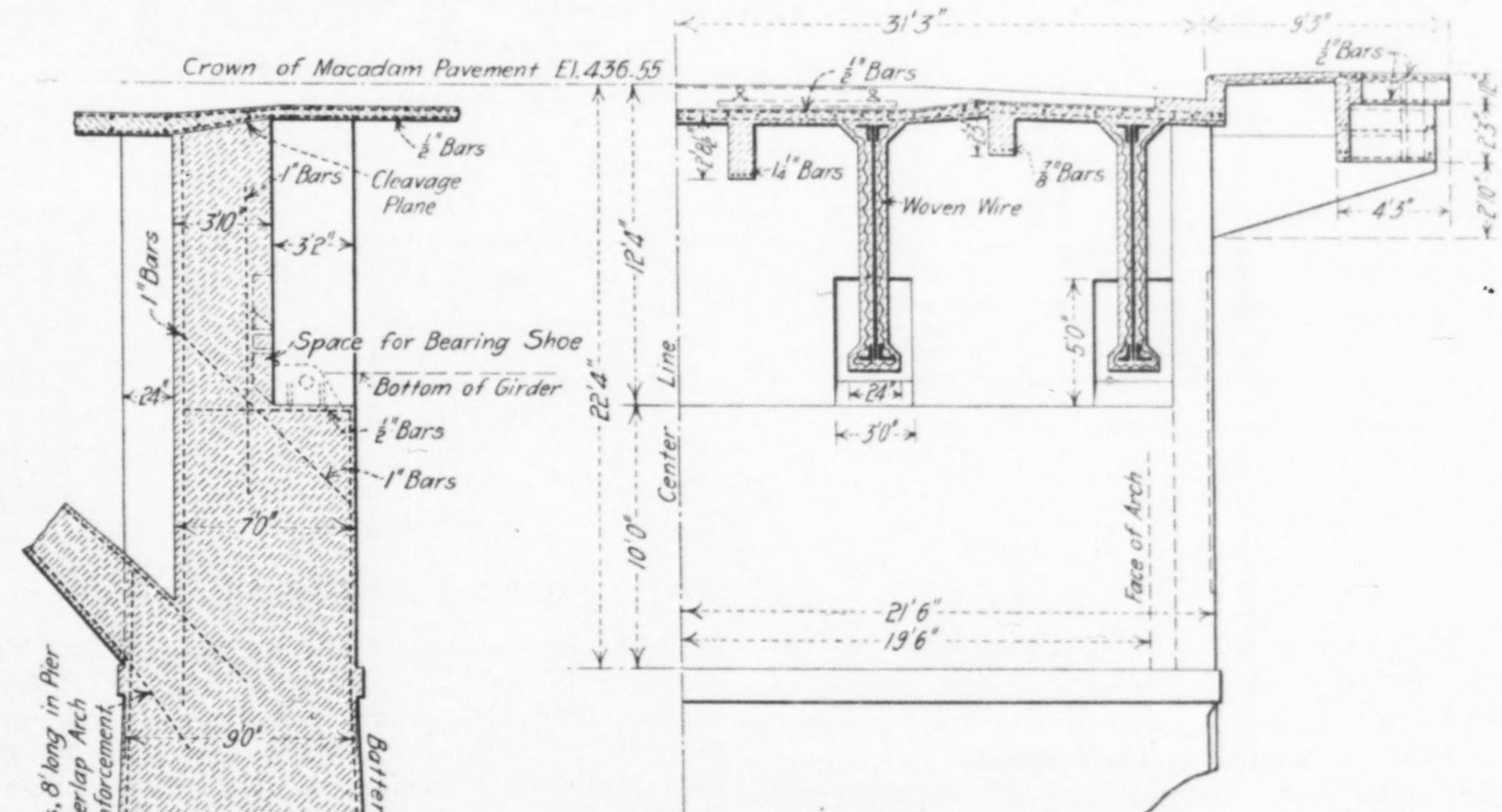


FIG. 5. DETAILS OF PIER FOR SUPPORT OF MIDDLE GIRDER SPAN, DALLAS-OAK CLIFF VIADUCT.



for encasing the metal work for the steel span over the Trinity River, the concrete shall be composed of one part Portland cement, two parts sand, and four parts stone or gravel to pass in any direction through a 1½-in. ring.

Concrete in all cross spandrel walls (below the plane of bottom of cantilevers), in the arch piers (below a plane 3 ft. 6 ins. below springing line of arches), and in the two end abutments and retaining walls shall be composed of one part Portland cement, three parts sand, and five parts stone or gravel to pass in any direction through a 1½-in. ring.

Concrete for curbs, wheel guards, sidewalks and parapets or copings shall be composed of one part Portland cement to three parts of a mixture of clean sand and gravel to pass a ½-in. screen.

The bases of all piers, columns and abutments shall be composed of concrete mixed in the proportions of one part Portland cement, three parts sand and five parts gravel to pass in any direction through a 2½-in. ring, or if broken stone be used the proportions shall be one part Portland cement, three parts sand and six parts of broken stone to pass a 2½-in. ring.

The reinforcing bars are of round corrugated shape and of medium open-hearth steel, having an ultimate strength of 60,000 to 68,000 lbs. per sq. in.; elastic limit not less than the ultimate strength; elongation of 22%; and the capacity to bend cold 180° to a diameter equal to the piece tested without fracture on the outside of the bend.

Loads and Stresses.

The live loads for the final design were the same as required in the case of the competitive plans. The electric railway loads were increased to allow for impact, 50% for floor slabs and about 13% for arch ribs. No impact allowance was made for the road roller concentrations or the uniformly distributed loads on roadways and sidewalks. The piers were designed for two conditions of loading; first, four 100,000-lb. cars, 80 lbs. per sq. ft. on a width of 22 ft. of roadway outside of the car tracks, and 60 lbs. per sq. ft. on sidewalks, all symmetrically disposed on the two adjacent spans so as to produce balanced horizontal thrusts; second, four 100,000-lb. cars, 40 lbs. per sq. ft. on a width of 22 ft. of roadway, and 30 lbs. per sq. ft. on sidewalks, all on one of the two adjacent spans and producing an unbalanced horizontal thrust of 7,000 lbs. per lin. ft. of width of arch ring. For the latter condition, which is considered quite unlikely to occur, it was computed that 2,000 lbs. could, if necessary, be carried through the adjacent arch rib to the next pier, thus leaving an unbalanced live load thrust of 5,000 lbs. per lin. ft. of width of arch rib to be taken care of by the pier, or 195,000 lbs. altogether.

WORKING STRESSES.

Maximum compression on concrete in arches, excluding temperature stresses	500 lbs. per sq. in.
Maximum compression concrete in girders	600 " " " "
Maximum compression on concrete, including stresses due to 80° F. variation in temperature	750 " " " "
Maximum shear on concrete, plain	30 " " " "
Maximum shear on concrete, reinforced for shear	100 " " " "

No tension in concrete will be allowed.

When temperature stresses are included, the steel reinforcement takes the entire bending moment without being stressed in tension above its elastic limit.

The total cost of the structure, including engineering and all incidentals, will be about $570,000, or about $2.10 per sq. ft. of floor. The contract is held by Corrigan, Lee & Halpin, Kansas City, Mo., who are doing the work, and the concrete piles are being put in place by the Gulf Concrete Construction Co., Houston, Tex. Field work is being carried out under the supervision of Hedrick & Cochrane, Consulting Engineers, Kansas City, Mo., and J. F. Witt, County Engineer. E. N. Noyes is Resident Engineer, representing the Consulting Engineers. Work was begun October, 1910, and should be completed by Dec. 1, 1911.

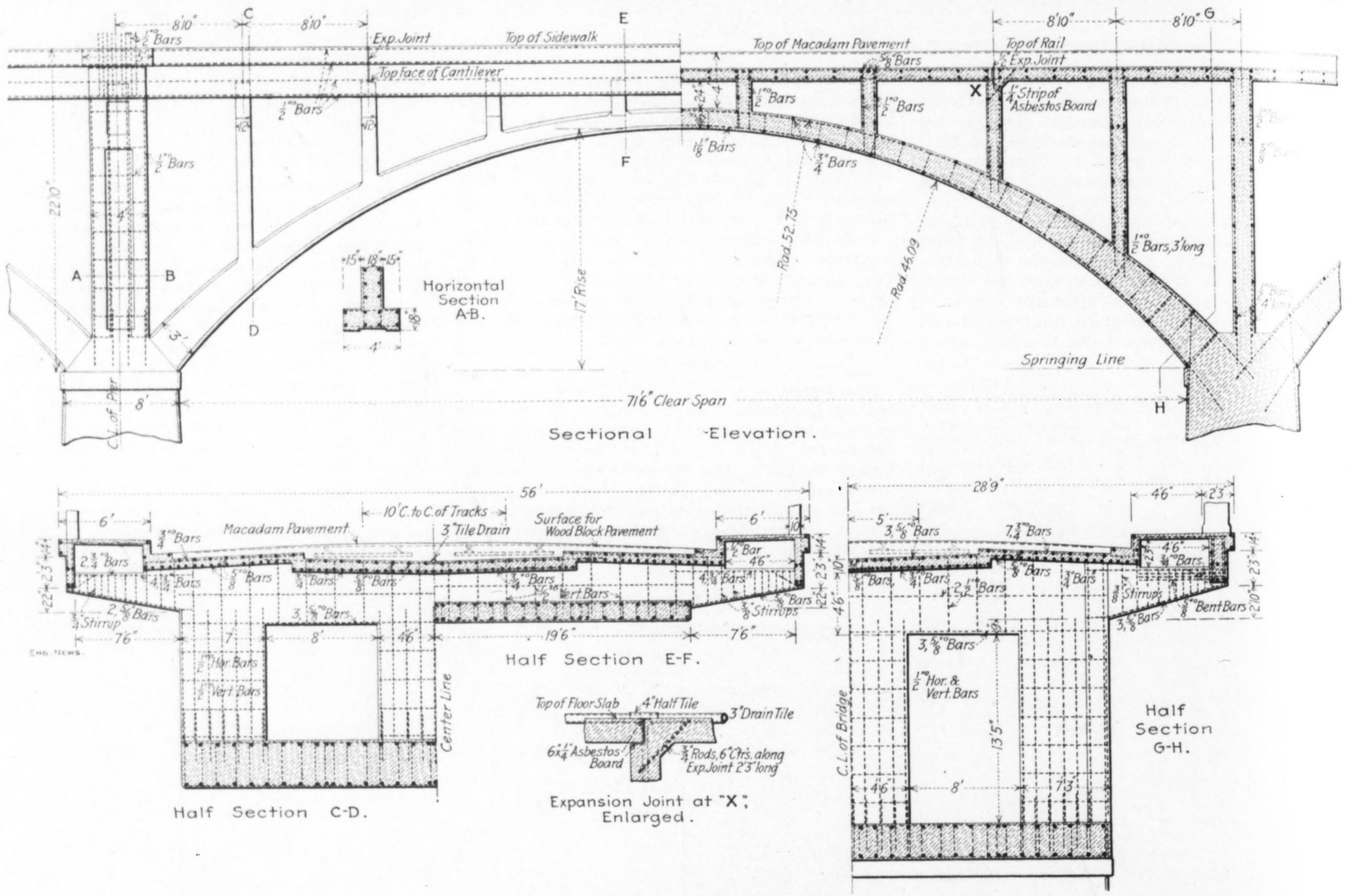


FIG. 6. DETAILS OF ARCH SPAN.

THE LARGEST CANTILEVER CRANE in the world, according to London "Engineering," was recently installed in the Imperial Japanese Navy Dockyard, Yokosuka, by Messrs. Cowans, Sheldon & Co., Ltd., Carlisle, Eng. It is designed for a working load of 200 tons, carried at the end of a 95-ft. radius, and has been tested with 250 tons at that radius. The same firm has under construction for the Japanese navy yard at Kure, a similar crane, designed for a safe load of 200 tons at 105-ft. radius. The capacities of some large cantilever cranes now in existence are as follows: Kure, 26,250 ft. tons; Yokosuka, 23,750 ft. tons; Devonport, Eng., 22,800 ft. tons; Clyde, Eng., 18,750 ft. tons.

Lighting The Double-Hump Yard of the Missouri Pacific Ry. at Dupo, Ill.*

At Dupo, Ill., about 13 miles southeast of St. Louis Union Station on the St. Louis, Iron Mountain & Southern R. R. of the Missouri Pacific Ry. system, is located one of the largest double-hump freight yards in the West, designed to handle both north and south-bound traffic, each hump having a capacity of 120 cars per hour. The yard is about three miles long and about 800 ft. wide at the classifying yards, with connections to the railroad company's Illinois division, East St. Louis, Ivory Ferry, the Terminal Railroad Association, and other systems. The yard is divided into receiving, classification, forwarding, storage, caboose and repair yards. The two humps, the roundhouse, repair yards, coal and water stations, and the power house, together with a hotel and other facilities, are located at practically the longitudinal center of the yard, making it very nearly symmetrical.

Under the direction of the Construction Department of the Missouri Pacific Railway Co., Westinghouse, Church, Kerr & Co. in 1906, acting as Engineers and Constructors for the Railway Co., designed, constructed and equipped their Dupo power house. The following year the Engineers were instructed to investigate and recommend a system of artificial illumination for the yard. All of the commercial systems of out-door arc lighting were investigated, but as the power house contained alternating-current equipment, the direct-current systems could not be given serious consideration. Owing to business conditions, all work in the yard was suspended from 1907 to 1909, in the Fall of which year the subject was again taken up, and, as many improvements had been made in the different lighting systems, another investigation and report was made which included flaming arcs, in addition to the systems previously considered.

Several installations of flaming arcs were visit-

*Description prepared by Mr. W. S. Austin, for Westinghouse, Church, Kerr & Co., 10 Bridge St., New York City.