Engineering News-Record

Vol. 111 New York, Sept. 28, 1933 No. 13

Self-Anchored Suspension Bridge Built in Missouri

Crossing an arm of the Lake of the Ozarks on a farm-to-market type road and having a main span of 225 ft., the unusual structure was built for $37,000—Stiffening girders are 33-in. rolled beams, and cables are prestressed twisted strands

By Howard Mullins
Missouri State Highway Department, Jefferson City, Mo.

ONE OF the most notable recent projects of the Missouri state highway department is the completion of a self-anchored suspension bridge of 450-ft. total length and 225-ft. main span, located in Camden County, about 100 miles south of Jefferson City, on one of the so-called "farm-to-market" routes. It spans the inundated bed of the Little Niangua River, now an arm of the Lake of the Ozarks, formed by the construction of Bagnell Dam, and is believed to be the fourth self-anchored suspension bridge built in the United States, the other three being the Sixth, Seventh and Ninth St. bridges over the Allegheny River at Pittsburgh, Pa. The new bridge is the first stiffened suspension bridge built in Missouri since the completion in 1890 of the Grand Ave. braced-cable suspension structure in St. Louis. Unique features of the design include the application of prestressed strands to the self-anchored type; an unusual arrangement of the strands in the cable; the use of rolled-beam sections for the stiffening girders; the use of a built-up or laminated cable clamp; and the method of handling the expansion movement. The principal dimensions are shown in Fig. 2.

Fig. 1—Short-span self-anchored suspension bridge built by Missouri highway department on a farm-to-market road.

The funds available for the construction of the bridge were limited. It was therefore desirable to select the most economical layout and since the bridge is located in a somewhat scenic region, with much camping and boating near by, it was felt that a design of reasonable esthetic fitness should be used. With these considerations in mind, comparative designs were made using 50-ft. I-beams throughout, three 150-ft. truss spans with one 70-ft. girder span, and an unstiffened suspension layout with the same span arrangement as was used for the self-anchored layout, with sand-filled anchorages.

The I-beam layout was entirely unsuited from an esthetic viewpoint. The truss layout was little better. The unstiffened suspension layout was considered inadequate for a bridge of such light weight and slender proportions. The self-anchored type proved the most economical and was adopted. One of the outstanding features of the self-anchored type is that the greater part of the construction is placed in the superstructure, where it can be seen, and not in expensive unseen anchorages. It is believed that the design as built presents a reasonably pleasing appearance.

The very low cost of the design is believed to be ample justification for its adoption. The total cost of the bridge was $36,914 for a length of approximately 523 ft. This gives a cost of $70.50 per linear foot, or a cost of $3.52 per square foot of floor area.

Basis of design

The bridge was designed for a live load of H10 on one lane only, with an impact allowance not to exceed 30 per cent of the live load. For the design of the towers, which are the only part of the structure stressed by uniform change of temperature, a range of 60 deg. was assumed. The wind load used was 30 lb. per sq.ft. on $1\frac{1}{2}$ times the vertical projection of the structure.

For the concrete substructure a unit stress was used of 500 lb. per sq.in. direct stress and 650 lb. per sq.in. bending. The floor system was proportioned for a working stress of 18,000 lb. per sq.in. in bending. The stress in the stiffening girders was limited to 18,000 lb. per sq.in. for bending and direct stress combined, due to $D+L+I$, and 20,000 lb. per sq.in. due to $D+L+I+$ wind. The towers were proportioned for a working stress of 20,000 lb. per sq.in. due to $D+L+I+$temp.$+$wind$+$ bending. All other parts were proportioned for a basic unit stress of 18,000 lb. per sq.in., reduced for compression. The cable was proportioned for a working stress of 65,000 lb. per sq.in.

The design was prepared using the assumptions usually made in stiffened suspension-bridge design where the elastic theory is employed. The design formulas were set up by the method of least work to include the lack of symmetry of the structure and the work done by the stiffening girder.

Substructure

The east abutment is of U type in which the walls are carried about 2 ft. below ground line at the front, and only the columns at each corner and at the center of the tie beams in the rear are carried to rock.

The two main piers consist of two columns of uniform section converging into a rectangular section under the coping and connected by a pointed-arch

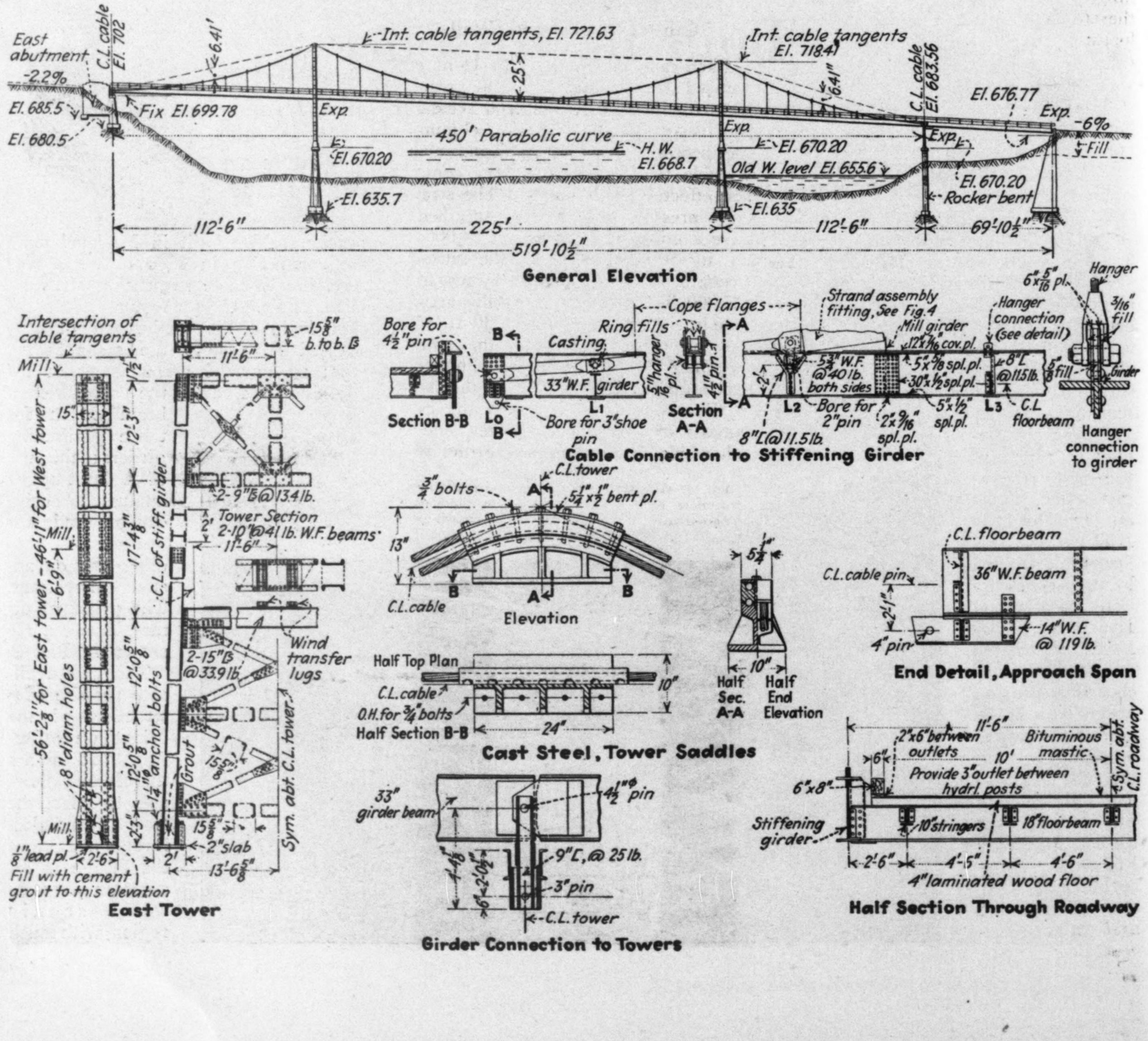

Fig. 2—Elevation and details of 225-ft.-span self-anchored suspension bridge. Stiffening girders are 33-in. rolled sections hinged at the towers, which are of fixed-base flexible type. Cables consist of four 1¼-in. twisted wire strands, pin-connected to the side-span girders by 9-in. eyeplates.

web. The footings are founded on rock 25 ft. below pool stage of the lake.

The rocker bent pier is an ordinary rectangular framed bent founded on rock and extended to 1.5 ft. above extreme high water.

The west abutment is of open type construction with back wall and wings just sufficient to retain the fill. It also is founded on rock.

Girders, floor and towers

The stiffening girder for a self-anchored suspension bridge is one of the major stress-carrying members. Great care therefore must be used in its proportioning and in the selection of suitable working stresses.

As no great rigidity was necessary for the traffic to be cared for, the stiffening girders were made of 33-in. rolled sections throughout, giving a ratio of depth to main span of about 1/82. In the side spans the girders weigh 141 lb. per foot, and in addition are provided with a coverplate over the top flange for the central 50 ft. The main span girders consist of 125-, 152- and 141-lb. sections. The girders are hinged at the towers and connected thereto as shown in Fig. 2 to permit longitudinal and prevent vertical movement.

At each floor beam the girder is stiffened by an 8-in. channel. The top flange of the stiffening girders at these points is slotted each side of the web to receive the hanger connection plates.

The floor consists of 2x4-in. creosoted pine laid on edge and fastened with metal clamps to the stringers, which are 10-in. W.F. sections spaced $4\frac{1}{2}$ ft. These stringers frame into 18-in. W.F. section floor beams spaced 12.5 ft. on centers and framed into the stiffening girders.

The towers are of the fixed-base flexible type, with legs made up of two 10-in. W.F. sections connected by stay plates. Field splices are placed just above the elevation of the stiffening girders. The wind reaction from the stiffening girders is delivered to the horizontal tower strut beneath the floor through lugs, as shown in the tower elevation, Fig. 2.

It was not desirable or practical to fix the structure to either of the slender towers, and since the depth of the foundations on the west end would require the addition of material if a fixed connection were used there, it was decided to make the broad, stable east abutment the fixed point and provide for expansion movement of the girders through the towers to the west abutment. This movement induces considerable bending in the towers, which was considered in their design. Provision is made at the west abutment for the expansion movement of the full length of the bridge.

Under the west end of the suspension structure and supporting the approach span a rocker bent was used. The wind load from the approach span is

Fig. 3—Bridge roadway 20 ft. wide consists of 2x4-in. creosoted pine boards laid on edge.

delivered to the end floor beam of the side span and then to the rocker bent through two 16-in. beam sections riveted between the floor beam and the top strut of the rocker bent. The rocker motion of the bent induces bending in the webs of these beam sections.

In order to continue the lines of the stiffening girders for the full length of the bridge, 36-in. W.F. sections are used for the girders of the approach span, with the same floor construction as was used for the suspension structure.

Cables, hangers and details

Each cable consists of four $1\frac{1}{4}$-in. galvanized strands (Fig. 5). The arrangement used makes each strand accessible for inspection and painting, eliminates the necessity of wrapping and makes possible the design of a very compact and effective cable clamp. The strands were prestressed and were specified to have a minimum modulus of 24,000,000 up to 50 per cent of the ultimate strength, which was shown by test to be 225,000 lb. on a gross metallic area of 1 sq.in. At approximately 30 ft. from the end of the side span the strands terminate in a modified Roebling compact strand assembly. From this point to the end of the side span the cable consists of two 9-in. eyeplates, one on either side of the stiffening girder web.

At the first panel point from the end of the side span the cable is below the top of the stiffening girder. Here the web of the stiffening girder was cut away and a casting, bearing on the web of the girder and on the pin connecting the eyeplates of the cable, was used to deliver the panel load to the cable. At the end of the side span the cable is connected to the stiffening girder by $4\frac{1}{2}$-in.-diameter pins.

A complete design was made using 9-in. eyeplates throughout for the cable, but as near as could be determined by estimates there was practically no difference in the cost of the two cable designs. The strand cable required slightly more metal for the stiffening girders and a comparatively greater cost for the hanger connection to the cable. However, the strand cable required no falsework or temporary cables for erection and reduced to a minimum the number of pieces to handle in the shop and field.

Hangers consist of $\frac{3}{4}$-in. prestressed galvanized strands meeting the same requirements as specified for the main cable strands. They were fitted with open sockets at each end for attachment

Fig. 5—Cable clamp and hanger detail is novel and compact.

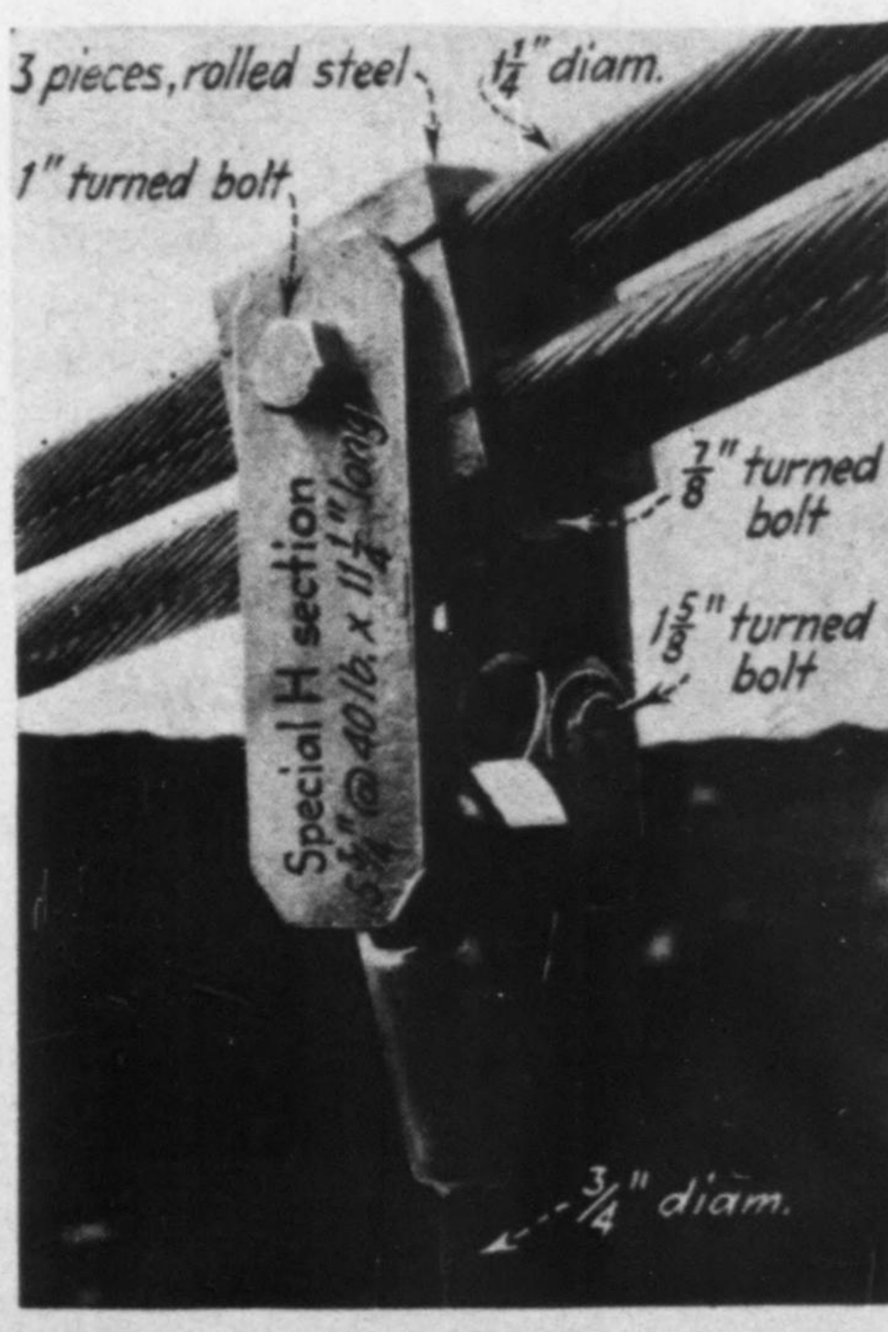

Fig. 4—Assembly detail where wire strands connect to eyeplates, which are pin-connected to the side-span girders as shown in Fig. 2.

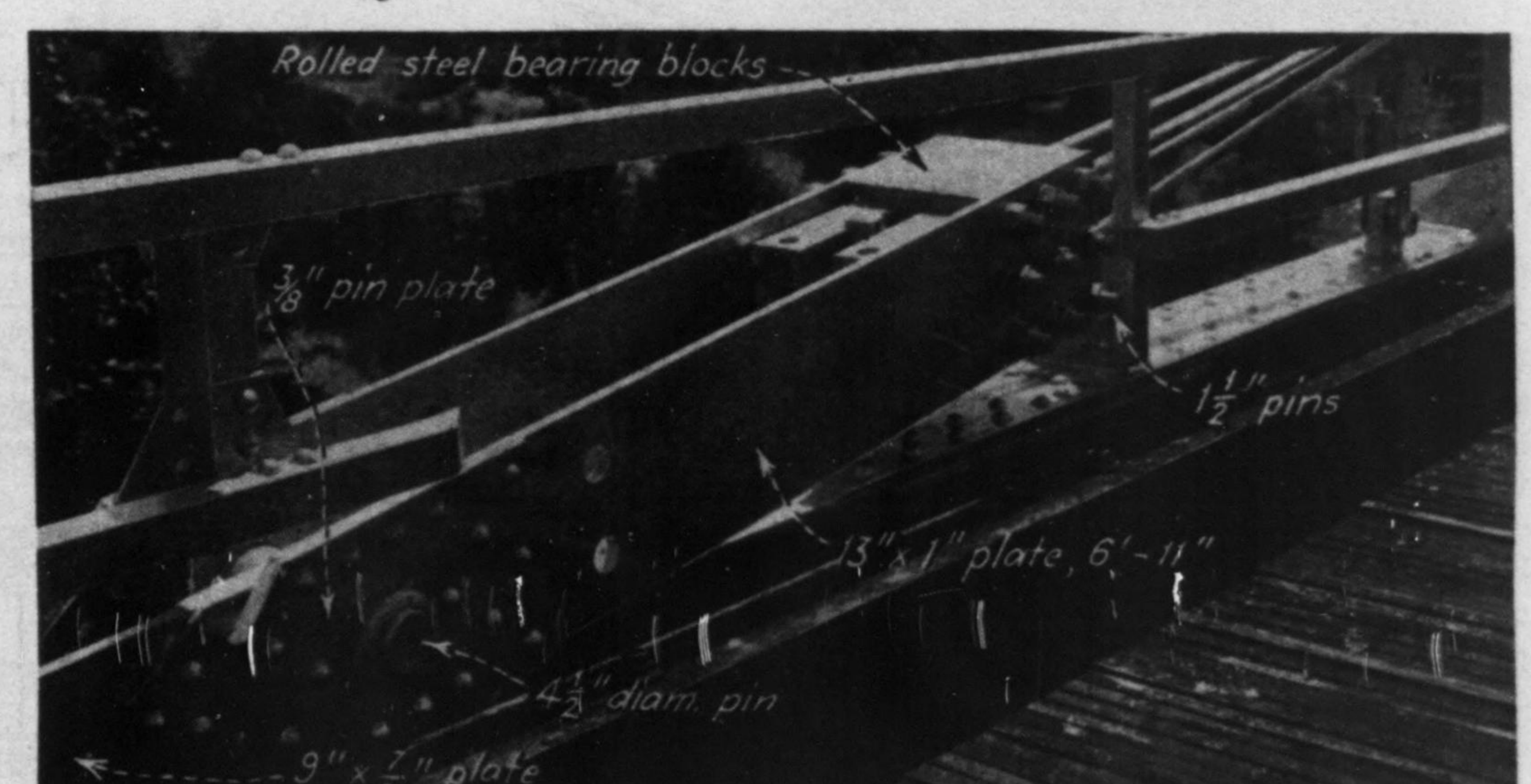

Fig. 6—Bridge superstructure, except cable system, was erected complete in one pass of a guy derrick.

to the girder and the cable. At the panel points near the center of the main span and at the splay point of the side span, built-up structural hangers were used.

The cable clamp consists of three sections of rolled-steel slab bored to receive the cable strands and bolted together with $\frac{7}{8}$-in. turned bolts set vertically between the two tiers of strands. The interior of the strand groove was given a rough cut of sixteen tool marks per inch at right angles to the axes of the strands, to assist in developing the required friction. As shown by test, this roughened surface aided materially in increasing the frictional resistance of the clamp. The center section is bored horizontally, on the center line of the cable, to receive the pin which connects a yoke, made from a W.F. section. The web of this yoke is bored for connecting the hanger sockets.

Construction

All construction operations were carried on from the west bank of the lake. The first task to be performed was the driving of a falsework trestle, consisting of three-pile bents spaced to suit the superstructure erection scheme and capped at El. 670.5. From this trestle all substructure work was done, beginning at the east abutment and working toward the west end. Work began on the substructure Aug. 1, 1932, and was completed March 10, 1933.

For the superstructure erection a guy derrick with a 70-ft. boom was set up on the lower tier of falsework and began setting steel at the east abutment. Moving toward the west end, this derrick set all upper-tier falsework and all steel complete, including the towers, in one pass. All superstructure material was delivered to position by a cableway operating along the north side of the bridge. After all riveting was completed, the strands were hoisted into position and adjusted. After placing of the cable clamps and flooring, the hangers were connected by pulling the cable down by a ratchet device connected between a sling over the cable clamp and one underneath the stiffening girder. In order to prevent kinking of the cable, it was necessary to connect several of the hangers on each side simultaneously. Work on the superstructure was started Feb. 1 and was completed March 25, 1933.

Personnel

The bridge was built by the Missouri State Highway Department, of which T. H. Cutler is chief engineer and N. R. Sack bridge engineer. The design was prepared by the writer, who also acted as resident engineer.

The Clinton Bridge Works was the general contractor. The cable material was manufactured and prestressed by the American Steel and Wire Co. at its Trenton plant.

John Reese was foreman on the substructure work, and M. A. Nunnelly was foreman on the superstructure erection.

Tree and Crop Damage in Flood-Detention Basins

Data from May flood in Miami District basins show $2\frac{1}{2}$ to $6\frac{1}{2}$ days' submergence required to destroy grain crops

By C. H. Eiffert
Chief Engineer, Miami Conservancy District, Dayton, Ohio

THE RESERVOIRS or retarding basins of the Miami Conservancy District hold water only during floods, while the ordinary river flow passes through the dams without any retarding action. As most of the floods occur during the winter and early spring, it has been feasible to keep under cultivation practically all the farm land in the basins. Only twice in the twelve years since the completion of the dams have floods occurred in the growing season and damaged crops and trees. These floods came in June, 1924, and in May, 1933. Observations after the flood of last May give quantitative data on the amount of submergence required to destroy crops.

During and after the flood careful observations were made of tree and crop damage in the Englewood basin, where the highest backwater occurred. Trees were practically in full leaf at the time of the rise, and many of them were completely submerged for about one week. Ironwood seems to be the only variety in the affected area that was killed by the water, but as this is considered a weed tree by foresters its destruction is a benefit rather than a damage. A few individuals of other varieties were found to be apparently dead after the recession of the water, but most of them made a quick and complete recovery. The trees submerged were principally elm, cottonwood and willow; however, there were some of a number of other varieties common to this section of Ohio. The

Crops damaged by $2\frac{1}{2}$ to $6\frac{1}{2}$ days' submergence, as shown on May, 1933, flood hydrograph for Englewood retarding basin of Miami Conservancy District, Ohio.