Lackawanna Makes Costly Line Change to Reduce Bridge Delays

New $3,000,000 Hackensack River Crossing Built to Speed Up Suburban Service—Interesting Flat Slab Approaches and Heavy Three-Track Lift Bridge Feature Design

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Undertaking a difficult and expensive main line improvement under traffic for the sole betterment of its passenger service, mostly suburban, the Delaware, Lackawanna & Western Railroad is completing one of the outstanding railroad projects of the year. Design features and a description of the project, the new Hackensack River crossing, are contained in the following article. —Editor.

Where movable bridges surmount a stream upon which there is a heavy flow of river traffic, there results a great loss of time due to the necessity of repeatedly opening the movable span to permit the passage of such craft as cannot clear the structure when closed.

This is the condition at present existing at the Hackensack River drawbridge of the Morristown line of the Delaware, Lackawanna & Western Railroad, to remedy which there is now under construction a new structure while the present one is still necessarily in operation. The construction of this bridge is the result of an order from the War Department, following its persistent refusal to grant relief to the railroads and therefore to the commuters in their petition to keep the drawbridges closed during the morning and evening rush hours, despite the fact that such closing would but little inconvenience the river interests. The cost of the new bridge, approaches and resulting line changes will be in excess of $3,000,000.

The Hackensack River is crossed by the railroad on a swing bridge about 2 miles west of the Hudson River between Jersey City, N. J., on the east and Kearny, N. J., on its west bank. The present drawbridge provides an underclearance of 12 ft. above high tide and permits the passage of only a small percentage of river traffic without opening. It was this condition that prompted the building of the new structure to afford some relief. A study of the number of openings required for various underclearances resulted in the design of a bridge giving an underclearance of 40 ft. above high water and 43 ft. 3 in. above mean tide. A typical week's river traffic is shown in the accompanying table.

HEIGHT OF VESSELS PASSING HACKENSACK RIVER BRIDGE
Week of Sept. 22-27, 1924

Height	No.	Height	No.
15 ft. to 20 ft.	12	40 ft. to 45 ft.	13
20 ft. to 25 ft.	6	45 ft. to 50 ft.	5
25 ft. to 30 ft.	27	50 ft. to 55 ft.	9
30 ft. to 35 ft.	48	55 ft. to 60 ft.	16
35 ft. to 40 ft.	31		

Of the total of 167 vessels over 15 ft. in height, 124 (75 per cent) were 40 ft. or less in height. Also, a 40-ft. underclearance will permit the passage of the highest tugboat smokestack as well as of all barges and scows.

General Conditions—In order to maintain traffic on the present line until the new bridge could be built, a new line was projected 65 ft. south of the present line, permitting the operation of the present swing bridge without interference with the new construction. This change in alignment is limited on the east end of the work by the Bergen Hill tunnels and on the west by the overhead crossings of the Public Service Corporation trolley line and the Newark Turnpike Viaduct known as Sanfords Crossing at Kearny, N. J.

The new bridge as well as the remainder of the new line will be built for three tracks, similar to the other improvements along the Morristown line, the center track being express eastbound in the morning and westbound in the evening with the flow of suburban traffic.

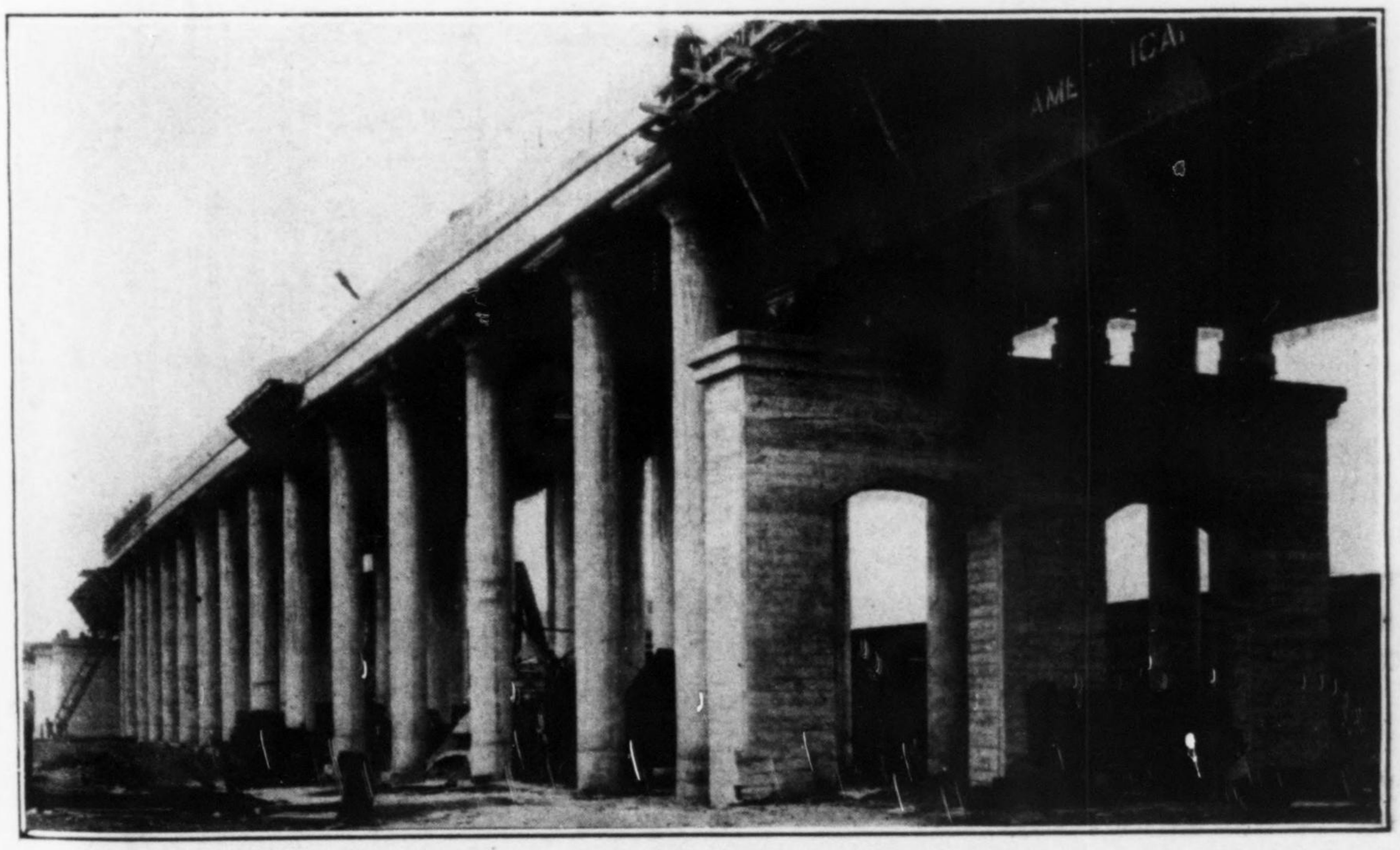

FIG. 1—EAST APPROACH LOOKING TOWARD RIVER FROM DUFFIELD AVE.

The clear width of channel prescribed by the War Department is 150 ft. between fenders measured at right angles to the current, which forms an angle of 76 deg. 2 min. with the new center line, so that the distance on this skew center to center of piers for the main span is 205 ft., resulting in a span of 198 ft. center to center of bearings for the movable bridge. Giving consideration to economy of first cost, facility and rapidity of opening and closing of the movable span, facility and cost of maintenance and operation, the vertical lift type was selected over a single- or double-leaf bascule type. The remainder of the river is spanned by structural steel deck girders, two for each track, of 110-ft. and 75-ft. spans center to center of bearings, reaching the pier-head lines on each bank of the river.

Approaches—The approaches to the river spans on either flank are of concrete of the girderless flat slab type, although the approach on the east bank, which is by far the more extensive, is interrupted by steel spans where Duffield Ave., Jersey City, is encountered. The construction on the east end is completed by a retaining wall 300 ft. long along the street line of Meadow St. to retain the fill for the new railroad embankment. On the west bank the viaduct approach is flanked by a high fill without retaining walls.

All the steel spans except the movable span are surmounted by solid concrete slabs to provide a continuous ballasted floor from the lift span to the end of the construction on either side. The length of the concrete approach spans, center to center of columns, on the west end is 22 ft. 6 in. longitudinally with the columns 18 ft. center to center transversely, the bents consisting of three columns for a 46-ft. width of slab to carry three tracks 13 ft. on centers, resulting in an overhang of 5 ft. on each side. These panels are all regular and rectangular, and continuous between expansion joints.

The section between the east bank of the Hackensack River and Duffield Ave., Jersey City, is also carried on a flat slab viaduct with the bents 4-E to 16-E all 22 ft. 6 in. center to center longitudinally, while 17-E is made parallel to Duffield Ave., the center line of which crosses the new center line of the railroad at an angle of 78 deg. 7 min. with the tangent to the curve at this intersection. These bents are all three-column bents 18 ft. center to center, similar to those of the west approach.

Inasmuch as it was compulsory to span the full width of 60 ft. of Duffield Ave., this span was made of steel. Because of the location of pipe lines under sidewalks, it was essential that the footings of the piers be kept within the street lines, making it necessary to set the piers supporting the girders back of the street line, which condition in combination with the skew of the angle of intersection increased the span to 75 ft. center to center of bearings. The conditions of design at the east street line were further complicated by difficulty in obtaining the vacation of Meadow St., which runs almost parallel to the tracks and intersects Duffield Ave. under the proposed structure; in addition to this the gas mains of the Public Service Company of New Jersey are carried under the Meadow St. roadway. These difficulties were met by a design of a pier with a heavy cantilever bracket, which supports the girders carrying one track over Duffield Ave. on one side and a similar pair of girders on the other side to a point where the face of the viaduct intersects the Meadow St. line, from which point the concrete viaduct could be continued. (Fig. 7.) This span is also 75 ft. center to center of bearings. Parallel to the latter span are four two-column bents carrying the girderless floor slab in panels more or less irregular and specially reinforced because of lack of continuity transversely. The viaduct is completed by four typical 22-ft. 6-in. bays in a counterforted abutment.

River Piers—One of the most interesting features of this structure is the variety of types of foundation design for the conditions encountered at the various sections. The piers supporting the lift and tower spans were

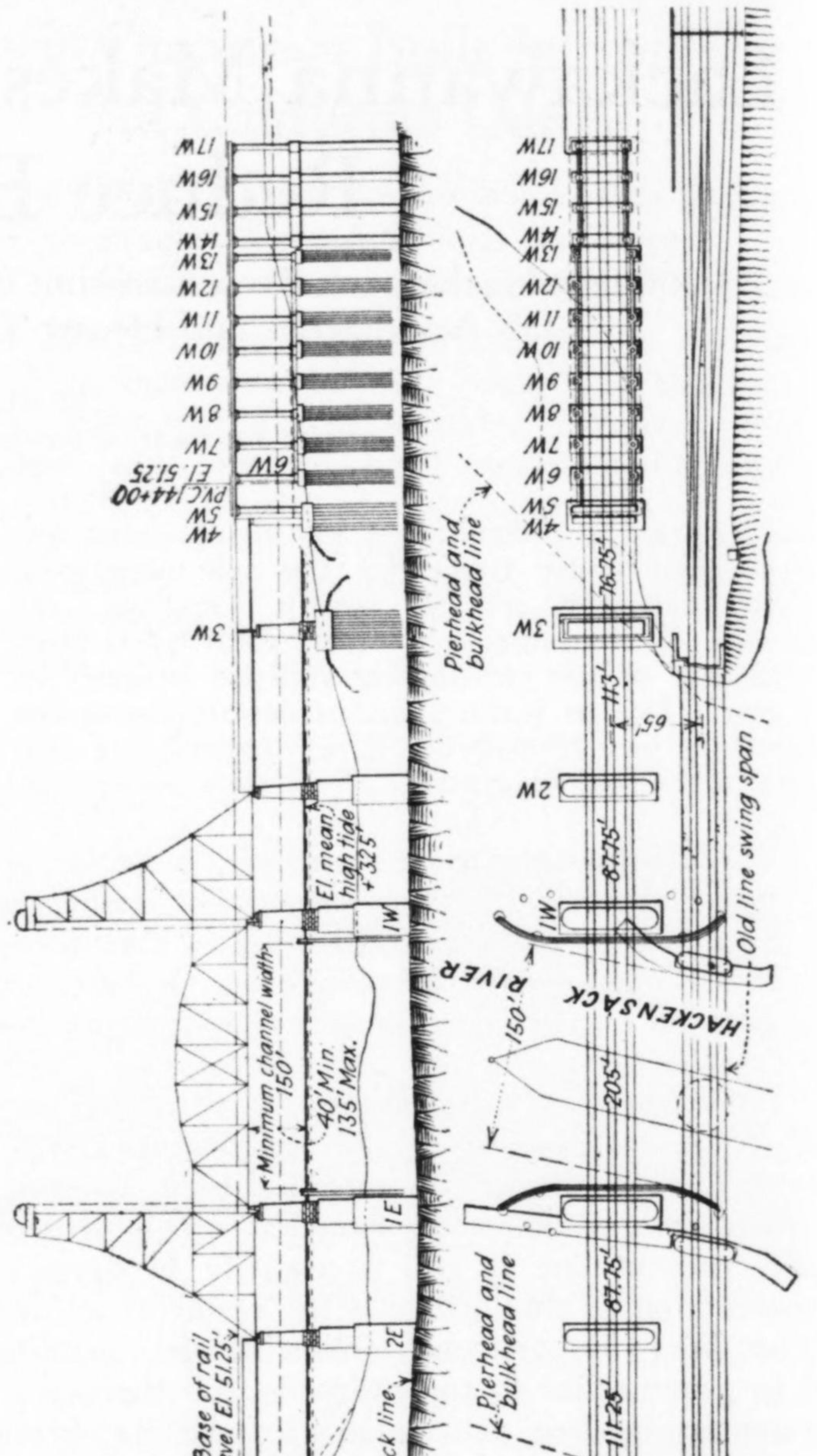

FIG. 2—GENERAL PLAN AND PROFILE OF HACKENSACK RIVER CROSSING

designed to be carried down to solid rock, and pneumatic caissons were chosen over open cofferdams because of the proximity of the present drawbridge abutments. Borings indicated rock at El. —75, but when excavated this proved to be a soft shaly variety, so the caissons were carried down about 15 ft. deeper, where a trap rock was found. (El. —91 for 1-W and El. —89 for 1-E.)

Piers 1-E and 1-W are identical; the dimensions at the base are 22 ft. 3 in. in width, by 71 ft. 3 in. in length, which dimensions are the result of design for full live load on all three tracks together with the tension at the top of towers resulting from the pull on the cables for signals, telephone, future electrification, etc., as well as wind load, which was figured at an angle of 45 deg. to the structure. The maximum resultant pressure at the base is 35 tons per square foot.

The bottom area of the piers is maintained for a depth of 9 ft. 6 in., at which elevation a 1-ft. $1\frac{1}{2}$-in. offset is made, giving a pier section of 20x69 ft. This section is maintained until El. —30 is reached, at which elevation it was thought that the removable cofferdam could be started and a 1-ft. 6-in. offset was provided on each side. Between El. —5.25 and +5.25, which are the extremes of highest and lowest tide ever encountered at this point of the Hackensack River, a protection of granite masonry was provided to guard against damage from

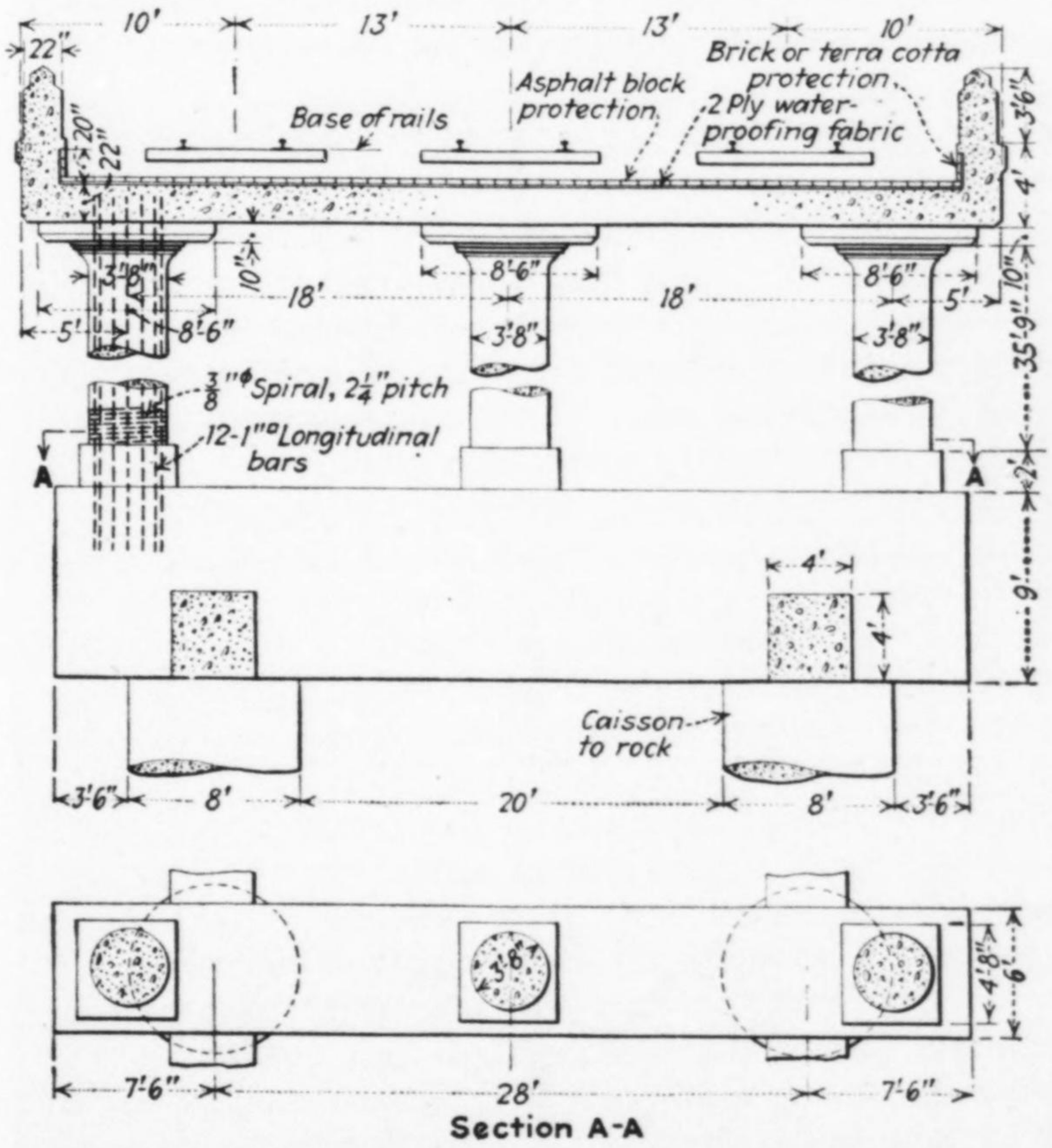

FIG. 3—DETAIL SECTION OF EXTREME WEST END FLAT SLAB

Remainder of flat slab approaches similar except for foundations.

ice or floating débris carried by the swift current. Piers 2-E, 2-W and 3-W, all of which are in the river, were similarly protected.

Pile Foundations—Pier 3-W is founded on timber piles driven to refusal and was designed to be constructed by the open cofferdam method, as were also piers 4-W, 5-W, 3-E and 4-E, which piers carry the steel deck girders of the river portion as well as the columns marking the beginning of the reinforced-concrete viaduct approaches.

All the piers east of 6-E carrying the concrete viaduct on the easterly approach, as well as those carrying the intervening steel girder spans, are carried by cast-in-place concrete piles 25 to 38 ft. long. At 5-E and 6-E, and at all bents on the west approach where lengths exceeding 40 ft. were required, timber piles were used.

Deep Concrete Piers—Because of the great weight of the fill flowing under the viaduct at the west end tending to cause settlement of the soil thereunder, which in turn might cause the settlement of the piles carrying these piers, it was deemed advisable to carry piers 14-W to 17-W inclusive to rock. They are about 8 ft. in diameter and were built by driving sheet piles to rock, excavating and filling with concrete. The piers average 70 ft. deep. This precaution was taken after the foundation failure of a small structure constructed in the early stages of the project some distance west of the river in the treacherous

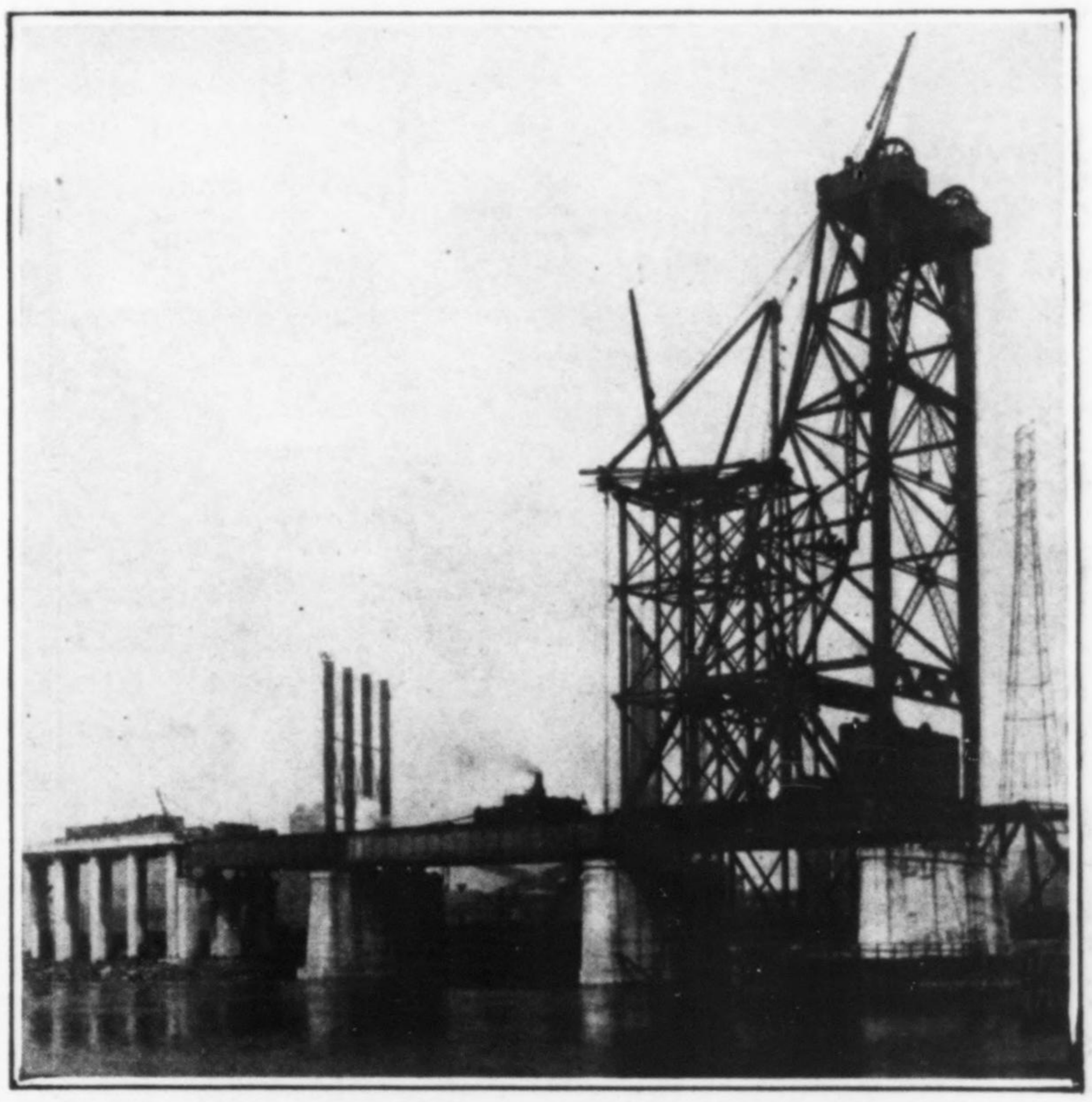

FIG. 4—WEST TOWER DURING ERECTION

Hackensack meadow soil. There are two piers for each bent, tied together by a heavy girder 9 ft. deep by 6 ft. wide, heavily reinforced to take the moments and shears due to the concentrations of column load at the center of the span and the loads of the side columns which are carried in cantilever as well as the superimposed fill carried by the girders.

Essential Details of Superstructures

Lift Span—The lift span trusses are of the Parker type, which is a modified Pratt type with inclined top chords. The counterweight cables are attached by means of adjustable eye-bolts to lifting girders which extend across each end of the lift span and are framed directly to truss hangers.

The trusses of the lift span as well as the towers are spaced 45 ft. center to center to accommodate three tracks 13 ft. on centers and at the same time provide minimum side clearance of 8 ft. from the center lines of outside tracks. The trusses of the lift span have a span length of 198 ft. center to center of bearings with a depth at the center of 48 ft., tapering down to 37 ft. at the hip and the horizontal end member. The floor depth is 8 ft. from base of rail to underclearance, with a depth of 16 ft. from base of rail to top of piers. The maximum movement of the lift span is 95 ft., which gives an open clearance of 135 ft. above high water and places the sheave centers 168 ft. above the top of masonry.

In order to reduce the load of the lift span to a minimum and thus reduce the equipment requirements, an open tie floor was selected rather than a solid concrete waterproofed deck with ballasted track, with which the remainder of the improvement from end to end is equipped. In order to reduce the load still further, silicon steel was specified for the trusses and the floorbeams, with an allowable stress of 22,500 lb. per sq.in. The resulting load requires the use of eight 2-in. diameter counterweighted cables on each of two sheaves of 13 ft. pitch diameter at each corner. The span is operated by means of winding drums located at the middle of the span, the operating ropes being hitched near the top and bottom of the towers and passing over deflector sheaves at each corner and on to the drums. The drums are rotated by

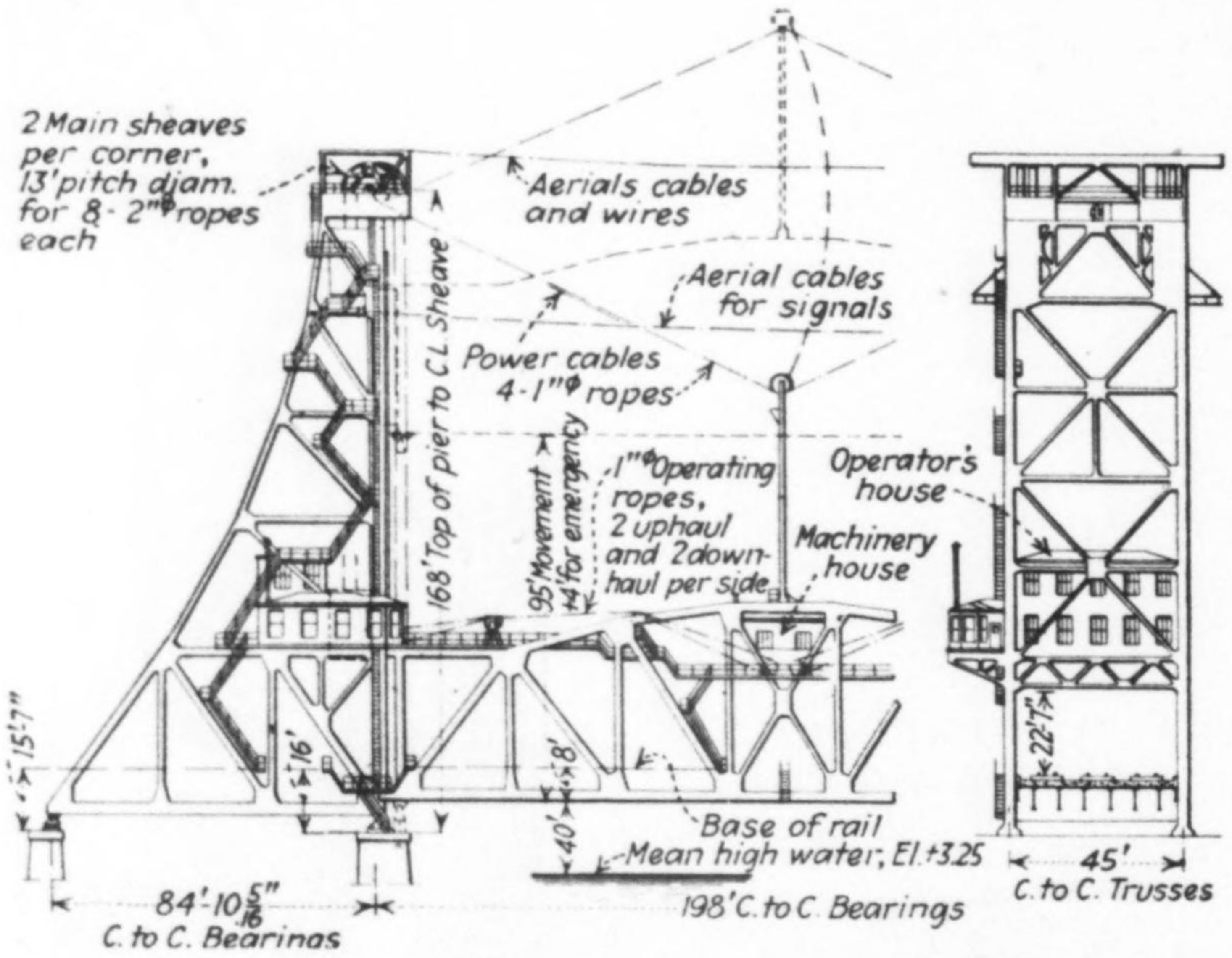

FIG. 5—DETAIL OF LIFT SPAN AND EAST TOWER

direct-current electric motors and a train of gearing which is located in the machinery house at the center of the lift span. The machinery house is also provided with a gasoline engine with mechanical drive for emergency operation of the lift span.

Air-operated locks to lock the lift span in its closed position are controlled by the railroad signal machine in the operator's house. The operator's house is a two-story building located in the east tower span above the vertical clearance line, the lower floor of which contains the bridge control bench and main control panel board, as well as the signal and interlocking machines, while the generator room houses the transformers, motor generator and gasoline-engine generator sets. The power service line comes from the east and has a connection with three 1,000-kva. transformers in the operator's house furnishing power at 440 volts three phase for operating the motor generator set. An emergency service line comes from the west across the lift span to the east tower. Transformers (5 kva.) provide current for lighting on the main service line and on the emergency line with automatic transfer switches on the secondaries of these transformers so arranged that lighting current ordinarily will be taken from the power service line, but in emergencies it will be switched to the other line. Direct current for lift span motors is supplied by motor generator set, but a stand-by source of direct current is provided by a gasoline-engine generator set.

For carrying the electric cables from the towers to the machinery house at the center of the lift span a steel rocker bent is mounted at the center on top of the lift span, the top of the bent being hitched to the east tower by means of a group of four wire ropes and to the west tower by a continuation of these ropes passing over a sheave on the west tower and held taut by a counterweight. In addition, this arrangement tends to compensate for the unbalancing effect of the rolling of the main ropes from one side of the sheaves to the other when the lift span is raised or lowered. This device is patented.

The signal machine controls the track switches, signals, derails, span locks, locks on span controllers and the locks on the mechanically driven gasoline engine in the machinery house. The span control is interlocked.

The counterweights for the lift span are designed to be of concrete poured into steel boxes, which will act as forms as well as permanent containers. There will be one counterweight on each side, weighing about 610 tons each.

The tower span, the length of which is 84 ft. $10\frac{3}{16}$ in. center to center of bearings, is of the floorbeam-and-stringer type surmounted by a solid concrete deck similar to that covering the remaining river spans, which are of the steel deck-girder type. The depth of girder for the the 110-ft. spans is 10 ft. $\frac{1}{2}$ in., while that of the 75 ft. spans is 9 ft. $\frac{1}{2}$ in., which same design was used for the spans over Duffield Ave. and adjacent to it. These girders were shipped in a solid train of 43 cars.

Flat Slab Design—The remainder of the superstructure is of reinforced concrete of the girderless flat slab type with four-way reinforcement. The various types of construction are separated by expansion joints. Whereas the steel spans were designed as simple spans center to center

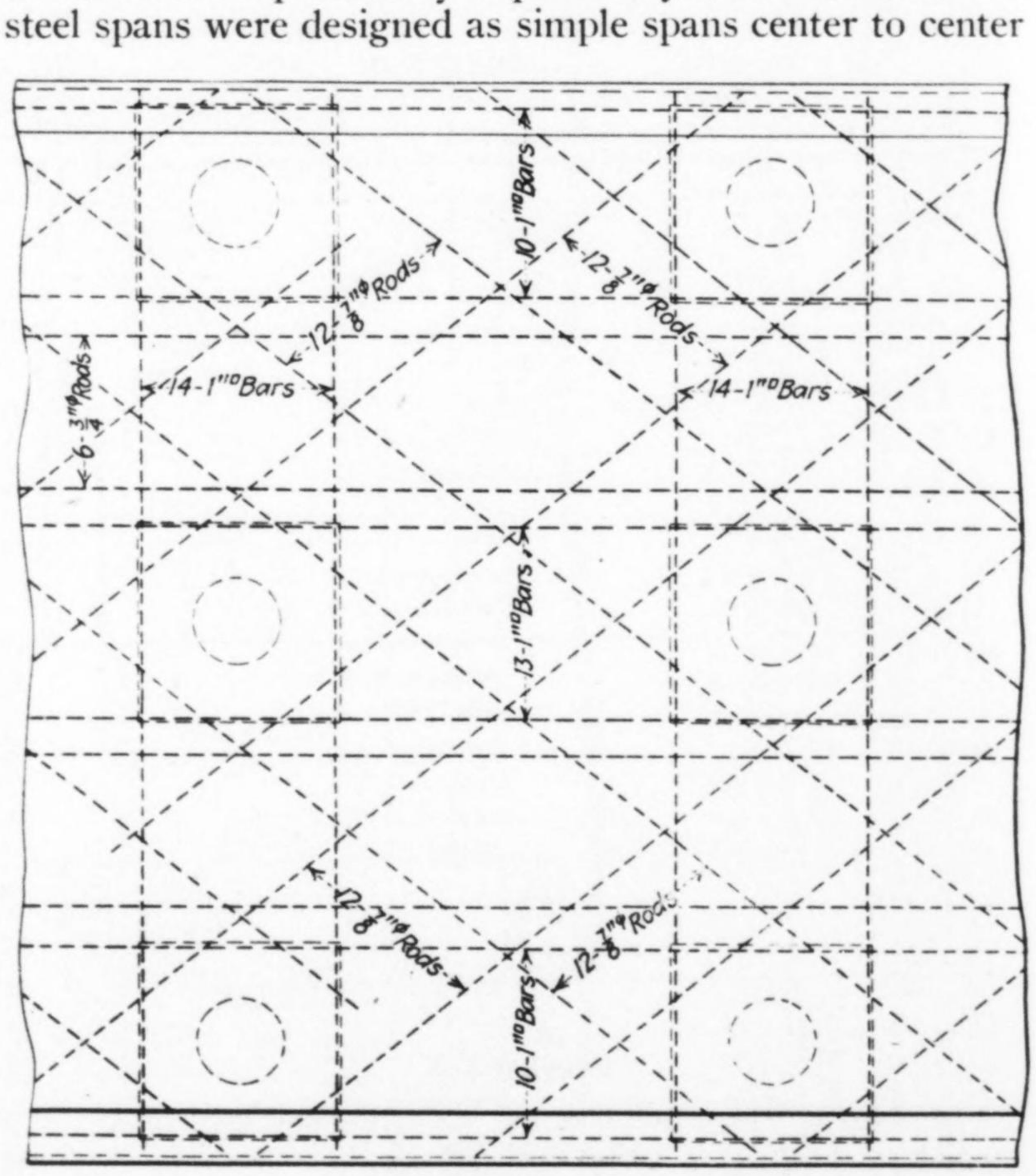

FIG. 6—TYPICAL REINFORCING DETAILS FOR FLOORS OF FLAT SLAB CONSTRUCTION

of their respective bearings, the concrete structure was designed as continuous between expansion joints. The system of spans from one expansion joint to the other was assumed subjected to the loading of two E-65 engines per track followed by a uniform load of 6,500 lb. per linear foot, so placed, respectively, as to give the condition of maximum column reaction for the row of columns adjacent to the end and for the intermediate rows of columns. The columns were made 3 ft. 8 in. diameter, not because of their loading but because of their unusual height, which varies from 36 ft. to 41 ft. These columns

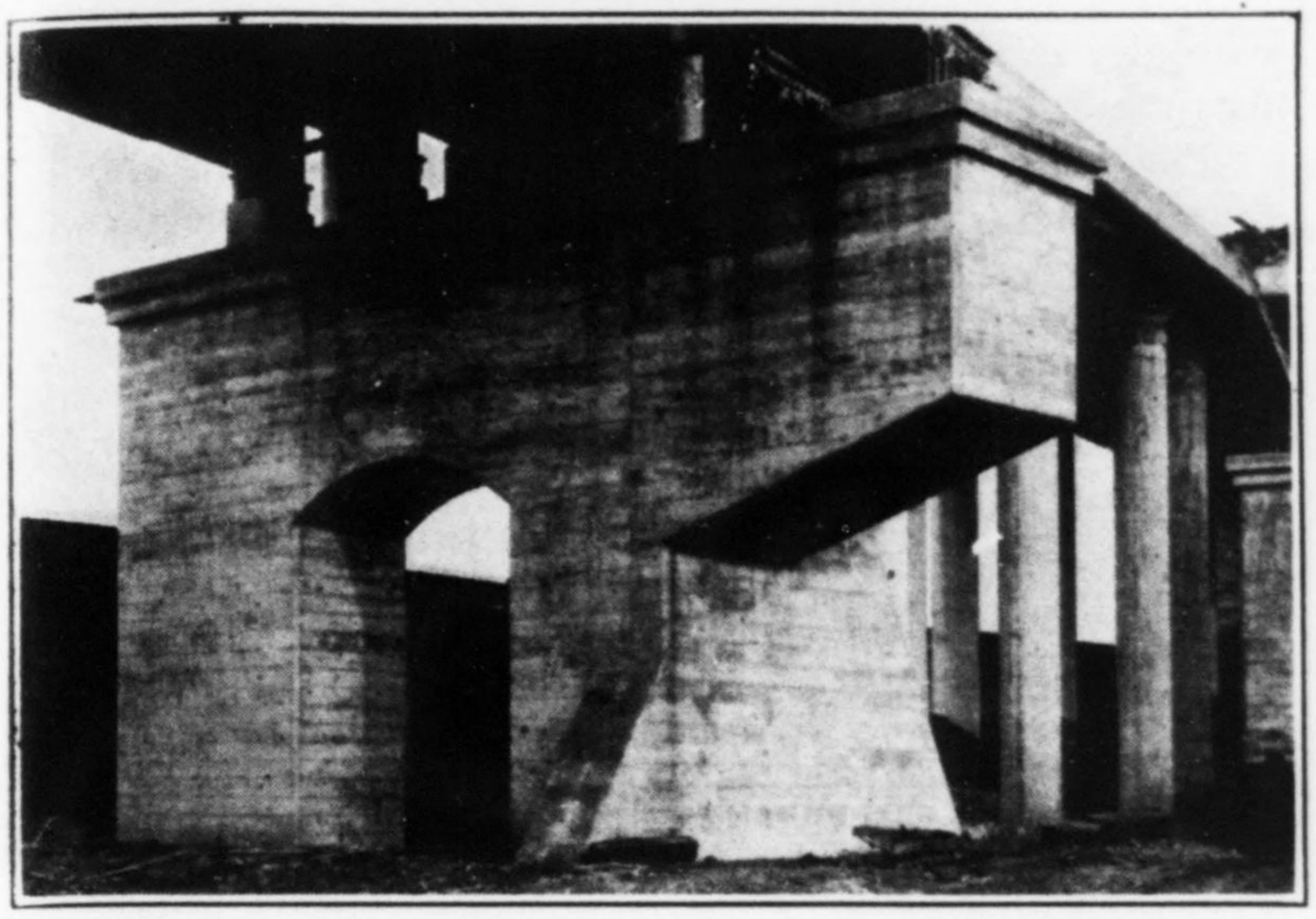

FIG. 7—CANTILEVER PIER AT DUFFIELD AVE.

are reinforced both spirally and longitudinally and each row or bent is supported on a continuous pier resting on either concrete or wooden piles as already described. All the piers are tied together by longitudinal struts of reinforced concrete along the outside rows of columns, and on the west approach, because of the softer soil, by a double set of ties both at the footing level and at the column bases.

For the design of the slab the loadings were then assumed and so placed as to result respectively in the maximum negative moments at the supports adjacent to the ends and over the interior rows of column as well as in the maximum positive moments near the centers of the end spans and at the intermediate spans. To compute the moments transversely a uniform load was assumed so placed along the line of tracks longitudinally as to produce the maximum moments corresponding to those found for the various conditions of locomotive concentrations indicated above, which moments were equated to those resulting from the concentrations, and the equivalent uniform load was thus determined. The track centers being 13 ft. transversely, this equivalent live load was assumed distributed over 13 ft. at the track location on the transverse column system and the maximum live load moments transversely were then arrived at by assuming the conditions respectively of one track loaded, two tracks loaded and all three tracks loaded, and the largest of these designed for. Moments were then found for the dead loads, and when combined with the impact the maximum conditions were obtained.

The next step was the distribution of these moments over the various bands. The maximum positive live load moment longitudinally was considered spread over 13 ft. and resisted by the two half longitudinal bands and the two diagonal bands in inverse ratio of the distance of the center of each band to the center of loading; the resulting moment in the diagonals was increased by the ratio of the spans. Since the diagonals also resist a portion of the moment transversely, one half the average of the transverse moments at the edge of each panel was considered distributed to the transverse and the diagonal bands in proportion to their spans. Where the diagonals were of unequal length, the sum of their portion of the longitudinal and transverse moments was distributed between them inversely as the square of the diagonal spans. The dead load moments were likewise distributed and impact added by formula $LL^2 \div (DL + LL)$.

For the application of these moments for the design of column capitals and the shortening of the span resulting therefrom, they were reduced on the basis of the square of the ratio of the clear span between points near the edge of the column capital and the span center to center of columns approximately similar to the coefficients used by the Chicago Building Code.

The slab was made 1 ft. 10 in. thick, with an additional thickness of 10 in. for the 8-ft. 6-in. square drop-panels; the reinforcing steel was selected to fit the moment requirements, longitudinally, transversely and along the lines of both diagonals, and the required number of bars bent up over the supports and carried into the adjacent panels. Temperature steel was provided in the upper plane in the longitudinal direction. (Fig. 6.)

There are also provided reinforcement and anchor bolts for future catenary supports in the event of the selection of that type of electrification in the future, as well as for signal bridges that are located on the viaduct.

Both for architectural reasons and for the purpose of aiding in the protection of trains in the event of derailment on the viaduct, the balustrades were designed with a heavy cross-section with enough space to accommodate electric light, signal and high-tension lines in the walls on both sides. The electric ducts have pull-boxes at the signal bridges and at the safety niches, which were provided at intervals to permit of protection of a hand car and crew.

Cantilever Pier—One of the features in the design of this structure is the cantilever bracket overhanging the pier on the east side of Duffield Ave. This cantilever acts as the support for the reactions of the girders spanning 75 ft. center to center of bearings on either side, for which purpose its depth was made 14 ft. at the pier, tapering to 10 ft. at the end for an overhang of 12 ft. 6 in. and a width of 7 ft. 6 in. To resist the moment of more than 130,000,000 in.-lb. resulting from the moment of the maximum live load reactions for both spans fully loaded, together with that due to the dead load of the steel girders and deck plus that of the weight of the cantilever itself as well as the impact, there were embedded in the top plane 35 $1\frac{1}{4}$-in. square bars in two layers spread over the 7-ft. 6-in. width—22 bars 3 in. below the surface and thirteen bars 6 in. below. To assist in taking the shear of $1\frac{1}{3}$ million pounds, eight $\frac{7}{8}$-in. round rods were bent down as diagonal reinforcement, while seventeen sets of stirrups of four each were installed, all to resist the diagonal tension resulting from the shear.

Counterfort Abutment—Another unusual feature is the

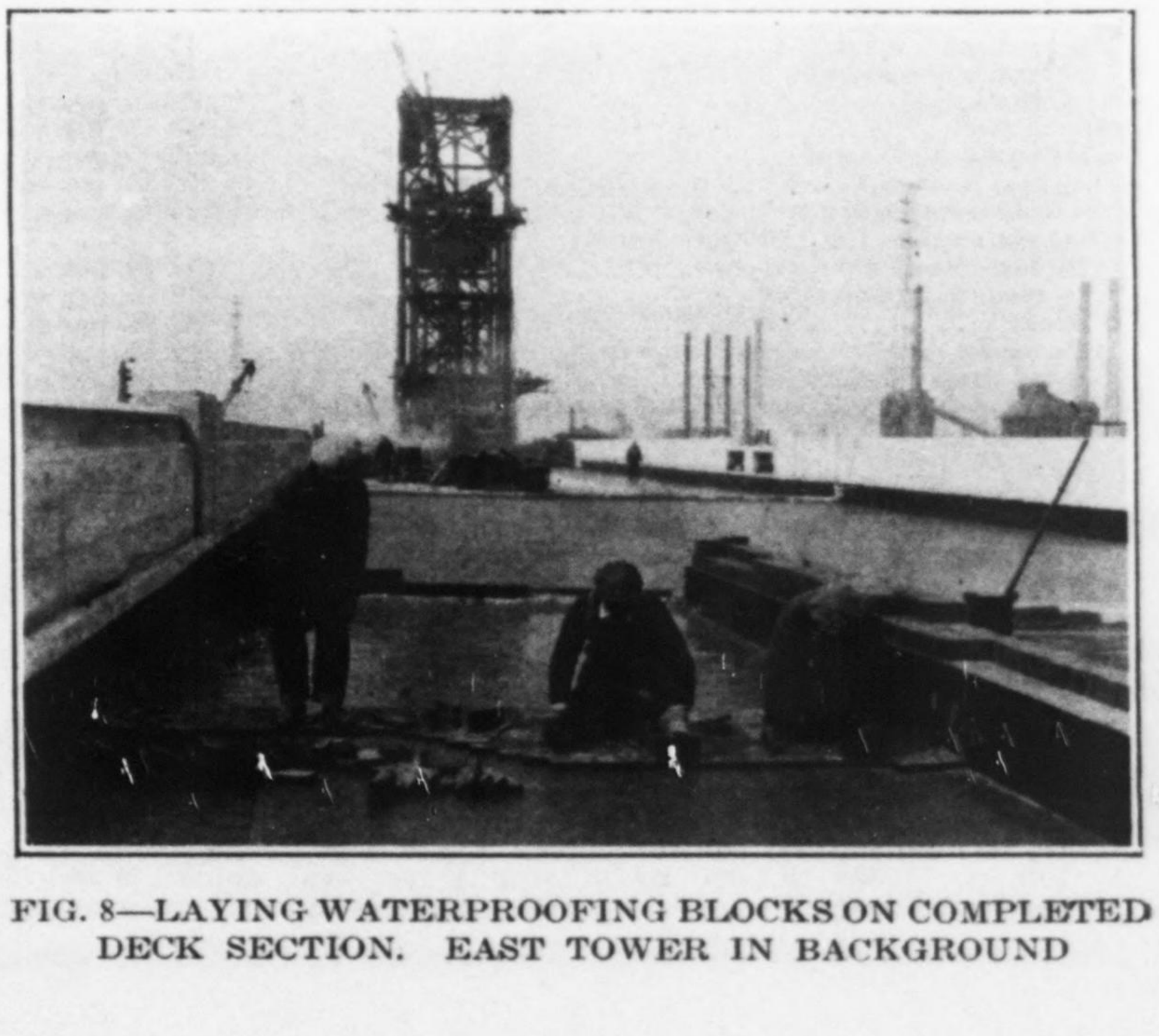

FIG. 8—LAYING WATERPROOFING BLOCKS ON COMPLETED DECK SECTION. EAST TOWER IN BACKGROUND

counterfort abutment completing the viaduct on the east end. To reduce the pressure against this high abutment and at the same time to reduce the load on the footing as well as the eccentricity of the pressure thereon and hence reduce the number of piles required, large openings were provided for the fill to flow through.

The heights of the openings were made to conform to the slope of the fill, and the slab between the counterforts and beyond was reinforced for the varying pressures, the counterforts for the overturning and shear, and the bottom slab to resist the upward pressures with the counterforts as supports. The wall along Duffield Ave. is of the semi-gravity type—that is, supplied with sufficient reinforcement to reduce the section which would otherwise be excessively heavy.

Retaining Wall—This wall is designed for the surcharge of the sloping embankment to the railroad roadbed, on which in addition the live load of engine loading is considered superimposed. The abutment and wall are founded on concrete piles. The assumed earth pressures used in the retaining wall design are shown in an accompanying table.

ASSUMPTION OF EARTH PRESSURES AGAINST RETAINING WALLS AND ABUTMENTS

Height	$\frac{1-\sin\phi}{1+\sin\phi}$	Angle of Repose
1 ft. to 20 ft.	0.286	33 deg., 41 min.
20 ft. to 30 ft.	0.300	32 deg., 33 min.
30 ft. to 40 ft.	0.315	31 deg., 24 min.
Above 40 ft.	0.333	33 deg., 0 min.

The live load concentrations of locomotives are distributed from actual concentrations at an angle of 45 deg. to the wall, the length of distribution being determined by the distance at the intersection of the line of distribution with back of the wall; the load divided by this distance will give the amount of surcharge when divided by the distance between tracks.

Waterproofing and Drainage—The entire deck slab of the structure, including the flat slab viaducts, the floor of the deck girder spans and tower spans, were waterproofed by the application of two-ply asphalt saturated open mesh cotton fabric laid in asphalt covered with an asphalt mopping and protected from damage by the ballast with 8x4x1¼-in. asphalt paving blocks on the flat surfaces and common brick on the vertical sides of the parapets. The slabs were pitched so as to drain off every two panels, on the level portions of the flat slab viaducts the water to be carried by 4-in. openings through the slabs and by pipes down the sides of the columns. The slabs

STRESSES USED IN DESIGN

Pile loadings—wood	15 tons
Pile leadings—concrete	30 tons
Structural steel	16,000 lb. per sq.in.
Silicon steel	22,500 lb. per sq.in.
Reinforced steel, tension	16,000 lb. per sq.in.
Diagonal tension in bent up bars (component of shear)	16,000 lb. per sq.in.
Shearing stresses in stirrups (per prong)	10,000 lb. per sq.in.
Concrete stresses for 2,000 lb. concrete:	
In flexure and compression	650 lb. per sq.in.
In straight compression	500 lb. per sq.in.
Shear	40 lb. per sq.in.
Punching shear	150 lb. per sq.in.
Bond stress—deformed bar	100 lb. per sq.in.

Allowable stress in columns:

$P = f_c \times A + nf_c\, pA.$

$f_c = f'_c\ (25 + 12\ p').$

P = total load.

f_c = allowable concrete stress in compression.

A = area of core of concrete column.

n = ratio of moduli of elasticity = 15 for 2,000 lb.; 10 for 3,000 lb. concrete.

p = ratio of longitudinal reinforcement.

p' = ratio of spiral reinforcement.

f'_c = ultimate strength of concrete in compression.

For 2,000-lb. concrete (new basis of design), f_c = 500 lb. + 240 lb. per 1 per cent of spiral.

For 1 per cent spiral reinforcement, f_c = 740 lb. per sq.in.; for ½ per cent, f_c = 620 lb. per sq.in.

Stress in longitudinal reinforcement = for 1 per cent spiral, 15 × 740 = 11,000 lb. per sq.in.; for ½ per cent spiral, 9,300 lb. per sq.in.

over the steel spans were similarly pitched and provision was made for taking off of the water from these decks.

The columns that will be buried in the fill, as well as the abutment and back of the retaining wall, were coated with a heavy coating of emulsified asphalt, to which was added asbestos fiber to give it additional body for troweling.

A feature of the construction methods used by the contractor on the land portion of the work is the forms

SUMMARY OF QUANTITIES

Timber piles	1,168
Concrete piles	1,137
Concrete, cu.yd	35,000
Reinforcing steel, lb	1,350,000
Structural steel, total tons	3,840
Silicon steel, tons	630
Embankment fill, cu.yd	600,000

used for the construction of the flat slab viaduct. The column forms were made of heavy sheet steel stiffened by angles at the joints and braced diagonally, transversely and longitudinally. The forms from the slab and drop panels were supported by bolts embedded in the permanent columns with structural steel members directly supporting the forms proper at the required levels. (This scheme is patented by the writer.) This obviated the necessity of using long upright members and the difficulty entailed in bracing them.

G. J. Ray, chief engineer of the D., L. & W., directed both the design and construction of all the work. The design of all the work, including that of the lift and tower superstructures and machinery, etc., which were designed by Waddell & Hardesty, consulting engineers, was done under the direction of J. L. Vogel, bridge engineer, and that of the writer, while the construction was supervised by M. H. Doughty as division engineer and W. H. Speirs as resident engineer.

The Foundation Company was the contractor for the pneumatic caissons and other piers of the river portion, while the H. F. Curtis Company constructed the remainder of the masonry work and foundations, except the concrete piles, which were driven by the Raymond Concrete Pile Company.

The lift span and towers as well as the steel girders were supplied by and are being erected by the American Bridge Company.

Elements in St. Francis Seepage Water

The tabulation of elements found by chemical analysis in samples of seepage water taken from the west (right) bank just below the St. Francis dam after the collapse was only partly given, due to an oversight, in last week's issue (May 10, 1928, p. 729). The complete tabulation is as follows:

	Aqueduct Water (P.P.M.)	Seepage From Right Bank (P.P.M.)	Ratio Second Col. to First Col.
Chloride	24.00	241.50	10.1
Evaporated solids	294.60	2,319.00	7.9
Silica	35.20	666.20	18.9
Oxides of iron and aluminum	3.00	4.00	1.3
Calcium	35.20	358.80	10.2
Magnesium	7.31	103.19	14.1
Carbonate radical	None	None	None
Bicarbonate radical	87.60	168.36	1.9
Sulphate radical	35.50	1,044.92	29.4
Alkalinity	143.60	276.00	1.9
Sodium and potassium	34.04	125.13	3.7
Hardness	117.97	1,320.08	11.2

In reference to the report published last week attention should be called to the error in the cut caption on p. 726. The caption should read, "Standing Portion and Left Bank After Failure," instead of "Right Bank" as printed.

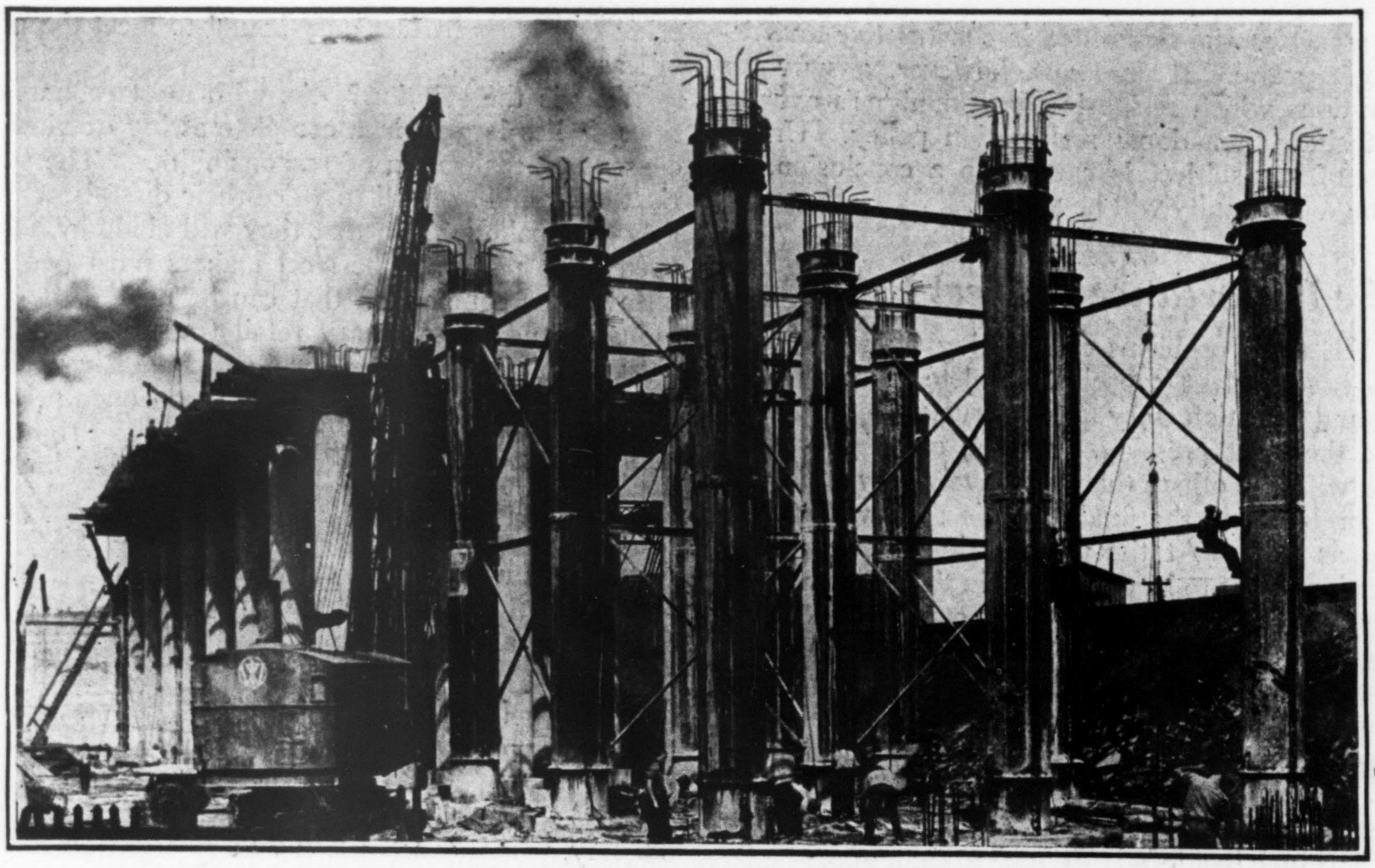

Fig. 1—Steel Column Forms and Structural Steel Beams for Slab Forms, Hackensack River Bridge, D.L.&W. R.R.

Construction of Lackawanna High-Level Crossing Over Hackensack River

Trucks Place 475,000 Cu.Yd. in Approach Fill Across Hackensack Meadows—Vertical Lift Span Erected in Place Without Interference to River Traffic—Slab Forms Supported Without Falsework

By W. H. Speirs

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A Description of the design and purpose of the new high-level crossing over the Hackensack River at Jersey City, just completed by the Lackawanna, appeared in the *Engineering News-Record*, May 17, 1928, p. 779. The new crossing was built solely to eliminate approximately 75 per cent of the number of bridge openings by increasing the closed clearance from 12 ft. to 40 ft., thus causing less interference with the road's 200 daily trains, mostly suburban. Construction features of the bridge, approaches and a concrete box subway are described in the following article. —Editor.

Many interesting construction features were involved in building the Lackawanna new three-track high-level crossing of the Hackensack River at Jersey City, a project totaling 1.4 miles in length and costing more than $3,000,000. Lack of space for construction operations complicated much of the work. The new three-track vertical lift bridge with a span of 205 ft. is parallel to and only 65 ft. distant, center to center, from the double-track swing bridge on the old line. During the construction of the lift span, river traffic had to be passed without interference and clearance provided for the swinging of the old bridge. Building up a 600,000-cu.yd. fill mostly with automobile dump trucks and constructing the west approaches on an underpass through the fill in the unstable part of the Hackensack meadows were difficult parts of the work. The project is now complete and traffic has been shifted to the new line. A general plan and profile of the bridge span are shown in Fig. 2.

Over the river proper the crossing consists of three deck plate girder spans, two tower spans and a vertical lift through-truss span with river clearances of 135 ft. and 40 ft. in open and closed positions, respectively. An earthfill 1,300 ft. long and a girderless, flat-slab concrete viaduct 550 ft. long, broken by one plate girder span over a street crossing constitute the east approach. The west approach is made up of an earth embankment 4,800 ft. long and a concrete viaduct 266 ft. long similar to that on the east side. In profile the crossing takes off the old main line at the extreme east end of the fill on a 0.49 per cent upgrade westward, for a distance of 0.3 of a mile, is level for 800 ft. across the river spans and descends on a 0.95 per cent grade for about 1 mile, where it strikes the old railroad grade.

The lift span will be electrically operated by direct current supplied by a motor generator operated by alternating current taken from the railroad's power line. On failure of this supply a gasoline engine-driven generator will furnish the power. As a second emergency equipment a direct-connected gasoline engine will be available for mechanical operation. The time required for the maximum 95-ft. lift is $1\frac{1}{2}$ minutes.

River Pier Construction—The four main river piers, *1E, 1W, 2E, 2W* (Fig. 2) were sunk to rock, 91 ft. below the water level, with pneumatic timber caissons with structural steel air chambers. No trouble was experienced in sinking the caissons except striking some of the timber grillage and riprap of two of the old piers, which required considerable work by divers to remove.

Steel sheet piling open cofferdams were used in constructing the three remaining river piers, *3E, 3W* and *4W*. Wood piling was driven inside the cofferdams for the pier foundations. To prevent any possible disturbance to the adjacent main line embankment the cofferdams were sealed with 5 ft. of concrete placed with tremies before pumping out.

All the concrete for the seven river piers was poured from a floating plant supplied by materials from barges (Fig. 4).

Approach Viaduct Construction—Along the east viaduct approach firm bearing ground was found at a depth of 10 to 20 ft. below the surface. The original meadow land on this side of the river had been filled in years ago as part of the industrial development of Jersey City. Most of the viaduct column bents on this approach and the Meadow St. retaining wall rest on 38-ft. concrete piles. The foundations for each bent consist of 56 piles, capped by a reinforced-concrete slab 11x47 ft. in area and 4 ft. thick. The caps are tied together at each end by concrete struts. On account of a dip in the firmer subsurface materials wood piling up to 75 ft. long was driven for the three bents nearest the river.

The west approach viaduct rests on timber piles 75 ft. long, excepting the four westerly bents, which are supported on circular concrete piers, 7 ft. 8 in. in diameter, extending to hardpan, a depth of approximately 70 ft. below the meadow level. This special circular pier construction was required to resist the thrust of the approach track embankment, as no abutment was built between the fill and the column viaduct because of the character of the foundation required. Each pier consists of two concrete cylinders supporting a reinforced-concrete beam 6 ft. wide and 9 ft. high on which the three columns of the viaduct bent are superimposed.

The circular piers were built by driving to refusal interlocked steel sheeting in two lengths. A crawler crane was used in setting up the sheeting and to handle the heavy steam hammer used in driving. When the sheeting had been driven to refusal, the caissons were excavated with an orange-peel bucket. The water was not pumped out, on account of the possibility of disturbing the material around the adjacent foundation piling. The concrete, amounting to approximately 120 yd. for each caisson, was placed with a covered bottom-dump bucket. The sheeting was not pulled, but the portions projecting above the top of the concrete level were burned off. Fig 3 shows the caissons after concreting.

FIG. 3—CIRCULAR PIER CAISSONS AT WEST END OF VIADUCT

Column and Slab Form Design—Several innovations in form design were used in the girderless flat slab viaduct construction. This part of the structure consists of a 22-in. concrete flat slab on three-column bents, each column 3 ft. 8 in. in diameter and about 40 ft. high. Steel forms were designed for the columns, each column unit consisting of two full length semi-cylindrical sections reinforced vertically and circumferentially with structural angles. The variation in column heights was allowed for by having the standard length of form the minimum height of column, with a few short extension sections of varying lengths up to 3 ft. Twelve sets of column forms, sufficient for four bents, were built. These could be set up in groups of six, nine or twelve and were braced with structural angle swaybracing. This bracing was adjustible, to allow the column forms to be placed in an exactly perpendicular position.

Expensive falsework necessary to support the heavy floor slab approximately 40 ft. above the ground was

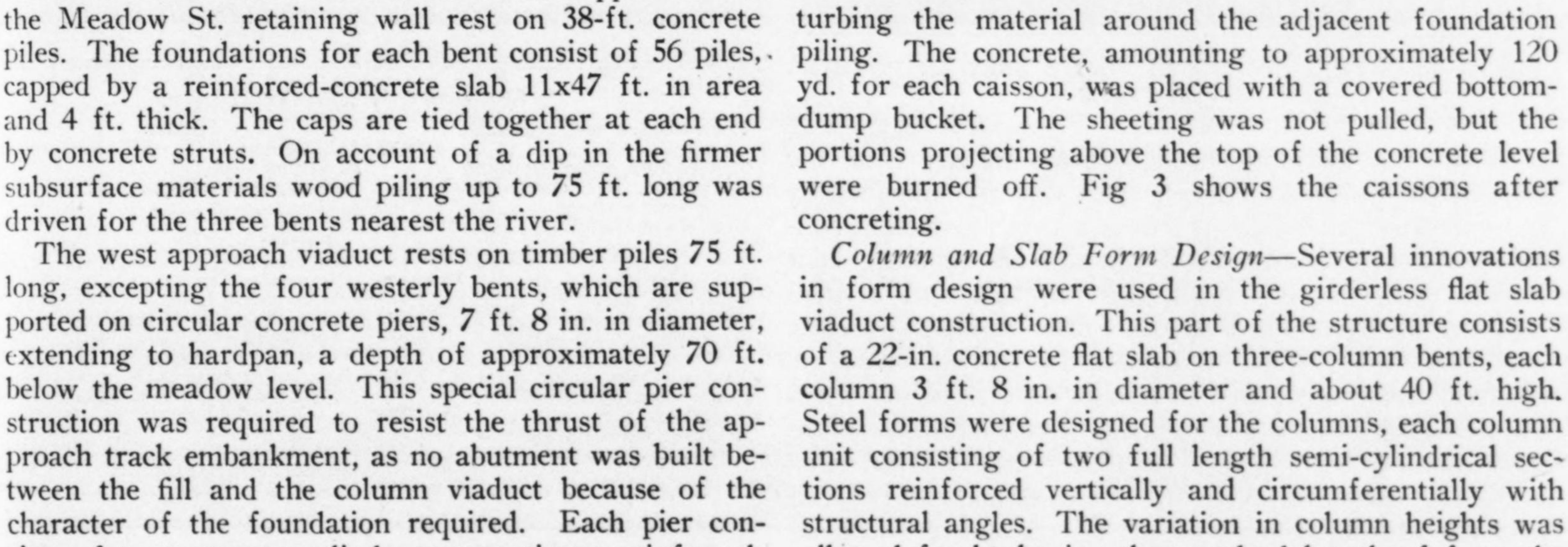

FIG. 2—PLAN AND PROFILE HACKENSACK RIVER CROSSING, D.L.&W. R.R.

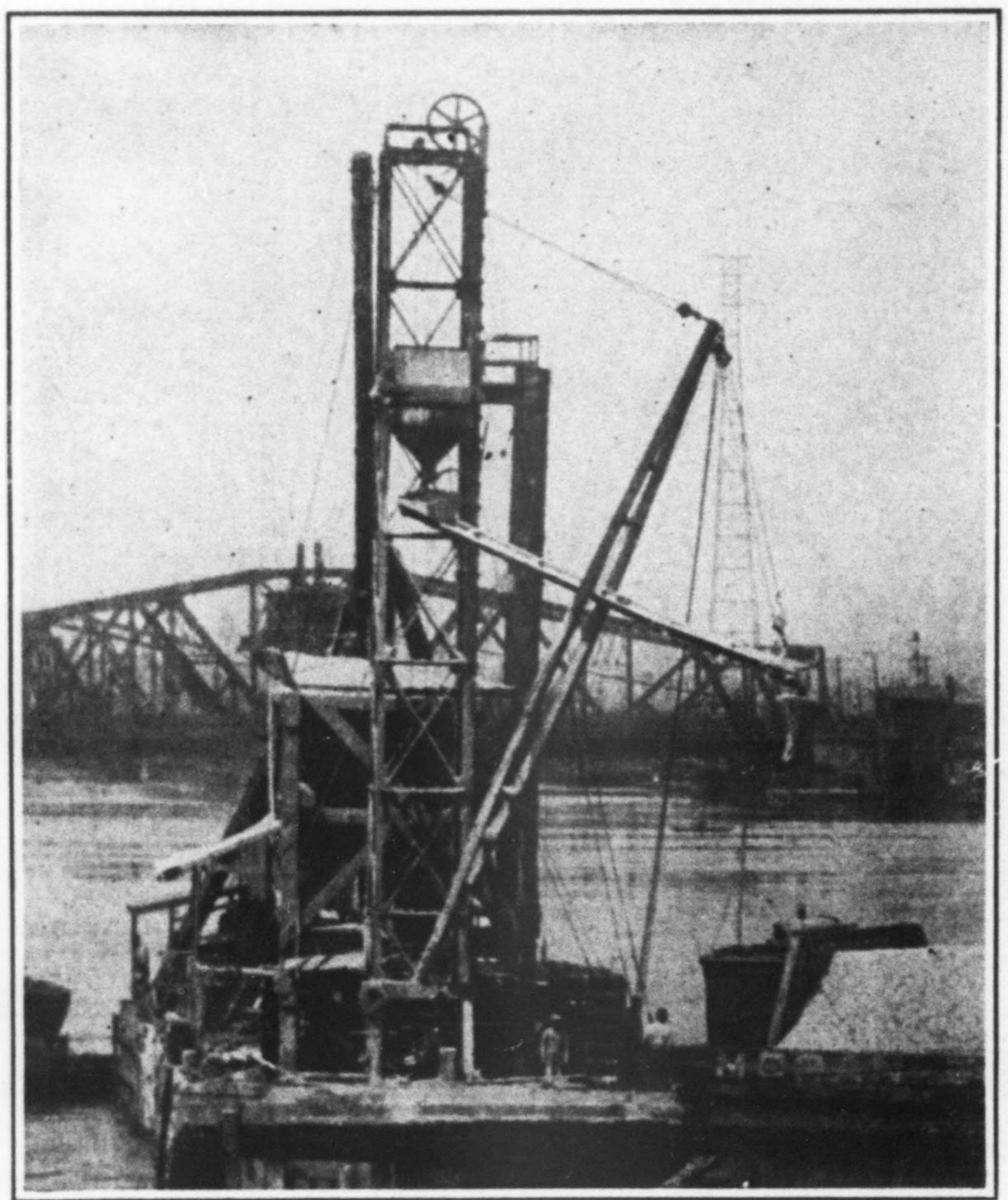

FIG. 4—FLOATING CONCRETE PLANT USED IN POURING RIVER PIERS

eliminated by hanging the slab forms directly from the tops of the finished columns. This was accomplished by designing the steel column forms with a top section consisting of a structural steel channel 15 in. deep, bent circular with flanges extending outward and divided into two sections for removal. This channel ring was anchored to the concrete column by bolts cast in the concrete and was allowed to remain on the column when the remainder of the form was removed. Structural steel channels were placed on these rings, one on each side of the bent tight against the columns, cantilevered past the outside column to extend the entire width of the slab. These cross-channels were used to support 12-in. I-beam stringers placed longitudinally and extending the full distance from center to center of bent. Wood joists, 2x12 in., placed across the I-beams supported the planking which completed the forms. Joints in the forms, provided for convenience in removal, were placed transversely along the center line of bents. In removing the forms, tackles were fastened to the ends of the transverse channels which projected beyond the forms at the side of the viaduct, and, using the finished concrete slab as an anchorage, the entire form section between bents, including the supporting steel members, was lowered to the ground by a steam hoist. All side forms were removed before this lowering took place. The supporting channel rings fastened to the columns were removed by burning off the anchor bolts. This system of supporting slab forms on the permanent column construction, used frequently on other construction of similar character, was designed and patented by M. Hirschthal, concrete engineer of the Lackawanna.

Concrete Plant and Placing Methods—The reinforced-concrete approach viaducts on both sides of the river were built from material storage yards and stationary mixing plants located at both ends of the elevated structure, before the approach embankments were placed at these locations. Crawler cranes, equipped with chamshell buckets, excavated the foundations and, fitted with various lengths of booms up to 60 ft., placed all forms and falsework and handled all concrete. The concrete was handled for the most part in dump-buckets by trucks from the mixing plant to the cranes. On the west viaduct foundations, hoppers and narrow-gage cars, operated by cable from a hoisting engine, transported the concrete to the cranes over an area inaccessible to trucks. A driveway along the new work, both east and west of the river, furnished the only area available for construction and transportation purposes which was not occupied by the elevated structure. Most of the construction material, excepting for that part of the structure over the river, was received by rail and delivered to side tracks, one on either side of the river.

Pouring concrete in the columns so as to retain a uniform mixture and to insure the thorough compacting of the concrete around the spiral reinforcing required special care. Because of the small space between the spirals and forms there was no opportunity to spade thoroughly or to rod the mixture in the form. A special $\frac{1}{2}$-yd. cylindrical bucket was used in placing the column concrete. This bucket was provided with a one-piece conical bottom, opening with a downward vertical movement and releasing its load slowly, thus insuring a comparatively uniform deposit over the area of the column.

SUMMARY OF QUANTITIES

		Steel	
Concrete piles, east approach viaduct	1,137	Lift Span:	Tons
Timber piles, river pier construction, east and west approach viaducts	1,168	Structural steel	840
		Stairways and walks	32
Reinforcing steel, lb	1,350,000		872
Concrete	Cu. Yd.	Towers:	
		Structural steel	1,287
Lift bridge piers (4)	19,400	Stairways and walks	33
Other river piers (3)	3,000	Counterweight boxes	94
East approach viaduct	6,200	Counterweight sheaves, shafts and bearings	115
West approach viaduct	3,800	Wire ropes and sockets	43
Girder approach span floors	1,100	Rope adjustment rods, span guides, etc.	35
Vehicle subway	1,200		1,607
James Ave. bridge, alterations to	300	Operating machinery	43
Retaining wall along Meadow Street	1,700	Approach girder spans	1,250
Total	36,700	Duffield Ave. bridge	300

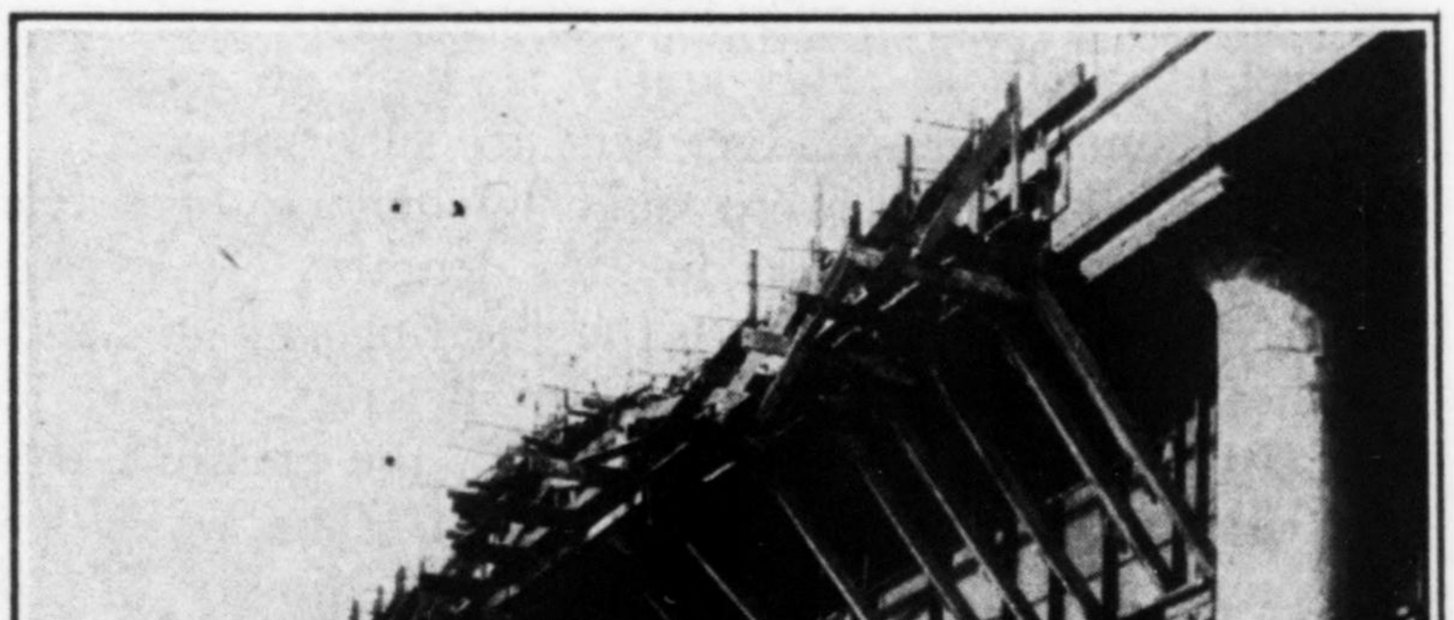

FIG. 5—SLAB FORMWORK ON PLATE GIRDER SPANS
Cantilever pier shown built to clear street right-of-way.

Aggregates not exceeding $\frac{3}{4}$ in. were used in the column concrete mix. It was found that hammering the outside of the forms at the concrete level while pouring bettered considerably the surface finish of the column.

Lift-Bridge Steel Erection—All the structural steel for the lift bridge and towers was delivered to the site in barges. The towers necessarily were erected first, one lower bay on each tower being placed by a large derrick boat. The balance of the tower erection was done by a stiff-leg derrick with a 110-ft. boom mounted on a temporary falsework about two-thirds the height of the tower and occupying the rear half of the tower area. The long boom of this erecting derrick extended approximately one-half of its vertical reach above the top of the tower, permitting easy erection of the top members and cable sheaves. The hoisting engines for the derrick were located on adjacent deck girder approach spans.

A lift span was erected at an elevation approximately 20 ft. above the closed position of the span, this elevated erection position being required to allow the swinging of the old drawbridge under the new span. The river was kept open for traffic during the erection of the lift span, excepting for one period of 72 hours. The river channel in use occupied approximately the westerly third of the channel to be provided under the new lift span, and the open position of the swing span of the old bridge over its pile protection pier occupied the middle third of the new channel, leaving only the use of the easterly third of the channel for erecting the lift span.

Erection of the first two-thirds of the span was accomplished by means of a supporting falsework pier resting on piles and located at approximately the center of the easterly two-thirds section of the span. This cantilevering of approximately half of the total span erected, as can be seen from Fig. 6, required the finished end of the span to be anchored down. The remaining one-third of the span was erected on a barge elsewhere (Fig. 7) and placed during the 72-hour period during which the river was closed to traffic. The one-third section was hoisted into place by a derrick boom mounted on the east side of the west tower. The section was fastened first at a special joint provided in the lower chord, with the free end resting on the permanent pier and with the section of upper chord over the joint in the lower chord omitted. After both sections had been placed, hydraulic jacks under the end floorbeam raised or lowered the end of the trusses to allow the omitted upper chord sections to be fitted. The placing of part of the top swaybracing between trusses and the lifting girder superimposed on end posts to which the lifting cables are attached completed the steel erection necessary before raising the lift span to the maximum height to open the river to traffic. The floor stringers and beams were placed after the span had been lifted to its maximum height by derricks.

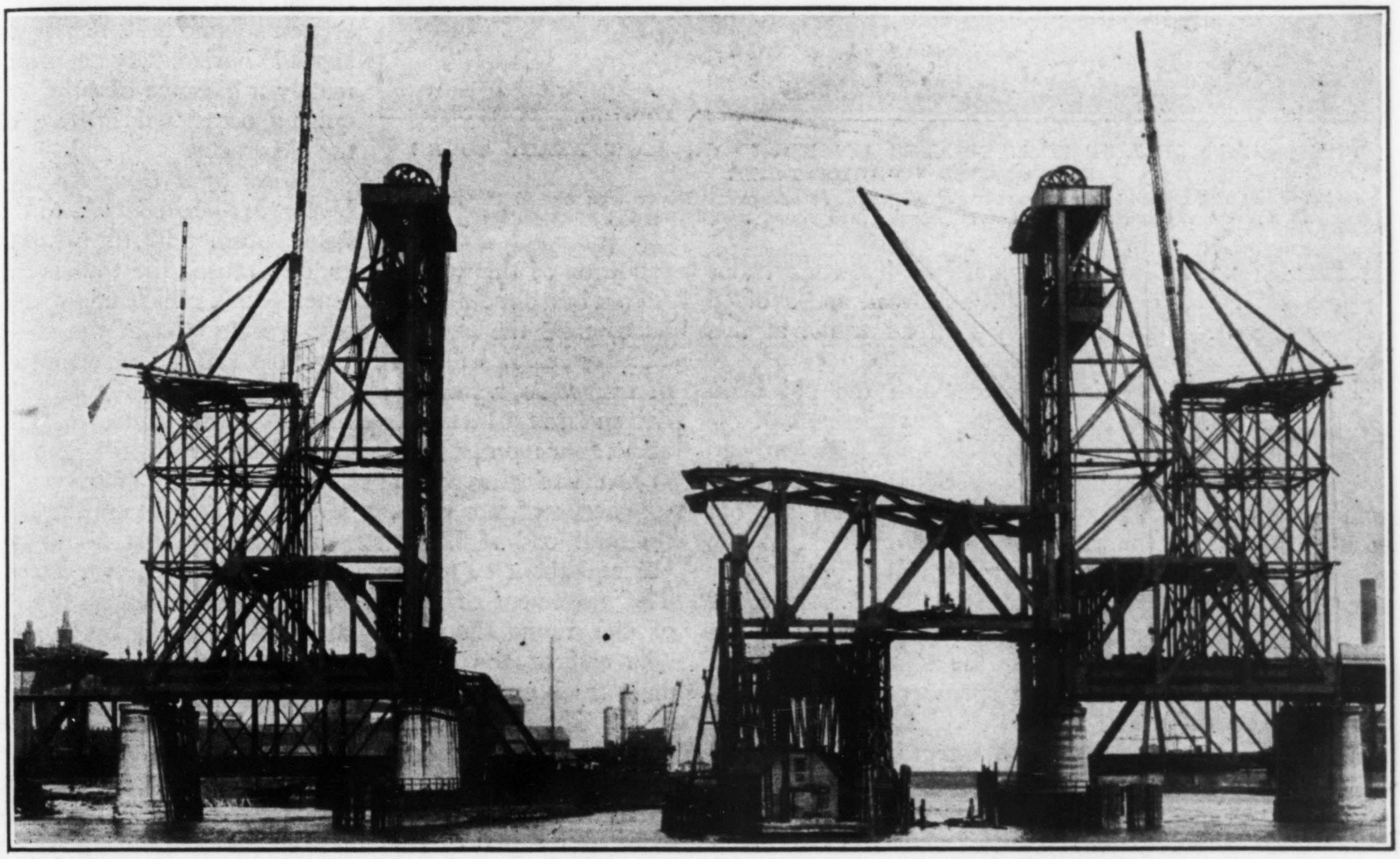

FIG. 6—ERECTION OF LIFT SPAN

Easterly two-thirds of span erected in place, above closed position to clear opening of old swing span. This part of span erected as a cantilever, resting on temporary timber pier and tied back to tower. Remaining one-third of span fabricated on a scow, floated to position and hoisted into place.

Approach Embankment Grading—The total grading in embankment required for the approaches was approximately 700,000 cu.yd., of which 75,000 cu.yd. was placed while traffic was being changed to the new line, as the new fills could not be completed without interference with the old main track. Approximately 475,000 cu.yd. of the fill was placed by automobile trucks from miscellaneous excavations within a radius of 10 miles. The railroad had previously dumped about 150,000 cu.yd. of cinders into the meadows over part of the new line area west of the river as part of the approach embankment. The yardage placed by trucks was at a rate of 50,000 cu.yd. per month, or an average of 2,000 cu.yd. per working day. This was as much material as could

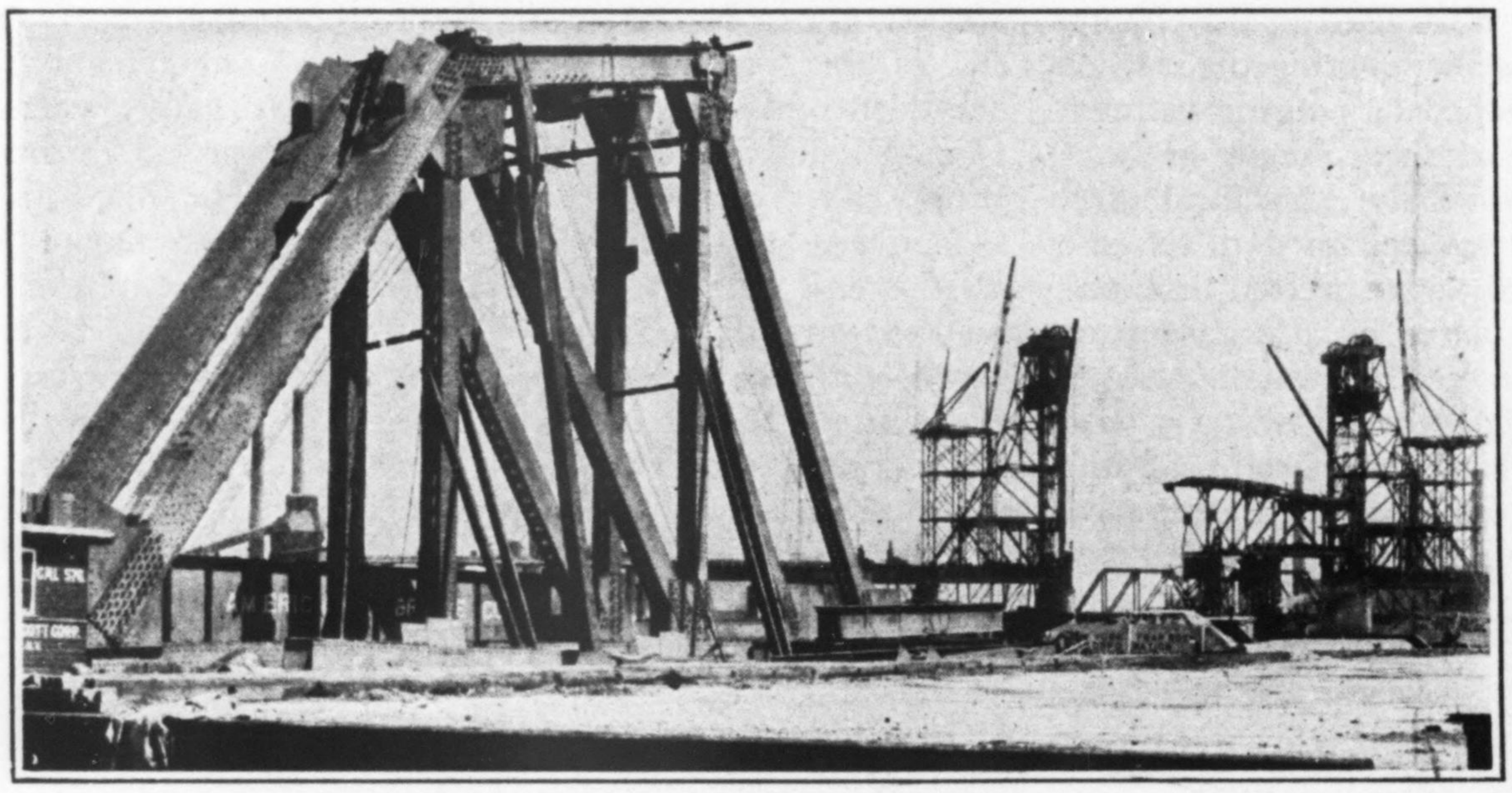

FIG. 7—PART OF LIFT SPAN TRUSSES ASSEMBLED ON SCOW READY TO BE FLOATED TO BRIDGE SITE

Background shows bridge ready to receive this part of span. River was closed to traffic 72 hours while trusses shown were hoisted into place, completing lift span trusses.

be handled under favorable conditions by work trains from a borrowpit located along the railroad and avoided the interference with the heavy railroad traffic at this location.

On account of the interference with the old main tracks, the new embankments at the extreme ends of the improvement were at first completed only sufficiently to accommodate one new track. Besides the additional filling required at the connecting points with the old main line, before the traffic could be changed to the second and third tracks, the approach embankments over practically their entire length were widened an average of 10 ft. This additional filling was brought in on dump-car trains and placed from the adjacent main track and plowed to the outside by a spreader car attached to the dump-car train.

Bearing Capacity of the Soft Meadow Mud—A study of the amount of subsidence and movement of the old and new fills placed in the soft mud of the meadows is interesting. Investigations revealed that the old main-line fill, west of the river, placed more than 40 years ago, had at no point subsided more than 15 ft. below the meadow level, although the soft mud reaches a depth of more than 30 ft. over the area investigated. The high bearing value under train loads without any further subsidence of this mattress of subsided material which has completely settled and stabilized, depending on the soft meadow mud for support, is surprising.

One side of the new track embankment rests on the old main-line fill, while the other side has nothing but the soft mud for support. On account of this uneven bearing, a rolling or tilting action of the entire mass of new filling as a unit occurred, which amounted to a sidewise movement at the top of the fill as much as 2 ft., varying with the depth of fill, which reached a maximum height of 42 ft. above the meadow level. The new fill has subsided about 10 ft. into the meadow at the outer edge of the fill, this depth decreasing toward the old embankment. The placing of the new embankment against and over the slope of the old has affected the old fill along a length of about 2,000 ft. where the new embankment is heaviest. No substantial change in grade of the old embankment along this section was noticed, but it was shoved horizontally approximately a distance of 6 in., requiring occasional shifting of the old tracks.

Failure of a Concrete Box Underpass—In contrast to the stable subsided fill, the unstable condition of the unfilled meadows caused the foundation failure of a double opening concrete box vehicle underpass, built across the new line about $\frac{1}{4}$ mile west of the river. This box, 32 ft. wide, 24 ft. high and 100 ft. long exclusive of wingwalls, rested on about 400 timber piles 75 ft. long driven through 30 ft. of soft mud. The structure was built about four months before any embankment was placed on or near it, during which period no settlement of the structure occurred, nor was any settlement noticed until nearly the total load of filling, extending about 20 ft. above the box and about 40 ft. above the meadow level, was placed. The movement of the structure that resulted in failure of the foundations was both vertical and horizontal. The end of the superstructure farthest from the old main track embankment settled a total of approximately $7\frac{1}{2}$ ft. The end adjacent to the old main track, at which point the piling had been driven through part of the old fill, raised about 3 in. Of the total settlement of $7\frac{1}{2}$ ft. at the one end, $1\frac{1}{2}$ ft. occurred as a gradual settlement over a period of about four months, during which the structure was carrying somewhat less than the total load. Shortly after the full load of the embankment

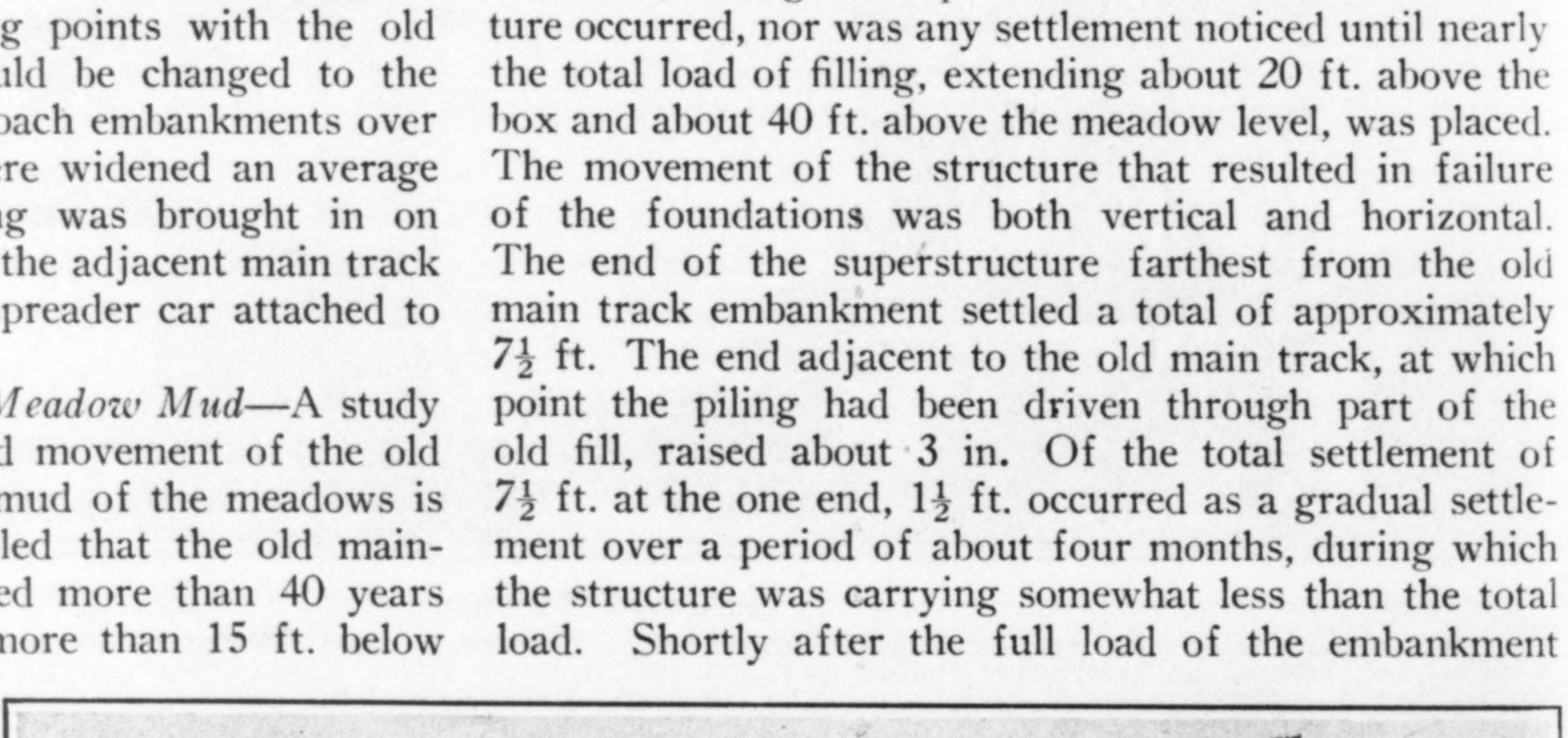

FIG. 8—WEST APPROACH VIADUCT AND GIRDER SPANS

Concrete columns have been painted with emulsified asphalt.

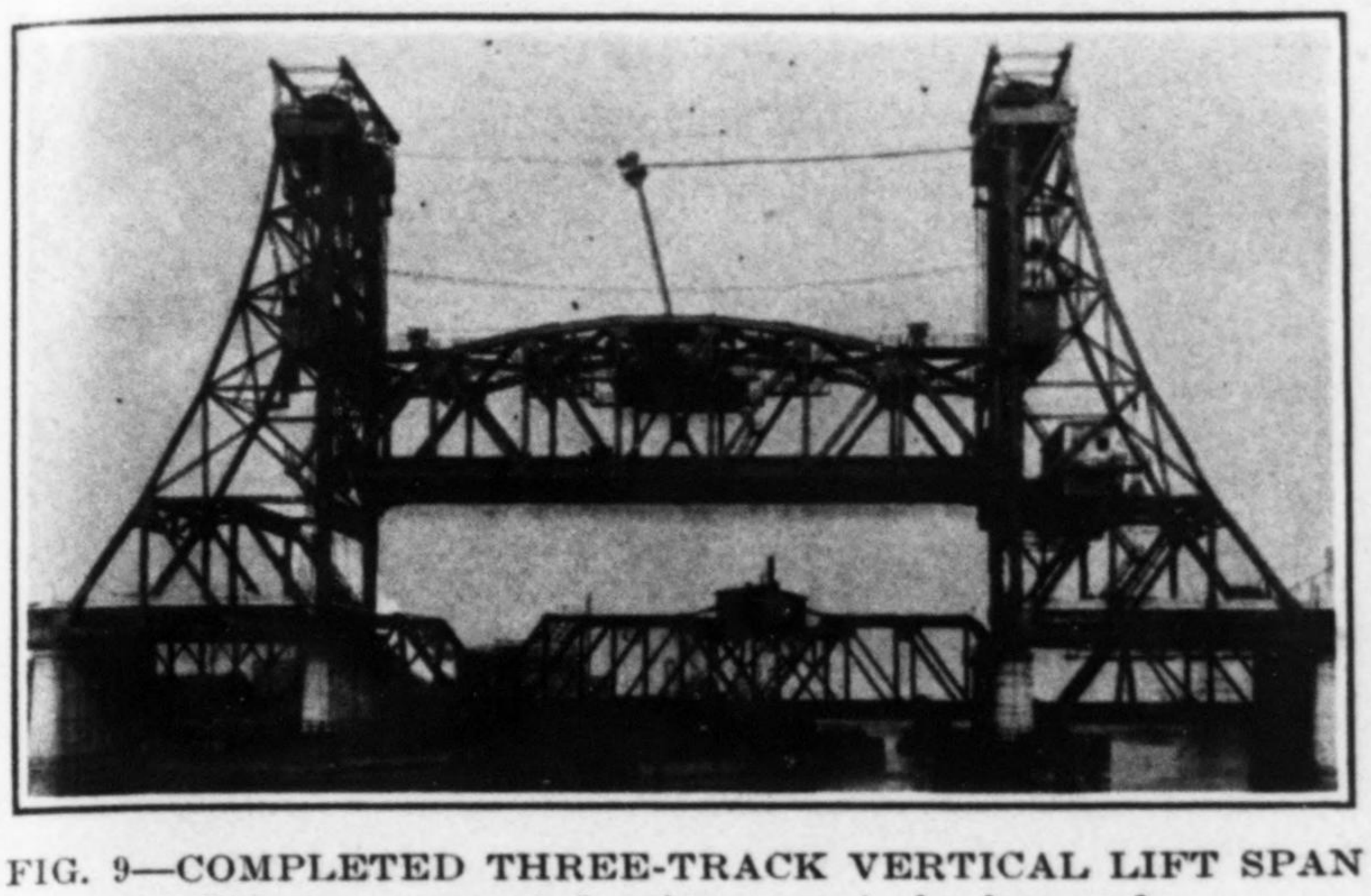

FIG. 9—COMPLETED THREE-TRACK VERTICAL LIFT SPAN
Old steam-operated swing span in background.

was placed, the foundations of the structure let go suddenly, and in three days the end had dropped approximately 4 ft., after which the rest of the settlement, amounting to about 2 ft., was gradual and extended over a period of four months. After this latter movement, the structure became completely stabilized and showed no further movement under train load. The structure was abandoned and a duplicate underpass was built about 80 ft. west of the original one. The entire concrete box during the period of movement held its shape perfectly, showing only minor cracks, which is surprising considering the abnormal stresses that the structure must have withstood. The entire structure above the horizontal construction joint in the bottom slab, which was waterproofed, moved directly endwise about 3 ft. away from the old embankment. This end movement was caused by the sidewise pull of the embankment as it subsided into the meadows. The piles, with that portion of the bottom slab below the construction joint, moved approximately 1 ft. in the same direction as the superstructure of the box. The superstructure moved 2 ft. more than the piles, sliding on the waterproofed joint. The greater portion of the sidewise movement of the piles occurred in the three-day period of rapid settlement. The absence of a subsided bed of filling material in the 30 ft. of mud permitted the movement of the piles sidewise, destroying most of their bearing value.

The second subway was built under the same conditions as the first, except that the site had been loaded by the complete embankment, which had subsided into the meadow. This fill had been in position for six months, and all settling and sidewise movement had ceased. The embankments were then excavated to the meadow level, 75-ft. piling driven and cut off at the surface. The effect on the pile penetration of the bed of subsided material was very noticeable. The piles drove very hard for the second structure, whereas no trouble has been experienced in driving the piling for the first structure. The new structure has been fully loaded and shows no settlement or other movement under train loads.

Track Construction—The railroad's most improved type of track construction was used on the new bridge and approaches, consisting of 130-lb. rail with 20-lb. tie-plates and 73-lb. tie joint plates on creosoted ties. Excepting for the lift span, on which standard timber bridge ties rest directly on the steel, the entire bridge structure will carry the standard ballasted track construction on solid reinforced-concrete floors. The use of No. 20 switches provides for high-speed train operation.

On the fills the tracks are ballasted for the present with cinders and will be raised to final grade elevation with permanent stone ballast after several months' operation has permitted any settlement of the fill to take place. To prevent scouring of the earth embankment slopes a cinder covering was used. This also added to the appearance of the slopes without resorting to expensive hand trimming. The cinders were brought in on dump cars and pushed over the edge of the bank by a spreader, more than 40,000 cu.yd. being used.

Construction of the project was begun in March, 1927, and was completed in October, 1928, requiring a total of about twenty months. The approximate time required to execute the important parts of the work is as follows: Construction of river piers, six months; erection of lift bridge steel, ten months; construction of approach viaducts and approach deck spans, twelve months; embankment grading, eleven months; track construction, four months.

Engineers and Contractors—The design and construction of the project were under the direct supervision of G. J. Ray, chief engineer of the railroad. Directly connected with the work for the railroad were H. M. Warren, electrical engineer; A. J. Neafie, principal assistant engineer; H. M. Doughty, division engineer; J. L. Vogel, bridge engineer, M. Hirschthal, concrete engineer; J. E. Saunders, signal engineer, and W. H. Speirs, resident engineer. Waddell & Hardesty were consulting engineers on the lift bridge. The contractors were as follows: American Bridge Company, structural steel; the Foundation Company, river piers; H. F. Curtis, approach viaducts and grading; Raymond Concrete Pile Company, concrete piles. The railroad company did all track and signal work, erected the approach span girders and took care of all temporary work. The dismantling of the old line, including the old bridge, is being done by the railroad company.

Rotating Rake on Finishing Machine Spreads Rock Asphalt

Rock Asphalt Laid Cold, Broken Down and Spread Evenly by Finishing Machine Used on Concrete Base

By John L. Hubbard
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A STANDARD concrete-finishing machine, with an added raking device, is being used in constructing a 9-mile pavement of rock asphalt on a concrete base in Cheatham County, Tennessee. Similar machines have been used to spread hot-mix asphalt, but this is the first time one has been used to spread natural rock asphalt laid cold. The asphalt is being laid on a 7-5-7-in. (parabolic curve) concrete base, with a 1¼x6-in. curb on each edge. The base, too, is finished smooth and a paint coat is applied ahead of the asphalt.

The rock asphalt is dumped in piles on the base ahead of the finishing machine as shown by Fig. 1. Men with shovels then roughly spread the piles, throwing the material back in front of the rake as shown by Fig. 2. It then contains many lumps, and the purpose of the rake is to break these down, even up the asphalt layer and provide a fine, homogeneous material ahead of the finishing machine screed. Except for the added raking