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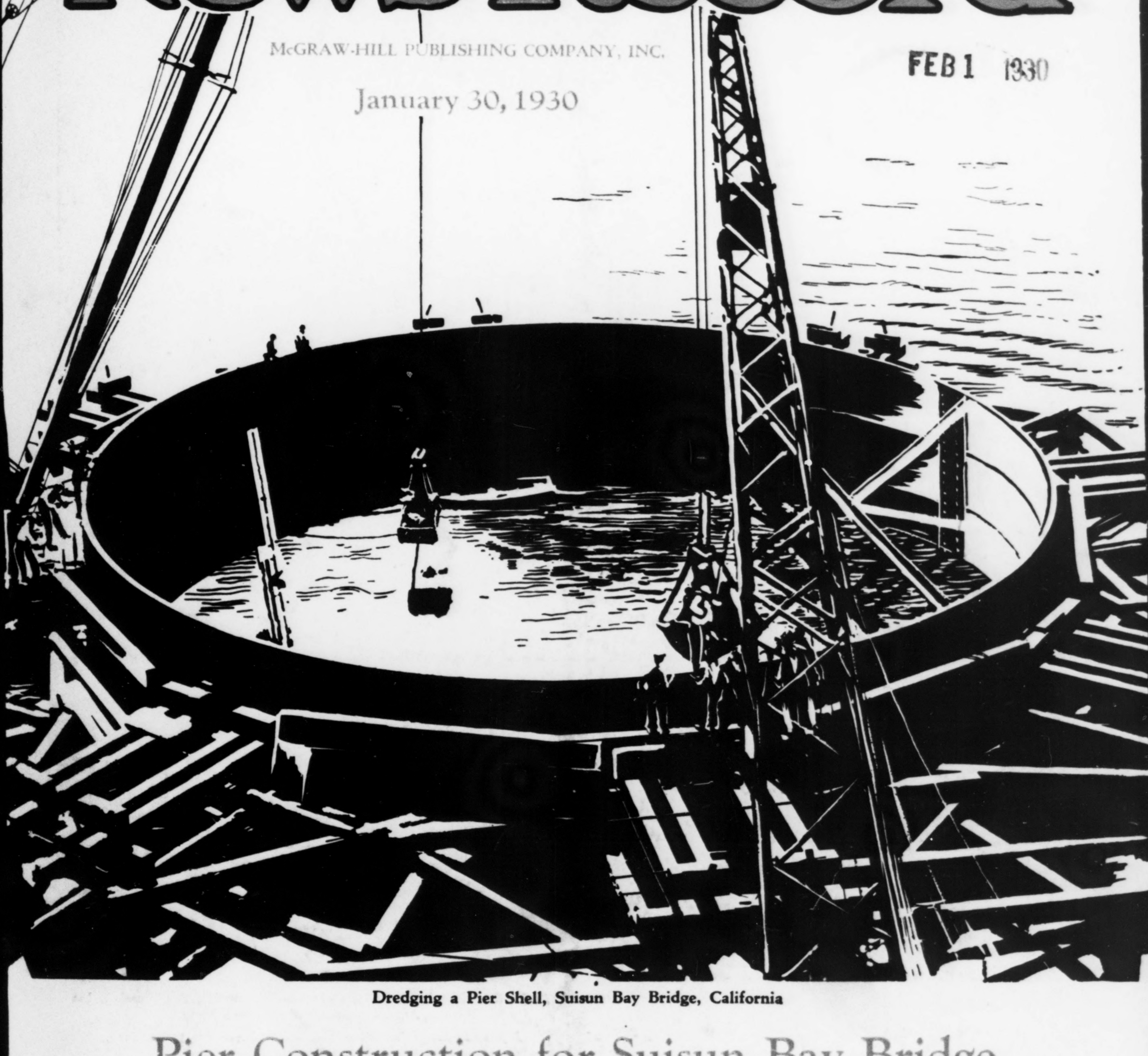


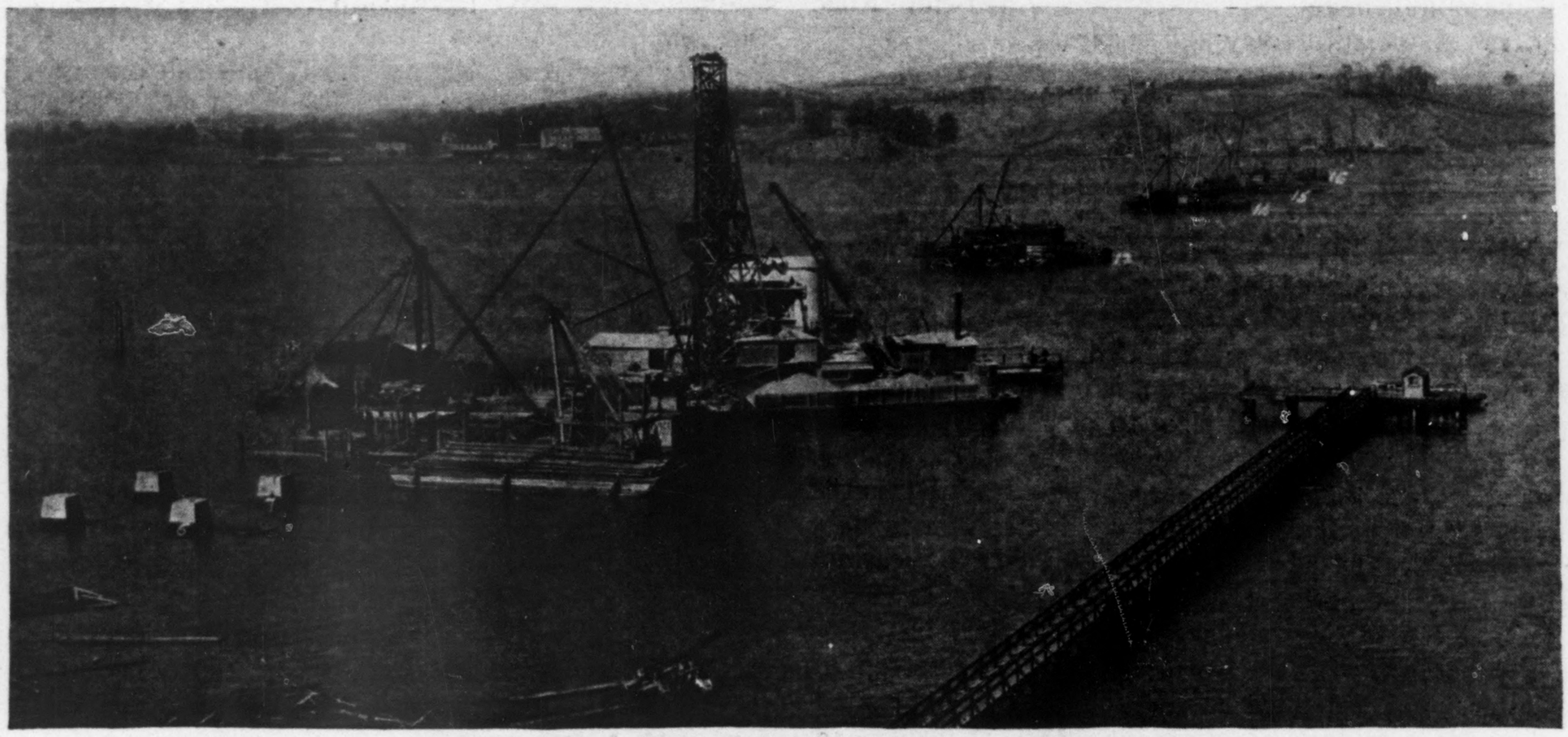
Dredging a Pier Shell, Suisun Bay Bridge, California

Pier Construction for Suisun Bay Bridge
Improvements Effect Economies on C&O Ry.
Steel Frames for Houses
Regional Planning for Los Angeles County
Depreciation of Railroad Property

Pier Construction for the S.P. Railroad Bridge Across Suisun Bay

Open Caissons Used in Depths of 143 Ft. Below Low Water—Artificial Sand Islands Within Steel Shells 81 Ft. in Diameter to Be Employed in Sinking Caissons for Eight Piers

By C. R. Harding
Assistant to the President, Southern Pacific Company



PIER CONSTRUCTION VIEWED FROM SOUTH SHORE
Shaft of pier 11, completed, in the foreground

CONSTRUCTION is well advanced on the substructure for the $12,000,000 double-track bridge which is being built across Suisun Bay in California to replace the car ferry that has carried Southern Pacific trains across that body of water since 1879. Bedrock depth of 143 ft. below low water made it necessary to program pier construction on this project with particular care. For the deeper piers a scheme that has come to be known as the sand island method was devised and thus far has proved wholly satisfactory in application. Open cofferdam construction to a depth of 58 ft. below high water also is being used. Descriptions of both methods are included in the following.

In determining the location and design of the bridge, consideration was given to the proximity of earthquake faults and the probable intensity of shock that may be expected therefrom. The findings of noted geological and seismological authorities engaged in this study revealed the presence of two faults not far from the ferry crossing, in whose general location the bridge was needed.

One of these, known as the Southampton fault, lies about a mile west of Martinez and extends northwesterly along the axis of Southampton Bay. As this is an important fault with large displacement, it was found extremely undesirable to locate the bridge across it.

The other, or Martinez fault, passes through the town of Martinez, but cannot be identified on the opposite shore. Its projected course under the water extends in a northeasterly direction toward Goodyear and, if existent, passes Army Point about ¼ mile from the shore line. Whatever displacement has occurred has been small and it was felt that the site selected for the bridge, across the lower end of Suisun Bay from Suisun Point to Army Point, was a safe one.

In order to determine the depth and character of underlying rock and to preclude the possibility of locating piers on a fault line, a series of borings was made across the proposed bridge site. These borings showed a maximum water depth of 55 ft., mud to a depth of about 90 ft. below low water, and rock at a maximum depth of 143 ft., averaging 116 ft. below low water. The material between mud and rock, in descending order, was sand or clay and firm gravel. Decision to carry the piers down to rock was prompted by the great depth of mud and the necessity of providing for earthquake effects.

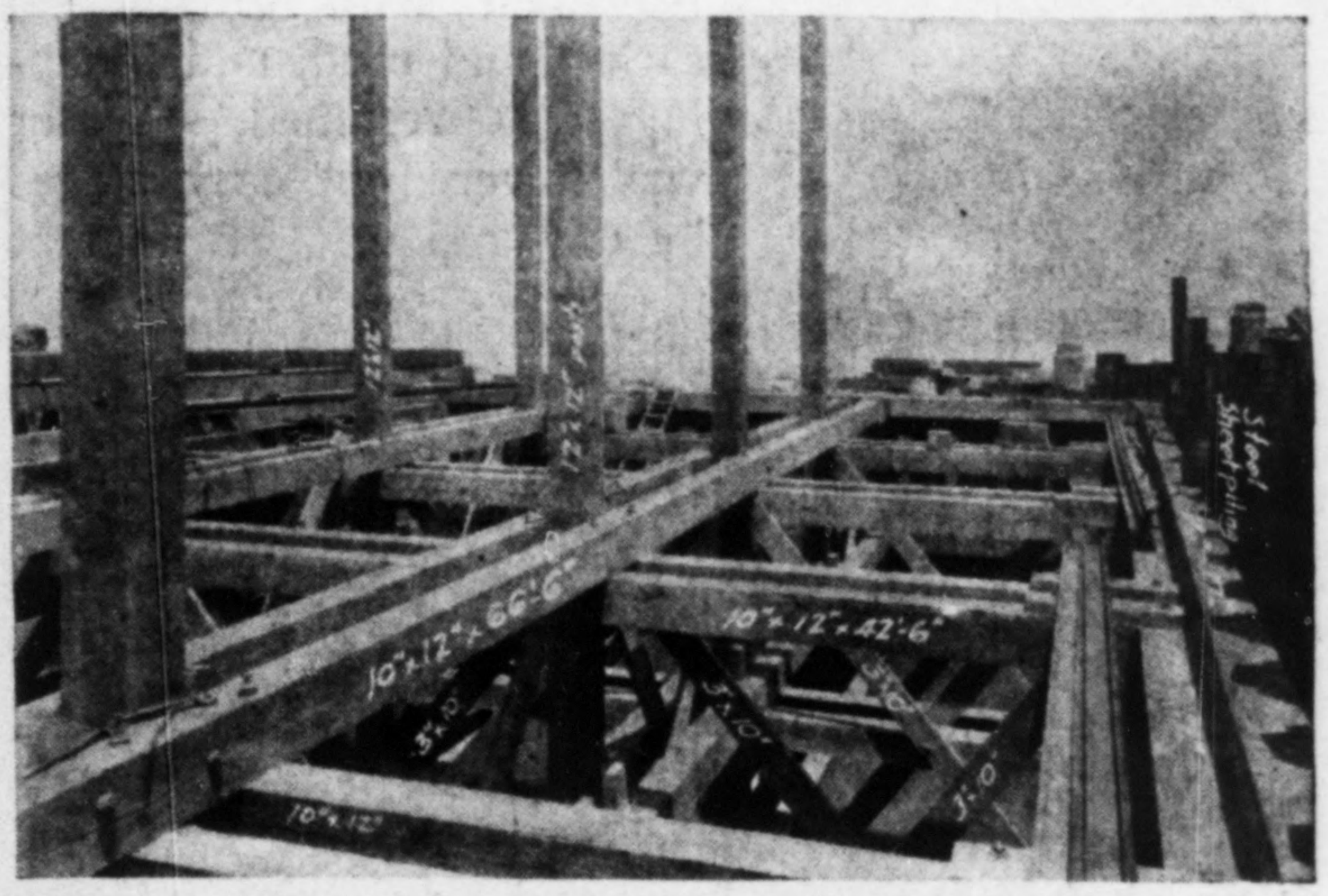
The superstructure of the bridge will consist of ten steel trusses with approach viaducts on either end, giving a total length of 5,603.5 ft. between abutments. Clearance above mean higher high water will be 70 ft. for the fixed spans and 135 ft. under the lift span when raised.

In addition to abutments and a total of 22 pedestal piers under approach viaducts, the substructure includes two piers (10 and 11) constructed in open cofferdam and eight piers (12 to 19, inclusive) constructed by a combination of the open dredging and open cofferdam methods.

A total of about 105,000 cu.yd. of concrete and 1,500 tons of reinforcing steel will be used in the piers. Pier

13, the deepest, will be approximately 214 ft. high from bedrock to bridge seat and will contain about 13,500 cu.yd. of concrete and 175 tons of reinforcing steel.

The bases of piers 10 to 19, inclusive, are to be 38x60 ft. in plan, except that the lift span piers (12 and 13) will be 40 ft. instead of 38 ft. wide. The sides will rise vertically to El. —20, at which height a shaft of smaller sections will begin. The abrupt change in section at the beginning of the pier shaft was unavoidable in view of the method employed in sinking the deeper pier bases, it being essential that the dredging wells have vertical faces so that free passage would be provided for the dredging buckets while removing material from beneath the piers.



BRACING AND STRUTTING IN COFFERDAM OF PIER 11
This bracing was framed and sunk within the cofferdam after closure of the latter.

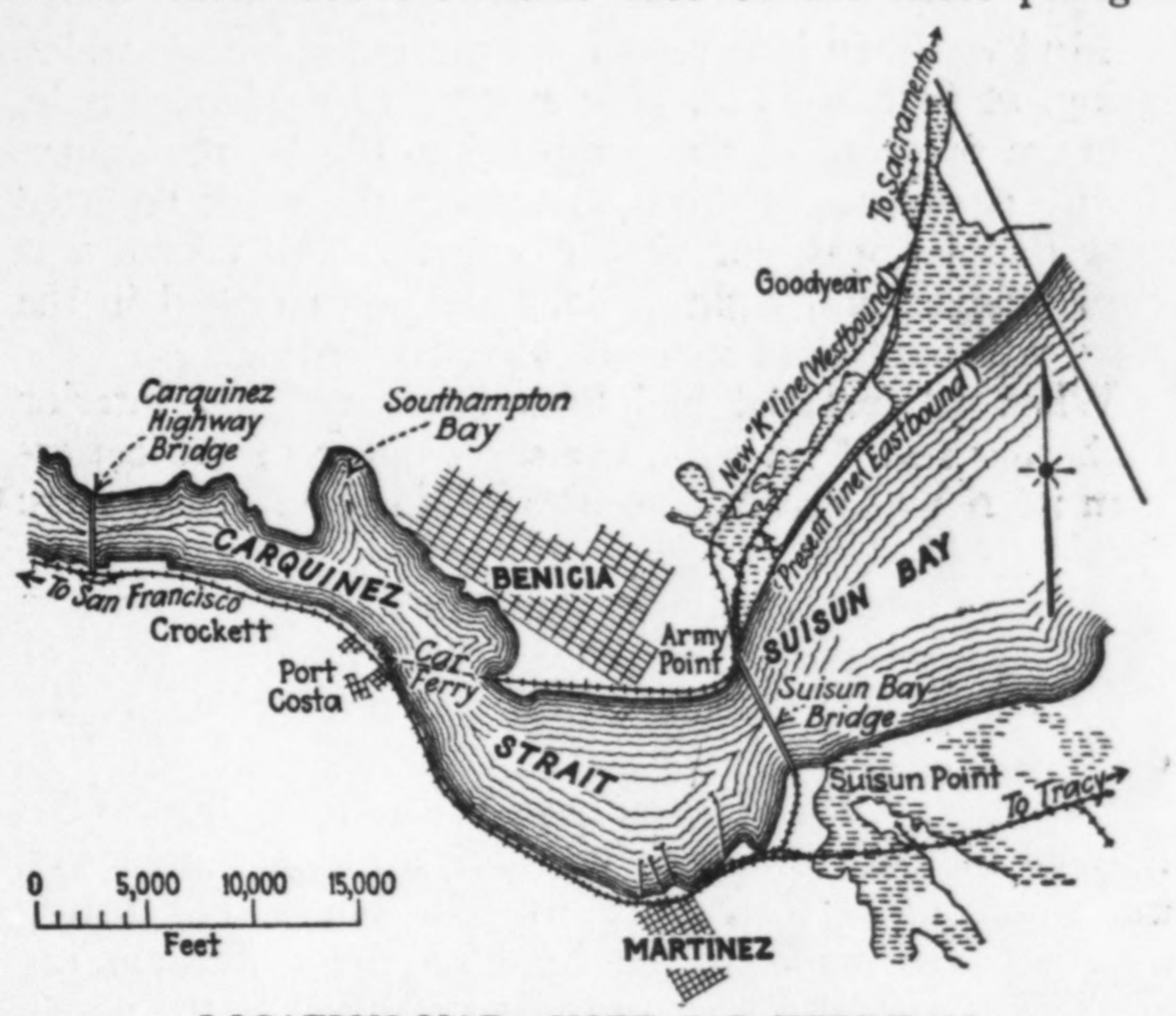
Earthquake history in the vicinity indicates that it is reasonable to expect shocks during the probable life of the bridge, although of course there are no definite data as to what forces such shocks would exert on the bridge. Ordinarily, pier bases of the size used for the Suisun Bay bridge would require no steel reinforcement, but in order to provide resistance against earthquakes it was decided to use not less than 30 lb. of reinforcing steel per cubic yard of concrete in the pier bases. Shafts likewise will be heavily reinforced.

Vertical and horizontal reinforcement is being placed around the edges of bases, dredging wells (piers 12 to 19) and shafts of the piers. At El. —20, where the abrupt change in section occurs, the steel will be extended from the pier base into the shaft above. Reinforcement consists of $\frac{3}{4}$-in. and 1-in. square and round deformed bars embedded 8 in. at edge of pier bases and dredging wells and 24 in. at edge of pier shafts.

Cofferdam for Pier 11—In the construction of pier 11 only a single wall of steel sheet piling was used in a rectangular cofferdam 58 ft. deep and 46x70 ft. in plan. First, falsework piling outlining the cofferdam was driven to rock in the form of a rectangle around the pier site, using second-hand Douglas fir timber that had been painted with a preservative compound. The vertical piles were 65 ft. long and were spaced about $8\frac{1}{2}$ ft. on centers across the current and $14\frac{1}{2}$ ft. on centers in the direction of flow. Brace piles 70 ft. long were driven to a 4 in 12 batter against the vertical piles, using two braces at corner piles and one brace at intermediate piles. The falsework was completed by bolting two sets of 12x12-in. wales to the piles, the first set being placed near the tops and the second set as far down as low tide permitted.

Deep-arch section steel sheet piling 65 ft. long was then driven around the inside waling timbers. Before closure of the cofferdam rectangle could be effected it was found necessary to use a wedge-shaped pile, due to the fact that the first piles placed had canted somewhat at the top and it was not possible to bring them to a vertical position. Two piles were cut, therefore, and their webs riveted together so as to form a wedge pile wider at the bottom than at the top. This wedge pile was placed in position and the remaining vertical closure pile driven, after which all sheet piling was driven to rock. The steel sheet piling was then bolted back to the outside walings and open excavation was made, almost to rock, by dredging through the water with clamshell buckets.

The next step, preparatory to unwatering, was the construction of the interior bracing wales, for which 12x24-in. timbers were used in combination with correspondingly heavy struts, posts and bracing, all timbers being well bolted together in eight tiers varying in height from 5 ft. at the bottom to 12 ft. at the top of the cofferdam. In order to facilitate forcing the wales down to position, a slight clearance was left between their outside faces and the inside face of the sheet piling.



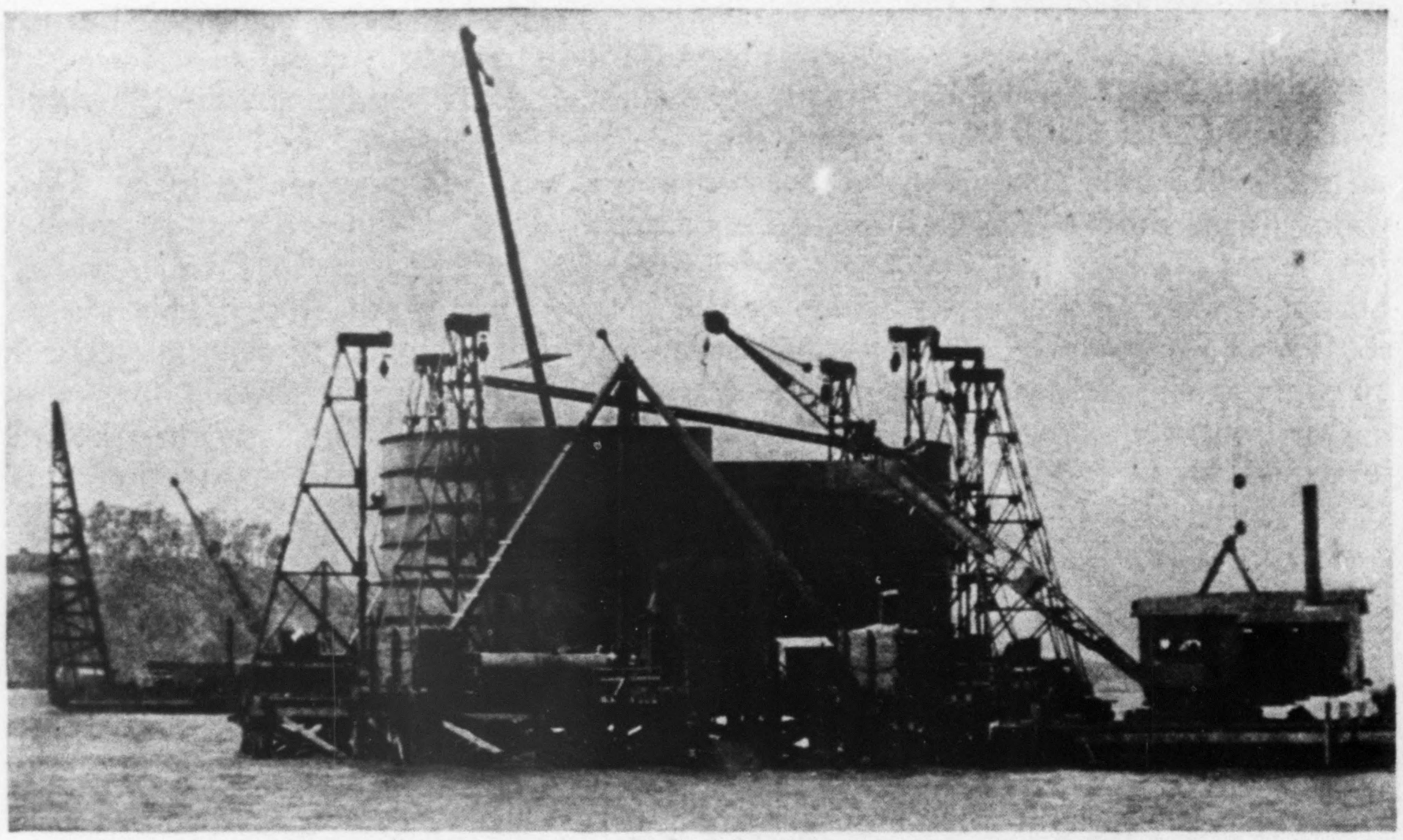
LOCATION MAP. NOTE CAR FERRY IN CARQUINEZ STRAIT

Wooden wedges were driven into this space after the wales had been put down to the prescribed level.

Struts were used in pairs, two pairs per set of wales being placed parallel with the long axis of the cofferdam and four pairs per set of wales parallel with the short axis. All struts were continuous, the longer ones being $66\frac{1}{2}$ ft. in length and laid directly on the shorter struts, which were $42\frac{1}{2}$ ft. long. Steel bearing plates, $\frac{1}{2}$ in. thick, were used between the ends of the struts and the wales.

Vertical posts were placed at the intersections of the two systems of struts and acted as spacers between each pair. It was decided to make the posts continuous only above the third tier in order that the lower waling sets and struts might be jacked down if necessary to take care of any great differences that might be found in elevation of rock at corners of cofferdam. It developed later, however, that the jacking was not required.

The timber cage formed by the interior bracing floated of its own buoyancy and no effort was made to sink it until all framing was completed. When ready to sink, 268 tons of second-hand rails were loaded on the top set of struts.



ERECTION OF STEEL SHELL TO INCLOSE SAND ISLAND
Towers aided in the erection of the steel, which was put together in assembled section 10 ft. wide.

that point than at the top, resulting in cracking of the interlocks when closure was made.

To correct this leak, the sheet piling on the south and east sides of the cofferdam was driven deeper into rock, the average increase of penetration being $1\frac{1}{2}$ ft. The outside of the cofferdam at the point of the break was backfilled with gravel (in sacks) and mud, and pumping was resumed. Water was lowered in easy stages by day and held constant at night, two of the 8-in. centrifugal pumps being placed at the second and third wales to take care of the lower water. When water level reached El. —20 use of the 8-in. pumps was discontinued and four 5-in. pulsometer pumps were installed, leakage being easily handled by one of these pumps. At the completion of the unwatering process, mud remaining in the bottom of the cofferdam was removed by dredging and by sluicing into the pumps.

The rock surface, when unwatered, was found to include shale, sandy shale and sandstone, the sandy shale predominating and being interspersed by ridges of sandstone which formed decided steps in the foundation. It was necessary to step the shale at several places in order to break up slope planes. The rock sloped from El. —46.4 at the southwest corner to El. —50.7 at the northeast corner. The rock formation was, in general, of excellent character, affording all necessary foundation requirements.

Pouring Concrete in Pier 11—Before pouring of concrete commenced the remaining 168 tons of scrap rail was unloaded. The foundation was carefully cleaned, and after footing forms were erected against the inside of the cofferdam wales, the surface of the rock was covered with 2 in. of grout.

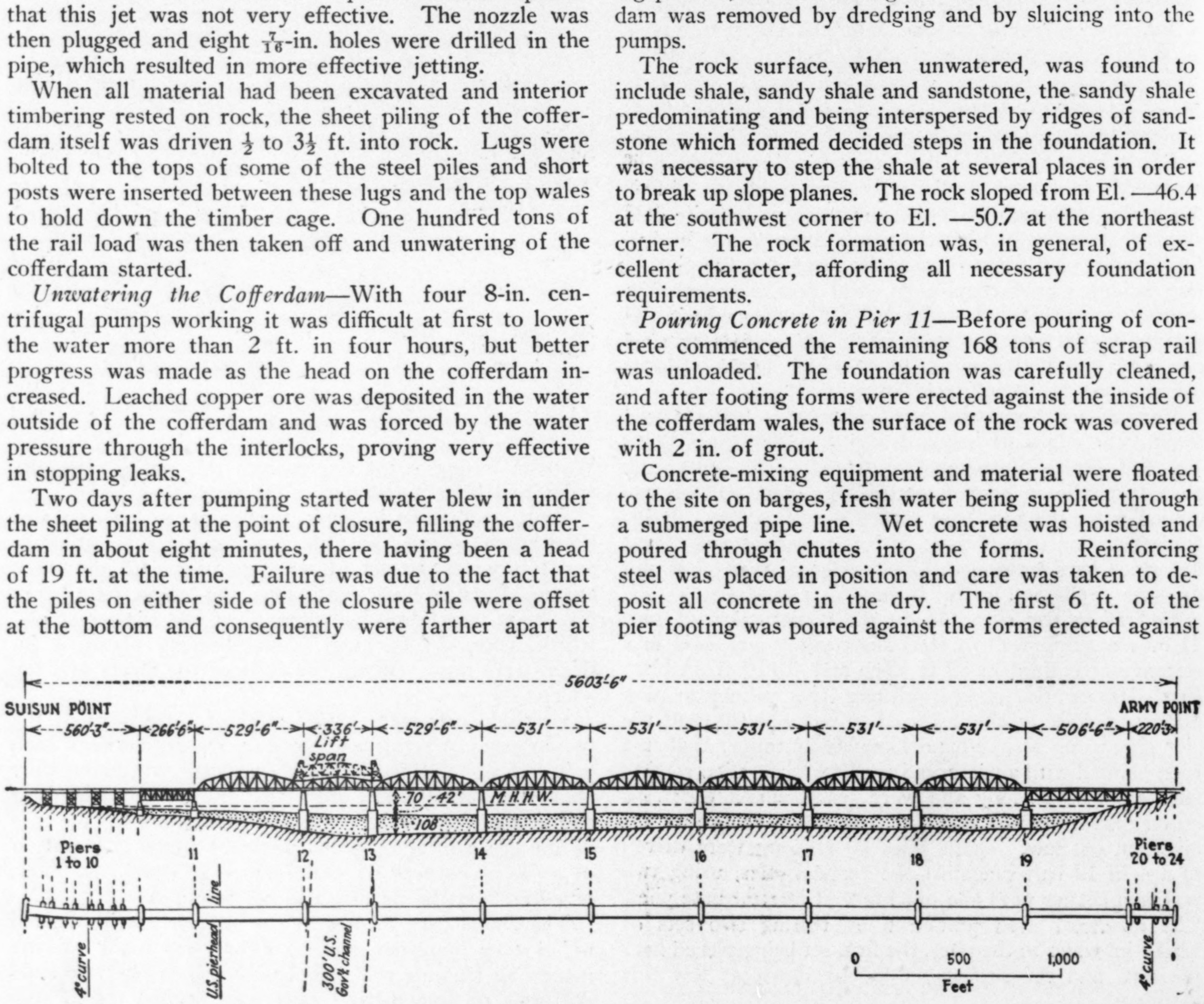
Concrete-mixing equipment and material were floated to the site on barges, fresh water being supplied through a submerged pipe line. Wet concrete was hoisted and poured through chutes into the forms. Reinforcing steel was placed in position and care was taken to deposit all concrete in the dry. The first 6 ft. of the pier footing was poured against the forms erected against

Mud was then jetted out from under the bottom wales, using, at first, a $2\frac{1}{2}$-in. pipe tapered to a $1\frac{1}{2}$-in. nozzle, average pressure at the pump being 125 lb. per square inch. A diver sent down to inspect the work reported that this jet was not very effective. The nozzle was then plugged and eight $\frac{7}{16}$-in. holes were drilled in the pipe, which resulted in more effective jetting.

When all material had been excavated and interior timbering rested on rock, the sheet piling of the cofferdam itself was driven $\frac{1}{2}$ to $3\frac{1}{2}$ ft. into rock. Lugs were bolted to the tops of some of the steel piles and short posts were inserted between these lugs and the top wales to hold down the timber cage. One hundred tons of the rail load was then taken off and unwatering of the cofferdam started.

Unwatering the Cofferdam—With four 8-in. centrifugal pumps working it was difficult at first to lower the water more than 2 ft. in four hours, but better progress was made as the head on the cofferdam increased. Leached copper ore was deposited in the water outside of the cofferdam and was forced by the water pressure through the interlocks, proving very effective in stopping leaks.

Two days after pumping started water blew in under the sheet piling at the point of closure, filling the cofferdam in about eight minutes, there having been a head of 19 ft. at the time. Failure was due to the fact that the piles on either side of the closure pile were offset at the bottom and consequently were farther apart at



ELEVATION AND PLAN OF BRIDGE

the waling, but above that height the base was stepped back to the prescribed dimensions of 38x60 ft.

The four lower sets of struts were left in the completed footing, as they were well below the mud line and would have been difficult to remove. All timber above was taken out as the concrete was poured, struts being boxed out to form large keys at each construction joint. Wales were braced to the concrete before struts were removed. All concrete surfaces were cleaned and washed and construction joints covered with 2 in. of grout before pouring the next lift.

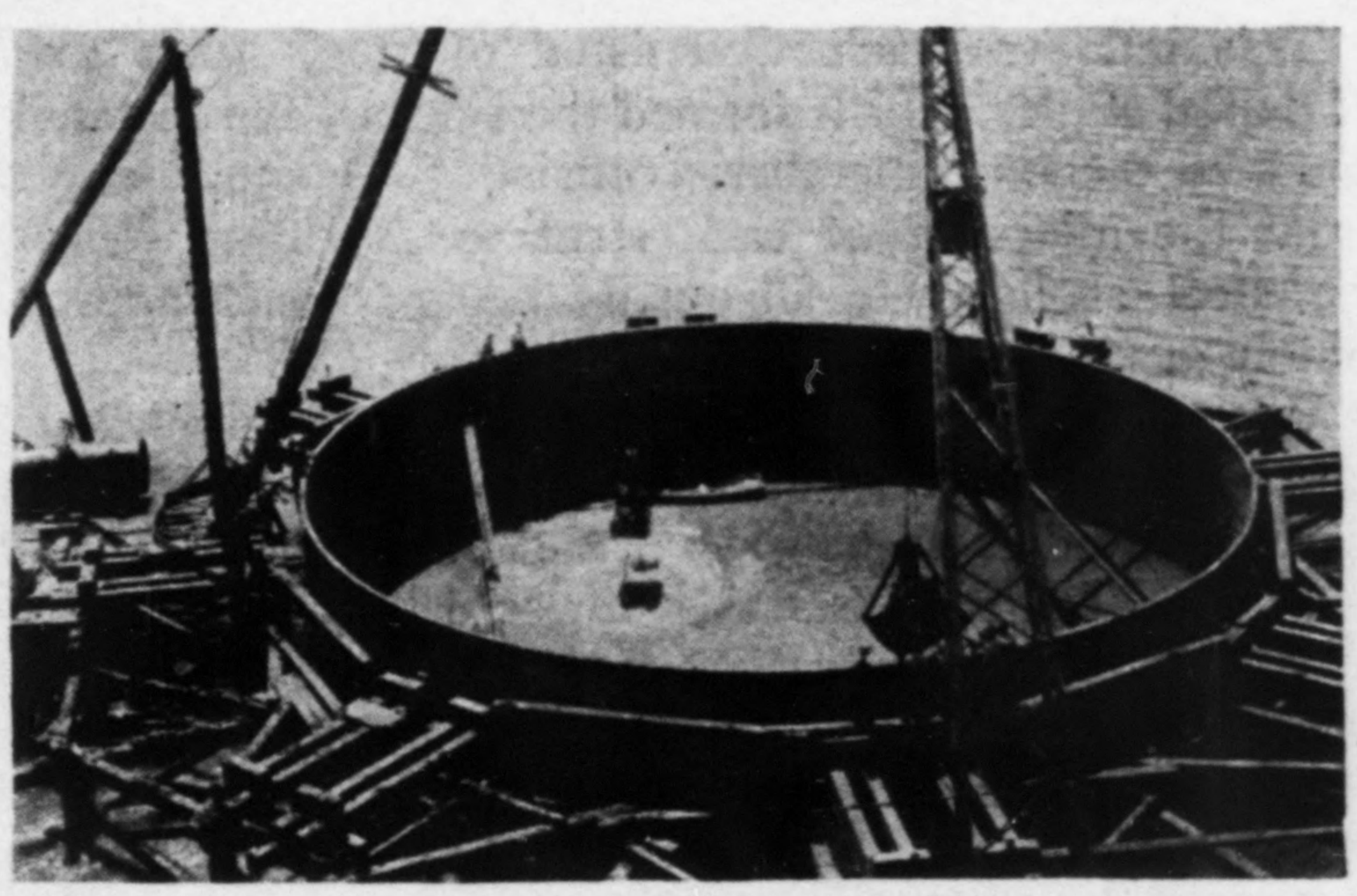
At El. —20 construction of the pier shaft was started, using steel panel forms on the north and south sides of the shaft and wooden forms on the semicircular ends. The shaft was battered 1 in 24 on the sides and 1 in 48 at the ends.

Between El. —8.0 and El. +8.0 the pier shaft was given two coats of waterproofing, after which the gate pile was pulled and the cofferdam filled with water. Pulling of sheet piling began immediately and was accomplished without difficulty, although most of the piles showed a decided bend outward in the lower 10 ft. This deformation was due to the fact that when driven the sheet piling was not exactly vertical but had an outward inclination from top to bottom. When the cofferdam was unwatered, hydrostatic pressure on the outside forced the lower portions inward. As the piles were embedded in rock, they could not be forced in at the toe, resulting in the bending described. The average penetration in rock was found to be approximately $2\frac{1}{2}$ ft. This piling was used again in the construction of pier 10.

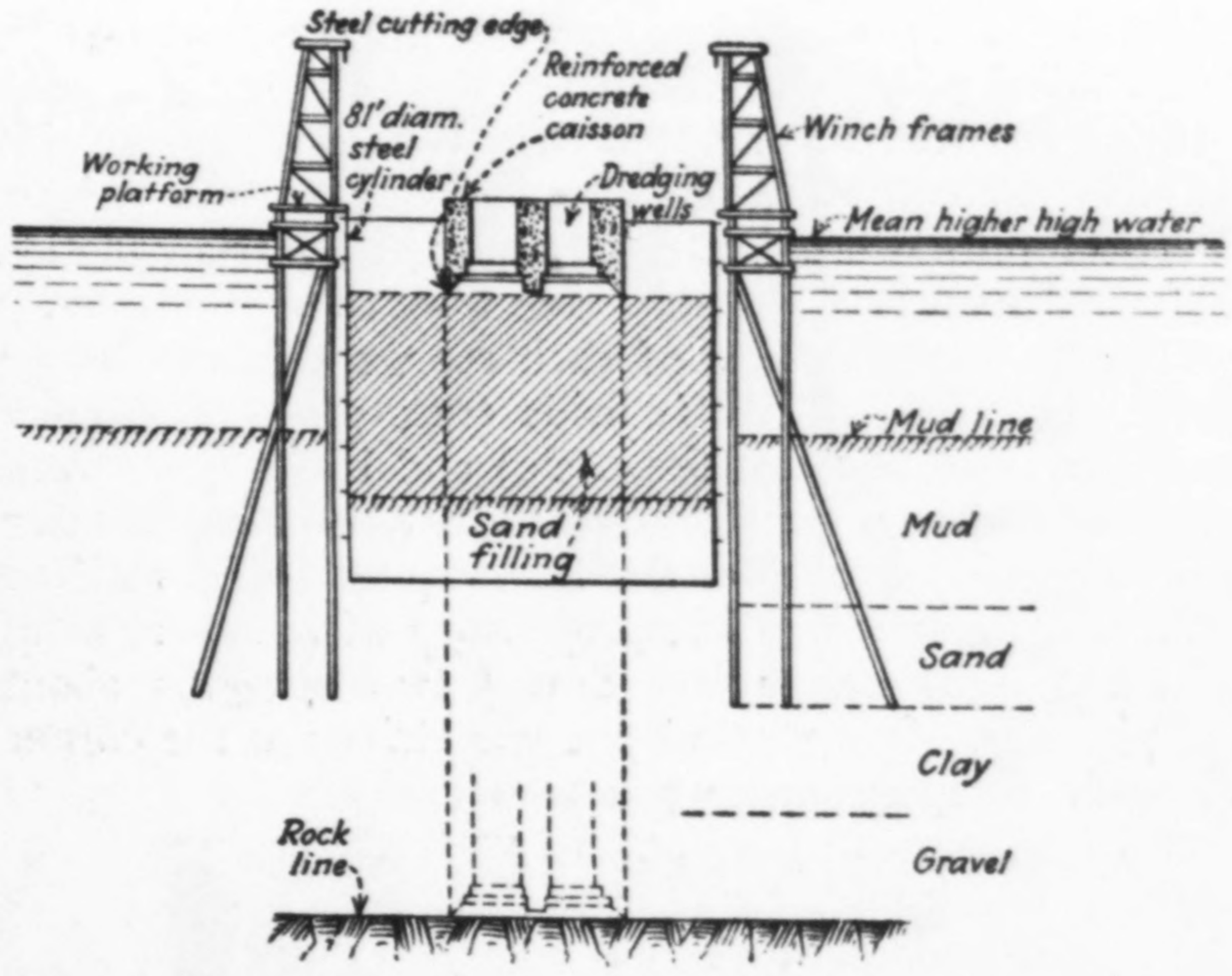
Pouring of the coping and setting of the anchor bolts completed the construction of the pier. Location of the anchor bolts was checked by triangulation, a comprehensive system having been established as a preliminary step in the work. After stripping the forms, the face of the shaft was brushed over with neat cement.

Construction of pier 11 required 4,111 cu.yd. of concrete made up of 5,582 bbl. of cement, 1,951 cu.yd. of sand and 3,426 cu.yd. of rock. Reinforcing steel weighed 43 tons, averaging about 21 lb. per cubic yard of concrete.

The first 6 ft. of footing was poured with 1 : 2.4 : 4.6 concrete, using $5\frac{1}{2}$ sacks of cement per cubic yard of concrete and $6\frac{1}{2}$ gal. of water per sack of cement. Above this and up to El. —4 a 1 : 2.6 : 4.8 concrete was used, with five sacks of cement per cubic yard of concrete and $6\frac{3}{4}$ gal. of water per sack of cement. From El. —4 to El. +10 the concrete proportion, by volume, was 1 : 2 : 3, using seven sacks of cement per cubic yard.



SINKING THE STEEL SHELL ON PIER 12
Dredging under way inside the shell. Note steel bracing to stiffen walls of the shell.



METHOD OF SINKING CAISSON ON SAND ISLAND

Above El. +10 and excluding the bridge seat, the concrete was the same as used below El. —4. The bridge seat was poured with a 1 : 2.2 : 3.8 mixture, using six sacks of cement per cubic yard.

The work on pier 11 was started May 16 and completed Sept. 21, an elapsed time of 128 days, of which eleven days were required for erecting falsework, 14 days for driving sheet piling, 24 days for framing, 22 days for excavating and lowering, 15 days for pumping and cleaning and 42 days for pouring concrete.

Sand Island Method—Eight piers (12 to 19, inclusive) are being constructed by a method never before employed, the author believes, in deep water work. Briefly, this consists in sinking a steel cylinder, 81 ft. in diameter, from an octagonal dock built around the pier site, the cylinder being erected in sections as it is lowered. This steel shell is allowed to sink of its own weight, and when it finally comes to rest, the mud is dredged out from the inside and the shell backfilled with sand nearly to the top. On the sand island thus formed a steel cutting edge is laid and steel forms are erected for the construction of a reinforced-concrete caisson containing six dredging wells. After the first 25 ft. of caisson has been poured, sand is dredged from beneath it through the wells, permitting the caisson to sink. This process is repeated, alternately building up the concrete pier base and dredging through the wells until rock is reached, each lift of concrete adding about 10 ft. to the height of the sinking pier base.

When the work has progressed in this manner until all the concrete of the pier base has been poured and while the pier base is still above water level, an open cofferdam is erected on top of the base and dredging is then resumed. When the cutting edge reaches rock, the dredging wells are sealed, and after water has been pumped out, are filled with concrete. The pier shaft is then constructed in the open cofferdam. After the cofferdam has been built the steel shell is unbolted approximately at the mud line and the upper sections are used again in the construction of other piers.

This method has the advantage that all concrete is placed in the dry, no timber is left in the pier, reinforcing steel can be properly set and the work can be carefully inspected at all times, thus insuring the highest grade of reinforced concrete in these piers.

Putting Down the First Sand Island Pier—Construction of the falsework in pier 12 consisted in driving eight sets of piles, 110 ft. long, at equal intervals outside the circumference of a circle 81 ft. in diameter, six vertical piles and two batter piles being used in each set, except that at the two points directly in the line of the current an additional batter pile was driven. Vertical piles were accurately spotted by triangulation before driving.

The six vertical piles of each set were driven in the form of a rectangle, 12 ft. wide (in a direction tangent to the circumference of the circle) and 15 ft. long. Four piles were driven on the inner short dimension and two piles on the outer side, the batter piles being used to brace the inner row. The projections of the short sides of these eight rectangles formed two octagons about the pier site. One vertical pile was driven at the corner of each octagon, making a total of 64 vertical piles and eighteen batter piles for each pier. In addition, seven vertical piles were used to support a stiff-leg derrick.

The piles were then capped about $3\frac{1}{2}$ ft. above high water. The platform thus formed was about 15 ft. wide and the distance between inner faces of caps across the diameter of the circle was approximately 85 ft. In order to avoid interference between plumb piles and steel shell during sinking, 6x10-in. wales were placed on the inner faces of the octagon.

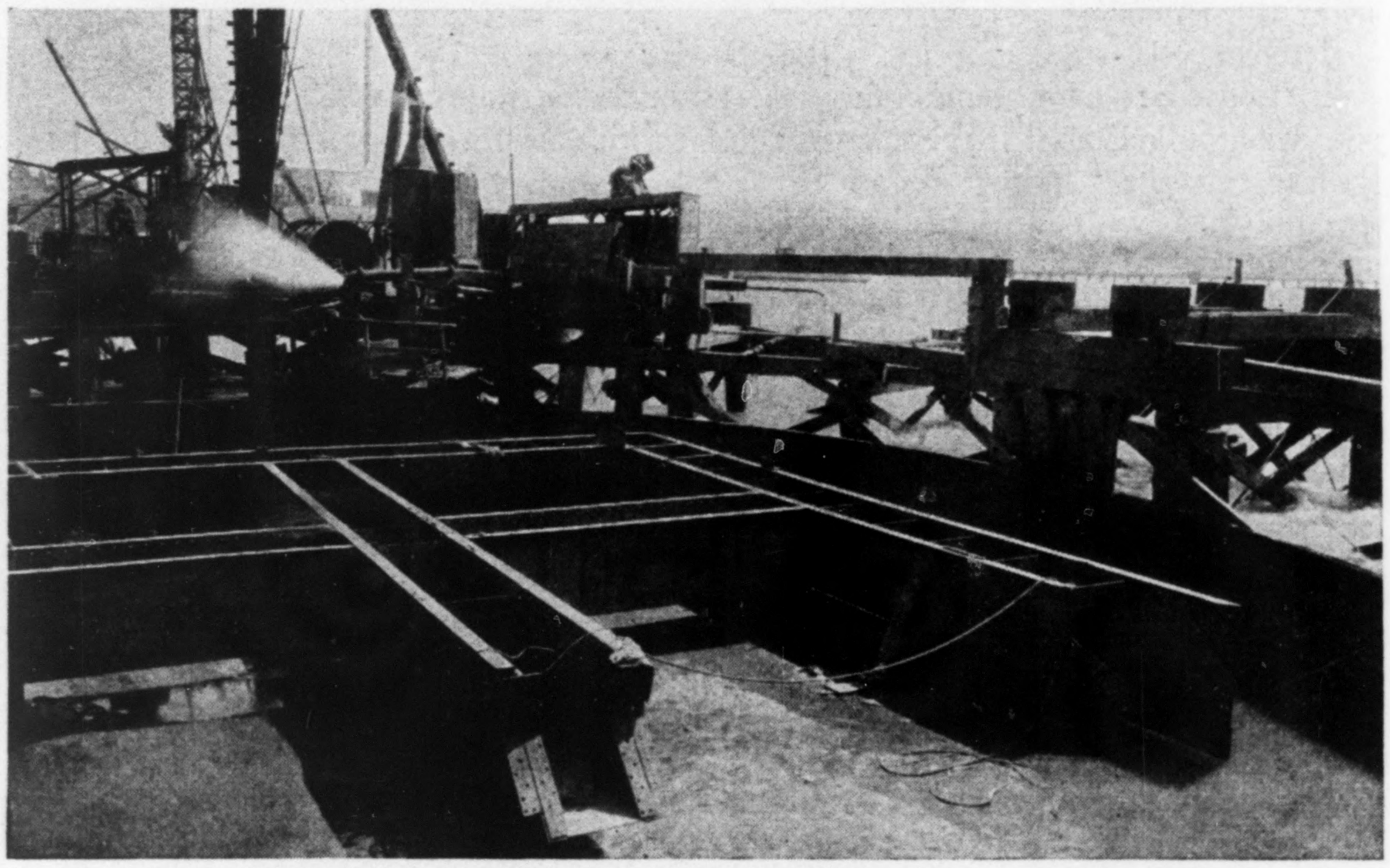
The next step was the construction of the grid seats for eight steel winch frames, which were built of heavy timbers over the eight sets of piling at the midpoint of each side of the octagonal dock. The winch frames were made in the form of steel towers surmounted by a 21-in., 104-lb. I-beam from which a hoisting pulley was suspended. The towers were about 44 ft. high with inner faces vertical. For the inner legs 6-in., 22.5-lb. H-beams were used, while the outer legs consisted of $4x4x\frac{3}{8}$-in. angles. The towers were of rugged construction, well braced with struts between towers, and each one was anchored to the falsework by tie bars and rods.

Mounted on suitable supports within the base of each tower and radial to the circle was placed a 27-in., 112-lb. I-beam, movable toward and away from the steel cylinder and resting on a 2-in. bearing plate at the inner end. These beams served as supports for the timbers on which the first sections of the steel shell were erected and also held the shell in position after lowering or during erection of additional sections, it being possible to connect the beams to the shell at any elevation by removing bolts in the shell splice plates.

The outer ends of these beams were held down by two 1-in. tie rods passed through a 10-in., 20-lb. channel on top of each beam. Timber supports for the bottom course of the cylinder consisted of two sets of 12x12-in. timbers, one set resting upon the inner ends of the movable I-beams spanning from beam to beam and the other set resting on the pile capping, these two sets being spanned in turn by four shorter cross-members upon which the steel shell rested.

Erection of the Steel Shell—Then followed the erection of the steel shell, material for which was brought to the pier site as an assembly of steel plates 10 ft. wide and of a length equal to one-eighth the circumference. Each assembly was made up of two $\frac{1}{2}$-in. plates 5 ft. wide, lapped 3 in. and provided with a $5x3\frac{1}{2}x\frac{7}{16}$-in. horizontal reinforcing angle along the lap. Similar angles also were riveted to the top and bottom of each section, field connections at horizontal joints being made through these angles, which were on the outside of the shell. Vertical field joints were butt connected, using $23x\frac{1}{2}$-in. plates on both sides of the shell. All field connections were bolted and all other connections shop riveted.

The bottom of the shell was reinforced with a $9x\frac{1}{2}$-in. plate extending around the inside circumference, the horizontal reinforcing angle being omitted at this point. In addition, the bottom 5 ft. of the shell was strengthened with $3x3x\frac{3}{8}$-in. vertical stiffener angles on the outside, six stiffeners being used in each 45 deg. section. In order to keep the shell truly circular and to prevent distortion by reason of possible unequal pressure during sinking, interior horizontal truss bracing was placed halfway up each of the 10-ft. sections, with the exception of the bottom section being so arranged that an unobstructed rectangle, 69x50 ft. in plan, was left at the center of the shell.



ERECTION OF STEEL CUTTING EDGE OF THE CAISSON
Steelwork was assembled on the sand island within the protection of the cylindrical shell.

This interior bracing consisted of steel struts made of paired angles, which formed the sides of the rectangle, and two diagonal members, consisting of single angles, from the midpoint of each strut to the inner face of the steel shell, these diagonals being placed approximately at right angles to each other.

When the first 30 ft. of shell had been erected and suspended from the winch frames, the timber supports were removed from beneath it and the movable I-beams slid back. The shell was then lowered by means of hand winches mounted in each tower, until the first 15 ft. was under water. It was held in this position by sliding the I-beams forward and bolting their inner ends to the shell.

Three more 10-ft. sections, including the interior bracing, were then added and the shell again lowered, sinking of its own weight into 21 ft. of mud but leaving its top a few feet out of water. An additional 10-ft. section was added later in order to prevent extremely high tides from washing into the cylinder.

When the bottom of the cylinder was about 50 ft. below lower low water, clamshell buckets were used to dredge a 10-ft. depth of mud from inside the shell, after which the interior struts were removed by a diver and the cylinder was backfilled with sand to an average elevation of about mean lower low water, approximately 9,500 cu.yd. of sand being required for this purpose. The top of the sand was leveled with clamshell bucket and by jetting, after which the cylinder was unwatered to a depth of about 12 ft. below high water.

Putting Down the Caisson—The next step was to erect on the sand island the cutting edge of the caisson which was to constitute the bottom of the outer edges of the pier base as well as of the cross-walls separating the dredging wells. Around the outer edges of the pier the caisson walls in this first section, or cutting edge, were $3\frac{1}{2}$ ft. wide at the top, tapering to a width of 6 in. at the bottom and 4 ft. 8 in. deep, a 6x4x1-in. angle being riveted to the bottom of the cutting edge proper to facilitate sinking. Under the cross-walls the steel members had a uniform width of 3 ft., but did not extend down to the level of the cutting edges under the outer walls, the latter being 13 in. lower than the intermediate members.

Structural steel used in the cutting edge consisted principally of $\frac{3}{8}$-in. web plates with 6x6x$\frac{1}{2}$-in. angles at top and bottom. Diaphragms of 9x$\frac{3}{8}$-in. plates with 4x4x$\frac{3}{8}$-in. connecting angles were placed at about 4-ft. intervals in the outer shoe and at the midpoint of the cross-members. At the four corners of the cutting edge, 8x8x$\frac{1}{2}$-in. angles were used on the outside and 6x6x$\frac{1}{2}$-in. angles on the inside. Horizontal corner braces, consisting of one 3x3x$\frac{3}{8}$-in. angle, were riveted to the top angles of the outer shoes. Total weight of steel required for the construction of the cutting edge was approximately 98,000 lb.

Included in the assembly of the cutting edge was a bracing and reinforcing system adequate to secure rigidity as well as bond with the structure above. Pouring concrete was the next step. Steel forms were erected around the cutting edge and around the faces of the dredging wells. The base of the pier was 40x60 ft. in plan, and the six well openings were each $10\frac{1}{2}$x$11\frac{2}{3}$ ft. The thickness of the outer walls of the caisson was $6\frac{1}{2}$ ft., while the walls separating the dredging wells were 6 ft. thick. The inside of the cutting edge was stepped inward in two steps to a height of 18 in. above the top of the shoes, these forms being removed when concrete had been poured to 25 ft. above the cutting edge.

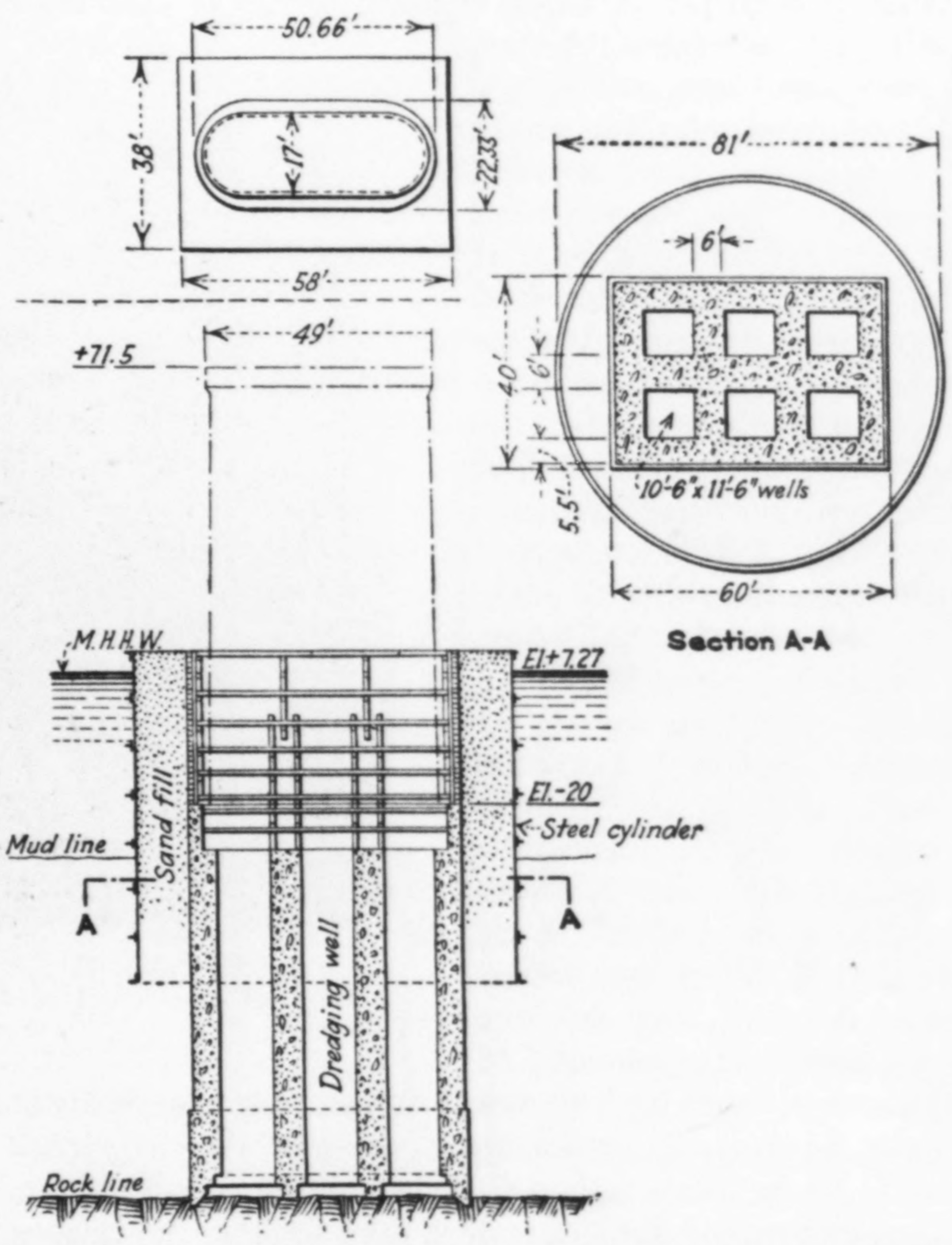
The first pour of concrete was a 15-ft. lift, including material poured into the shoes. Subsequent lifts were 10 ft. high. After each pour the caisson was lowered by dredging through the wells, well forms being removed before dredging started. Construction joints were well keyed by sinking wooden troughs in the top of each pour, and surfaces were cleaned and grouted.

At an elevation of 20 ft. above the cutting edge the pier base was stepped back 1 ft. on all sides, giving a 38x58-ft. section from this point up to El. —20. This was done in order to reduce skin friction on the sides during sinking.

For the first 6 ft. of concrete poured a 1 : 2.1 : 4.1 mixture was used, averaging $5\frac{3}{4}$ sacks of cement per cubic yard and $6\frac{1}{2}$ gal. of water per sack of cement. The remaining concrete, up to El. —20, was a 1 : 2.6 : 4.8 mixture, using five sacks of cement per cubic yard and $6\frac{3}{4}$ gal. of water per sack of cement.

Pouring was done by day and dredging was carried on at night. A 10-ft. lift, averaging well over 500 cu.yd. of concrete, could be completed at one pouring. The steel forms were easily raised and bolted in position for succeeding lifts. With two rigs working, about sixteen hours were required to sink the caisson 10 ft. through sand or mud.

Some difficulty was experienced in keeping the caisson vertical and in exact position during sinking, but tendency to tilt was corrected by regulating the dredging. If the pier was out of position, it could be inclined



PLAN AND SECTIONS OF TYPICAL SAND ISLAND PIER

slightly by dredging through the wells on one side only, thus bringing it back to proper location, and then plumbed by dredging through the opposite set of wells. The maximum deviation was 12 in. out of line and 19 in. out of level, all of which was corrected. No abrupt change was noted during the transition from sand to mud or when the caisson passed below the steel shell.

At an elevation of 73 ft. above the cutting edge, a recess was left in the top of the pier base for the distributing block, which will be poured after the dredging wells have been filled with concrete. This block will be 53 ft. long, 33 ft. wide and 10 ft. thick, leaving a wall $2\frac{1}{2}$ ft. thick around the block. Anchor bolts were set at about 6-ft. intervals in the top of this wall for the purpose of holding down the timber cofferdam in which the pier shaft is to be constructed.

Two sets of struts, to act as part of the cofferdam, were wedged tightly against the inside faces of the walls which formed the upper 10 ft. of the caisson. Each set of struts consisted of two longitudinal and four transverse 12x12-in. timbers, placed above the partition walls of the dredging wells in order not to interfere with dredging operations.

Cofferdam Attached to Pier Base—Construction of the cofferdam started when the top of the caisson, at a point 83 ft. above its cutting edge, was only about 2 ft. above high water. Dredging was discontinued during construction of the cofferdam.

The cofferdam walls, which had a total height of 30 ft., were made up of 12x12-in. timbers, in horizontal layers calked and treated against teredo attack. Successive courses, or sections, varied in height from 5 to 10 ft. Each successive layer of timber was secured to the preceeding layer by ¾-in. drift bolts, 18 in. long, spaced about 3 ft. apart.

At about 12-ft. intervals, 10-in., 15.3-lb. channels, 20 in. long, were placed flange upward on top of cofferdam wall and connected by two 1-in. square bars to the anchor bolts previously set in the top of the concrete caisson. These bars passed through holes in the webs of the channels and were held in position by nuts on the upper ends. They extended down on either side of the caisson wall to the anchor bolts and, when tightened, served not only as a means of anchorage but also to close the joints in the timbering.

With the cofferdam completed, dredging was resumed. Sinking proceeded slowly, as only one rig was available at that time, the height of the cofferdam wall making it impossible to use the derrick barge for dredging. At this stage of the work mud was excavated to an average depth of 5 ft. below the cutting edge without producing caisson settlement. It was deemed inadvisable to dig deeper than 6 ft. beyond the cutting edge. Three charges of dynamite were fired to start the caisson moving, and although it did not settle immediately, soundings showed that the mud had caved into the pockets dredged below the wells. Dredging was continued and the pier lowered in easy stages, dynamite being used whenever the caisson did not show satisfactory movement by the time excavation had been carried 6 ft. below the cutting edge.

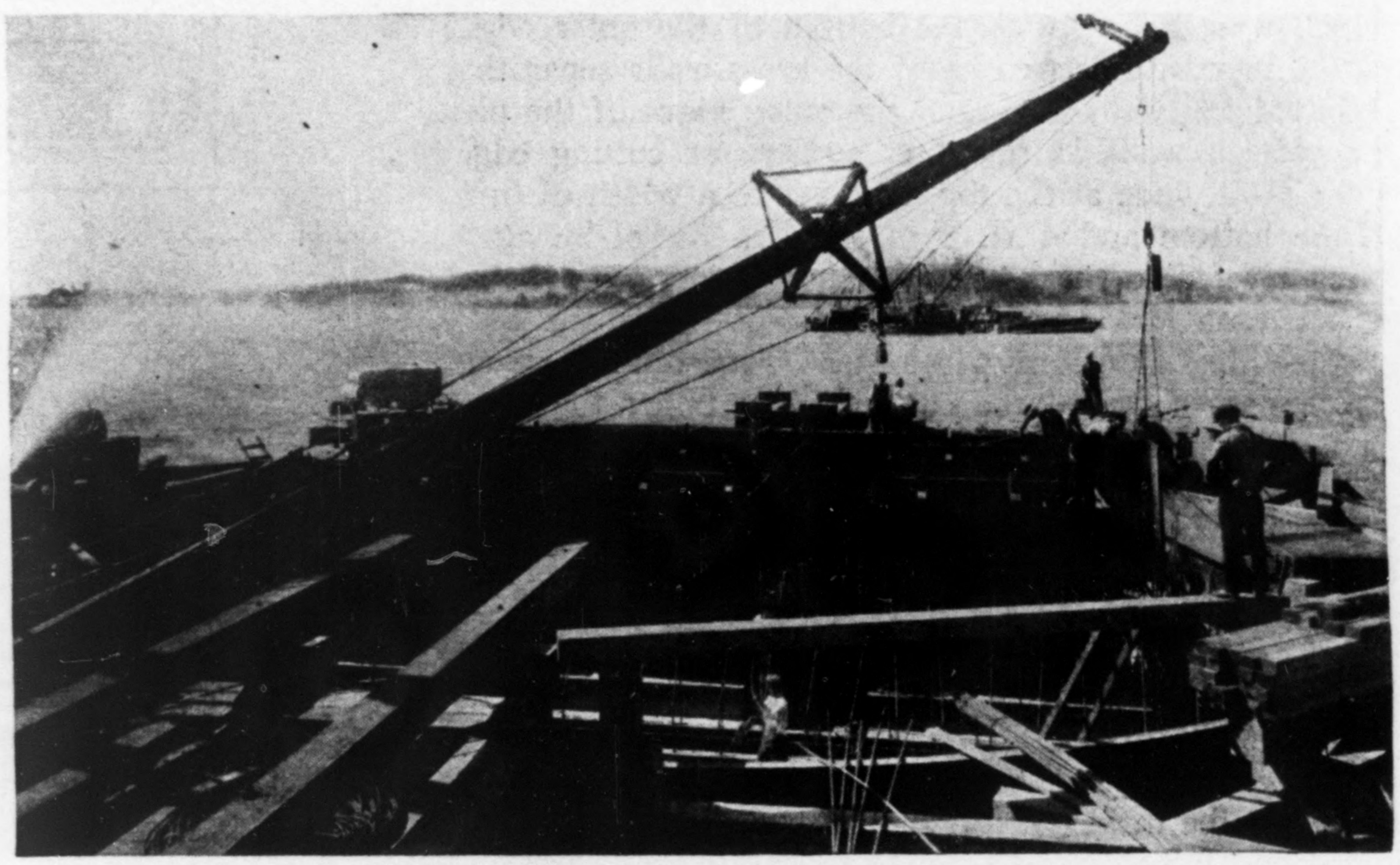
The sinking proceeded through blue clay and sand until the cutting edge reached El. —99.3 with holes in dredging wells about 4 ft. below cutting edge and on rock. At this stage of the work a diver was lowered to report on bottom conditions. Each well was carefully explored and the depths of rock were reported by telephone as the diver proceeded around the sides of the well.

Material was then jetted from under the pier and several charges of dynamite were used to assist in the sinking, the cutting edge finally coming to rest at El. —105.5 (106.3 ft. below lower low water). The diver reported that the outer shoes had penetrated to a depth of about 1½ ft. in the hard shale and that approximately 30 per cent of the partition walls rested upon rock. Foundation conditions were entirely satisfactory. The dredging wells were then sealed with concrete to a height of about 30 ft. above the cutting edge, the concrete being placed under water through tremies.

Later the dredging wells were unwatered and concrete was placed in the dry, being keyed to the caisson by means of recesses left in the walls of the dredging well during construction. When the wells were closed, the distributing block was poured and work on the shaft was started.

Progress on other parts of the project up to Jan. 1, 1930, included completion of the south pedestal piers, and piers 10 and 11. Pier 12 had been sunk to bedrock, piers 12 to 17 inclusive were in various stages of construction and the remaining piers were to be started soon.

The Suisun Bay bridge is being built under the direct supervision of the writer and W. H. Kirkbride, engineer, maintenance-of-way, with the assistance of G. W. Rear, engineer of bridges. All field work is in charge of H. I. Benjamin, assistant engineer of bridges. Design of foundation and superstructure is being checked by S. A. Roake, chief designer.



ERECTION OF CONCRETE FORMS FOR THE CAISSON IN PIER 12
Edge of steel shell in foreground. Rectangular steel forms for the dredging wells are removed before caisson sinks.

Moran & Proctor are consulting engineers for foundation work; Waddell & Hardesty are designing the lift span and towers; Ralph Modjeski was consultant in the preliminary study of the project. Substructure is being constructed by Siems, Helmers & Schaffner, Inc., of St. Paul, under the direction of N. F. Helmers, vice-president; M. F. Clements has been engaged as this concern's consulting engineer. The American Bridge Company will furnish and erect the superstructure.

Canadian Railroad to Extend

The Temiskaming & Northern Ontario Railway will be pushed on to James Bay. Construction is to start in the spring and the line will be completed to the projected terminus at Moose Factory before the close of 1930 if practicable. The total distance is a little short of 100 miles. The first intention is to rush construction from Coral Rapids, present northerly terminus, to Blacksmith Rapids, heart of the new lignite coal field, a distance of 30 miles. The road will then be extended on toward James Bay with the hope that Moose Factory can be reached by the end of the year.

Engineering News-Record

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June 4, 1931

Rapid Subway Construction in Buenos Aires

The Southern Pacific's Suisun Bay Bridge in California

Civil Engineering as Practiced in Siam

Geological Survey Forecasts Low Autumn Streamflow

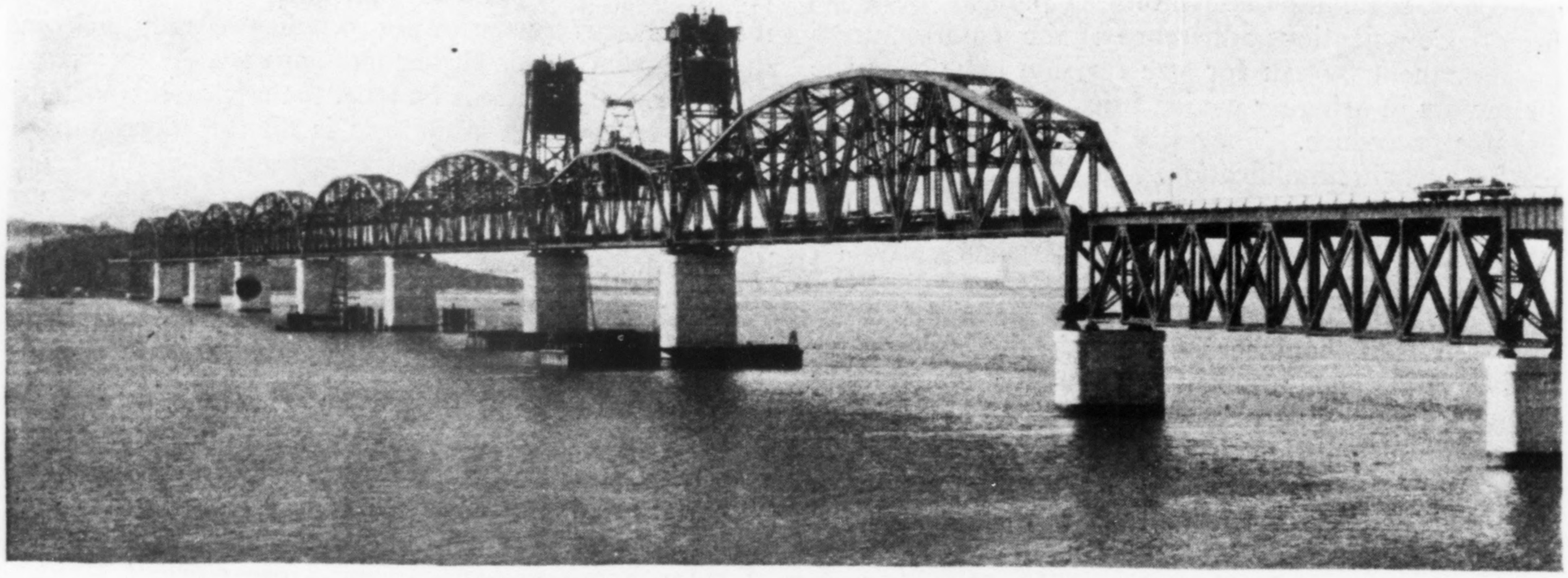
Sewage-Works on Flood Plain Site at Ottawa, Kansas

First of a Series of Articles on Wind Design for Tall Buildings

Report of American Water Works Convention

Pinning up the bottom chord of the Suisun Bay bridge

25 cents
$5 per year



Martinez-Benicia bridge of the Southern Pacific Railroad
In the lift span a load of 1,580 tons is raised 65 ft. to provide a 135-ft. clearance.

Design and Erection of the Martinez-Benicia Bridge Superstructure

Special steels used extensively—Earthquake-resistant provisions—Spans erected on movable steel-truss falsework—Lift-span design and safety provisions

By S. A. ROAKE
Chief Designer, Southern Pacific Co., San Francisco, Calif.

FORMAL OPENING of the new Southern Pacific double-track railroad bridge across Suisun Bay on Nov. 1 completed this $12,000,000 improvement project and terminated the half century of train-ferry service that the company has maintained across the upper waters of San Francisco Bay between Port Costa and Benicia, about 2 miles west of the site. Outstanding features in the superstructure construction are the economic use of silicon steel and heat-treated eyebars pretested to full-load capacity, consideration of earthquake forces in the design, the erection scheme and the 1,580-ton 328-ft. vertical lift span providing a 135-ft. clearance over the main channel, together with the notable provisions for its safe operation. Erection of 20,000 tons of steel in the nine truss spans, the lift span and towers and the viaduct aproaches was accomplished in eleven months. Foundation construction was described in detail in *ENR*, Jan. 30, 1930, p. 174.

Design Characteristics—Essential data on the major divisions of the superstructure, including span lengths, weight and the proportionate use of special steels, are shown in the table on p. 921. Stress calculations were based on an assumed live load of two Cooper E-90 locomotives followed, preceded, or both, by a uniform load of 7,500 lb. per linear foot on each track. For maximum stresses with both tracks simultaneously loaded 90 per cent of the specified live load was used. This loading represents ultimate permissible motive power and train loads and is far in excess of present requirements.

Sectional areas for members were determined by the following unit stresses given in lb. per sq.in.:

1. Axial tension, net section: structural grade (carbon) steel 21,300, silicon steel 32,000 and heat-treated eyebars 36,000.
2. Axial compression, gross section: structural grade steel $20{,}000-70\frac{l}{r}$, but not to exceed 16,500; silicon steel $30{,}000-105\frac{l}{r}$, but not to exceed 24,750.
3. Shear: structural grade steel 16,000, silicon steel 24,000.
4. Power-driven rivets: shear 16,000, bearing 32,000.

The length and type of spans adopted were such that customary methods of analysis were followed in the design calculations. Top-chord joints of the truss spans are riveted, and pin connections are used at the bottom-chord joints. Secondary stress analysis resulted in slight reductions of unit stresses to specified limitations.

The possibility of earthquake-induced stresses was included in design considerations assuming a horizontal acceleration of $2\frac{1}{2}$ ft. per second per second. No additional sectional area was provided in truss members, but the spans were anchored to the piers against this horizontal movement. Longitudinal and transverse ribs projecting downward from the bottom of the steel-truss shoes were grouted into slots left in the concrete pier tops. The anchor bolts are not relied on to resist horizontal forces of this magnitude. The concrete piers carrying trusses are heavily reinforced with steel to resist shattering due to earthquake disturbances.

The method of erecting the superstructure at the 70-ft. clearance above mean higher high water was one of the major construction problems of the project. The scheme adopted by the contractor of floating a low-level deck span successively into each opening, using it as a falsework span upon which to erect the permanent through spans, and finally using this erection span as a part of the completed structure proved most satisfactory. Clearance between the piers of the main 526-ft.

through spans permitted the use of the 504-ft. deck span, ultimately used on the north end, to fit between the shafts and rest on the projecting rectangular bases with the aid of partly submerged steel erection bents. The end shoes of the deck span were landed on this bent just above high-water line, which placed the plane of the top-chord at the proper elevation for the timber blocking required to give the correct camber to the through spans.

Truss panel lengths were identical for the seven 526-ft. through spans and the 504-ft. deck span, except at the ends of the latter. For erection purposes no stringers were used on the deck span, which reduced the floated weight to about 1,750 tons; this load was safely within the capacity of the two 1,000-ton barges used. When in position, first the floor beams of the through span were set in place on the timber blocking, which rested on floor beams of the deck span; then stringers were erected to support the deck. This procedure enabled the locomotive crane to move forward to set the panel of steel in advance. With the deck in position the bottom-chord eyebars were placed, the verticals and diagonals were set up and the pins driven, followed by the top chords and swaybracing. Four 500-ton hydraulic jacks at each pier lifted the through span until it was free of the blocking, permitting the erection span to be floated out and warped into position for the next span. The through span then was jacked down to its permanent position.

For erecting the 328-ft. lift span, the same deck span with five of its twelve panels left off was utilized in the same manner. The operations of placing and removing the erection span were performed at favorable tidal elevations, but final placing on and lifting from the supports was accomplished by the use of 500-ton hydraulic jacks on the barges. The storage yard for bridge steel was located near the south abutment. Erection, with the exception of the northerly 220-ft. viaduct, progressed continuously toward the north, locomotive cranes handling the material brought up on flat cars as required. Both tracks were available.

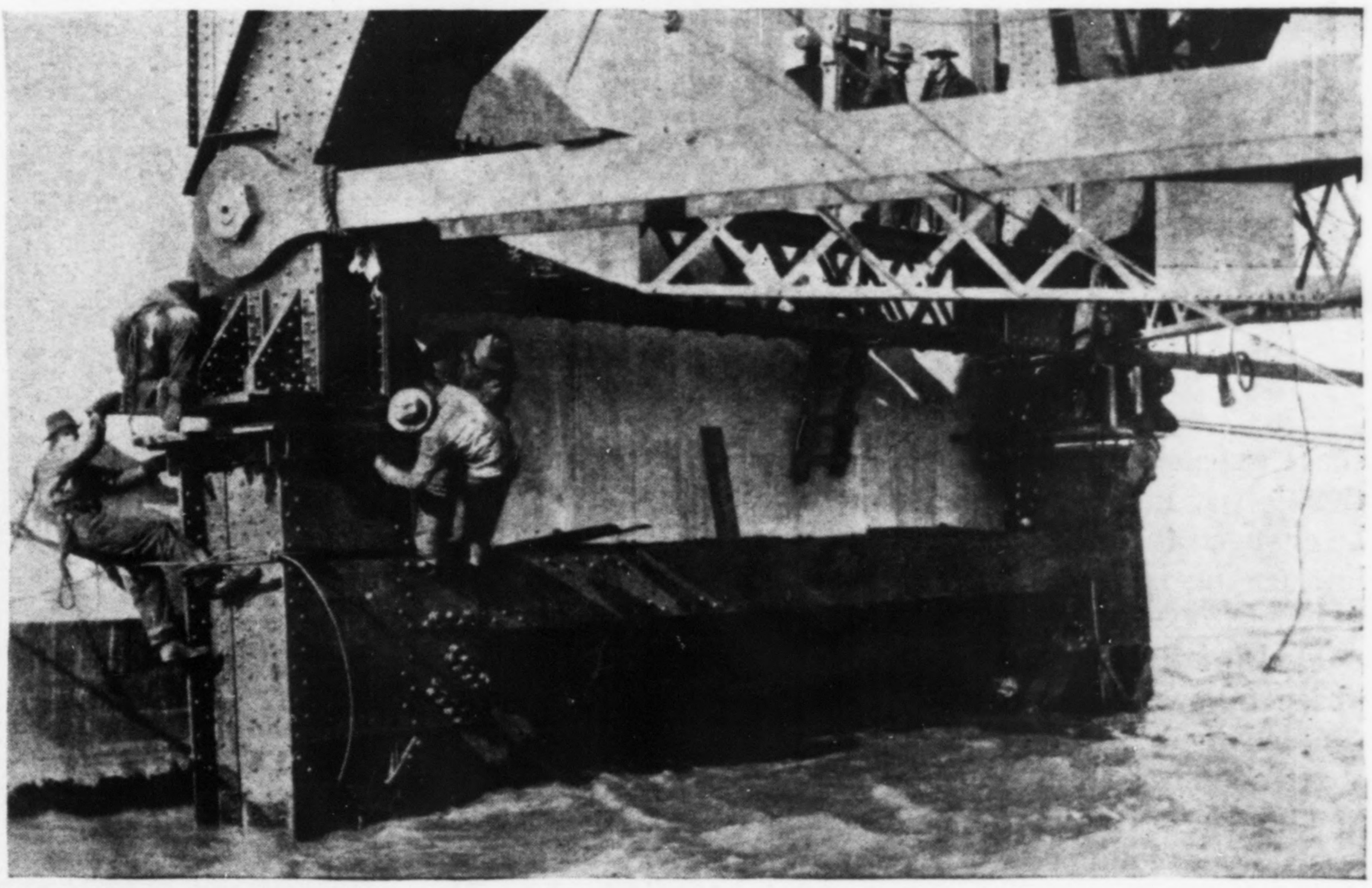
Lift-Span Design — The vertical lift span designed by Waddell & Hardesty, consulting engineers, is a double-track through riveted Warren-truss span, 328 ft. c. to c. of end bearings, with trusses 45 ft. deep at the end panel and 62 ft. deep at the center panel. From the low position providing a 70-ft. clearance it can be raised 65 ft. to a high position with a 135-ft. clearance above high water. The machinery and control apparatus are housed in a two-story structure within the trusses and above the tracks at the center of the span. The concrete counterweights are suspended in towers built up from the first panel of the adjacent fixed spans.

Complete with all machinery, timber deck and tracks, the lift span weighs about 1,580 tons and is carefully balanced by the two counterweights to reduce power consumption for operation to a minimum. To overcome the inertia of the span and counterweights and to lift or lower the span against frictional resistance a distance of 65 ft. in 90 sec., two 150-hp. electric motors are provided; also, for emergency, a 200-hp. gasoline engine is installed.

Each end of the span and its concrete counterweight are suspended by means of thirty-two $2\frac{1}{4}$-in. plow-steel cables, which pass over four 15-ft. cast-steel sheaves (eight cables to each sheave). These cables are fastened to adjustable eyebolts attached to a lifting girder that is riveted to the top of vertical end posts, thus suspending the span at each of its four corners.

To secure equalization of the loads carried by each counterweight cable, the tension in each was tested and adjustments made by means of the eyebolts. The tests were made from platforms in the towers near the mid-point of the stretched length of cable when the span was down, and the counterweights were lowered from the temporary supports until the cables were pulled taut but not heavily loaded. Each cable was pulled sideways to cause it to vibrate and then adjusted until its vibrations, during a selected time interval, were in agreement with vibrations of the others. For this bridge the cables were adjusted to about 24 vibrations in 15 sec.

The sheaves are supported by girders riveted to the main columns at the top of towers transmitting the dead-load reaction from half the span and one counterweight directly to the pier through the two massive steel columns, which also serve to guide the span during lifting or lowering. In raising and lowering, the span is restrained from lateral movement at the expansion end and from either lateral or longitudinal movement at the fixed end by means of roller guides located on the moving span in the plane of the top and bottom chords and traveling on guide surfaces attached to the vertical tower columns.

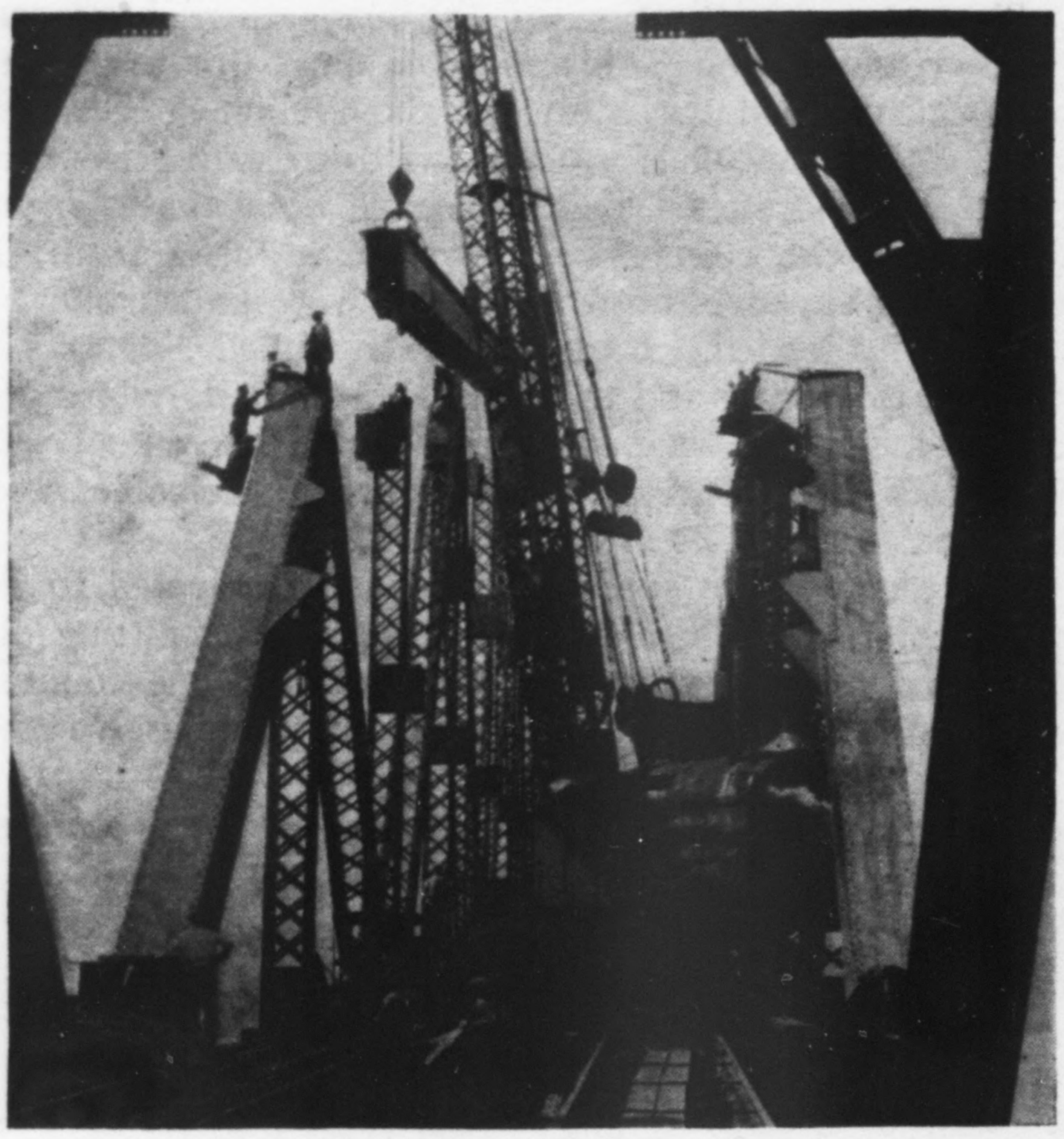


Landing erection span on temporary bent

Deck truss that fitted between main piers was moved by barges.

Lift-Span Operation—The span is raised or lowered by means of sixteen $1\frac{1}{8}$-in. plow-steel operating ropes; eight ropes are used for uphaul and eight for downhaul. These ropes are attached to take-up devices consisting of long threaded eyebolts that are connected to the tower columns, the uphaul near the top and the downhaul near the bottom of towers.

The operating ropes hang vertically along the tower columns, passing over deflector sheaves at the span ends to a horizontal direction and wind around the four 54-in. diameter operating drums located outside the machinery house, two downhaul and two uphaul ropes on each drum. To each drum is bolted a spur gear meshing with spur pinions on the main transverse shaft, which passes into and through the house, and a spur gear keyed on this shaft meshes with the two pinions located on the shaft extensions of the main motors. If for any reason the electric motors become inoperative, a jaw clutch may be engaged to shift control to a 200-hp. six-cylinder gasoline engine installed for auxiliary power and geared to the main operating shaft.



Placing top-chord sections

Main 526-ft. through trusses are 80 ft. deep at center and 33 ft. c. to c.

Each of the two motors is a 150-hp. 2,300-volt three-phase 60-cycle unit with full-load speed of 575 r.p.m., coupled to the driven machinery by means of a flexible shaft coupling. On the other end of the rotor shaft is mounted a solenoid brake of the spring-set magnet-released type equipped with an air cylinder for releasing the brake by compressed air. A third solenoid brake with identical equipment is mounted on the operating shaft independent of the motors.

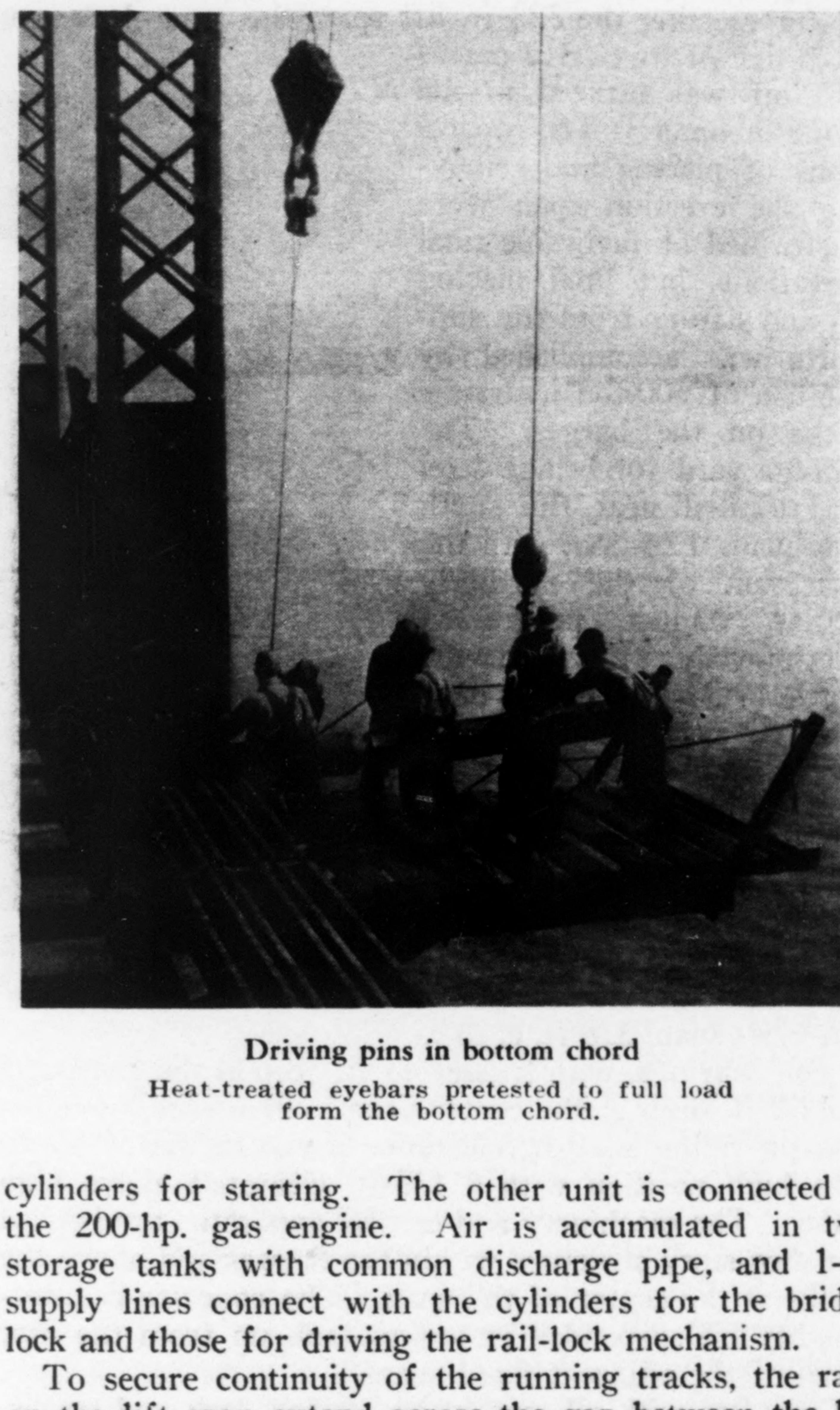
Railroad-signal and electric-power cables are brought from the south tower, supported along steel cables hitched to a steel rocker bent at midspan, and down the rocker bent to the machinery house. An auxiliary counterweight weighing 37½ tons attached to the steel cables at the north tower holds the rocker bent upright and, in addition, tends to overcome the unbalancing effect of the weight of the 64 main counterweight ropes in various positions of lift.

To retard descent of the span at final seating, two air buffers have been placed on each end floor beam. They consist of a piston with protruding shank working vertically in a cylinder closed at the top and fitted with an adjustable needle valve to regulate air exhaust. When the span lifts, the piston drops by its own weight to the low end of the cylinder with shank protruding, and as the span approaches the pier in descending the shank strikes on a bearing plate on the pier with cushioning effect.

Accurate transverse alignment of the closed span is insured during the last 18 in. of its descent by means of centering castings with tapered jaws, which fit a wedge-shaped tongue mounted on steel struts between the main bearing castings. The span is precisely centered longitudinally at the fixed end bearings by tapered jaws that seat on a half-round bearing surface cast integral with the main-pier bearings. At the expansion end each end pin carries a single segmental rocker of 36-in. radius, which bears on the main-pier castings.

The span locks consist of a horizontal bar driven from the plunger of a 10-in. diameter cylinder by compressed air. The air is controlled by electro-pneumatic valves located within the lock-bar housing and operated by electric push button in the control room. This control assembly includes a latch that holds the lock bar in the drawn or driven position.

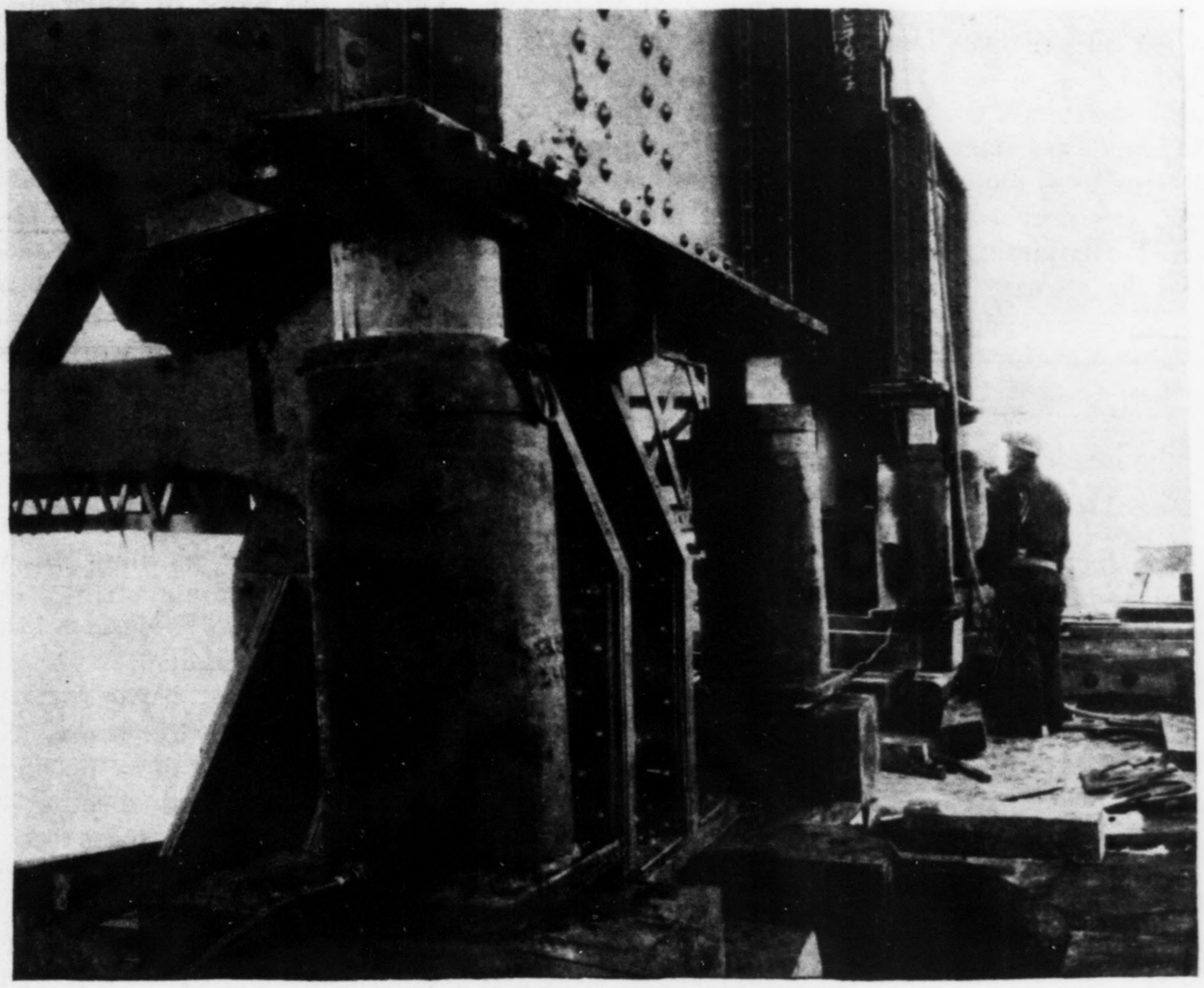
For operating the bridge locks, air-compressor units were installed in duplicate in the machinery room. One unit is directly connected to a 5-hp. electric motor and fitted with automatic pressure regulator and unloading cylinders for starting. The other unit is connected to the 200-hp. gas engine. Air is accumulated in two storage tanks with common discharge pipe, and 1-in. supply lines connect with the cylinders for the bridge lock and those for driving the rail-lock mechanism.



Driving pins in bottom chord

Heat-treated eyebars pretested to full load form the bottom chord.

To secure continuity of the running tracks, the rails on the lift span extend across the gap between the lift and the fixed spans and are tapered to form slip joints in the track at the expansion end of the lift span. Each protruding rail on the expansion end of the lift span



Four 500-ton jacks ready for final lowering

Jacks were used to lift main spans for removal of erecting deck span.

carries a conical spud cast on the rail base, which fits into a circular hole in the rail-supporting chair casting on the fixed span. This spud provides a constant clearance gap between the rail ends of the lift span and the rail ends on fixed span; and all longitudinal movement of track, due to temperature changes in the length of the lift span, takes place in the slip joints.

The ends of the running rails on the lift span, which extend across to the fixed span, are locked down to the supporting rail chairs by means of a rail lock located on the end tie of the fixed spans. The rail locks are set and released by a system of links and levers, which are driven by compressed air controlled from the operator's house on the lift span.

Storage tanks for water, gasoline and compressed air are located outside of and above the machinery room. A jib crane for handling material from track level and a traveling crane of 4-ton capacity in the machinery room are provided.

Safety Provisions—A complete set of navigation signals has been installed, consisting of colored lights on spans and piers, in addition to audible signals for use during foggy weather. Also, a revolving airway beacon of 600,000 cp. has been placed above the top of the south lift-span tower, and flashing airway lights are set on the top of the north tower and each of the fixed spans. To insure against failure of these signals, an electric generator operated by a small gas engine has been provided.

The control room commands a view of the channel and the tracks at the end of the lift span. Operation of the span is controlled by a railway signal machine in the control room interlocked with the controls that raise the span. The main shaft is geared to the limit switch to cut off automatically either the electric motors or gas engine during raising or lowering, at the proper stage to prevent overhaul on the operating cables. The position of the bridge, vertically, is shown to the operator on indicator boards, one in the machinery room and one in the control room.

The railway signal machine is so arranged that in operating the span the following sequence must be used: (1) set all necessary track signals; (2) release the rail locks; (3) release the span locks; (4) set a selector that will determine whether the span is to be operated by the electric motors or by the gas engine; (5) release the lock on the controller or the engine, as determined by the selector; (6) raise the span.

The signal machine performs all these operations except the actual raising of the span. In lowering, the sequence is reversed. To maintain this sequence the interlocking is arranged as follows:

1. Track signals can be set clear only when all locks for the rail joints are engaged.

2. Rail-joint locks can be operated only when all necessary track signals are set against trains and both end locks of the span are engaged.

3. Span locks can be operated only when all locks for the rail joints are disengaged and the span is seated at all four corners.

4. Neither the lock on the controller nor the lock on the gasoline engine can be operated except when the selector is set for that particular lock and both the controller and the reversing lever for the engine are set at the neutral position.

5. Either the controller or the reversing lever on the gas engine can be unlocked, but both cannot be unlocked at the same time; the span can be operated either by electric motor or by the engine, according to the setting of the locks.

PRINCIPAL DATA ON SUPERSTRUCTURE

Item	Description	Span Length, Ft.	Distance From South Abutment, Ft.	Weight, Tons	Types of Steel — Per Cent of Weight: Silicon	Heat Treated	Structural
1	Viaduct approach (plate girder)	40 (four) 80 (five)	560	826	56	..	44
2	1 deck Warren truss (40 ft. deep 19.5 ft. c. to c.)	264	826	655	65	11	24
3	1 through Warren truss (curved top chord, 80 ft. deep at center, 33 ft. c. to c.)	526	1,356	1,912	59	14	27
4	1 through Warren truss (lift)	328	1,692	1,225	61	..	39
5	Lift-span towers	...	.....	940	9	..	91
6	Counterweight boxes	...	...	155	8	..	92
7	Main sheaves and bearings	...	...	170	..	..	..
8	Wire rope and connections	...	...	999	..	..	..
9	Operating machinery and guides	...	.....	89	..	..	..
10	6 through Warren trusses (same as item 3)	526	4,877	11,472	59	14	27
11	1 deck Warren truss (60 ft. deep, (33 ft. c. to c.)	504	5,383	2,007	54	18	28
12	Viaduct approach (plate girder)	40 (three) 100 (one)	5,603	298	49	..	51

6. The air release for the span brakes is so interlocked that power cannot be turned on the span motors when the brakes are released by air.

The entire bridge project was constructed under the direct supervision of C. R. Harding, assistant to the president, and W. H. Kirkbride, engineer, maintenance-of-way, assisted by G. W. Rear, engineer of bridges. All field work was in charge of H. I. Benjamin, assistant engineer of bridges, and matters of design were in charge of the writer.

The United States Steel Products Co. furnished the steel, and the American Bridge Co. erected the superstructure.

Mill and shop inspection of all the material used in the superstructure was made by Robert W. Hunt Co., and, in addition to usual tests, the heat-treated eyebars received a special inspection. This included inspection of the material, the heat treatment and, to give positive assurance of the superior stress-carrying properties imparted by this heat treatment, *every* eyebar was proof-tested to full-load capacity. These bars showing no permanent set, after being pulled, were then bored to exact length required.

Collapse of German Arch Bridge Traced to Tremie Concrete in Piers

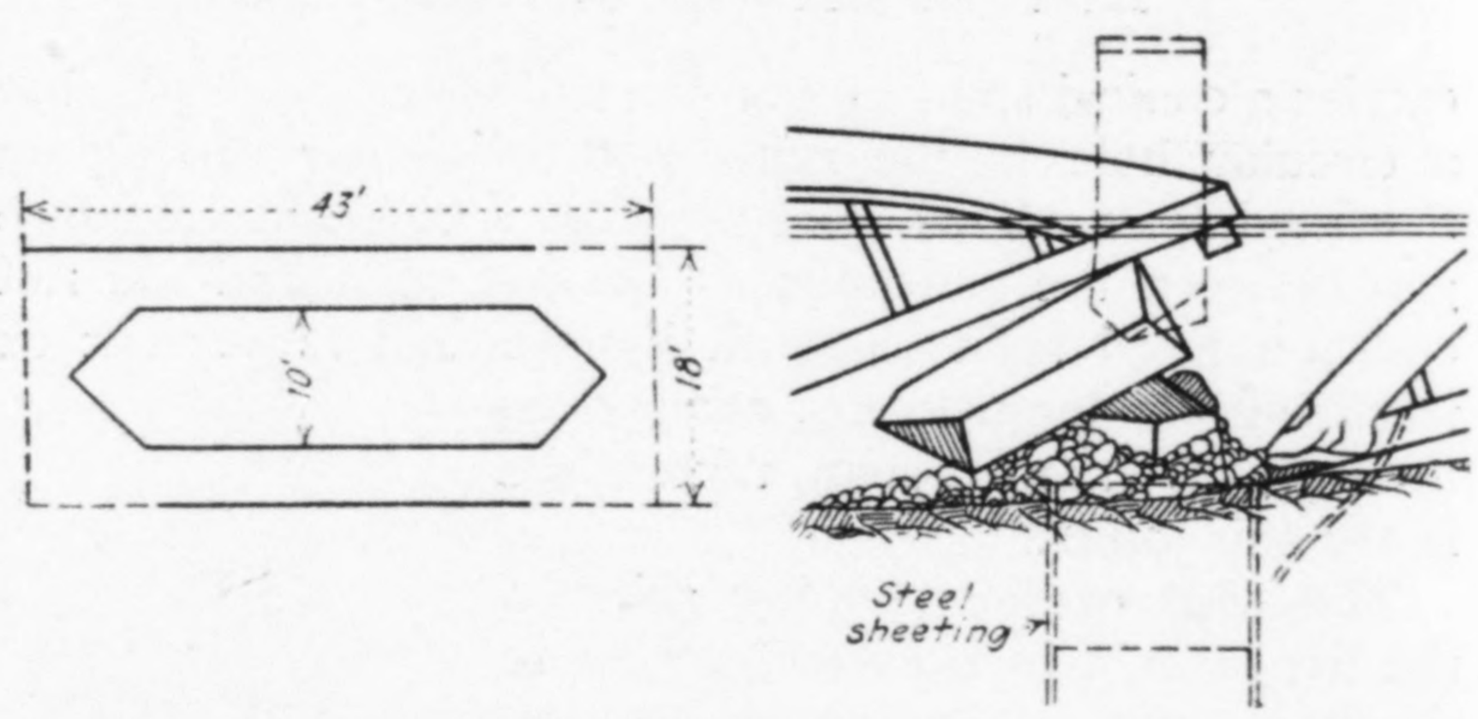
FOUR YEARS AGO a concrete arch bridge over the river Oder at Gartz, Germany, collapsed a week before it was to be opened for traffic, and three workmen were killed. Judicial investigation and subsequent criminal trial of four members of the contractor's organization have just led to their acquittal. The trial brought out that weakness of the lower parts of the piers, which were concreted under water by tremie, was responsible for the collapse. The case was reported in *Der Bauingenieur* in December, 1930.

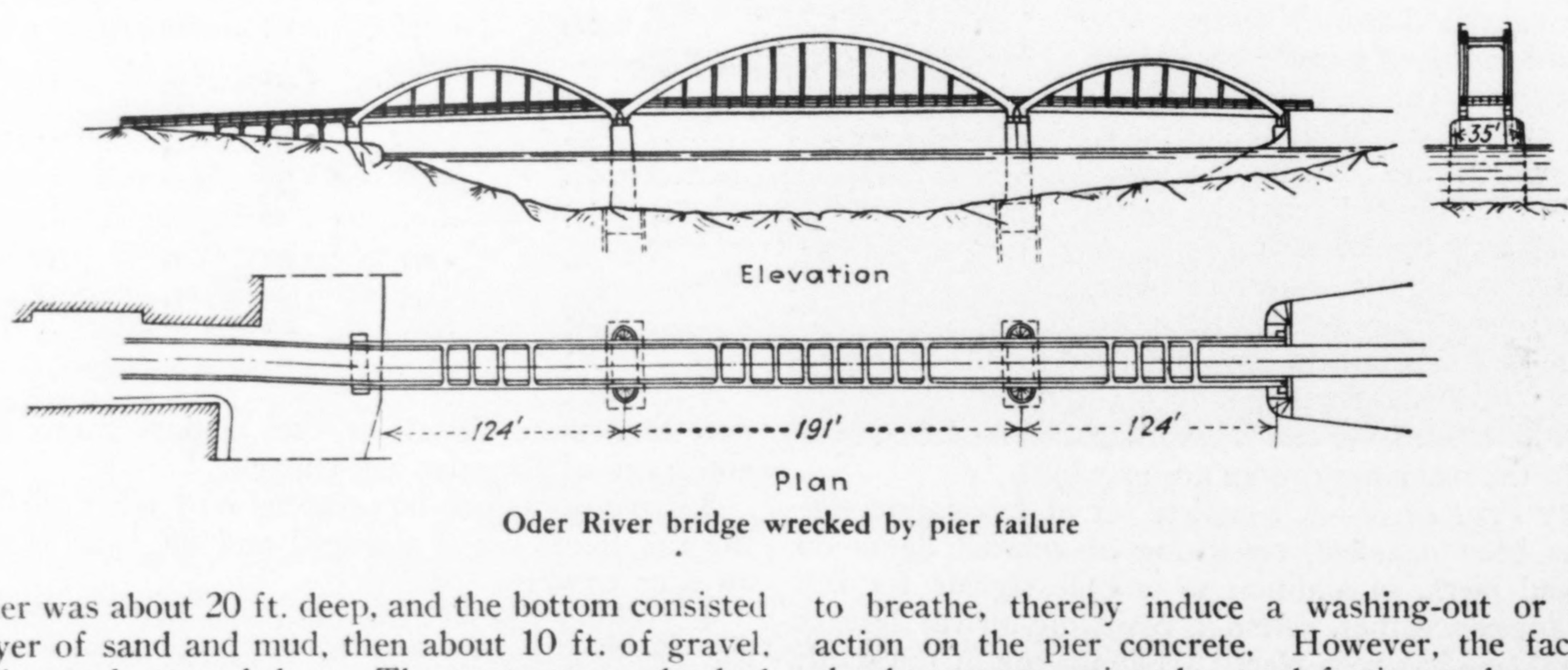
Three concrete spans of tied-arch type, 124, 191 and 124 ft. long, formed the bridge. At the two pier sites swelling of the wood. During the work of concreting the first or left-hand pier it was found that concrete flowed out through the form joints into the space between form and cofferdam wall, and work was stopped when the top of the concrete was a short distance above the river bottom. The cofferdam was pumped out, and a large amount of concrete material was found in the cofferdam outside the form. Later, apparently, the rest of the pier was concreted in the dry. The right-hand pier, the one which later failed, was concreted through water all the way up to river surface. The work was done with a tremie pipe that was shifted progressively over the whole area of the pier. Concrete was charged into the tremie in 10-cu.ft. batches. The tremie is stated to have lost its charge at least occasionally.

After the entire superstructure was completed the right-hand pier collapsed by shearing or crushing below water level and dropped the two adjoining spans. A section of this pier, about 10 ft. high and beginning just above river bottom, was crushed into fragments.

Subsequent examination of the concrete of the surviving pier showed extensive segregation, very low resistance at many points (this lack of strength being more marked in the lower parts), and between pier and sheeting a deposit of about 45 cu.yd. of a weak, light mass (evidently hardened laitance). The fragments of the failed pier indicated that it was similar to the other.

At the trial there was much expert testimony on chemical action of river and groundwater, and on the effect of wave wash from passing vessels to cause the cofferdam





Oder River bridge wrecked by pier failure

the water was about 20 ft. deep, and the bottom consisted of a layer of sand and mud, then about 10 ft. of gravel, and tight marl strata below. The contractor, who had also designed the bridge, built the piers by inclosing the site of each with interlocking steel sheeting driven down into the marl after the bottom had been dredged to footing level. Pier forms extending down to the river bottom were then suspended in this inclosure, and concrete was placed by tremie.

The form planks were spaced ¼ in. apart to allow for to breathe, thereby induce a washing-out or leaching action on the pier concrete. However, the facts point clearly to segregation due to defective underwater concreting method as the responsible cause.

Indictments were found against the contractor's manager, engineer and two foremen. They were acquitted, the court holding that, while the method adopted was not in accord with recognized good practice, approval of the method by representatives of the city had relieved the contractor of responsibility.