Bridge of Unusual Design Replaces Ohio River Crossing of Big Four at Louisville

Design of 547-Ft. River Spans Influenced by Erection on and Within Old Trusses—Separate Wind Chord—Pin-Connected Hangers Carry Floor beams Below Bottom Chord—Foundation and Viaduct Reconstruction

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FIG. 1—UTILIZING AN OLD E-30 BRIDGE AS FALSEWORK FOR ERECTING A NEW E-70 STRUCTURE
The Ohio River crossing of the Big Four at Louisville, Ky. Travelers are of timber and run on a track on the top chord of the old spans.

DURING the past year the Cleveland, Cincinnati, Chicago & St. Louis Railway Company (Big Four) has been engaged in renewing the Ohio River bridge of the Louisville & Jeffersonville Bridge & Railroad Company, as well as the Indiana and Kentucky approaches. The structure provides a route for the interchange of important freight traffic between the Big Four and its southern connections and has also been used by the traction cars of the Interstate Public Service Company. It was started in 1889, but completion was delayed by the panic of 1893 and later by several accidents, the most serious of which was the destruction by a hurricane of two of the long spans which had been nearly completed. The bridge therefore was not put into service until 1895.

The bridge consists of three 547-ft. single-track through-truss spans over the channel, flanked by two 338-ft. spans on the Kentucky side and one 208-ft. span on the Indiana side. The Indiana approach consists of a single-track viaduct 5,000 ft. long, while at the Louisville end of the bridge there is a Y approach consisting of a double-track viaduct 2,600 ft. long leading to the Louisville & Nashville yard at East Louisville and connecting with the tracks of the Interstate Public Service Company and a single-track viaduct connecting with the bridge company's freight house and yard in Louisville as well as with the passenger station at Seventh St.

The three approach viaducts and the truss bridge, with the exception of the four channel piers, were completely renewed. Owing to the character of traffic handled, a double-track structure was not considered necessary, and it was possible to take the bridge out of service during the reconstruction, both of which factors influenced the new design.

The loading used in the original design was about equivalent to E-30, but due to many poor details and the lack of consideration given to other loads and forces, the structure, even with the rating stresses, had a computed capacity of less than the designing load. For many years it had been necessary to reinforce various local failures, and in 1923 all heavy traffic was removed and detoured. Even under the lighter loading, defects continued to develop at an equally rapid rate, particularly in the viaducts, and later all traffic was suspended except that of the traction cars.

Principal interest in the renewal work centers on the three 547-ft. spans over the river channel. Erected on the old bridge as falsework and necessarily placed within the old trusses, the new trusses are unusually narrow. It was therefore necessary, in order to provide lateral rigidity, to use a separate wind chord outside of the main trusses. The use of this wind chord made necessary some form of articulation between it and the main bottom chord, since the elastic actions of the two chords would be dissimilar, and the problem was solved by carrying the floorbeams on pin-connected hangers below the bottom chord. This article covers principally the design and erection of these three unusual spans. It also touches briefly on the foundation work and viaduct reconstruction.

Foundations—The four channel piers, while comparatively small, were in excellent condition. They are of concrete, faced with oolitic limestone, the two north piers being founded on rock and the other two on cemented gravel. The three shore piers consisted of steel shells filled with concrete, the one on the north end being on rock and the two at the south end on clusters of wood piling. These steel piers were very slender and there was evidence that they were not in the best of condition. They were therefore completely incased and extended, being so designed that the incasement would carry all of the future load independent of the old pier. The new footing of the north pier was carried to rock, while the south footings on the Kentucky shore were erected on wood piling. The shafts were heavily reinforced, and the incasement on both sides was tied together by $1\frac{1}{4}$-in. rods spaced about 4 ft. on centers, passing through the old piers. The upper 10 ft. of the shell and the filling were removed and replaced with concrete.

The upper course of two of the stone piers in the river had cracked, probably as a result of ineffective expansion bearings under the old spans. To insure against further failure of this sort and also to afford resistance to a too rapid lateral distribution of the new and heavier imposed loads on the bridge seats, the tops of these four piers were incased or girdled for a depth of 12 ft. This incasement was heavily reinforced and hooped, being tied together by rods set in channels cut in the top course, and lower down by rods passing through the pier. All of the concrete used was proportioned on a strength basis, maintaining the specified water-cement ratio and controlling the workability with variation of the amount and proportion of aggregates.

Approaches—The total length of the new viaducts is about 7,000 ft., and they range in height from 25 to 65 ft. Although no unusual difficulties were encountred in their design or construction, several details are worthy of note. The tower spans are all 30 ft., while the intermediate spans, because of street crossings, etc., vary in length from 40 to 75 ft. The top surface of the bearing plates under the intermediate spans is curved to give line bearing and to fix, within narrow limits, the point of application of the load to the column cap. By this means eccentric loading of columns can be reduced to a minimum.

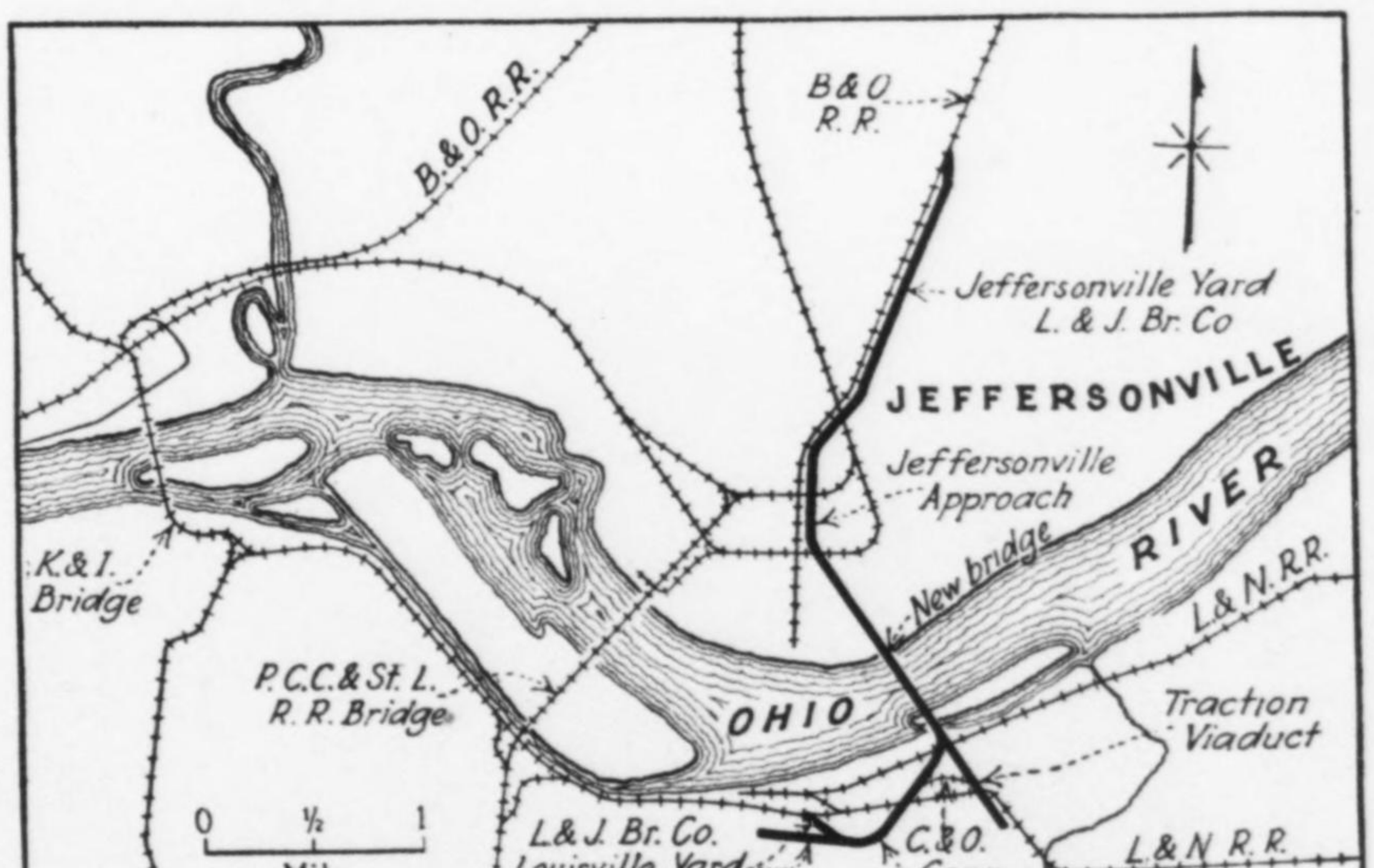
In one of the four columns of a tower, the holes in the sole plates were made to fit the anchor bolts snugly, while in the sole plates of the three other columns the holes were enlarged and the contact surfaces between the plates well lubricated when the bents were erected. This simple method of providing for expansion in the tower bases is noticeably effective up to the present time, and it is hoped that it will serve to eliminate the splitting of pedestals so frequently observed where no provision is made for expansion.

Another unusual detail is used on the curved portions of the viaduct, where the entire structure, including girders and bents, is tilted to correspond with the superelevation of the tracks, the use of tapered ties being thus avoided; at the same time lateral forces at balanced speeds are reduced.

Superstructure Design—The 207-ft. and 338-ft. spans are of usual form and have eight and ten panels respectively. The trusses are $24\frac{1}{2}$ ft. center to center, with curved top chords, and have depths at the center of the spans of 40 and 60 ft.

The design and construction of the three 547-ft. spans was influenced to a considerable extent by the method of erection to be used. The existing piers were too short to permit placing the new trusses on the outside of the old span and erecting them as cantilevers, and the depth of water, about 45 ft., and requirements of navigation made the use of falsework somewhat hazardous and most undesirable. There was left the alternative of building the new spans within the old trusses, using the existing spans as falsework. While it was not the intention to fix in advance the method of erection to be used by the contractor, it was assumed with considerable assurance that the suggested method would appeal to him as the cheapest and most feasible, and the design was based wholly on this assumption.

The old spans had eighteen panels of equal length and were 546 ft. 6 in. center to center of end pins. The trusses were 84 ft. deep and 30 ft. on centers. The new spans were made 90 ft. deep, which brought the new top chords well above the old, and sixteen panels were used, to avoid interference at panel point splices and at floorbeams. To provide the necessary clearance for erection, the new trusses could not be made more than 24 ft. 6 in. center to center. This distance gave ample width for a single track, but was considerably less than that desirable for a span of this length and depth. However, by using a separate wind chord which could be placed after



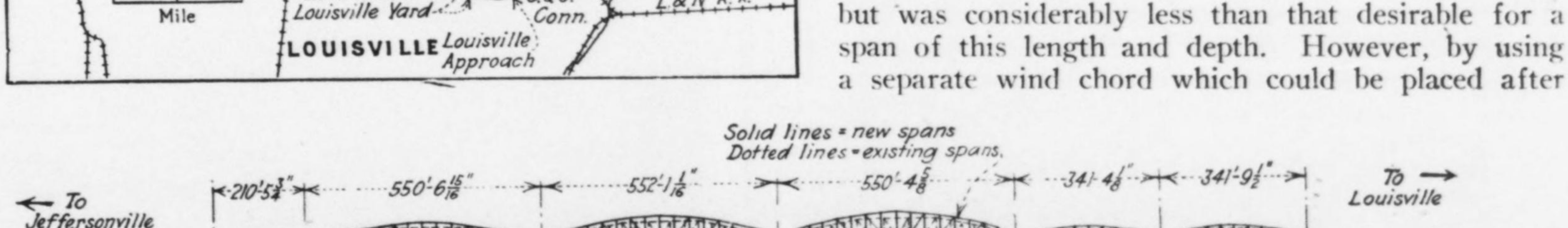


FIG. 2—LOCATION PLAN AND ELEVATION OF OHIO RIVER CROSSING OF BIG FOUR AT LOUISVILLE, KY.

FIG. 3—CONSTRUCTING THE TWO 338-FT. SIDE SPANS ON FALSEWORK

Note holes drilled in old steel pier for tie rods. Incasement of near pier has been completed.

the old spans were removed, a bottom lateral system of sufficient stiffness could be provided, as the old structure would not control its width. By extending the lower portions of the floorbeams, which hang entirely below the main bottom chord, it was possible to support these independent chords at a reasonable distance out from the end of the beam, and they were made 30 ft. center to center. This distance is reduced at the end panel, the chords being drawn in until they pass inside of the main bearing shoes. The bottom laterals are in the plane of and attached to this wind chord.

The use of this auxiliary chord to resist lateral forces only, necessarily introduced other unusual features. The main chord under full live load will theoretically elongate about $\frac{1}{8}$ in. per panel, or 2 in. in the length of the span, while there will be no corresponding elongation of the wind chord. This condition called for some means of articulation between the two chords, and the floorbeams are therefore suspended from pins at the panel points, except at the center of the span, where a stiff connection is provided to transfer to the main chords the longitudinal force which will be carried through the floor system to the wind chords. The pins from which the floorbeams are suspended are 8 in. in diameter and are designed for only 50 per cent of the usual permissible bearing stress, to prevent wear or cutting. They do not pass through the bottom chord, but are suspended from jaws attached to the posts. The pins are provided with nuts to clamp the jaws, and another pair to jam against the inner face of the leaves of the bottom chord. They can be lubricated by oil holes in the tops of the floorbeam hangers which go over the pin and between the jaws on the post.

To compensate partly for the difference in elongation of the two chords and to reduce transverse bending in the floorbeams, the wind chord was fabricated $\frac{1}{16}$ in. longer per panel than the main chord. However, provision for expansion was required at both ends of the wind chord because of the rigid connection between the wind chord and the truss at the center of the span. The ends of the wind chords, which are brought in between the main shoes, bear against vertical segmental rollers, attached to the castings. These take the lateral reaction, while the dead-load reaction of the half-panel is taken by a rocker arm on the pier top.

It was also necessary to control and account for the relative deflections of the top and bottom lateral systems. By using a low unit stress of about 8,000 lb. in designing the wind chords, the lateral deflection of the lower system under maximum wind loading will never be more than 1 in. greater than that of the upper system, and the web members of the trusses were designed to take this 1 in. bending about the bottom struts of the sway bracing. In fact, the force required to bend these members transversely is about equal to or less than the maximum wind load at any lower panel point, so that by their inherent stiffness they will immediately begin to transfer a portion of the wind load from the lower to the upper system, maintaining equilibrium; the top laterals, portals, etc., were designed to take care of this overload.

The design of these spans contemplated placing the new bottom chords above the old floorbeams and erecting

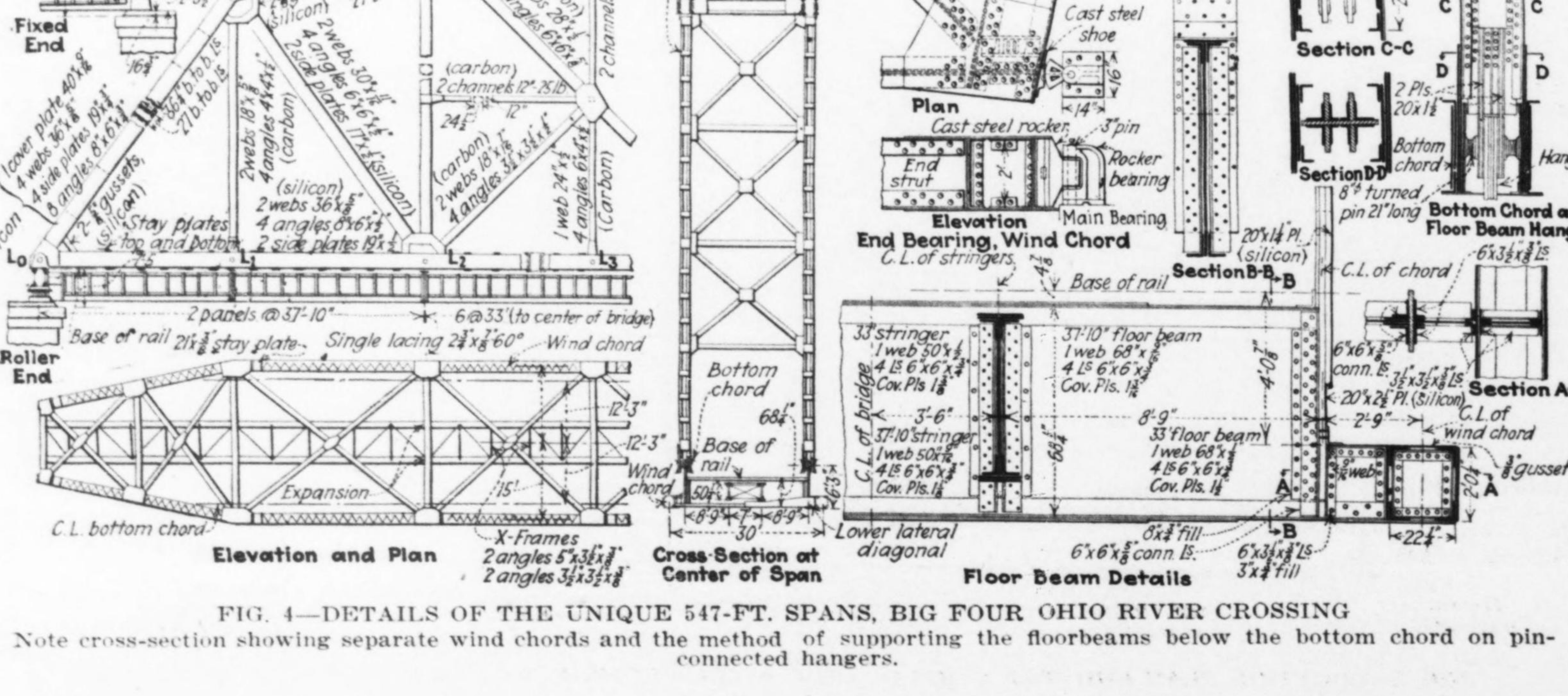


FIG. 4—DETAILS OF THE UNIQUE 547-FT. SPANS, BIG FOUR OHIO RIVER CROSSING

Note cross-section showing separate wind chords and the method of supporting the floorbeams below the bottom chord on pin-connected hangers.

the span complete except the floor system and wind chords. When the new spans were swung, they in turn could be used to carry the old spans while they were being removed. The new and the old trusses were so close together that the new shoes must necessarily occupy the position of the old and they could therefore not be placed until the bearings of the old trusses were out of the way. To provide support for the new spans during this interval and to afford a convenient means of raising or lowering them, the end floorbeams, of box girder section because of the restricted depth, were designed to support the weight of both the old and the new structure, with the jacks or cribbing supporting the floorbeams on 15 ft. centers.

The use of the separate wind chord probably added some weight to the structure, but not so much as might be supposed. Without it, the section of the main chords would have been increased, and with the close spacing of trusses, considerable weight was saved in the top lateral system, sway frames and floorbeams. Its use also permitted swinging the spans at their minimum weight, which was most important. The new spans will undoubtedly look narrow, end on, and after the old structure is gone the reasons for their unusual design will be less obvious, but certain conditions had to be met, and it is felt that the new trusses will function quite as well as those of any other design which might have been adopted had there been no restrictions imposed.

Carbon steel of structural grade was used in the viaducts and in the 207- and 338-ft. spans. In the 547-ft. spans silicon steel was used in the main members of the trusses with the exception of the posts, and carbon steel was used in the floor system, wind chord and bracing. The use of silicon steel in these spans not only was economical but the saving in weight, about 1,000 tons, was most important, as the old spans were loaded

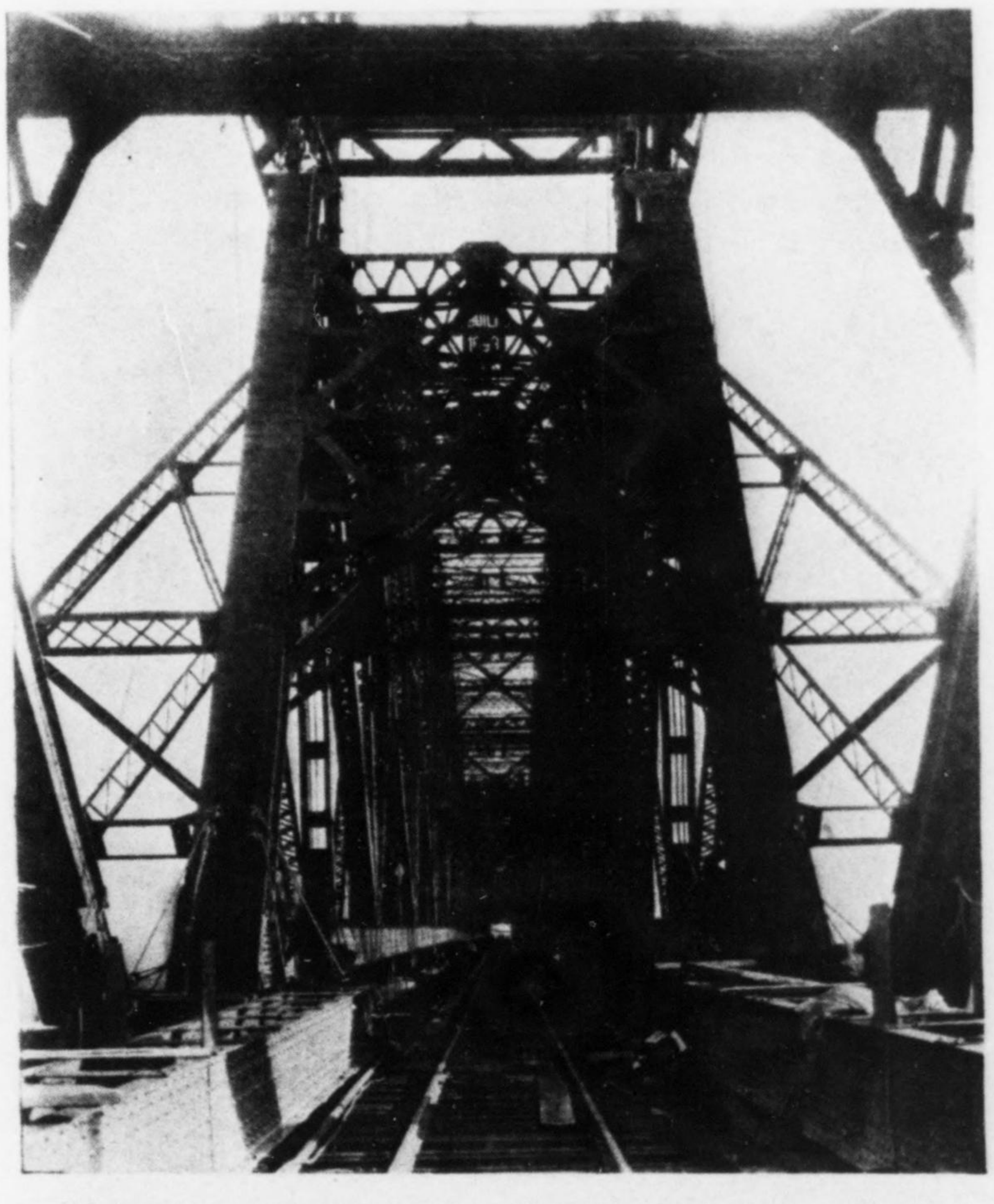


FIG. 5—NEW 547-FT. TRUSS BEING ERECTED WITHIN THE OLD TRUSS
Note kingpost bracing extending outside of old end posts.

to their full capacity during erection. A total of 16,000 tons of steel was used, of which 3,400 tons was silicon steel.

Loads and Stresses—The viaducts and the 207-ft. span were designed for E-70 loading and a basic unit stress of 18,000 lb. For the 338-ft. and 547-ft. spans, a loading of E-70 was used for the floor system and web members and E-65 for the chords. For carbon steel, a basic unit stress of 16,000 lb. was used for the live load, 20,000 lb. for dead load. For silicon steel, the corresponding unit stresses were 22,500 and 28,000 lb. After the designs were completed, the spans were investigated for 50 per cent overload, and the section of any member in which the unit stress exceeded 26,000 lb. for carbon steel or 36,000 lb. for silicon steel was increased. Several members were increased, at small expense.

Erection—The three shorter truss spans, being over shallow water, were erected on falsework. For the 547-ft. channel spans, the old bridge structure was utilized as falsework. The weight of each of the new spans without the floor system amounted to about 6,000 lb. per foot, whereas the old spans had been designed for a live load of only 3,000 lb. per foot. Computations showed that with all the expected loads and wind on the old and the new structure the stresses would be well up to the elastic limit, but since the old structure was in excellent condition it was felt that under a static load properly distributed it was not so far beyond safe limits but that adequate reinforcing could be done at a nominai cost. The stress in each member and the theoretical deflection of each panel point under a unit load at any panel point was computed, so that by multiplying the unit deflections by the proper load factor, and summing them, the total theoretical deflection at any or all points could be quickly determined under any condition of loading. It was felt that any considerable deviation of the actual from the computed deflection would give ample warning of overloading throughout or of an unexpected concentration at any point. Checking by deflections proved more positive than by measuring the stresses. As an example, the last pair of top chord sections placed before closing produced an additional deflection of $\frac{1}{2}$ in. at the center, which could be easily checked, while the corresponding increase in chord stress was only 600 lb. per sq.in., which would be difficult to measure. As the applied load approached the maximum, levels were frequently taken, and under no condition of loading did the measured deflection at any point differ from the computed deflection by more than $\frac{1}{4}$ in. The total deflection at the center just before the spans were closed amounted to about 7 in. As a further precaution, the old top chords and end posts were stiffened by the addition of kingpost trusses, bolted in place, and the floorbeams were reinforced to some extent.

The lower chords of the new trusses were set on blocking at the panel points and the load was carried to the floorbeams of the old spans by the stringers. Since the structure was being erected on yielding supports, deflecting progressively, the camber blocking was made of such height that it would be certain that the top chord joints at the center would be slightly open, so that it would be necessary only to raise the ends in order to close and swing the span. The blocking was adjusted from time to time to maintain the true dead-load camber in the bottom chord from the center to each end.

The lower chords and lower half of the diagonals were set with locomotive cranes. The end posts, top chords, upper diagonals and posts were set with a wood traveler running along the top chord of the old span. These travelers were double ended and so arranged that, in the lifting position, the legs came at the old panel points. Power was supplied by motor-driven hoisting engines set on platforms rigged out from the pier tops, one on each side. A traveler for each span was provided so that work on all three could be carried on simultaneously.

Each half of the lower chord and the lower connections of the diagonals and posts were riveted in their true dead-load position. The center splice and top chord connection were lightly pinned and bolted. After all of the steel was in place four 500-ton hydraulic jacks were set under each end floorbeam, and the span was raised.

The first jacking operation was to close the top chord joint. This required a rise of from 4 to 5 in. and was usually made in three lifts to permit the adjustment of wedges at the quarter points, thus relieving the concentration at the center. After the span was closed it was raised to clear the blocking on the old structure. This required a lift of about 10 in. to take out the deflection of the old spans and to allow for deflection of the new. This left the new spans about 4 ft. above their final elevation, in which position the other joints, except at the top chord center, were made up and riveted. The stringers and upper portion of the floorbeams of the old span were then cut out and the new floor system was placed.

The next operation was to hang the old trusses to the new spans and by means of the jacks to lift them clear of their old bearings, following which the new span, together with the old, was lowered to its final position. The work had reached this stage at the time the accompanying photographs were taken. The lower portion of the old trusses was next removed, the wind chords were erected and finally the remaining portion of the old steel was taken down.

A departure from precedent was involved in placing the deck. It has been frequently observed that the maximum deterioration of girder flanges takes place under the ties rather than between them, and with a creosoted deck the period between renewals is so great that the flanges under the ties can get no beneficial paint protection unless the ties are shifted or temporarily removed. On this long deck it seemed advisable to apply as much paint as possible before the ties were placed, and two layers of 10-oz. duck soaked in red lead and oil were therefore placed in the dap between the tie and the stringer flange, the hope being that the canvas would carry and retain the excess paint and seal the opening. These pads carry about 1 lb. of red lead per tie, and being applied while wet, form themselves over the rivet heads, adhering to the girder rather than to the tie.

Administration—The total cost of the work was approximately $3,250,000. It was carried out under the direction and supervision of C. S. Millard, general manager of the C. C. C. & St. L. Railway Company and general manager and chief engineer of the L. & J. Bridge & Railroad Company. W. S. Burnett was chief engineer of construction, E. A. Humphreys district engineer, and A. A. Keever resident engineer. The plans for the work were prepared in the general office of the Big Four at Cincinnati, L. H. Schaeperklaus, assisted by A. M. Westenhoff, supervising the designing force. The masonry work was done by the Walsh Construction Company, of Davenport, Iowa. The superstructure was fabricated and erected by the McClintic-Marshall Company, Pittsburgh, Pa.

Lloyd Dam in India, the Largest in the World

Forms Major Part of Great Irrigation Scheme Built by the Bombay Presidency as a Protective Work

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LLOYD DAM, at Bhatgar, in the Bombay Deccan, which was officially opened on Oct. 27 of last year, though notable for its large volume, 21,500,000 cu.ft., is really more interesting as a part of one of a series of great projects than as an achievement in construction. The great size of the dam is in nowise the result of an effort to achieve magnitude, nor does it contain a cubic foot of masonry in excess of that needed to provide a reservoir duly related in position, size and cost to the runoff of the catchment area, to the areas which it commands, to the value of the crops to be grown and to those earlier projects in the series of great protective works of which this project is the latest.

The number and magnitude of these protective works may largely be ascribed to the effect produced in the minds of administrations and engineers by the great famine of 1889-1902 and to the recommendations of a commission thereafter convened by Lord Curzon. They were begun in 1903 and include the Wilson dam and Pravara canal project, completed in December, 1926. How effective these works have been and how completely the specter of famine has been laid may be judged, in the first place, by comparing the cost of fighting the famine with the increases in the values of the crops now grown in the irrigated areas. Apart, of course, from the losses inevitably incurred in other respects, the expenditure of the Presidency in combating the effects of the drought was three crores of rupees, or $11,000,000, whereas the total increased value of the crops grown in the irrigated lands is about six crores, or $22,000,000, annually, as the result of a capital expenditure totaling only twelve crores, or $44,000,000.

Features of the Project—The general character of the sites of the dam and reservoir may be realized from the data which follow. The dam had to be 5,333 ft. long and 190 ft. in height above lowest foundations in order to impound water to a depth of 143 ft. over the lowest sluices and to create a reservoir of 24,198 million cubic feet, or 555,555 acre-ft. The site of the reservoir, situated in a catchment area of 128 square miles, is such that this volume of water is impounded in an artificial lake $14\frac{1}{2}$ square miles in area and therefore nearly 60 ft. in average depth. The topography is such that the lake is 17 miles in length and has a perimeter of 46 miles. To regulate the water level in the reservoir there are 81 gates, 45 of which are automatic. Although the hydro-electric power which can be made available as the result of building the dam is about 1,200 hp. this will not be utilized at present.

The two main canals of the Nira canals system, supplied by the reservoir, are 106 and 100 miles in length and the area commanded is 834,000 acres, of which 202,000 acres will be irrigated annually, producing crops of the increased value of $11,900,000. The dam cost $6,300,000 and the whole project $20,700,000. In order to keep pace with the progress and development of the canals, the work