FIG. 1. TRAIN OPERATING OVER PART OF NEW BRIDGE AS STEEL ERECTION WORK CONTINUES ON REMAINDER.

Simple Truss Bridge Lengthened Into Long Cantilever

HARRY J. ENGEL
Assistant Engineer, Modjeski and Masters, Harrisburg, Pa.

***Contents in Brief**—Widening the Atchafalaya River for flood control required lengthening the Missouri Pacific R.R. bridge at Krotz Springs, La. Scheme was to separate two of the existing 300-ft. simple truss spans by 720 ft. of new steel in the form of twin cantilevers having a common anchor arm, the 300-ft. spans being hung from the ends of the new cantilevers. The 300-ft. span to be removed to the far end of the new bridge was rolled out of position simultaneously with the roll in of 300 ft. of new steel, so as not to interfere with train operation. While remainder of new steel was being erected trains operated over a detour trestle. Work involved removal of two old piers and the construction of two new deep caisson piers of special design, incorporating a jet system and sounding wells in the walls and three dredging wells.*

PRELIMINARY TO WIDENING the Atchafalaya River as a part of the federal flood control project on the Lower Mississippi, the four-span single track Missouri Pacific R. R. bridge over the Atchafalaya at Krotz Springs, La., has been lengthened by the process of building into the bridge a new 721-ft. cantilever structure, and suspending two of the existing 300-ft. simple spans from the ends of the new cantilever arms. This unusual transformation required sinking two deep open caisson piers of interesting design and rolling out one of the existing simple spans for relocation at the far end of the new steelwork together with the simultaneous rolling in of a like length of new cantilever structure, all without interrupting railway traffic.

The original railway bridge at this site, consisting of a 300-ft. swing span flanked by two 300-ft. simple spans, was constructed in 1907, and shortly thereafter was extended on the west by the addition of an identical 300-ft. simple span. This was necessary due to erosion on the west bank of the river. This structure served until a 1936 amendment to the Federal flood control act of 1928 provided authority for doubling the channel capacity of the Atchafalaya River and authorized federal expenditures for any necessary bridge extensions.

Comparative cost estimates made by the consulting engineers for two methods of lengthening the bridge showed a saving in cost of about 15 per cent in favor of the adopted plan, as against adding two new 300-ft. spans on deeper piers and replacing one of the old piers. Fewer substructure operations and considerably less hazard were involved in the adopted plan (Fig. 2), which contemplated the use of a twin cantilever with common anchor arm, to the cantilever ends of which two of the old 300-ft. spans were to be attached.

In detail, the plan required that the principal reactions were to be carried by two new caisson piers, while a new pile pier would support the westerly end of one of the 300-ft. spans after it was shifted to the west end of the new steelwork. Panel lengths of the new steelwork were so arranged that a 300-ft. length of the new steelwork could be erected alongside the old 300-ft. span which was to be shifted, and rolled into place while the old span was rolled out. The additional steel of the new structure was then to be extended westward from this rolled-in portion over the proposed new channel, while rail-

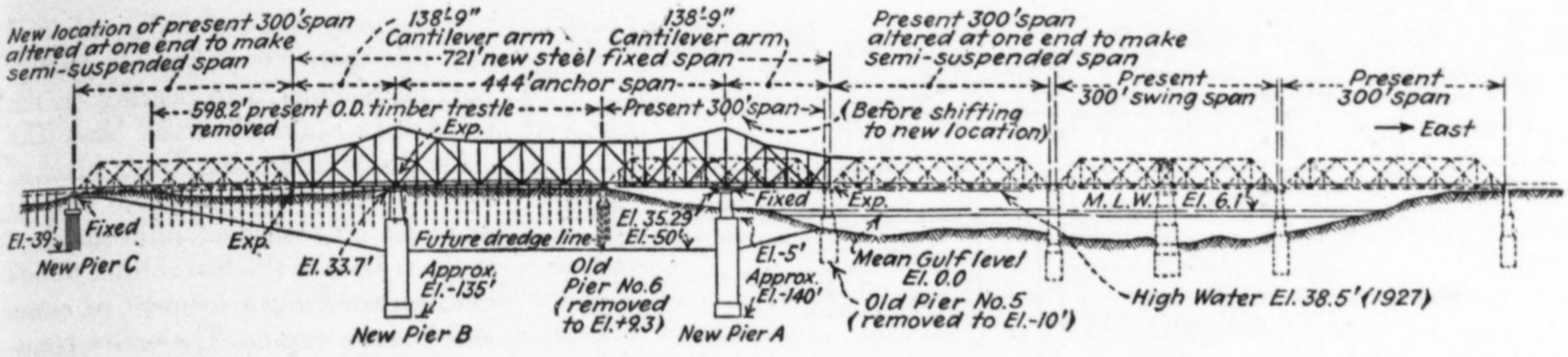

Fig. 2. Plan of reconstruction of Krotz Springs bridge showing how an existing 300-ft. span was moved 720 ft. and utilized at the end of the new steelwork that filled in the gap.

way traffic would be detoured from the new construction through an opening to be left in the north anchor arm truss until the time of final closure.

The piers required

The two deep cellular caisson piers (shown in Fig. 3) are of a unique design, employing triple dredging wells which extend upward into the shafts of the piers, thus saving the expense of a cofferdam and permitting the weight of the shaft concrete to be utilized in overcoming skin friction while the caisson is sinking. The three wells were also used, instead of the single-well circular design that is common in this southern vicinity, to permit the contractor to dredge in the outer wells without interference from the trusses of the existing bridge spans. As a further aid in maintaining the caisson in position during sinking, jet systems were designed to operate in quadrants, and closely spaced 8-in. dia. sounding wells were provided in all caisson walls. Due to the hollow cellular construction of these piers, the pressure imposed on the foundation strata was only 4 tons per sq.ft greater than that imposed by the weight of existing earth.

How the piers were sunk

Pier construction operations were begun at the site of new caisson Pier A, located on the west bank of the river, the contractor utilizing a stiffleg derrick placed on a platform at El. + 27 alongside the bridge from which the derrick worked with flat boom under the steelwork. The placing of concrete lifts was alternated with open dredging, until the caisson cutting edges had sunk to El. −60, when a list of the top of the caisson was noted toward the river. Some correction of this list was accomplished at this point by jetting through the sounding wells on the bank side of the caisson.

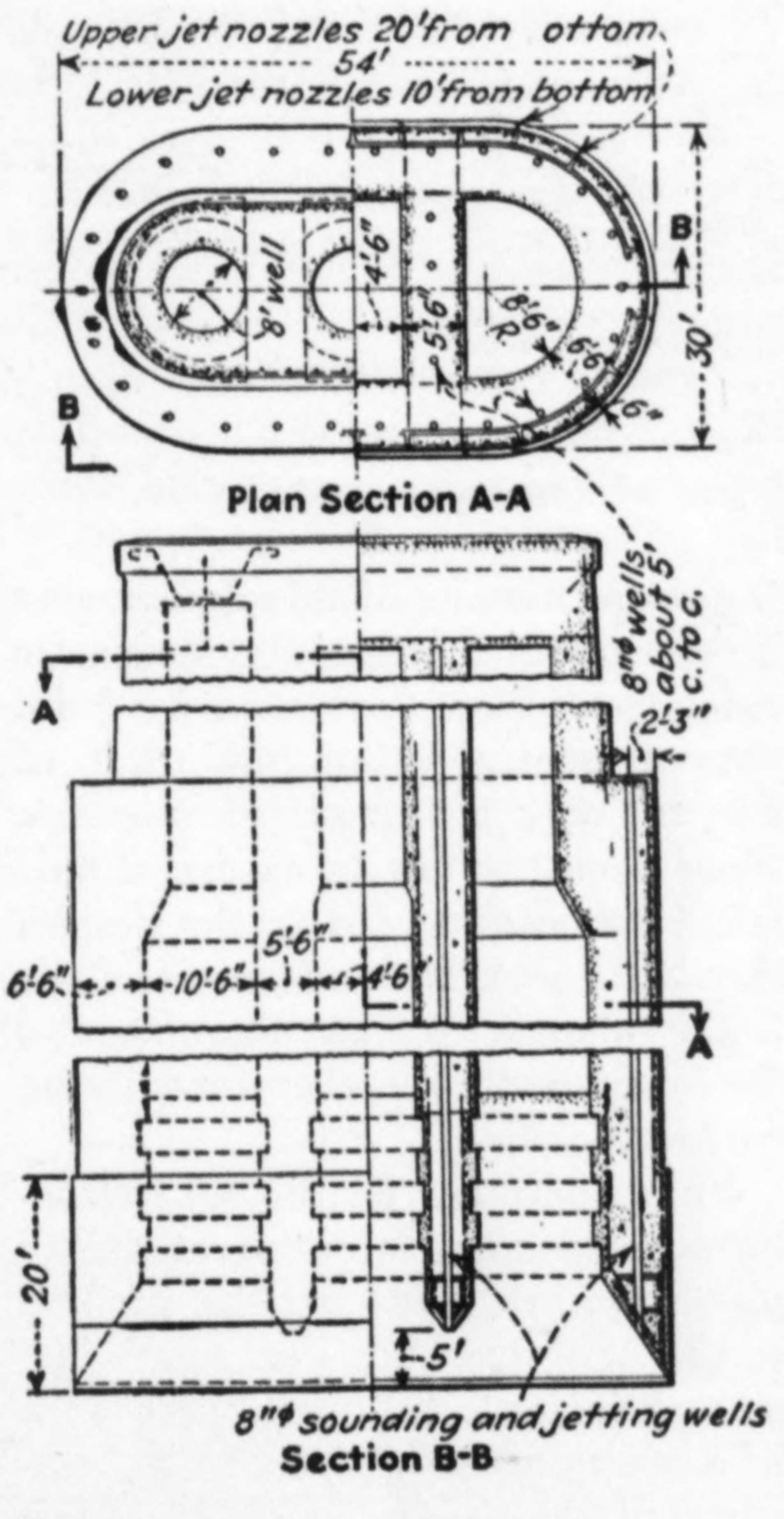

Fig. 3. Details of new deep caisson piers notable for their jetting systems and three dredging wells.

When Pier A caisson had reached a cutting edge elevation of −77, the list had reached 9½ in., and a shoreward cable pull was applied, augmented by air jetting through the northwest and southwest quadrants, until the list was entirely removed during the next 20 ft. of sinking. Thereafter the pier sank vertically, following closely upon the dredging, until its shaft had been poured and the cutting edge had reached the plan elevation of −140, in a formation of coarse sand and gravel. The bottom seal was placed by tremie.

Operations on Pier B, located on high ground to the west, had meanwhile been carried on intermittently with Pier A operations. The detour embankment and trestle had previously been completed, and railway traffic was being run over this detour free of Pier B operations. Some of the initial work on Pier B was slow because of the necessity for clearing the pier site of old piles from the former mainline trestle.

A tendency presently developed in Pier B caisson to tilt toward the west.

Fig. 4. Ready to roll out the 300 ft. truss span and roll in the new steelwork, which is part of a long span cantilever.

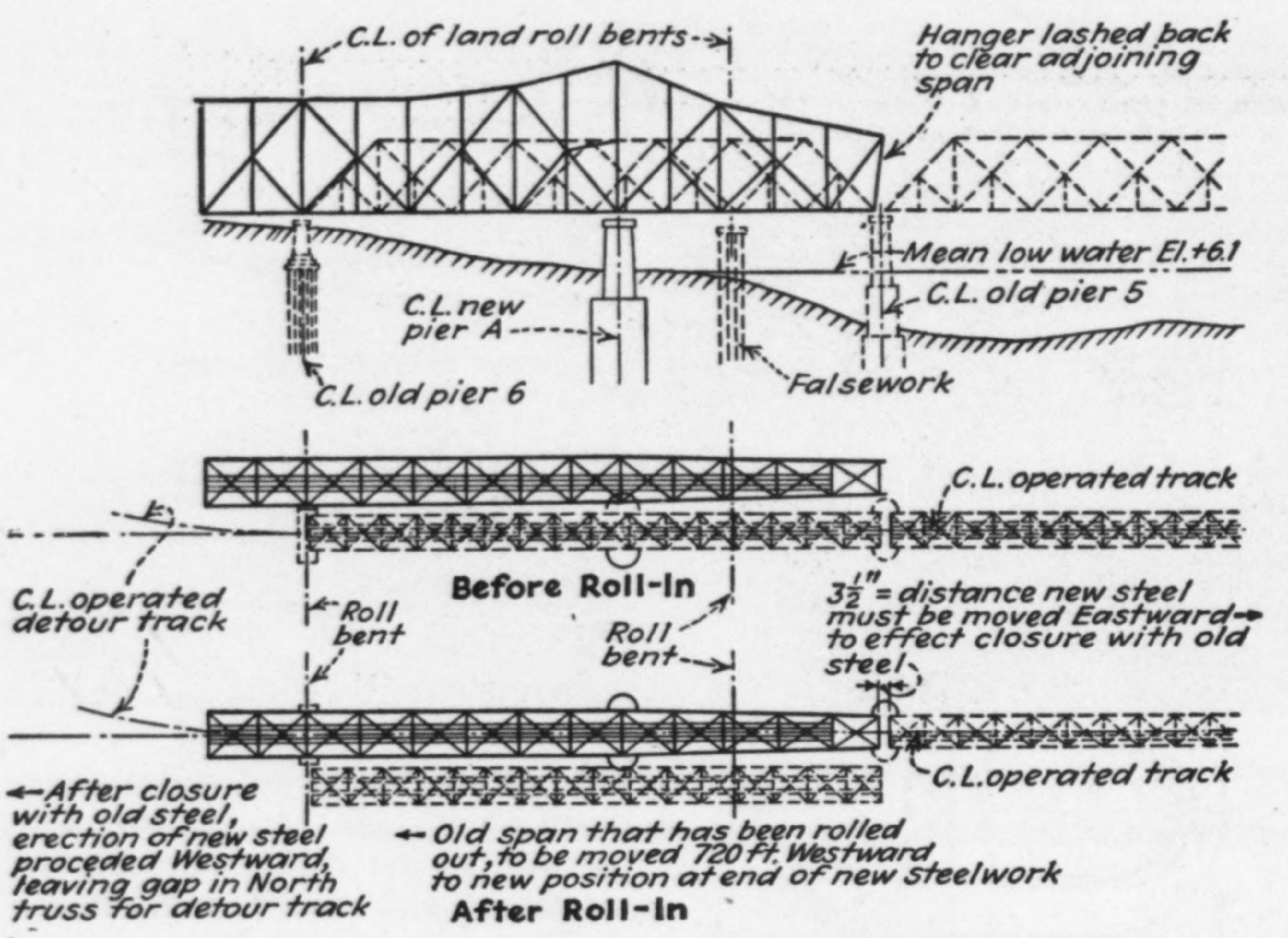

Fig. 5. How old truss was rolled out and part of new span was rolled in.

Accordingly air was turned into the east jetting quadrants, and timber kickers were applied on the west, by which means this caisson was maintained plumb throughout the sinking.

When the cutting edge of Pier B caisson had reached El. −100, considerable skin friction was retarding the sinking, and dredging had to be carried below the interior cutting edges to cause movement. The caisson was now in about 50 ft. of coarse sand, overlain by blue silty clay. Very little seepage of water into the wells occurred, and water was run into them from the top during the sinking. The shaft concrete was placed early to gain weight for the sinking, and in addition 22 ft. of sand, amounting to 350 tons, was placed inside the forms above these shafts; yet the caisson "froze" tight with its cutting edge at El. −135.5, about $4\frac{1}{2}$ ft. above the desired elevation. At this point, the buoyant weight of pier concrete plus the superimposed sand load was exerting a force to overcome skin friction amounting to 620 lb. per sq.ft. of external caisson surface. A 3-in. diameter jet was now worked outside the pier on the north side, and dredging was carried to 6 ft. below the cutting edges at well centers in an effort to resume sinking, but a blow-in occurred causing sand to rise 5 or 6 ft. inside the caisson and the effort had to be abandoned. The caisson was, therefore, sealed above plan elevation in good coarse sand and gravel.

The possible accuracy of positioning deep caissons of this design may be seen from the fact that the lower caisson of Pier A was only $9\frac{3}{4}$ in. out of plan position, and that of Pier B only 6 in. out of position. Since eccentricities in excess of this had been allowed for in the design, the shafts were built off center of the lower caissons by those amounts, and the bridge seats are, therefore, in the correct positions.

First operation on the superstructure contract was to construct timber pile falsework bents for the support of the new steelwork that was to be erected (prior to rolling it into place) in a position north of the most westerly old 300-ft. span (Fig. 2). Two principal roll bents were established, one opposite old pier 6 and the other opposite the far quarter point of the old 300-ft. span. It was upon runways mounted on these two bents that the rolling out of the old steel and rolling in of the new was accomplished.

The various motions of the roll-in are shown diagrammatically in Fig. 5. The time available to the contractor for this operation was the 9-hr. interval between the last morning train at 4 A.M. and the first afternoon train at 1 P.M., and careful preparations had been made to carry out the move as expeditiously as possible. Railroad forces were on hand to break track and remove a portion of the adjoining old trestle as soon as the last morning train had crossed. The shore end of the old 300-ft. span had already been set up on roll points on a transverse runway laid over old pier 6, and now this span was jacked off its support on old pier 5 to bearing on the roll beams at the river roll bent. The old and new steel being firmly blocked together, they were made to roll as one unit by means of lines attached to the trusses at the two rolling points and operated by a locomotive crane and a shore engine. The entire transverse roll was accomplished in half an hour and was completed by 7:30 A.M.

It was now necessary to install the floor stringers in the end panel next to the adjoining 300-ft. span, and to roll the new steel $3\frac{1}{2}$ in. longitudinally riverward to seat these stringers and close the connection between the old and new steel. To jack the new steel on and off the rollers for this last longitudinal move, to complete the hanger and floor steel connections and to jack the new steel to the required elevation for its position in the final structure were time consuming operations; but they were accomplished successfully with but slight delay to the afternoon train.

This completed the most difficult phase of the superstructure erection. During the next month, the 300-ft. span that had been rolled out of the old structure was mounted on longitudinal roll beams and moved 720 ft. westward, and then transversely northward to its final position. Erection of the remaining new steel was meanwhile taking place, and as a last operation those members where the detour track passed through the north truss (Fig. 1) were erected.

Personnel

Lt.-Col. Wm. F. Tompkins was district engineer of the Second New Orleans District, Corps of Engineers, during the progress of these bridge alterations. C. S. Kirkpatrick is chief engineer and R. Owen construction engineer of the Gulf Coast Lines, Missouri Pacific Railway. Otto F. Sorgenfrei was resident engineer for the firm of Modjeski and Masters and the J. F. Coleman Engineering Co., who were associated as engineers on the work and prepared plans for and supervised construction of the bridge alterations. Paul H. Galbraith was superintendent for the general contractor, Missouri Valley Bridge and Iron Co., whose bid for the work was $723,779. The superstructure work was handled by the Mt. Vernon Bridge Co. on a subcontract.