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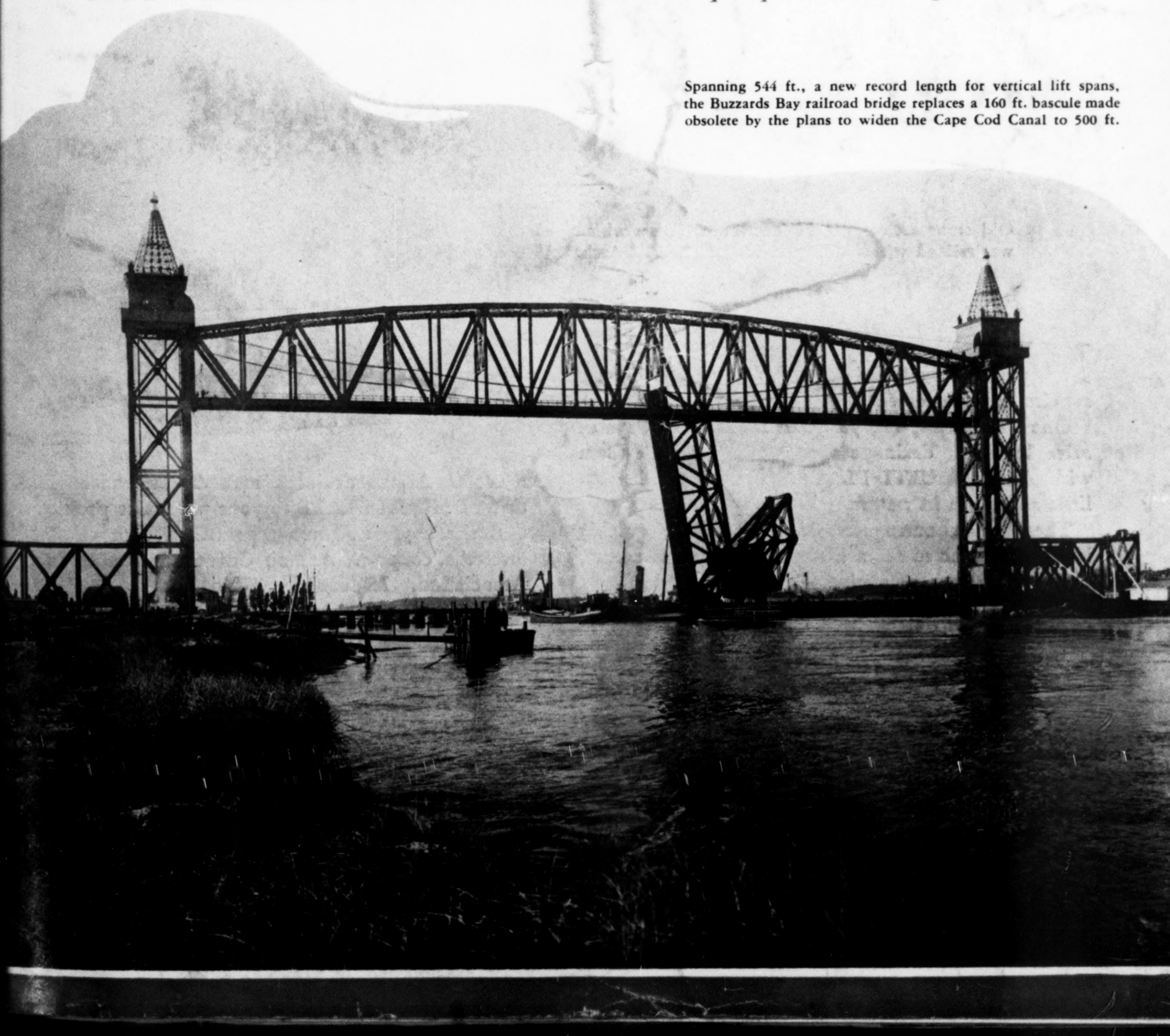

Spanning 544 ft., a new record length for vertical lift spans, the Buzzards Bay railroad bridge replaces a 160 ft. bascule made obsolete by the plans to widen the Cape Cod Canal to 500 ft.

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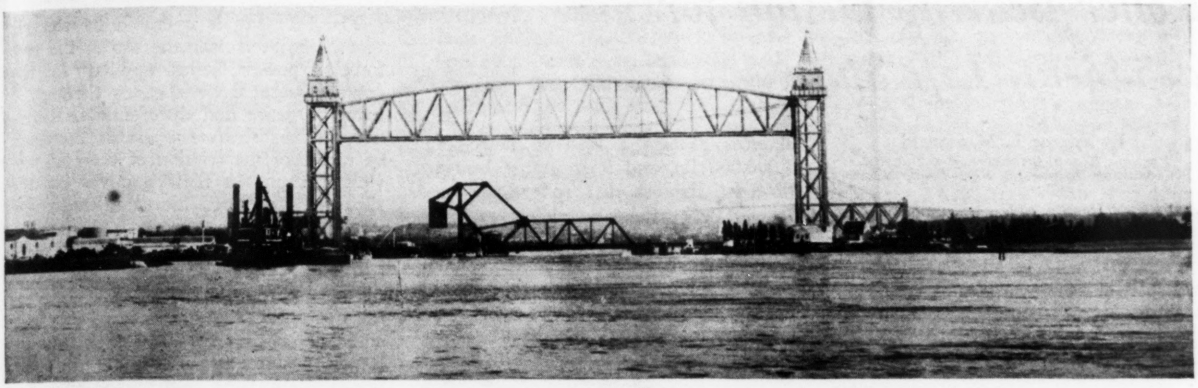


FIG. 1—MARINER'S VIEW of the new Buzzards Bay lift bridge, whose 544-ft. opening frames the present 160-ft. bascule and in comparison gives visual evidence of how much the Cape Cod Canal is to be widened.

Lift Span Over Cape Cod Canal Sets New Precedents

Railway structure at Buzzards Bay, Mass., is longest vertical-lift span in the world, first to use roller bearings, one of highest lifts on record, one of first to utilize architect's services and pioneer in adoption of photo-electric cell control of lights

AMONG the extensive improvements being made to modernize the Cape Cod Canal in Massachusetts for coastwise shipping, the new lift bridge built by the federal government to carry the New York, New Haven & Hartford Railroad is most notable. The two high-level highway bridges of arch continuous-truss type at Bourne and Sagamore *(ENR,* Jan. 25, 1934, p. 107) have their counterparts in at least two previous structures. Dredging the canal to greater width and depth as provided for under the present plan involves a large but otherwise routine operation. The single-track railway lift bridge at Buzzards Bay, however, sets new precedents both in span and design, and its erection was characterized by an interesting procedure planned to meet high tidal currents and the needs of shipping. In a previous article *(ENR,* Jan. 3, 1935, p. 1) the difficult foundation construction on this bridge as well as on the highway bridges was outlined, so that this article completes the chronicle of a notable undertaking.

The Cape Cod Canal lift span measures 544 ft. c. to c. of end bearings and is thus the longest vertical lift in the world, exceeding by 10 ft. the span of the highway bridge over the Delaware River between Burlington, N. J., and Bristol, Pa. It has a width of 27 ft. c. to c. of trusses and is 69 ft. deep at the center. In the open position it provides a clearance of 139 ft. above mean sea level and a clear canal width of 500 ft. between fenders. It is planned to maintain the bridge in an open position most of the time, since there are only a few train movements across the canal daily. Noteworthy features of the lift span are the synchronous hook-up of the operating motors that are located at the tops of the towers and the use of roller bearings for the main counterweight sheaves. Both of these features tend to reduce both the first cost and the maintenance and operating costs of the bridge, while the elimination of operating machinery from the span itself aids in improved appearance, which was one of the prime considerations throughout the entire design.

The lift span is flanked by two tower spans similar in design, except that the north tower span supports an operator's house. These spans are 128 ft. long and 33 ft. wide c. to c. of trusses. The towers are 260 ft. high from pier level to the stainless-steel ball that is used for a finial.

A total of 5,037 tons of steel and machinery is used in the superstructure. Of this, 2,620 tons are in the tower spans and 1,870 tons are in the lift span, which is of silicon steel except for the chards in the end panels. There are 547 tons of machinery and wire rope. As between carbon and silicon steels, the respective tonnages are 2,144 and 2,248. The total moving load, exclusive of machinery, is 4,450 tons.

The bridge was designed and its construction supervised by Parsons, Klapp, Brinckerhoff & Douglas (with whom J. A. L. Waddell and Shortridge Hardesty are associated) who acted as consulting engineers to the Corps of Engineers of the U. S. Army. Col. John J. Kingman, Boston district engineer, is in charge of all of the Cape Code Canal work. McKim, Mead & White were the architects. The Phoenix Bridge Co. fabricated and erected the bridge. P. H. Zipp was the subcontractor on the electrical work. The following two articles by members of the staff of the consulting engineers and of the contractor describe the work in detail.

Roller Bearing Design for a 544-Ft. Lift Span

By Eugene L. Macdonald
Parsons, Klapp, Brinckerhoff & Douglas, Consulting Engineers, New York, N. Y.

THE features that distinguish the vertical lift span over the Cape Cod Canal, in addition to its record span of 544 ft., are the use of roller bearings on the counterweight sheaves, the high development of the synchro-tie method of driving and the retaining of architectural services in an attempt to beautify this usually homely type of structure. The bridge constitutes the western gateway to the canal, and the towers were treated as shafts marking the canal entrance. As the mariner moves up Buzzards Bay the aerial beacons at the tops of the towers and the arched top chord of the span itself give him the first intimation that he is approaching the canal, since its entrance is hidden by a point of land. One of the distinguishing characteristics of the bridge and one which contributes to its pleasing appearance, is the use of the "tower drive," which permitted the placing of the operating mechanism within closed houses at the tops of the towers, keeping the structure entirely free of machinery and appurtenances. The outline of the towers, the proportioning of the primary elements and the ornamentation were the work of the architects.

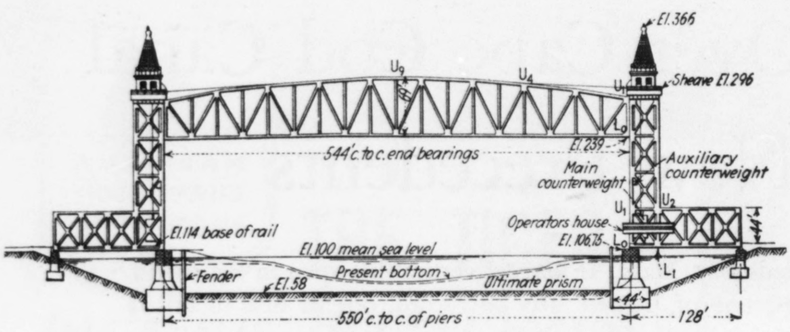


FIG. 2—A NEW RECORD SPAN for vertical lifts is set by the Buzzards Bay lift bridge over the Cape Cod Canal.

Adoption of roller bearings

The lift span is suspended by 80 2¼-in. diameter wire ropes, which pass over four 15-ft.-diameter sheaves located on each tower. Each of these 10-rope sheaves carries a load of more than 1,000,000 lb., and it is in the bearings of these sheaves that rollers are used. It was only after an exhaustive investigation of the use and results obtained with roller bearings in heavy installations that they were admitted to this design. The principal field for heavy frictionless bearings has been in steel rolling mills. Fortunately for the investigation, these installations have frequently received such severe service in the way of overload and shocks as to have broken down completely, thus establishing the ultimate strength to be expected. The research indicated that the roller-bearing manufacturer who claimed that the bearing which he recommended for this bridge would be good for 100,000,000 revolutions might be right. Since this number of revolutions represents about a thousand years of life of the bridge, the designers felt safe in utilizing roller bearings.

The excess of cost of roller bearings over the usual bronze bearings was about $35,000, which is more than offset by the capitalized cost of the annual saving in power. This type of bearing permitted the reduction in the total horsepower of the driving motors from 600 for bronze bearings to 300. The saving in power cost was also magnified by the method of computing the bill, which takes into account a stand-by cost for connected horsepower.

The bearing on each end of each counterweight sheave shaft has an outside diameter of 45 in., the diameter of the shaft is 22 in., and the bearing contains 32 2½ in.-diameter rollers 14¼ in. long. These rollers are, however, cut into two lengths—8 and 6¼ in.—so placed in the bearing that the joints are staggered. Each one of these bearings complete weighs a little more than 9,000 lb.

Synchro-tie operation

The lift span is operated by electric motors, located near the top of the two towers, power being applied to the counterweight rope sheaves through a train of gears and differentials. Rotating the big sheaves operates the span by virtue of the friction of the counterweight ropes in their grooves on the sheaves. It is not anticipated that there will be any slippage of the ropes on the individual sheaves, but differentials were installed as a precaution. The large differential on the shaft that connects the two corners on each end of the span could not be left freely moving because it might encourage unleveling the span transversely. For this reason this differential on each tower is so arranged that it can be brought into play only at the will of the operator, to correct any out-of-level condition, and is so interlocked that it can be used only when the span is within 6 in. of being seated on the piers.

The two ends of the span are kept in step while in motion by means of a 150-hp. synchro-tie motor installed on the same shaft with each of the 150-hp. driving motors. Referring to Fig. 4, motors A and D (on opposite towers) may be used as power motors and B and C as synchronous tie-in motors; also B and C may be used as power motors and A and D as tie-ins. If any one of the four motors is cut out, its mate on the opposite tower can drive the span, and the remaining two motors will act as tie-in motors; also with one motor out of service on each of both towers the remaining two motors can operate the span. If the normal power source is cut off, the span may be operated by auxiliary motors E and F on each tower, supplied with power by a gas-engine generator set.

Besides the synchro-tie for keeping the span level, there have been provided two skew indicators equipped with rheostatic leveling, one to operate in connection with the main power control and one with the emergency power. In conjunction with these automatic leveling devices, there has been installed a hand controller by which the operator can govern the leveling manually. If the operator, in using this hand control, releases the control handle before the automatic system has caught up, the handle will be returned to the automatic point through the action of a spring which is part of the controller operating head.

Each of the six motors is equipped with a thrustor-operated brake designated by the same letters as the motors. In addition to the six motor brakes, there are four drag brakes, G, H, J and K, on the shafts turned by the motor pinions which are operated by springs and held off by thrustors. Their inclusion in the design resulted directly from the use of roller bearings, which have a friction coefficient of only ½ per cent as compared with the 4 to 12 per cent of a bronze bearing. The drag brakes have a capacity a little greater than the difference in friction between roller bearings and bronze bearings, so that, when applied, they will not ordinarily stop the span but merely make its movement as sluggish as if bronze bearings had been used. In normal operation the first point on the controller releases brakes B and C, the second releases brakes A and D, and this point also applies power to the synchronous-tie motors in proper sequence to insure locking the two ends of the span together. This is accomplished by first applying single-phase power to the primary of the synchronous-tie motors in two steps, followed by 3-phase power, all on the one controller point. When the 3-phase power is applied, it is in the proper phase rotation relative to the main driving motors, to insure the torque characteristics that give stable operation over the speed range.

Moving the master controller to the third, or drift point, releases the rest of the brakes, G, H, J and K, and on the next position of the controller power is applied to the main driving motors and the span is operated in the normal manner.

Once the span is under way, the synchro-tie is maintained in operation continuously except that in the down direction the power supply to the synchro-tie motors is cut off when the span is about 2 ft. from the piers. This permits the sure seating of the two ends of the span independently of each other.

When operating without the synchronous-tie motors connected as such, the master controller is used to operate the span, with a leveling controller to maintain the span in its proper position. The hand-operated leveling controller and the skew indicator provide five points of automatic leveling in each direction, the last point cutting off all power and applying the brakes if the span becomes out of level as much as 2 ft.

The brake circuits are equipped with time relays, so that it is never possible for all brakes to go on at once. The drag brakes, G, H, J and K, always go on first whether power is cut off by the controller, the limit switches or through failure of supply. A and D go on next, followed by B and C. A and B, and C and D are interchangeable, however, with their respective time relays. Only one of each of these pairs is required in operating with emergency power.

Near either end of the span's travel the current is cut off and all brakes set, in sequence, by limit switches, and the span is arrested, after which the return of the master controller to the off position and then to points one, two, three, etc., releases brakes A, B, C and D; or, with emergency power, it releases brakes A, D, E and F and starts the motors again. Going down, when the span has been seated at all four corners, as shown by indicator lights, these brakes can be set by a single manually operated switch while power is still on the motors. Going up, the power is cut off first at a point 3 ft. from the top. The operator then starts the span going up again, and this time each end of the span is stopped independently when it reaches its fully open position and the upper-span locks fall into place.

The power for emergency operation is derived from an alternating-current generator driven by a gasoline engine. The emergency system duplicates in arrangement the control for normal operation except for the synchro-tie feature.

The need for the drag brakes was particularly brought out in the study of the method of keeping the span level when operating with emergency power. For this condition of operating, the leveling is taken care of by the automatic rheostatic leveling. This system is completely satisfactory when the motors driving the span are operating under power. Should, however, the span start coasting with sufficient momentum to overhaul the machinery and the motors, the rheostatic device would operate in the wrong direction—that is, it would allow the leading motor to continue to accelerate. To overcome this, the drag brakes are connected electrically to the leveling device so that when this device is called into play and moves to its first point of correcting the level it also ap-

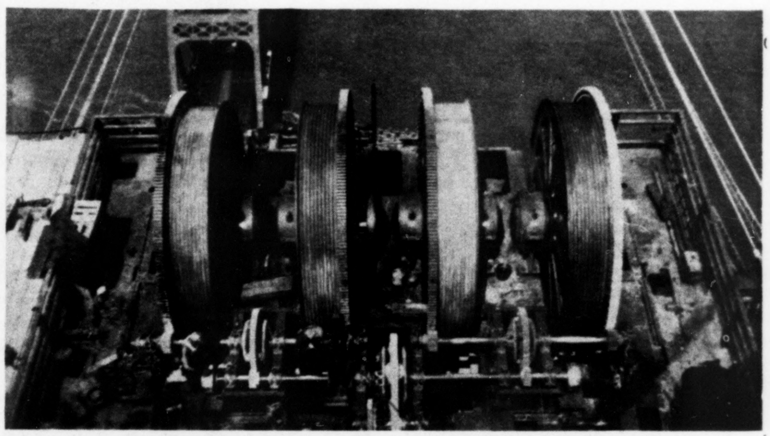
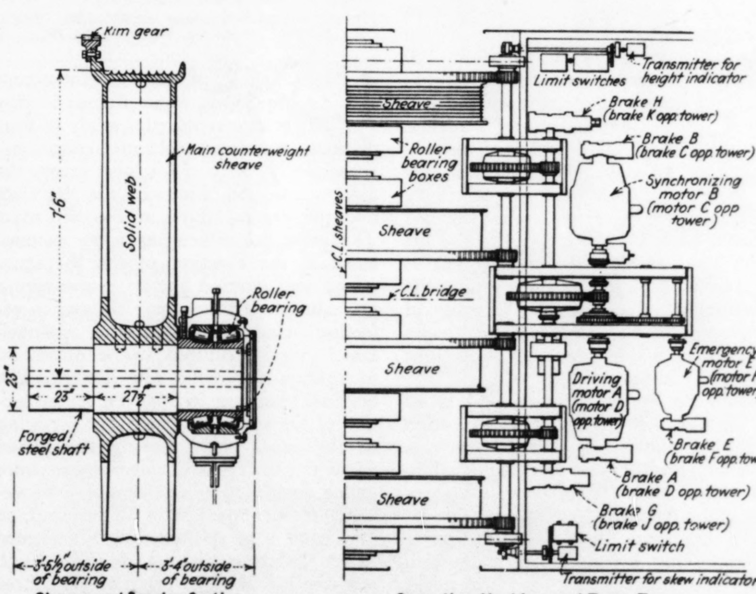


FIG. 3—ROLLER BEARINGS for the main counterweight sheaves (in the boxes on the hubs) provide a design element new to lift-bridge practice.



Sheave and Bearing Section **Operating Machinery at Tower Top**

FIG. 4—DRIVING MACHINERY on the tops of the two towers is tied together electrically by synchronous motors that maintain the span exactly level. The extensive braking system shown is partly a result of the use of roller bearings and their low friction characteristics.

plies the drag brakes. This will slow up the span and machinery sufficienty so that power will be required to drive the motors forward, and the leveling device will take effect in the right direction.

As an additional safeguard in connection with the leveling, a mechanical limit switch is geared to the machinery, which will cut off all power from the motors and apply the brakes should the span in any emergency get out of level by as much as 3 ft. in its length. This gives a 1-ft. margin over the range within which the electrical safeguards to leveling are supposed to work.

Structural considerations

The problems encountered in the designing and detailing of the structural steel differed from smaller structures only in the size of members and stresses to be accommodated. The introduction of double intersection bracing in the tower spans (for architectural reasons) provided a minor problem, however, in one panel—the one containing diagonals U_0L_1 and U_1L_0. Here the fact that the main tower column with a section of 371 sq.in. forms one side of the panel prevented equal distribution to the two diagonals. This was overcome by leaving the upper end of U_1L_0 unconnected until two panels of the tower had been erected. This applied sufficient dead load in diagonal U_0L_1, so that in sharing the balance of the shear in this panel with U_1L_0 equal distribution was attained.

The usual specification for wind load —15 lb. per sq.ft. for the open position —was increased to 30 lb. for this bridge because it is anticipated that normally the lift span will be in its raised position, since more ships than trains have to be accommodated. With the span raised and a quartering wind of 120 miles per hour blowing, the tower to which the span is tied longitudinally will deflect a calculated amount of 2 inches. The towers are also deflected toward each other at each raising of the span due to the action of the auxiliary counterweight ropes, which are attached at panel point U_4 at each end of the lift span. This auxiliary counterweight is a patented device for overcoming the unbalancing effect of the shifting of the weight of the counterweight ropes from the span side to the counterweight side of the sheaves.

Since the operating machinery is on the tops of the towers at an elevation of 180 ft. above the railroad track, a one-man elevator has been installed in each tower for convenience in maintaining the operating equipment.

The lift span is held to its course while moving by guide rollers, which come in contact with the main tower columns. This is the usual practice but, in this case, with the combination of the extreme length of the span and of the doubled specification for wind, the rollers are greater in size and in number than on past structures. At the south tower there are six rollers provided at the elevation of the bottom chord of the lift span, to resist longitudinal motion and four rollers to guide the span transversely. At the north tower there are only four rollers to take transverse load—one at each of the top and bottom chords. All of these rollers are equipped with self-lubricating bronze bushings.

Air buffers are provided at each of the four corners of the span, to facilitate its being seated gently on the piers. Buffers are also provided to ease the span into the open position, at which point it comes against structural-steel stops. This definite limit of opening was used to facilitate the locking of the span in the open position, as it was thought best to add to the reliability of the brakes the positive safeguard of a lock, so that vibration could not by any chance start the span on its downward travel.

The rail locks used on the Cape Cod span embody the principles used in previous designs but include radically different details due chiefly to the length of the span and the resulting amount of expansion that has to be taken care of. In brief, the alignment of the rails is secured by a wedge-shaped adapter on the lift-span rails that engages a slot in the enlarged base of the first lengths of rails on the tower span which are left floating. The shifting of these floating rails is permitted by providing long tapered expansion joints at their far ends.

The counterweight ropes, as usual in recent installations, were not provided with any equalizing device other than the threaded eyebolts by which they are attached to the lifting girder of the lift span. The equal distribution of stress in the 40 ropes at each end was achieved by plucking them at a point part way up the tower and timing their oscillations.

Photo-electric control of lights

The span is equipped with all the lights and signals required for shipping and air traffic. These include lights on the stainless steel pinnacles of each of the towers and on the highest point in the center of the lift span. All of these lights are operated by a photo-electric cell, which is supposed to recognize darkness as it advances in the evening and disappears in the morning. Power for the lights on the span comes through a loop of cable swung from the aerial cables that cross the canal on each side of the lift span. To avoid the necessity of running the large generator set at times when the principal source of power has failed and when it is not desired to operate the lift span, a smaller set has been provided to furnish sufficient power for the lights, the elevator motors, the oil-burning furnace in the operator's house, and the various pumps and warning signals. This set starts automatically when the normal power fails.

Erecting a 544-Ft. Lift Bridge

By William A. Ellis
Assistant Engineer of Erection, Phoenix Bridge Co., Phoenixville, Pa.

ERECTION of the superstructure of the 544-ft. span Buzzards Bay lift bridge required a study of four distinct erection problems concerned, respectively, with the tower spans, the towers, the end thirds of the lift span and the central third of the lift span. The principal difficulties were encountered in the erection of the lift span, which was carried out by cantilevering procedure aided by a derrick boat in effecting closure across the channel, which was permitted to be closed to navigation for only 120 hours. The original plan was to float the center section of the span into place, but dredging of the canal prism during the period when the towers and tower spans were under construction caused the tidal velocity to increase nearly 50 per cent, so that such a procedure was not deemed safe. Tidal velocities as high as 10 ft. per second lasting for two or three hours were recorded as compared to the 7-ft. velocity previously considered to be the maximum. Even with the erection method used, work was made more difficult by this high-velocity current caused by a difference of three hours in the tides at the opposite ends of the 8-mile canal and by the difference in tide range, which is $4\frac{1}{2}$ ft. in Buzzards Bay at the west end and 11 ft. in Cape Cod Bay at the east end of the canal.

Tower spans and towers

Erection work began with the tower spans, the equipment consisting of two locomotive cranes of 60 and 75 tons capacity, which on the first pass set the floor system on sand jacks supported on cribbing. They also erected one panel of the lift-span floor on falsework, to provide more room for crane operation. This was followed by the erection of the lower sections of the main tower columns weighing 57 tons each, and then on the return pass by the erection of the tower span trusses and bracing, part of the permanent tower, and part of the temporary erection towers.

These temporary erection towers were supported on the tower spans extending upward 144 ft. above the top chord to a platform carrying a 40-ton steel stiff-leg derrick with a 75-ft. boom, which

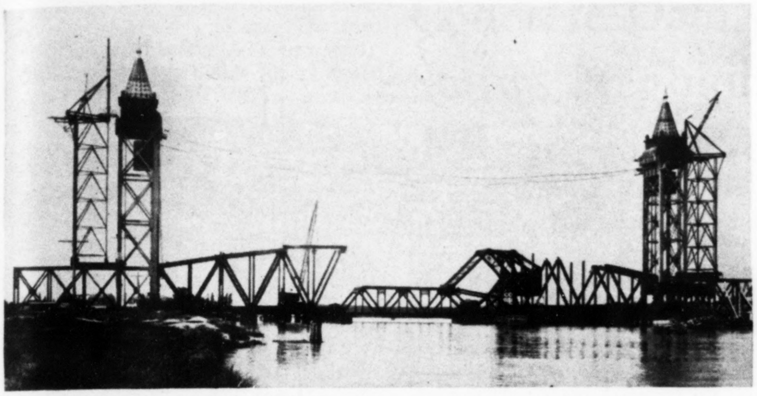


FIG. 5—ERECTION of the Buzzards Bay lift span was accomplished by tower derricks and locomotive cranes, the latter erecting the end thirds of the span on falsework.

was used to erect the permanent tower. The derrick mast was located 12 ft. behind the permanent tower. Each erection tower and derrick weighed 137 tons, and bending in the top chord of the tower span was eliminated by supporting the erection towers on heavy I-beams clamped to the top chord at the panel points.

The derricks on these erection towers set the permanent towers, the sheave girders and the auxiliary counterweight boxes. While this was being done the locomotive cranes assembled the three counterweight girders into one cross-shaped unit, and all material making ends or sides of the counterweight boxes was assembled into large units and riveted as far as possible. The stiff-leg derrick then erected the counterweight girders, supporting them on links provided at the top of the towers for the emergency support of the completed counterweights. Each side and end of the counterweight boxes was then erected as a unit, and the bottom was erected last. This method eliminated much of the expense of erecting, fitting up and riveting in the air, much of the danger to men working below and in general expedited the work. After the main and auxiliary counterweight boxes were filled with concrete, the counterweight ropes, machinery at the tower tops, machinery houses and spires were erected.

Lift-span procedure

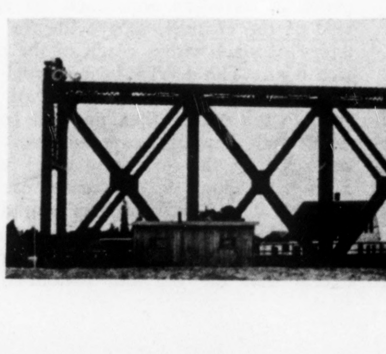
Erection of the lift span was influenced by local conditions and by the design. Specifications provided that "during erection the contractor will be required to maintain a navigation channel 160 ft. wide, in line with the channel through the existing bridge, with a vertical clearance of 134 ft. above mean sea level." The existing channel is at the center of the new span. The lift span could not be erected at a low level and floated in as a unit because the ends of the lift span dovetail between the main columns of the tower spans. Erection at a high level on falsework, with cantilever erection over the channel, was considered expensive and dangerous because of the difficulty in protecting the falsework from the impact of large boats operating in the swift current in the canal. Erection at a low level would decrease the time during which the span might be damaged by navigation and during which the timber piling might be attacked by marine life known to be active in the canal. Therefore, it was decided to erect six panels of the lift span at each end on falsework at a low level, and to cantilever the central six panels over the channel, working from each end and using a derrick boat.

Six End Panels—The bridge design provides emergency hangers at the top of the towers on which the main and auxiliary counterweights may be suspended. There is no provision for jacking or releasing the counterweights when so suspended because the operating machinery is located directly above the emergency hangers. The main counterweights may be suspended at elevations corresponding to a rope stretch of 6, 12, 27 or 48 in. with the span in its lowest position. The auxiliary counterweight may be similarly supported except that provision is made for supporting it in one position only, rope stretch being cared for by means of long threads on the eyebolts at the span end of these ropes. No counterweight can be released from its emergency support unless the weight on the two ends of the counterweight ropes is equalized, permitting removal of the link pins.

Because of the unknown stretch in the ropes, it was decided to erect the span 24 in. above its lowest position, assuming a 3-in. stretch in the ropes and allowing for a maximum stretch of 27 in. In deference to the possible rope stretch, the caps of all falsework bents were kept below the underclearance of the lift span when in its lowest position. The bents under panel paints 1 and 3 contained sixteen piles each, while those under panel points 2, 4 and 6 contained eight piles each. At panel point 5 a 73-pile bent was driven, to take care of jacking loads.

On this falsework the permanent floor, trusses and bracing of the lift-span ends were erected out to the canal channel on both ends of the span and swung free of all supports except the main piers and the heavy bents at panel point 5. The span ends were jacked up and the counterweight ropes connected to the lifting girders. After the ropes had stretched, the end floor beams were blocked up on the piers to relieve

FIG. 6—TOWER OUTLINE reflects a lighthouse motif based on the landmark character of the bridge at the entrance to the Cape Cod Canal.

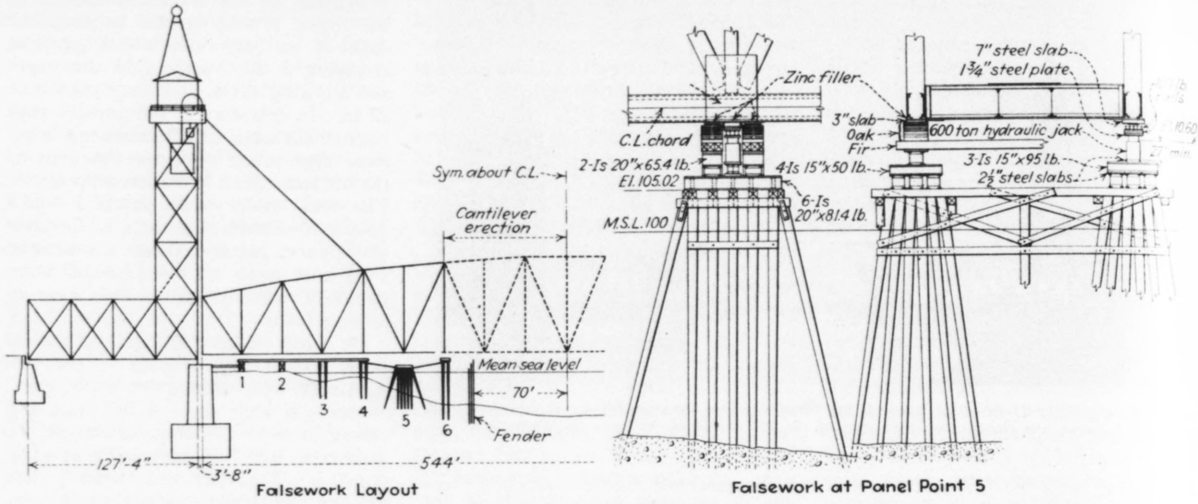


FIG. 7—SUPPORTS for cantilevering the center third of the span to closure were provided by 73-pile bents at panel point 5 at either side.

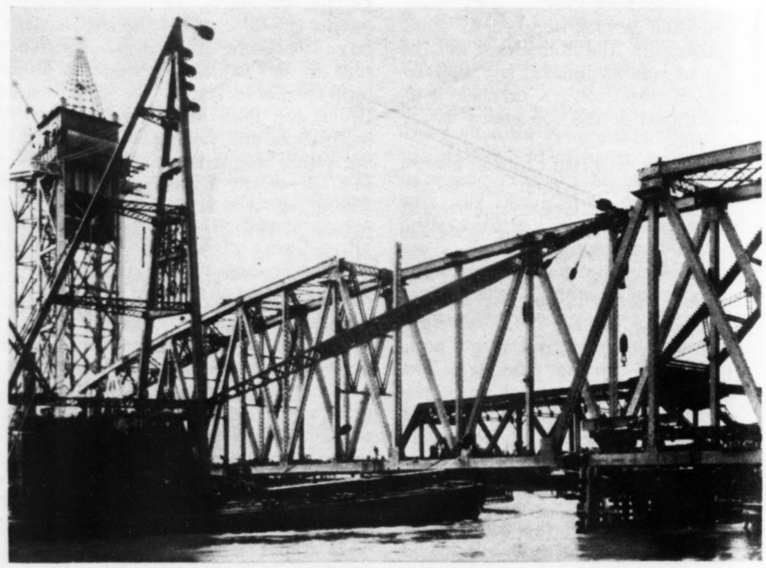


FIG. 8—DERRICK-BOAT ERECTION of the closing members of the span eliminated the need for falsework in the navigation channel.

strains on the machinery caused by the operation of locomotive cranes on the span. On the bent under panel point 5, heavy steel grillages, supporting 600-ton jacks and oak blocking, distributed the weight of the span into the falsework. By means of these jacks, the span was moved to accurate line and erection camber, after which the gap between the ends of the steel on opposite sides of the channel was checked for distance, and preparations were made to erect the central portion of the lift span.

Center Six Panels—Construction of the central portion of the lift span with a minimum interference to navigation was important. The contract provided for a period of 48 hours, during which the canal could be closed to navigation. However, because of the increased tidal velocity caused by dredging, this time was extended to 120 hours. During this period it was planned to complete the erection of the lift span by the cantilever method, using a derrick boat for members in the central part of the span.

The derrick boat, prepared especially to combat the strong tide, dropped two large ship anchors and ran a cable to each of the old bridge piers, to resist tidal flow to the east. Two large ship anchors, a smaller anchor and a line to a deadman on each shore provided anchorage against current running west.

Artificial light was provided in order to permit two eight-hour shifts to work. Ten floodlights of 750- and 1,000-watt capacity supported high on the towers were arranged so that two illuminated the material yard on each side of the canal while the other six were directed at the central portion of the lift span. In addition, eight reflectors were distributed throughout the length of the span, to light the walks; one was suspended above the central portion of the span, and two were arranged to light the outside of the west truss. Extension cords lighted points where the general illumination was not sufficient.

Navigation was officially stopped at 4:25 a.m., Monday, Sept. 16. The derrick boat maneuvered to a position east of the lift span, ran out anchors and was ready for work at 9:30. During this time, locomotive cranes erected one panel of stringers and track on each side of the channel, to provide for delivery of steel within reach of the derrick boat. The derrick boat erected the westerly truss of the south half of the span to the center, then the east truss, floor and bracing, cantilevering the south half of the span from panel point 5 to panel point 9, the center of the bridge. The boat was then moved to the north side of the channel, where it completed the erection of the west truss except that the top and bottom-chord splices at the center of the span were not connected. The closing bottom chord on the east truss was then erected, the north end of the span jacked 1 in. south, to close the bottom chords, and all remaining steel was erected. The last piece of steel was in place at 9:13 a.m. on Thursday, Sept. 19. Steel was then fitted up, temporary track and equipment were removed, and the span was balanced by the addition of portions of the permanent track.

At noon, men were sent to their stations, and the span was lowered by means of the two 600-ton hydraulic jacks under panel point 5 at the north end of the span, and four 300-ton jacks were similarly placed at the south end of the span until the full weight of the span was on the counterweight ropes. The pins were then removed from the main counterweight emergency links, permitting the span to be lowered so that the auxiliary counterweights could be freed from their supports. The span was then raised about 15 ft., to permit erection of the four guide rollers at the top of the span which were inaccessible when the counterweights were up. As a final operation, the span was raised to its upper position and lashed at 7:10 p.m., Sept. 19. The total time navigation was stopped was 95 hours, 7 min., divided as follows: maneuvering the derrick boat, 13½ hr.; night shutdown, 24 hr.; steel erection, 47½ hr.; and balancing and raising the span, 10 hr.

During the time that the canal was closed to navigation, 280 pieces of steel weighing about 600 tons were erected.

The falsework bents under panel point 5 were the keys to successful cantilevering of the span. They were designed to carry a load of 900 tons per bent on 68 piles, and to resist horizontal forces due to tide, wind and rolling. The 68 piles were driven in five parallel rows, spaced 30 in. on centers. The piles were 55 ft. long, with 8-in.-diameter points, and were driven in 25 ft. of water to a penetration of 7 to 20 ft. in sand. Observations indicated a settlement of the top of the caps amounting to ¼ to ⅞ in. under full load.

The camber of the span over the jacking bent was determined after a careful consideration of the permanent camber of the span, its elastic deflection, an allowance for falsework settlement and erection camber, and an allowance for rope shortening due to reduction in load. The sum of these camber allowances was 17½ in.

The minimum vertical motion required of the jacks was estimated to be about 11 in., of which about 3½ in. was under full load and 7½ in. under a diminishing load. These estimates proved to be closely correct. In order to secure approximately equal loads on all jacks, they were operated simultaneously until a vertical movement of the span of about ½ in. was attained. Gage readings were then adjusted to close agreement by lowering the jacks carrying excessive loads. After this had been done, the operation was repeated until the pins connecting the main counterweights to the tower tops could be removed.

During this jacking the top of the hard-wood blocking between the grillages and the bottom chord was never allowed to be more than 1 in. below the steel. This blocking was so arranged that 1-in. layers of it could be removed at a time when the jacks were under their greatest strain. As the load on the jacks decreased, the blocking was increased in thickness, since the danger of jack failure was greatly reduced.

The work was fabricated and erected by The Phoenix Bridge Co.; John F. Kinter was general superintendent of erection, Harry A. Archinal field superintendent, and Frank W. Peirce resident engineer. The method and details of erection were designed by the writer.

Concrete Dam, Twice Raised, Shows Negligible Leakage

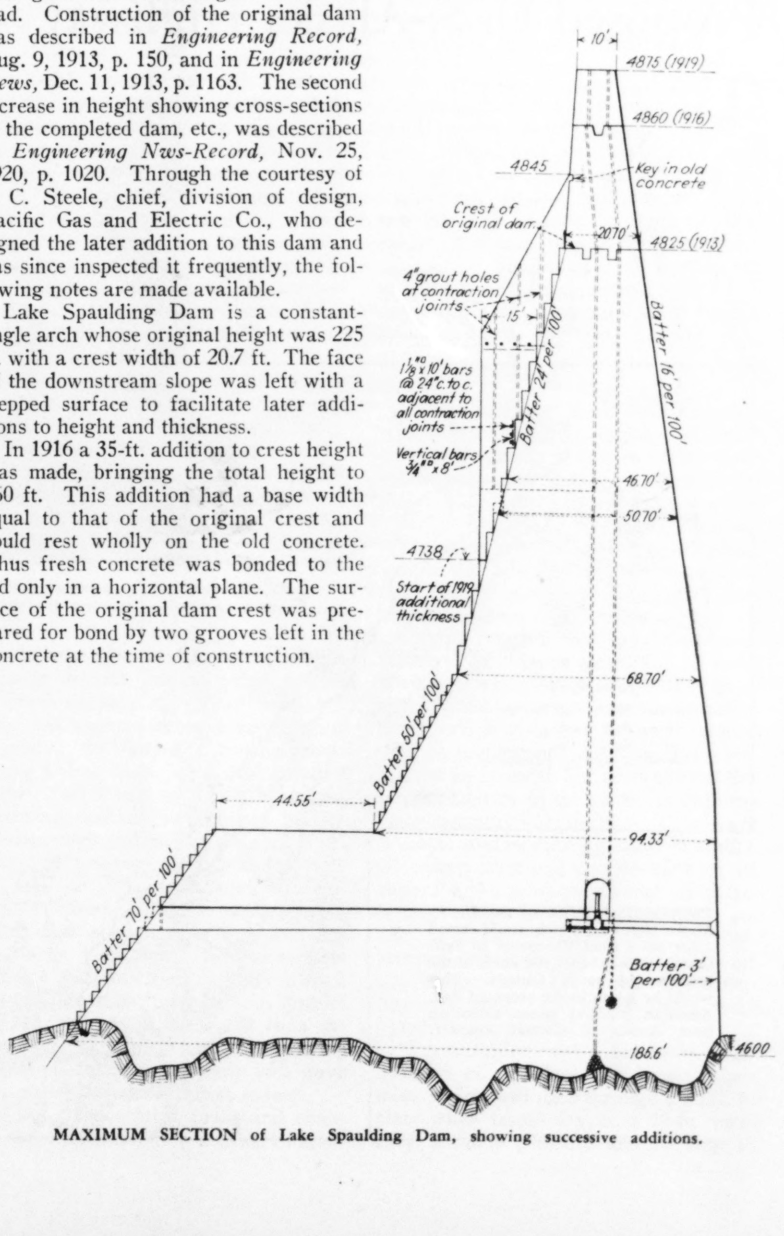
Lake Spaulding arch dam in California, built 225 ft. high in 1913, raised 35 ft. in 1916, 15 ft. more in 1919, shows only slight leakage along vertical and horizontal joints

QUESTION about the watertightness of bond between new and old concrete when additional thickness and height are added to a large dam has focused attention on dams to which such additions have been made. One of these which affords a particularly good example is the Lake Spaulding Dam on the south fork of the Yuba River in California. This structure has been raised twice, and in the sixteen years since additional thickness was added, frequently has undergone extensive changes in the water load. Construction of the original dam was described in *Engineering Record,* Aug. 9, 1913, p. 150, and in *Engineering News,* Dec. 11, 1913, p. 1163. The second increase in height showing cross-sections of the completed dam, etc., was described in *Engineering Nws-Record,* Nov. 25, 1920, p. 1020. Through the courtesy of I. C. Steele, chief, division of design, Pacific Gas and Electric Co., who designed the later addition to this dam and has since inspected it frequently, the following notes are made available.

Lake Spaulding Dam is a constant-angle arch whose original height was 225 ft. with a crest width of 20.7 ft. The face of the downstream slope was left with a stepped surface to facilitate later additions to height and thickness.

In 1916 a 35-ft. addition to crest height was made, bringing the total height to 260 ft. This addition had a base width equal to that of the original crest and could rest wholly on the old concrete. Thus fresh concrete was bonded to the old only in a horizontal plane. The surface of the original dam crest was prepared for bond by two grooves left in the concrete at the time of construction.

In 1919 the height again was increased, this time by a 15-ft. addition, bringing the total height to 275 ft. In this second addition it was necessary to increase the thickness of the arch ring. This was done over a total height of 107 ft. on the downstream face of the dam. This increase or addition to the arch had a maximum thickness of 15 ft. at a level 25 ft. below the original crest and was 3 ft. wide at the lower edge of the addition, 89 ft. below original crest level.



MAXIMUM SECTION of Lake Spaulding Dam, showing successive additions.

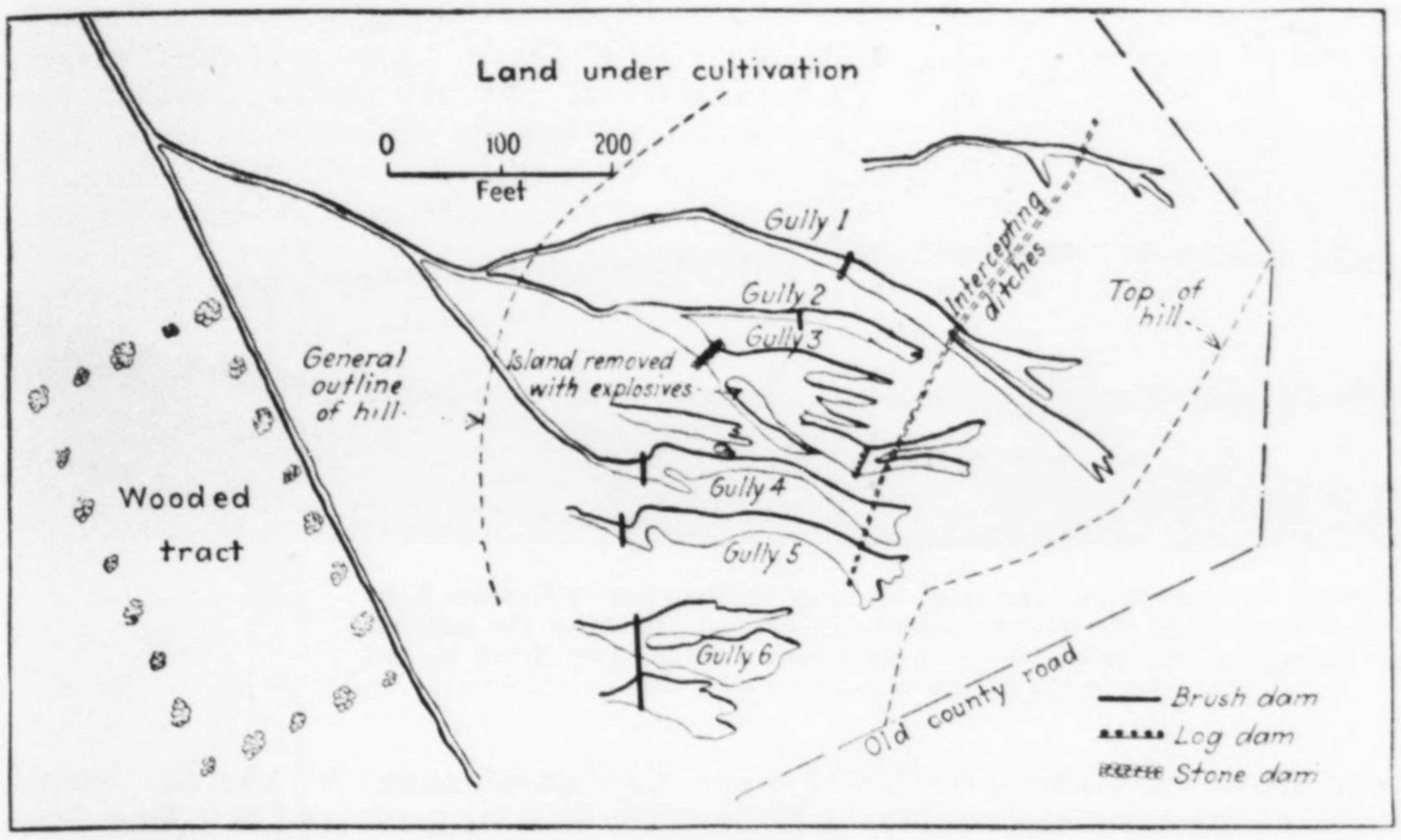


FIG. 4—MAP of a typical case of gully-erosion treatment. Several gullies are drained into one by means of intercepting ditches and a check dam built at the throat. Log dams are placed near lower ends of the other gullies.

This typical project lies in the basin of the Tradewater River and contains bottomland, "high bottomland" and hilly land. The large gullies shown in the map, Fig. 4, have developed in the last fifteen years solely through neglect and poor farming methods. The slope of the land averages 14 per cent. The deepest gully is about 13 ft., the remaining ones average 7 ft. in depth.

Intercepting ditches were installed to drain most of the gullies into gully No. 3, as shown in Fig. 4. These ditches are 1 ft. deep, 4 ft. wide at the bottom and have side slopes of 1½:1 on the low side and 3:1 on the high side. The excavation was cast on the low side to form a dike. The gradient on the bottom is 0.5 per cent, which has been demonstrated to be the grade below which erosion is negligible. At the junction of these ditches with gully 3, the bottom and slopes have been paved with riprap for protection. This riprap protection takes care of a 4-ft. drop from the ditches to the flow line of gully 3. The ditches enter gully 3 just above a substantial log dam 4½ ft. high. Considerably below the log dam a stone dam has been constructed in the throat of gully No. 3 (Fig. 2). This structure is 6 ft. high and 30 ft. long at the crest level, and is built of hand-placed loose stone masonry laid without mortar. The dam is backfilled with earth, and after very heavy rains it has proved to be watertight. A section of this structure is shown in Fig. 1.

Near the lower end of each of the remaining gullies was built either a log or a brush dam to retain material that may creep down after the banks were sloped for planting. With flow intercepted at the heads, this is felt to be ample treatment. Vegetation is assumed to hold the banks sufficiently before these silting basins have been filled.

The banks of each gully were graded down to permit seeding and planting. At first this bank grading was done entirely by hand, but was subsequently aided by the use of plows and scrapers to speed up the work. Applications from land owners far exceed the possible number of projects that can be completed with present forces, so equipment was introduced to make the work as widespread as possible.

On these projects the land owner does not relinquish title to the treated area, but the Commonwealth of Kentucky reserves the right to the timber grown on the reclaimed tracts.

Organization and personnel

There are about 200 men of the Civilian Conservation Corps in each erosion-control camp. About 150 workers in each camp are available for field work. These men are divided into work parties of about 25 men each, working under the direction of a forester foreman of the Kentucky State Forest Service.

All work on the project is under the general direction of W. E. Jackson, Jr., state forester and collaborator, U.S. Forest Service. All technical details of laying out work, inspecting construction and record mapping are handled by the writer.

Three New Bridges for Cape Cod Canal

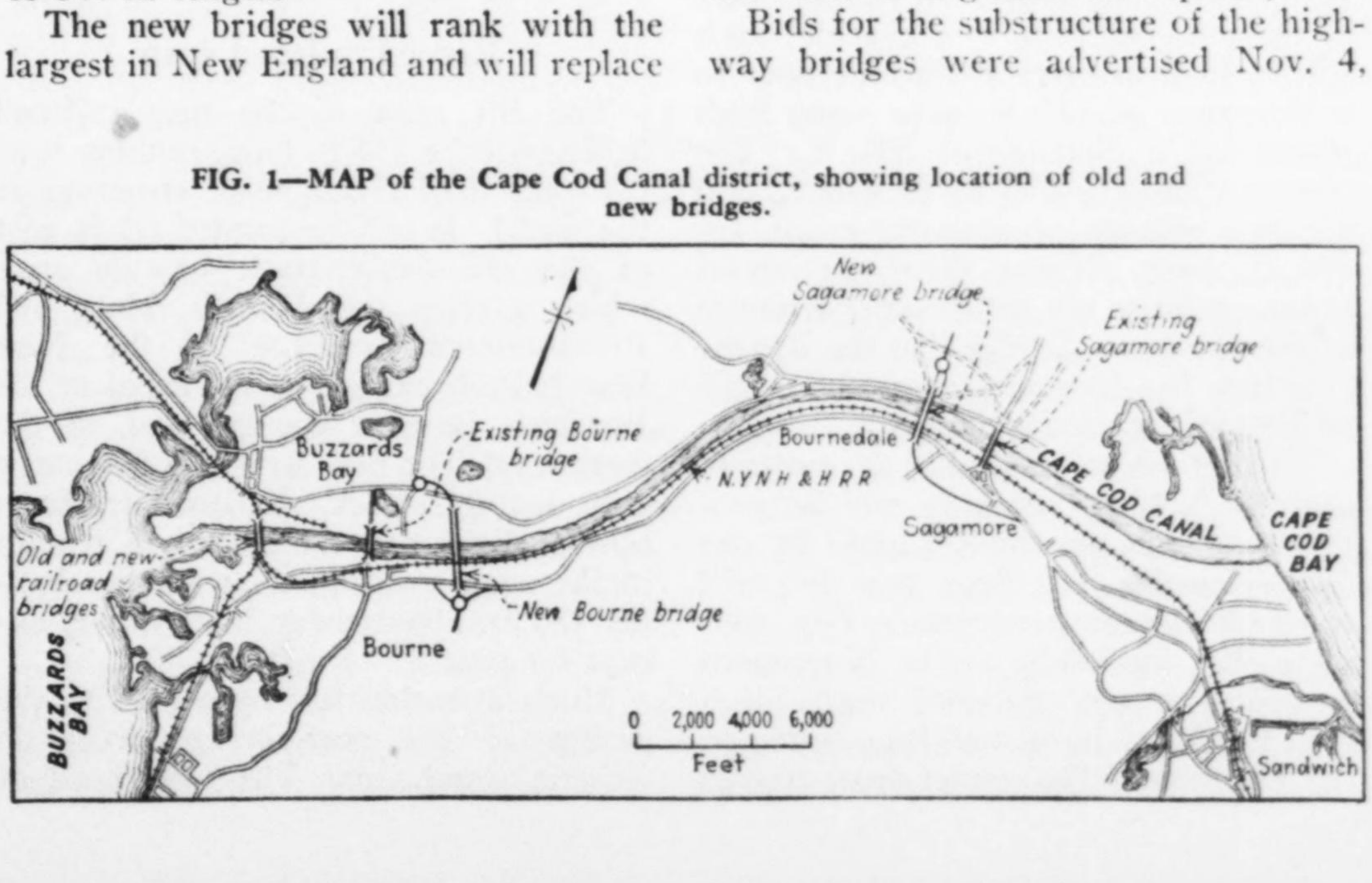
Start of construction follows close upon PWA allotment for two high-level highway crossings of three-span continuous type and a vertical-lift railroad span of record length

UNDER an appropriation to the U. S. Army Engineers from PWA funds, three large bridges are under construction across Cape Cod Canal in southeastern Massachusetts, two of them fixed continuous-truss highway crossings that will clear the canal 135 ft., and the other a vertical-lift railroad structure with a record lift span of 544-ft. length.

The new bridges will rank with the largest in New England and will replace three low-level structures that now hamper navigation, rail and highway traffic alike. An allotment of $4,600,000 was made on Sept. 14, 1933. Contract for design was at once let to private engineers, and the basic plans were rushed to completion. These were approved by the Boston district engineer, the North Atlantic division engineer, and the Chief of Engineers on Sept. 24, 1933.

Bids for the substructure of the highway bridges were advertised Nov. 4,

FIG. 1—MAP of the Cape Cod Canal district, showing location of old and new bridges.



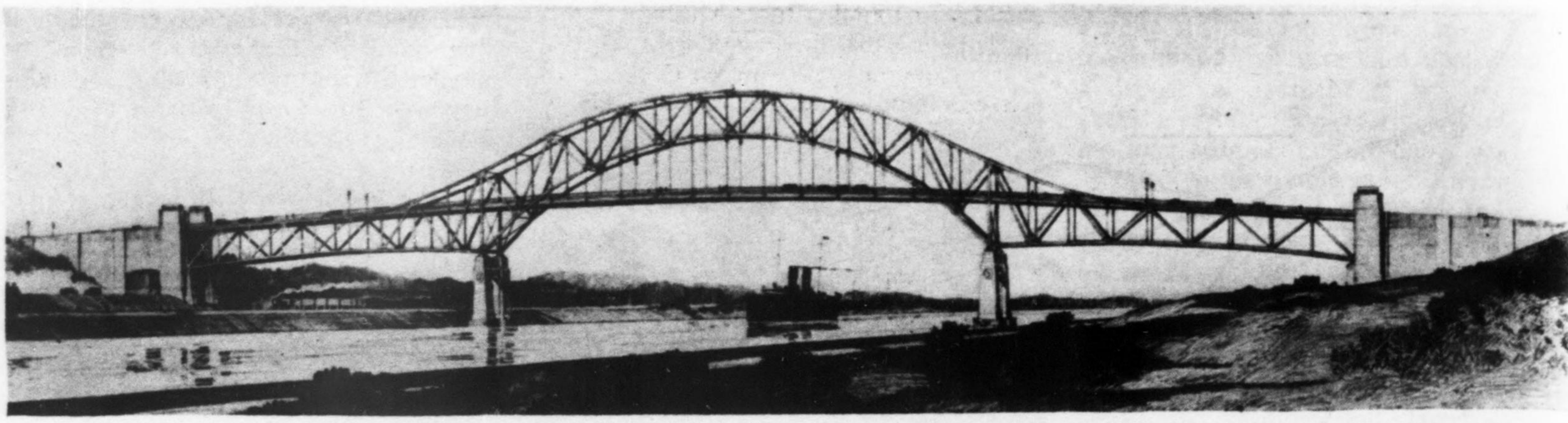


FIG. 2—ARTIST'S DRAWING of one of the two new highway bridges that will span Cape Cod Canal. This is the Sagamore Bridge; the Bourne structure is identical except for the addition of side spans. The main crossing is a three-span continuous truss with a center span of 616 ft. and flanking spans of 396 ft.

opened Nov. 20, and contract was awarded Nov. 27. Proposals for the railroad bridge foundations were advertised on Nov. 6, opened Nov. 29, and contract was awarded Dec. 5. The construction schedules call for all foundations to be ready for steel erection early this spring. Contract for the steel superstructure of the two highway bridges was awarded Jan. 6. Bids for the railroad bridge superstructure will be advertised shortly.

Highway bridges of graceful design

The two highway bridges will be located in the villages of Bourne and Sagamore, toward the western and eastern ends of the canal, respectively, and about 3 miles apart. The Bourne Bridge will be 3,235 ft. east of an existing bridge that it is to replace, and the Sagamore Bridge will be located 2,500 ft. west of a present bridge which it will replace. Both of the old structures are double-leaf bascule spans, the span length being too short adequately to pass the increasing number of boats now using the canal. Furthermore, they are low-level structures, causing frequent interference with highway traffic because of the numerous openings required.

For the main spans across the canal the two new bridges are identical in size and design. Though designed as a three-span continuous truss, the center span of the crossing rises in a graceful arch 270 ft. high to carry the bridge floor to a clearance of 135 ft. over mean high water for a distance of 500 ft. The center spans are 616 ft. in length, and the two adjacent continuous spans are 396 ft. long. These three continuous spans comprise the entire superstructure at the Sagamore Bridge, but the Bourne structure has four additional side spans of simple deck-truss design.

The trusses are spaced 51 ft. center to center. A 40-ft. roadway will be provided on each structure, flanked by one 6-ft. sidewalk. The floor structure will consist of I-beams carrying a 7-in. concrete slab, topped by a 2-in. bituminous wearing surface. Stepped curbs of a total height of 16 in. will keep traffic on the roadway. The center-span trusses rise above the roadway level, and over most of this span the roadway will be suspended by prestressed cables instead of rigid hangers. The entire roadway will be lighted, and an airway beacon will be provided on top of the center span at the highest point. The structures are designed for a 20-ton truck loading.

The four main-channel piers, two for each bridge, are identical in design and size except for a slight variation in height. The bases of the piers will be 108x50 ft. in plan and 25 ft. in thickness, resting on a compact gravel formation at an elevation from 52 to 54 ft. below sea level. Upon each monolithic base section will rise two shafts at each pier, each shaft being 32x32 ft. in plan at the bottom and tapering at the top. The upper 71 ft. of each shaft will be hollow. A granite facing will be placed around the shafts at water line.

All four abutments are hollow U-shaped structures with thin reinforced-concrete walls. A concrete T-beam deck will carry the roadway over each abutment. Because of the similarity of substructure on both highway bridges, bids were taken for both jobs together as one contract on a lump-sum basis. The only unit price asked for was that for extra concrete, in case the footings of any of the structures were carried below the depth originally contemplated, the cost of concrete to include the cost of extra excavation and shoring.

Record railroad span

The lift span of the new railroad bridge will be 544 ft. long, ranking it as the longest of this type of structure in the world. It will be located 60 ft. east of the old double-track bascule span which carries the Wood's Hole and Provincetown branches of the New Haven Railroad across the canal at the Buzzards Bay, or western end, of the waterway. The new structure will carry only a single track. Railroad traffic is now so light that the span will remain in the open position, clearing the channel 135 ft. above mean high water, except for passage of a train.

Much attention has been paid to the design of the new bridge from an esthetic standpoint. The top chord of the main span is slightly curved. The flanking spans are 128 ft. long. Some consideration was given to towers of reinforced concrete, but final plans designate the use of steel. Omission of all redundant members adds to the appearance. All operating machinery will be located in the towers, leaving the lift span free of an unsightly operating house. Submarine cables will take the place of the usual overhead wires across the span.

Two electric motors at the top of each tower will provide the operating power for the lift span. These will be geared to counterweight rope sheaves. Two 10-in. rope sheaves will be placed at each corner of the span. If the usual bronze bearings are used in all sheaves, the size of the motors will be 250 hp. Consideration is being given to the use of roller bearings throughout, capable of handling 300-ton loads each, which would reduce the size of the motors to 150 hp. The unbalanced weight of the counterweight ropes will be compensated by auxiliary counterweights hanging above the main counterweights. A storage battery and an auxiliary gas-engine generator unit will be installed for emergency operation.

The towers will rest on masonry piers, each having a monolithic base 44x88 ft. in plan, 25 ft. thick, with bottom resting at an elevation 62 ft. below mean sea level. The pier shafts, 44 ft. high, are 18x68 ft. at bridge seat level, tapering outward on a ½-in. to 1-ft. batter toward the base. Because of their depth it is expected that the construction of the railroad bridge main piers will be the most difficult part of construction of all three projects. The abutments for the railroad bridge rest on bases 46x22 ft. in plan, carried to an elevation 20 ft. below sea level. Granite facing is provided at water line on abutments and piers.

Canal improvements

The Cape Cod Canal, opened in 1914, has had a long and interesting history. It was considered for more than two centuries before actual construction began. The first official recognition of the need for the canal was in 1697, when the General Court of Massachusetts ordered a report made. Work was actually started on the cut in 1883, when about a mile of canal was excavated. In 1899 a charter was granted to the Boston, New York and Cape Cod Canal Co., with amendments to this charter being made in 1900 and 1910. Construction was again commenced June 19, 1909, utilizing the previous excavation. Removal of 15,000,000 yd. of material was required be-

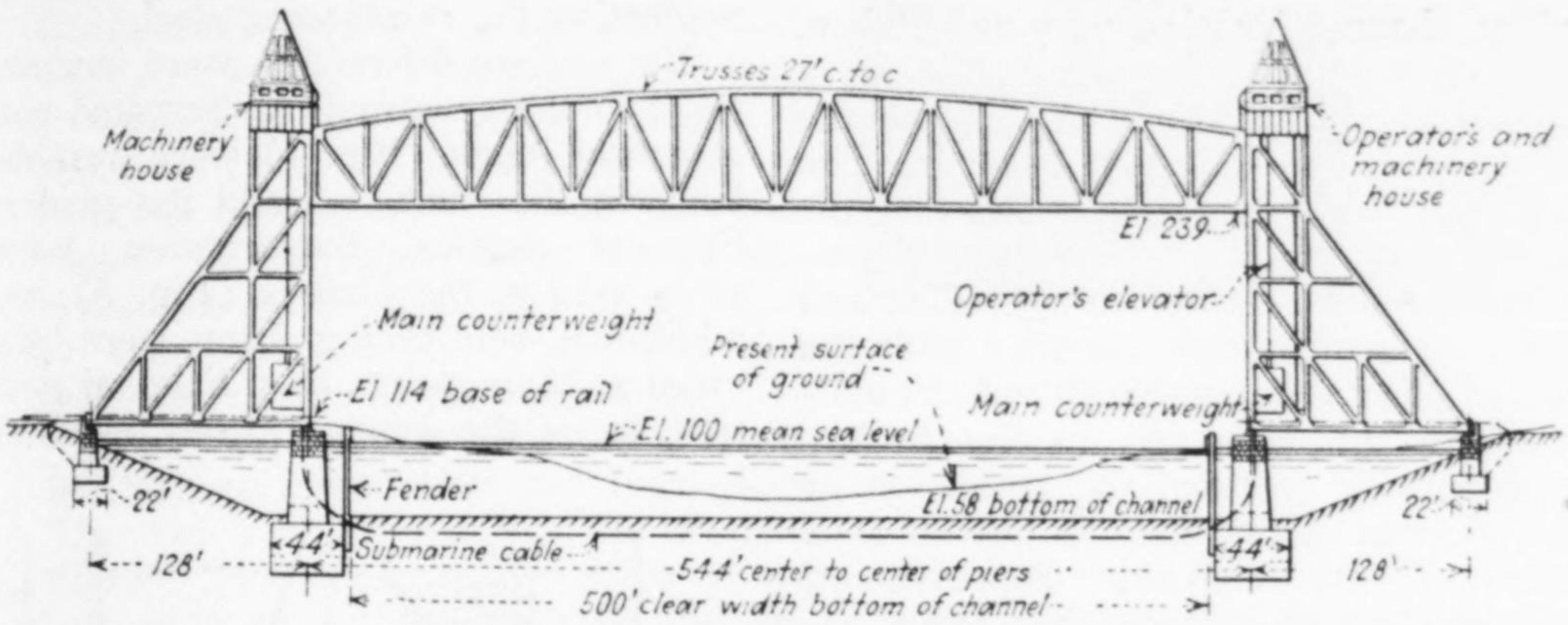


FIG. 3—THE RAILROAD BRIDGE over the canal will be the longest vertical-lift span yet built, 544 ft. c. to c. of piers.

fore the job was finished. The site of the canal is a glacial terminal morain, consisting of sand, gravel and granite boulders.

The original construction provided a waterway 7.7 miles long across land and an additional 5.3 miles of sea channel, with a minimum bottom width of 100 ft. and a minimum depth of 25 ft. The canal was purchased by the federal government under the Rivers and Harbors Act of Jan. 21, 1927, at a capital cost of $11,500,000. The government took possession March 31, 1928. Traffic through the canal has increased rapidly in the past few years, now amounting to 10,000,000 gross tons of shipping annually.

Variable and non-synchronous tides exist at each end of the canal. At the Cape Cod Bay end the mean tidal range is 8.9 ft., and at the Buzzards Bay end the range is 3.6 ft. This creates a reversing current of 4 to 7 miles per hour velocity. Because of this tidal current and the narrowness of the waterway, the canal has long been a one-way channel; that is, there is no room for large vessels to pass within the land section. The federal government is now widening the canal to 170 ft., with a minimum depth of 25 ft. Half of this widening operation has been completed, and the other half is under way at present. Before the canal without locks can be used as a two-way waterway for larger vessels, it will be necessary to widen it out to at least 300 ft. and deepen it to 30 ft. The government hopes to start widening and deepening to 250 ft. in the near future. A logical subsequent step in improving the canal would be to widen it to 400 ft. and possibly to install twin locks at one end 110 ft. wide and 1,000 ft. long, with 40 ft. of water over the sills. Opinions differ as to whether locks would be necessary if the canal were widened to 400 ft. The widening will increase the tidal current, but with more room available, boats might be able to navigate safely. If so, the trouble and expense of installing and operating the locks would be unnecessary.

The present 170-ft. widening program was adopted Aug. 20, 1932. An allotment of $277,000 was made to widen the canal to 170 ft. from the east entrance to Bournedale. Subsequently a second allotment of $316,500 to continue dredging operations and remove boulders from the channel between Bournedale and the western end of the canal was made. This work is now in progress.

Old bridges

At the time of construction of the existing bridges, widening the canal was evidently considered, for the channel draw spans were built 160 ft. from center to center of pier, although the canal is only 100 ft. wide, and pier construction permitted deepening to 30 ft. The present need for widening and the desirable elimination of troublesome delays to all traffic by operation of the old bridges prompted the government to go ahead with the new bridge program. Since it is not known at this time what the future development of the canal will be, provisions are being made in the new structures for a possible 500-ft. width of canal and a depth of 40 ft.

The old railroad bridge is an electrically operated, single-leaf trunnion bascule, clearing mean sea level by only 12 ft. The substructure consists of three piers only, the approaches being pile trestles. The piers were founded on wooden piles. The structure carries two tracks between trusses spaced 29.6 ft. c. to c.

The two existing highway bridges are double-leaf bascules, 160 ft. c. to c. of piers, and are of the Scherzer rolling-lift type. The piers were not faced with stone, but creosoted form lumber was left on the exposed faces at water line, which has proved to be adequate protection. The piers are founded directly on gravel, and no piles were used. They were constructed by the pneumatic-caisson method.

Besides the 160-ft. main span at Bourne, this structure contains two 90-ft. truss side spans and three additional 40-ft. and six 38-ft. flanking spans. The Sagamore Bridge was similar to the Bourne structure, except for the nine additional flanking spans.

Engineers and contractors

All plans for the structures were prepared under the direction of Lieut-Col. Richard Park, district engineer, Boston district, U. S. Engineer Corps. This office will also supervise construction. Plans for the two highway structures were prepared by Fay, Spofford & Thorndike, consulting engineers, Boston. Cram & Ferguson, Boston, were consulting architects in the preparation of the highway bridge design. The railroad bridge was designed by Parsons, Klapp, Brinckerhoff & Douglas, consulting engineers, New York. McKim, Mead & White, New York, were consulting architects on the design of this structure. The contract for the substructure of both highway bridges was let to P. J. Carlin Construction Co., New York, for $1,327,700. C. W. Blakeslee & Sons, New Haven, and Blakeslee-Rollins Corp., Boston, received the contract for the substructure of the railroad bridge, amounting to $317,500. Superstructure for both highway bridges was awarded to the American Bridge Co. for $1,444,730.

New Highways Planned for Buenos Aires Province

A COMPREHENSIVE highway-construction program for the Province of Buenos Aires, Argentina, involving the expenditure over a period of five to fifteen years of 180,000,000 paper pesos was recently approved by the federal government national highway bureau. (Par value of paper peso equals $0.4245, U. S. currency.) The total amount will be distributed in more or less equal parts over three zones of the province, one being the neighborhood of the city of Buenos Aires, and the other two the northern and southern zones of the province.

The project calls for the construction of a total of 5,039 km. of highways, of which 1,921 will be paved, 2,553 improved and 785 ordinary dirt roads. The provincial government at the same time will undertake to construct roads totaling 20,000 km., consisting chiefly of short roads giving access to various towns and to main roads.

Congestion now existing on roads giving access to the city of Buenos Aires will be relieved by the construction of new highways in this area. Work on the Avenida General Paz will be completed, for which purpose 40 per cent of the proceeds of the gasoline tax in the capital will be appropriated.

ENGINEERING NEWS-RECORD

February 28, 1935

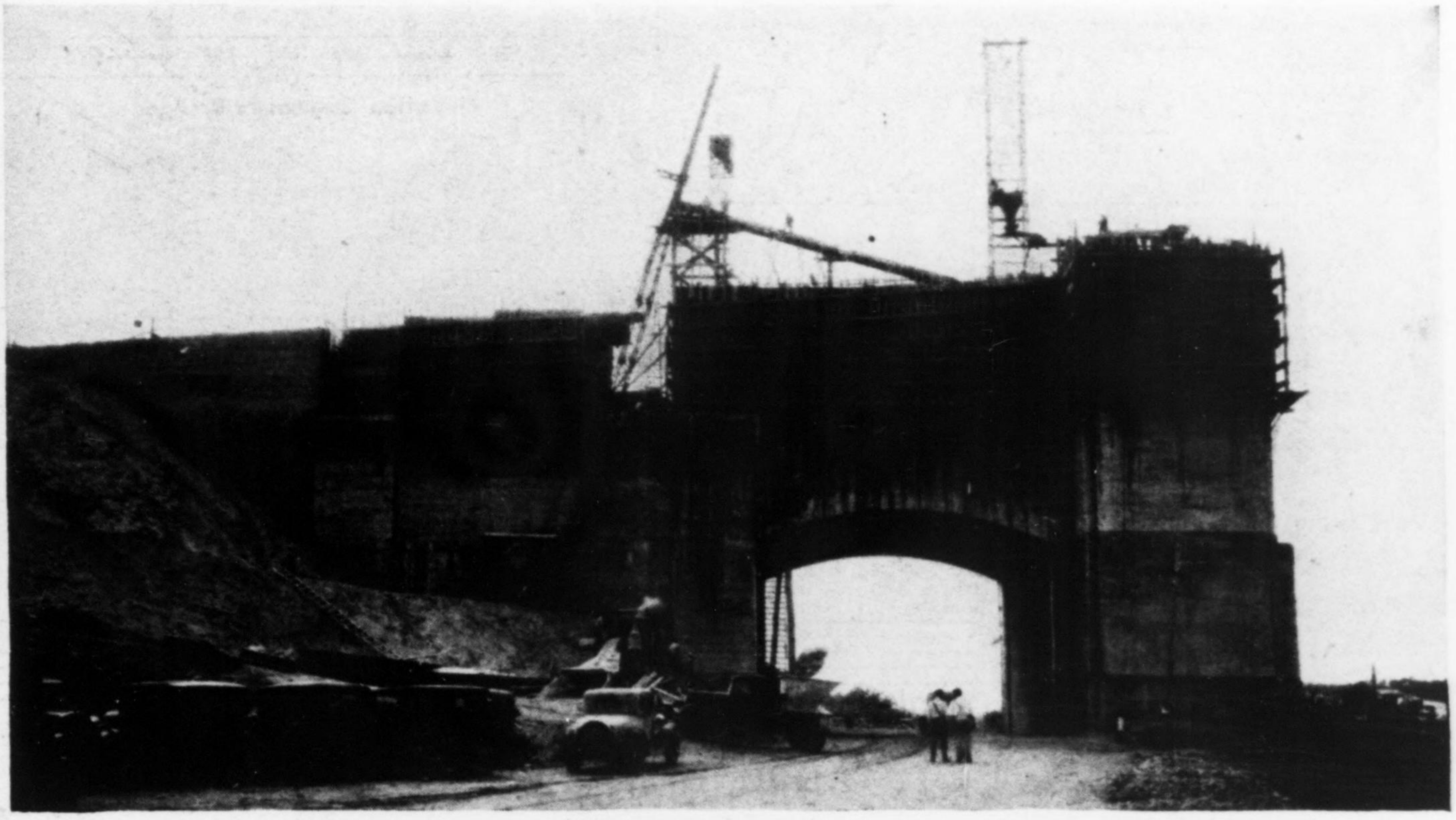


FIG. 1—CONCRETING the South Sagamore abutment with the plant units typical of the job—small tip-over-body trucks, paving mixers and a crawler crane to hoist the bottom-dump buckets. The elevator tower was required for the high pylons. Note how crane is set on an embankment to increase its reach.

Planting Bridge-Pier Work for Efficient Concreting

Demands for seal pours of 133 yd. per hour and irregular and small pours thereafter met by batching plants and pavers on a project involving two bridges over the Cape Cod Canal four miles apart

A FUNDAMENTAL QUESTION confronting the contractor who was awarded the substructure contract for the two highway bridges being built across the Cape Code Canal in Massachusetts was: What type of concreting facilities will most efficiently and economically meet the unusual demands of a 70,000-cu.yd. job involving a dozen land piers and abutments, some of which extend 100 ft. above the ground surface, on two bridges located 4 miles apart and where 4,300-cu.yd. continuous seal pours for each of the four piers at the water's edge must be made at an average rate of 133 cu.yd. per hour. Also two types of concrete, differing in mix proportions and in size of aggregates, might at times have to be furnished simultaneously. Two alternatives were studied: (1) a large central mixing plant with a fleet of agitator trucks and (2) several batching plants with small tip-over-body trucks delivering dry materials to mixers at the various pier sites. A theoretical economy was shown for the latter plan, and it was adopted. Paving mixers were chosen because of their mobility. For the same reason crawler cranes with 95-ft. booms were used to handle concrete into the high piers and abutments, although these had to be supplemented by hoist towers in one or two instances. With the work now completed, the contractor is well satisfied with the degree of efficiency obtained from this equipment and the method that was worked out for operating it.

The Cape Cod bridges (*ENR*, Jan. 25, 1934, p. 107, and Jan. 3, 1935, p. 1) for which the concreting facilities were developed are highway structures of continuous-truss type, providing a clearance of 135 ft. for canal shipping. They are located at the village of Sagamore near the east end of the canal and at the village of Bourne near the west end. Some 10,000 cu.yd. of concrete is also being supplied for the piers of the new railroad bridge at Buzzards Bay, but this is under a subcontract and did not figure in the original design of the concreting facilities.

Of the four possible locations for batching plants — North Sagamore, South Sagamore, North Bourne and South Bourne—plants were installed at all but North Sagamore, where there were no railroad facilities for receiving aggregate and cement.

At South Sagamore and South Bourne, sidings from the railroad that

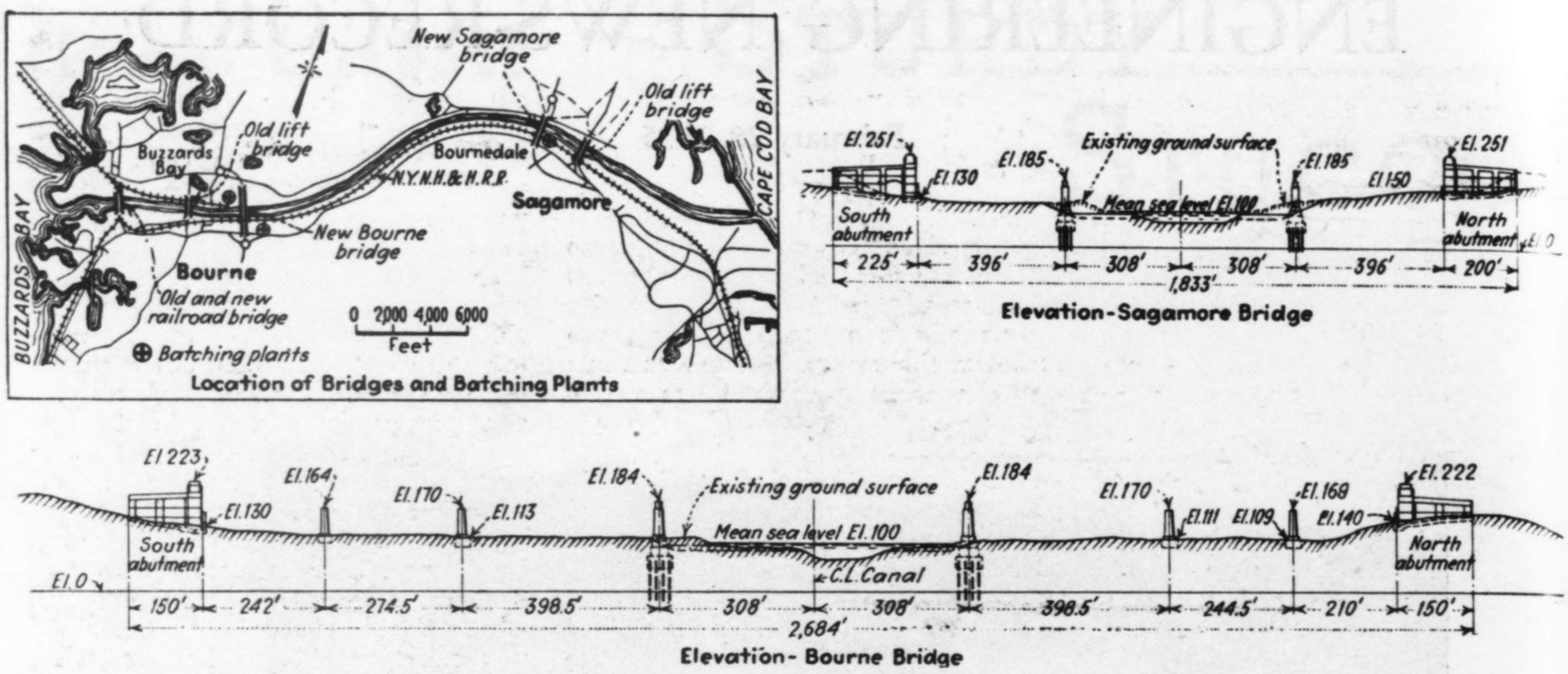


FIG. 2—THE TWO BRIDGES four miles apart, as shown on the map, presented eight piers and four abutments for concreting, as shown on the elevations.

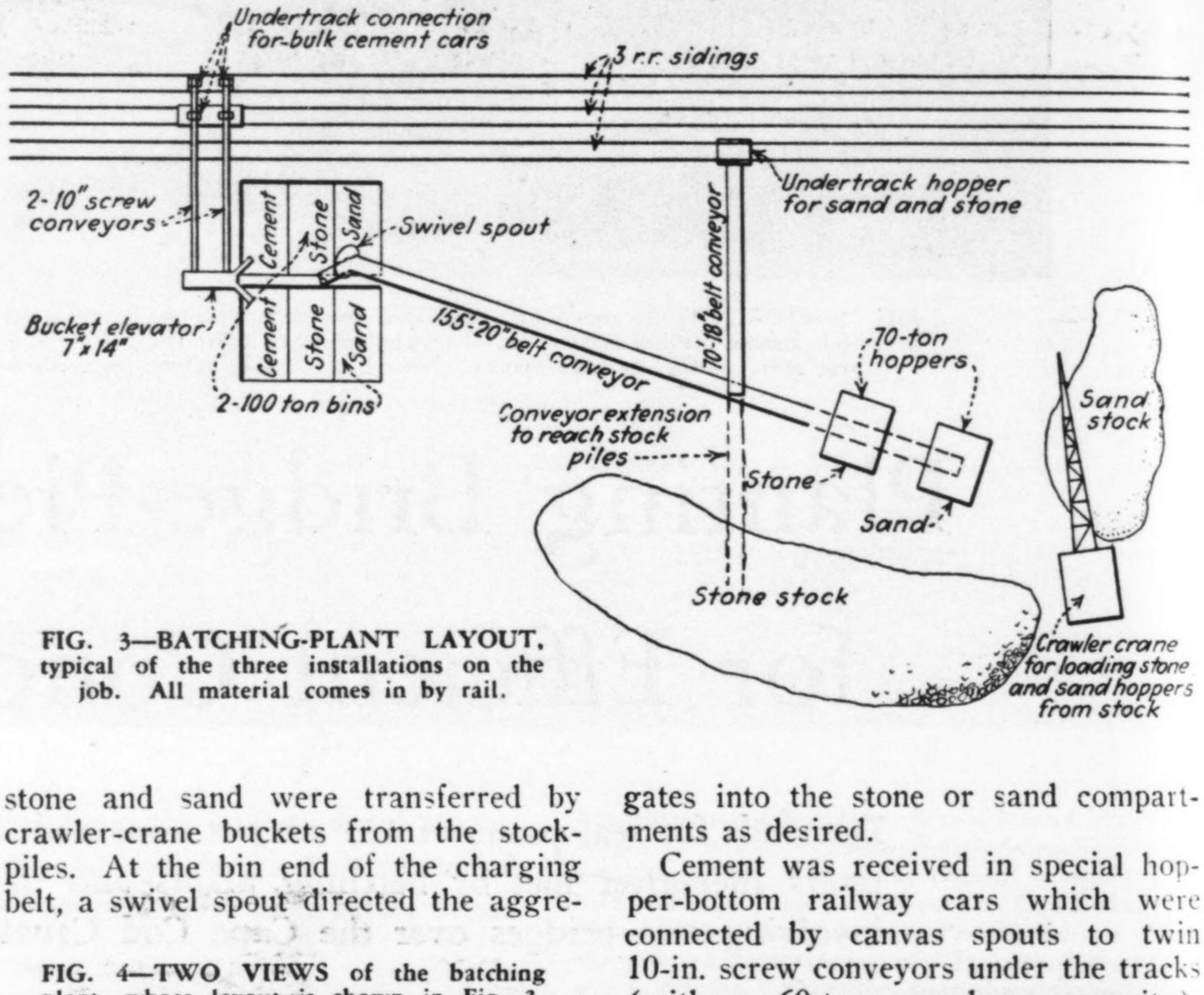


FIG. 3—BATCHING-PLANT LAYOUT, typical of the three installations on the job. All material comes in by rail.

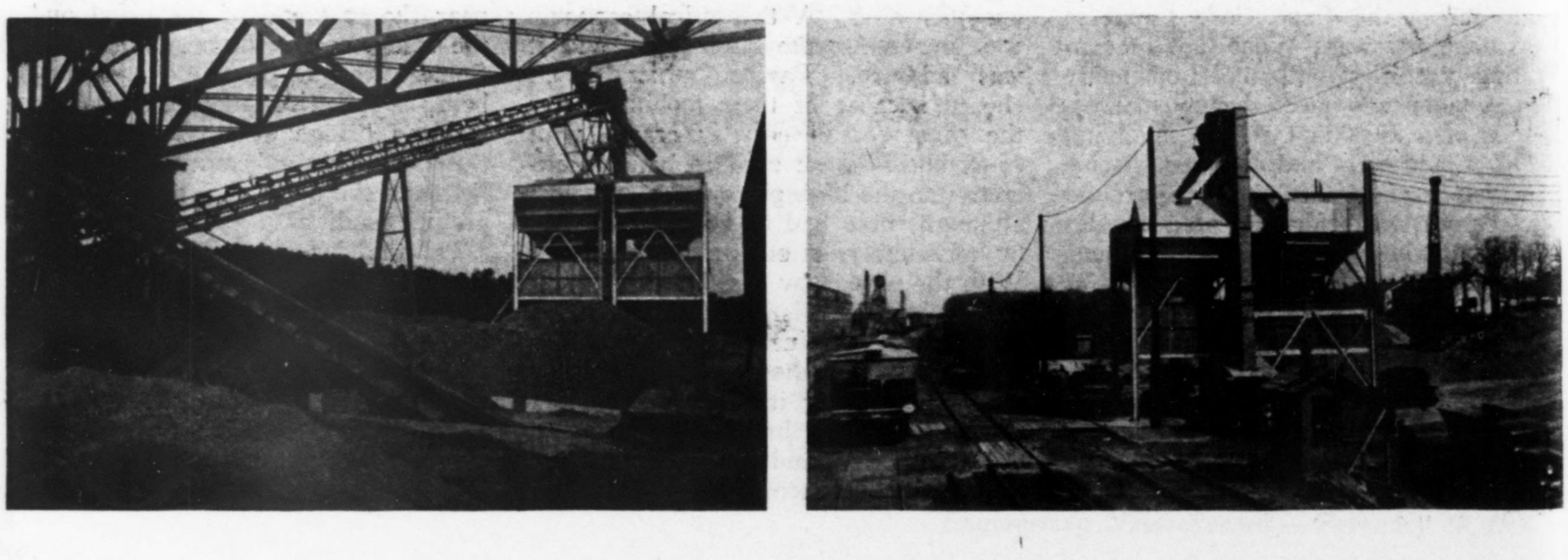
runs along the canal at these points were available. At North Bourne a mile of branch track was laid from Buzzards Bay to the batching-plant site. By placing a batching plant near each main pier, speed in handling materials to the mixers for the 4,300-yd. seal pours was assured, and the number of trucks required was reduced.

Batching plants

Each of the three batching plants followed the same general pattern, a typical layout of which is shown in Fig. 3. The two 100-ton bins were of three-compartment type, holding 40 tons of stone, 35 tons of sand and 200 bbl. of cement. A controlling reason for using two bins at each plant was the necessity at times of furnishing different mixes for Class A or B concrete simultaneously. Sand and stone, dumped into undertrack hoppers, were transported to stockpiles by an 18-in. electrically operated belt conveyor. Or this belt conveyor could be connected with the 20-in. belt conveyor that charged the bins. The charging belt also ran beneath two 70-ton hoppers into which stone and sand were transferred by crawler-crane buckets from the stockpiles. At the bin end of the charging belt, a swivel spout directed the aggregates into the stone or sand compartments as desired.

Cement was received in special hopper-bottom railway cars which were connected by canvas spouts to twin 10-in. screw conveyors under the tracks (with a 60-ton per hour capacity), which discharged into a 7x14-in. bucket elevator at one side of the bins. A gate at the top of the elevator permitted dis-

FIG. 4—TWO VIEWS of the batching plant, whose layout is shown in Fig. 3. Left: aggregate-handling conveyors with railway car at extreme right and bridge superstructure overhead. Right: cement end of plant with bulk-cement cars on left-hand track.



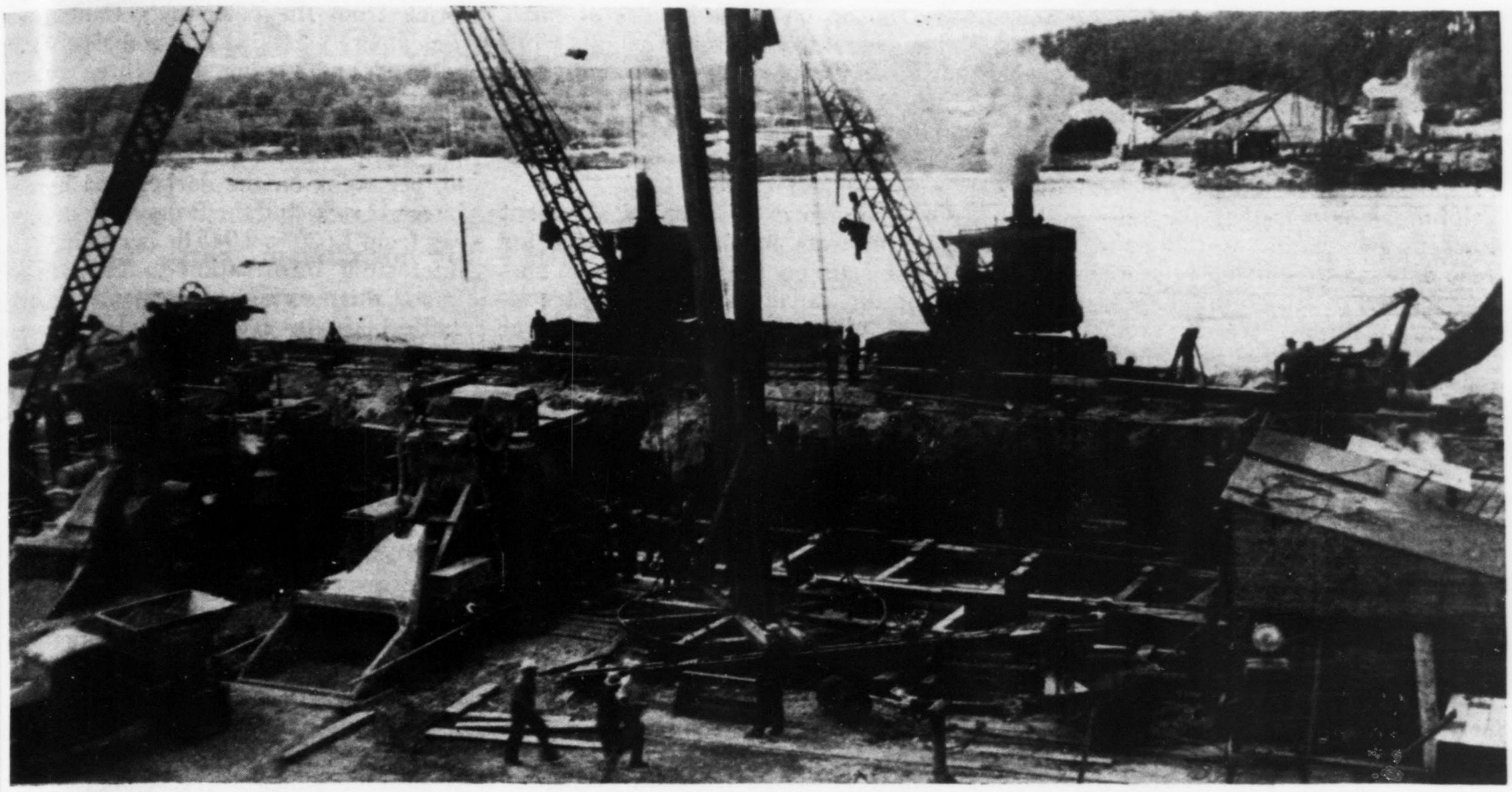


FIG. 5—BATTERY of five paving mixers making the 4,300-cu.yd. continuous pour on the seal course of the South Bourne main pier. The mixers discharged into bottom-dump buckets handled by the four derricks and cranes.

charge into the cement compartment of either bin. Such large-capacity cement-handling facilities were required because for each seal pour a total of 21 cars of cement was required in a period of about 32 hours. This is said to constitute one of the largest single shipments of bulk cement yet made in the eastern part of the country.

Batching was by weight, stone and sand going into the same weighing batcher, while a separate batcher was provided for the cement. The single-batch trucks were driven under the bins and loaded with stone, cement and sand in that order.

Seal concreting

At the main piers where the large seal pours were required a platform was built at one side of the cofferdam, to accommodate four 27-E pavers and a dual-drum mixer of the same size. Mixed concrete was chuted into the bottom-dump buckets of the subcontractor who built the bases of the main piers (Fig. 5). In making the 4,300-yd. continuous pours, two 2-yd. buckets and two 1-yd. buckets were used. For these pours seven of the single-batch trucks shuttled back and forth between the near-by batching plant and the pavers at the pier site.

This simple procedure in making the seal pours, which was used at both Bourne piers and at the South Sagamore pier, was not applicable to North Sagamore where there was no batching plant. To have trucked all of the materials in dry batches from South Sagamore around to pavers at North Sagamore would have required too large a number of trucks to meet the requirements of 133 yd. of concrete per hour for some 32 hours. Therefore, the usual procedure was supplemented by setting up a central mixing plant at South Sagamore near the batching plant, where a few dry-batch trucks would be adequate to serve it, and then to transport mixed concrete around to North Sagamore in large agitator trucks. Pavers were also set up at North Sagamore, and the remainder of the batch trucks brought dry materials to them as on the other seal pours.

The central mix plant layout was simple but effective. A convenient hillside provided an ideal location for a two-level layout. An elevated platform of timber was constructed for the charging and mixing floor. On this platform two paving mixers were set, so that they could discharge over the edge of the platform into the agitator trucks on a road at the lower level. A roadway was built to the platform level, and over this the small trucks from the batching plant brought in the dry materials for the pavers. On the seal pour for the North Sagamore pier five 4-yd. agitator trucks, six 4-yd. dry-batch trucks and twelve 1-yd. dry-batch trucks were used to transport concrete and dry materials. The round trip required about half an hour.

Pier and abutment concreting

For the work above ground, concreting demands were less severe, but the

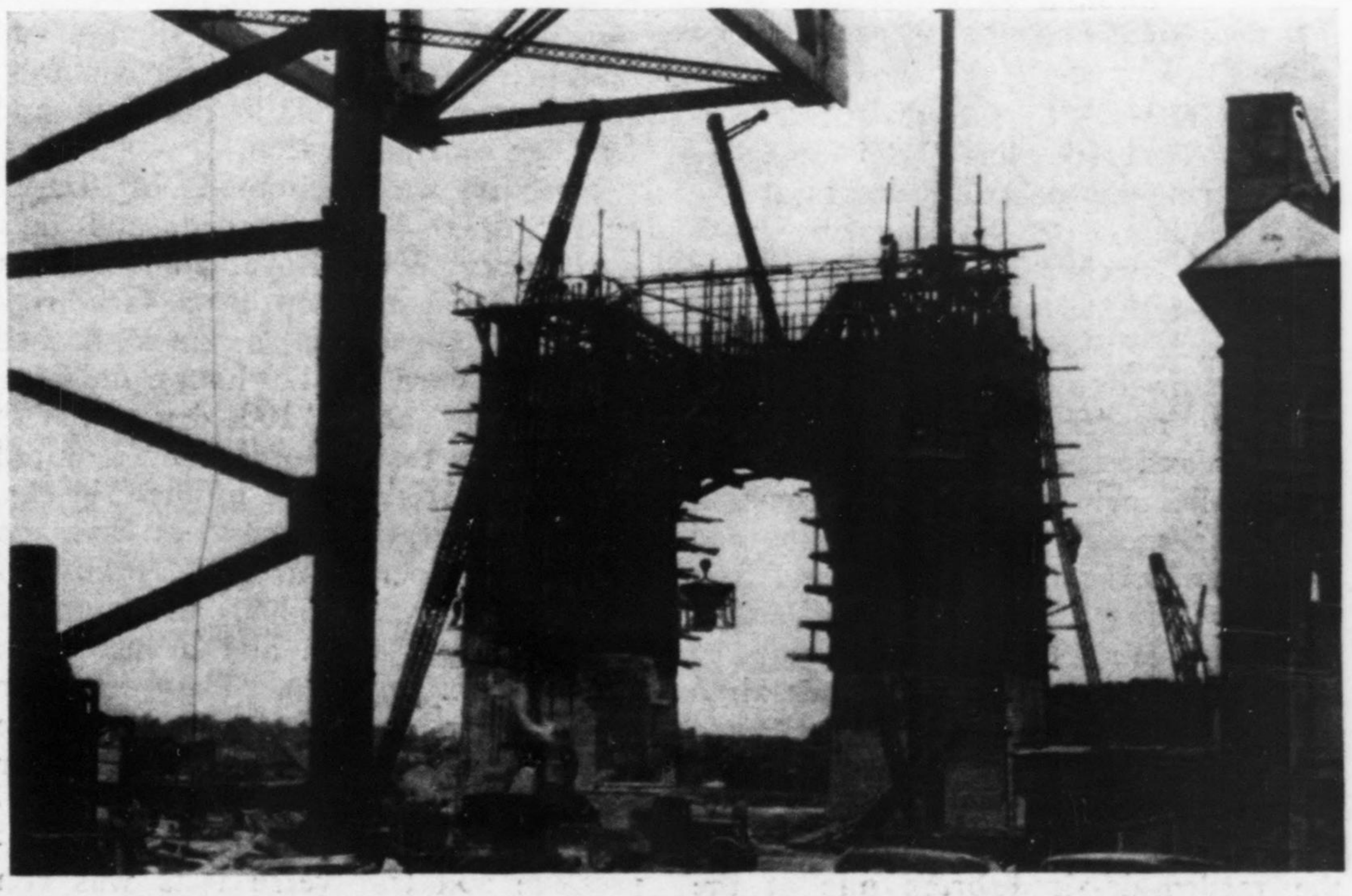


FIG. 6—CONCRETING the South Bourne main pier. The two pavers are charged by the small tip-over-body trucks that shuttle between the pier and the batching plant, a corner of which shows at the right.

handling of forms and reinforcing and the separation of operations into groups at each pier and abutment site added new complications. Mobility of the pavers and of crawler cranes provided the principal element that contributed to the success of this work. Another helpful factor was that the reinforcing steel in the shafts of the main piers (the only ones requiring reinforcement) was formed of structural shapes, on the recommendation of the contractor. This permitted the reinforcing to be set with a minimum of difficulty and hazard and also offered a substantial support from which the forms could be hung and adjusted.

Concreting on these piers and abutments was handled as it was for the bases of the main piers. Pavers placed near the work were charged from the batch trucks and discharged into bottom-dump buckets handled into the forms by the crawler cranes. The small trucks with tip-over bodies proved very effective in negotiating steep grades and loose-fill roads. For most of the work the 95-ft. booms of the cranes were of ample length, but on the highest parts of the piers and particularly on the pylons of the abutments the booms were too short when the cranes were at ground level. Accordingly, the cranes were run up on embankments of earth, sometimes of considerable height, while the mixer remained at ground level (Fig. 1). In one instance, however, the height was so great (about 130 ft.) that the cranes were supplemented by an elevator hoist and chutes.

On the North Sagamore abutment concreting was handled as it was for the seal pour on the main pier on this side of the canal. The agitator trucks charged with concrete at the central plant at South Sagamore delivered to cranes and buckets at the site.

Administration

The Cape Cod Canal bridges are being built under the supervision of the Corps of Engineers, of which Lt.-Col. John J. Kingman is Boston district engineer. Designs for the highway bridges were prepared and construction is being supervised by Fay, Spofford & Thorndike, consulting engineers, Boston. The contractor for the substructure work, for which the concreting facilities described in this article were developed, is the P. J. Carlin Construction Co., New York; J. P. Carlin is president, Samuel Kent chief engineer, Sylvia Bonelli superintendent, M. T. Staples resident engineer, and Paul Jones expediter.

Properties of De-Aired Brick Revealed by Tests

Investigations reported to National Paving Brick Association indicate greater strength and toughness and less absorption and abrasion for de-aired brick compared to ordinary paving brick

By R. B. Keplinger
Metropolitan Paving Brick Co., Canton, Ohio

EQUIPMENT for producing de-aired brick has been installed during the past three years at five of the plants of the company with which the writer is connected. The first strength tests were made in the fall of 1932 on brick selected from those manufactured by the first de-airing extrusion machine installed at the Imperial plant. The averages ran from 12,446 lb. per sq.in. and absorption of 7.47 per cent for the lighter-burned brick, to 19,575 lb. per sq.in. and 3.66 per cent absorption for the harder-burned brick. These results may be fairly compared with brick not de-aired from the No. S6 plant, which showed an average compressive strength of 11,686 lb. per sq.in. and 4.55 per cent absorption. Some brick selected from the rattled de-aired brick pile at the Imperial plant, which was the hardest-burned, was broken into halves and the fractures examined, to select specimens showing the best interior structure and texture; these samples gave a compressive average of 17,823 lb. per sq.in. and 1.36 per cent absorption.

Selection was then made of sixteen bricks, dried but unburned, and they were taken to the Royal plant, set six courses from the top in a face-brick kiln and burned in a smoke-flashed face-brick burn for 120 hours, finishing the kiln at about 100 deg. F. less temperature than is ordinarily required for paving brick. Five of these bricks, tested compressively, gave an average of 23,774 lb. per sq.in. and 2.70 absorption, with an individual minimum of 17,875 lb. per sq.in. and a maximum of 26,062 lb. per sq.in. These tests are cited as an indication of a possible optimum point in heat treatment, for high compressive strength.

In October, 1934, a series of compressive and transverse tests was run at the engineering experiment station, Ohio State University, on de-aired brick from the company's Canton and Bessemer plants. These bricks were selected and arranged in three groups, light, medium and dark. The average crushing strengths ran from 11,540 lb. per sq.in. on a light sample to 17,800 lb. per sq.in. on the darker and harder-burned brick, and the transverse strength ran from 1,620 to 2,945 lb. per sq.in. The interesting information to be gathered from this series of tests was that while both the crushing and transverse strengths increase more or less consistently, directly as the hardness of burn, it was impossible to correlate the two tests. Quite frequently, a specimen showing high compressive strength would show a lower transverse, and vice versa. It should be added that these samples in the medium and dark ranges were considerably lower in absorption than the samples selected in 1932, showing average absorptions as low as 0.48 per cent, with an individual minimum of 0.23 per cent. This was due to higher vacuums obtained in extrusion and not to greater severity in heat treatment. Here again is a faint indication that a possible optimum vacuum point has been reached and passed for these particular properties, in certain groups.

Quite recently, Dr. Reuckel, of the brick research bureau, collaborating with Mr. Litehiser, head of the Ohio state highway testing laboratory, has begun an investigation into the tensile and compressive strength of paving brick, both de-aired and otherwise, together with rattler and other possible abrasion tests. A glance over the results as tabulated (*ENR,* Feb. 14, 1935, p. 293) indicates that apparently some manufacturers have not standardized de-airing operations sufficiently. Second, there is apparently no possible correlation of rattler tests with flexure tests. And, finally, not only did the highest flexure test, both average and maximum, occur on an unde-aired product but also this same product, owing to some laminated samples, showed a 26.55 per cent loss when subjected to the standard rattler test.

Three de-aired paving-brick samples, ranging from medium to hard-burned, tested in the Deval rattler gave an average of 3.15 per cent wear and an average French coefficient of 12.77. Cores from similar samples tested on the Page impact machine gave an average toughness value of 12.33. Another test, using the Deval rattler and two samples of de-aired paving brick, resulted in an average of 2.78 per cent of wear and 14.45 French coefficient. Standard specifications for granite block require for heavy traffic a percentage of wear of not more than 3.6, a French coefficient of wear not less than 11 and toughness not less than 9.

These tests were made on selected bricks and over a very short burning range, so that they are far from being representative of a commercial shipment.

Weighing Bridge Reactions With Proving Rings

Instruments familiar in calibrating work on testing machines adapted to successful use for adjusting truss reactions during construction of continuous bridges

By C. M. Spofford
Hayward Professor of Civil Engineering, Massachusetts Institute of Technology, Boston;
and C. H. Gibbons
Baldwin-Southwark Corp., Philadelphia, Pa.

IT HAS LONG BEEN recognized that a definite knowledge of the actual pier reactions of continuous bridges would be of exceptional value to the bridge designer and constructor, since the assumed reactions are seldom if ever attained because of such things as changes in relative elevation of the piers, variation in modulus of elasticity of built-up steel members and differences in length of the various truss members as they come from the fabricating shop. Hydraulic jacks have been used to weigh reactions, but they have never been very satisfactory. Three years ago a calibrated elastic loop or proving ring was used with great success in determining the reactions of a bascule lift span. Within recent months such proving rings have been used on the new continuous-truss bridges over Little Bay in New Hampshire (*ENR*, Sept. 27, 1934, p. 387) and over the Cape Cod Canal in Massachusetts (*ENR*, Jan. 25, 1934, p. 107), and have proved to be wholly satisfactory and highly accurate. This article describes the elastic loop, whose most familiar field of usefulness heretofore has been in calibrating laboratory testing machines and presents the technique of its application to the problem of the continuous bridge.

Advantages of continuity

Bridges in which the trusses or girders are continuous over two or more spans have not been built extensively in the United States probably because, being statically indeterminate, complicated computations are required to determine the stresses in such structures and also because settlement of the piers or abutments modifies the computed stresses. In recent years, however, a number of important continuous bridges have been constructed in the United States and Canada, among which may be mentioned the Lachine Bridge over the St. Lawrence River; the Sciotoville Bridge over the Ohio River; the Bessemer & Lake Erie Bridge over the Allegheny River; the Hudson Bay Bridge over the Nelson River; the Cincinnati, New Orleans & Texas Railway Bridge over the Ohio River; the Lake Champlain Bridge; the Little Bay Bridge in New Hampshire, and the bridges across the Cape Cod Canal at Sagamore and Bourne, Mass. The Queensboro Bridge in New York City is also a continuous structure, although it is generally spoken of as a cantilever.

As compared with simple span bridges, either the cantilever or the continuous bridge is more economical for spans of considerable length and, moreover, both can be erected without the use of falsework in the main channel span, a condition that often must be met. The continuous bridge has the considerable advantage of fewer expansion joints than either a simple span or a cantilever bridge. Continuous bridges also have greater rigidity than cantilever bridges, since the maximum deflection is in general less in magnitude and occurs less rapidly. The lateral bracing can be made continuous throughout, and hinges with their troublesome details, such as are needed in a cantilever bridge, may be omitted. While the computations required for the continuous bridge are more complicated than for simple span or cantilever bridges, the methods of making them are well understood by recent graduates of our leading technical schools and offer no difficulty except that of additional labor.

With respect to the effect upon the stresses of the settlement of piers and abutments of continuous bridges, little concern need be felt unless the foundations are so bad that it would probably be unwise to construct any type of long-span bridge upon them. Moreover, most of the settlement of the foundations occurs during the construction period, as the weight of the masonry and superstructure is added. Inasmuch as the dead-weight reaction, due to the superstructure, can be established by jacking and weighing (as described in this article) after the trusses are completed, the effect of construction settlement may be accurately evaluated, and the bridge adjusted accordingly. A moderate settlement due to live loads does not greatly affect the total stress in the members of a long-span bridge, particularly a highway bridge with concrete deck.

Weighing reactions specified

In the erection of a continuous bridge, provided falsework is not used under the cantilever arms, the ends of the anchor arms generally must be jacked both longitudinally and vertically, to bring the free ends of the center span together. Since the value of the dead-load reactions is affected by the amount of this jacking, some method of establishing the desired reaction in the field is necessary. In the specifications for the Lake Champlain Bridge (*ENR*, Nov. 21, 1929, p. 796) it was required that the end dead-load reactions be established by weighing with hydraulic jacks and gages, but this method was not found to be satisfactory owing to the difficulty of determining the value of the friction of the hydraulic packing in the jacks and to inability in maintaining the hydraulic gages in accurate calibration when handling them under job conditions. Subsequently, however, proving rings or elastic lops had been used with complete satisfaction in weighing the reactions of a bascule span in New Jersey, and when the specifications for the Little Bay Bridge at Dover Point, N. H., and for the Cape Cod Canal bridges were being written, late in 1933, the engineers (Fay, Spofford & Thorndike) decided to include a paragraph requiring the accurate determination of the reactions on the three-span continuous trusses of the bridges, contemplating that proving rings could be used. This specification paragraph was as follows: "Devices satisfactory to the engineer, and of such degree of precision that the reactions applied to the trusses may be measured with an error not greater than 1 per cent, shall be used in the jacking process." The proving rings used by the contractors for establishing the reactions on all three of these bridges more than met this specification.

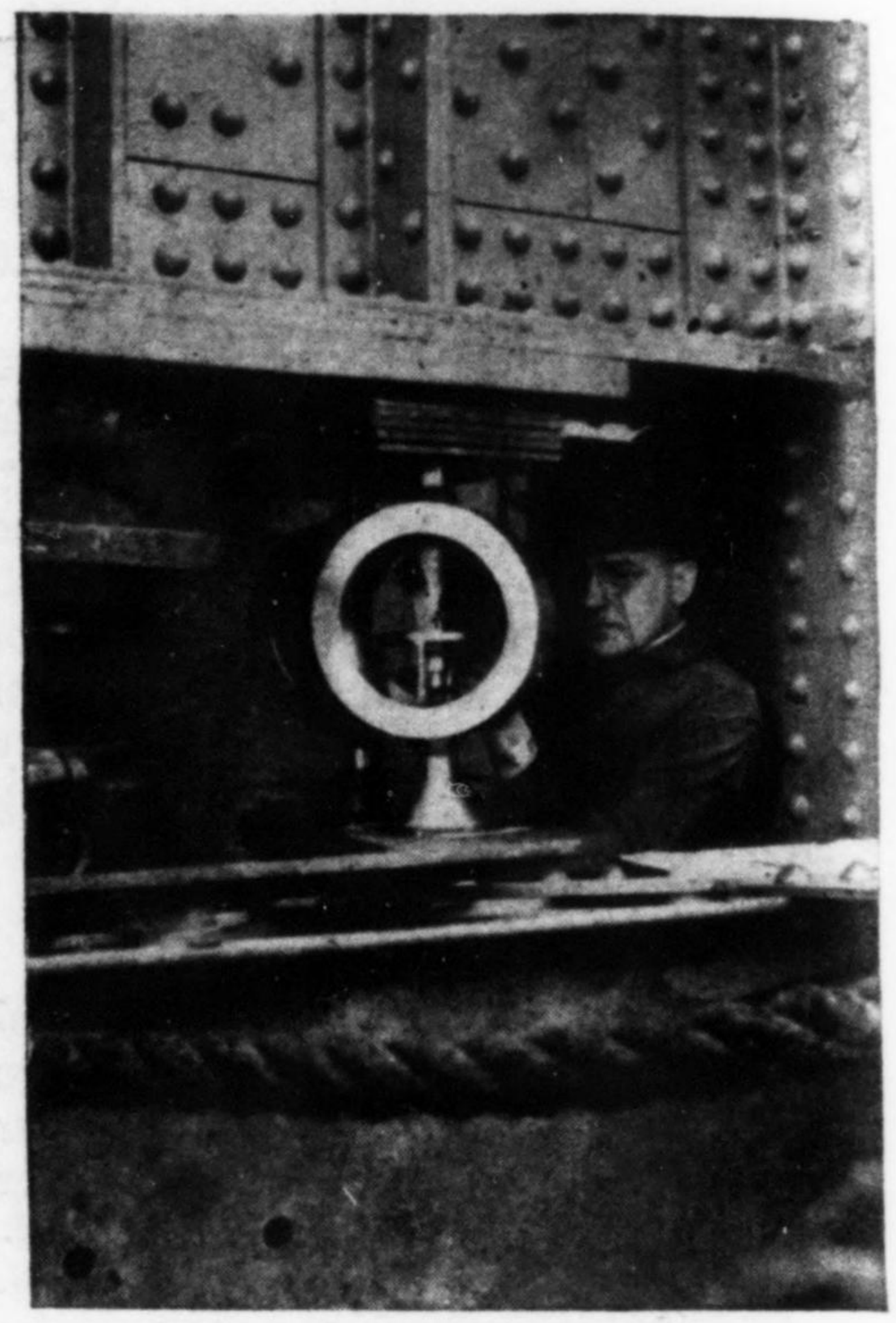


FIG. 1—TAKING DEFLECTION readings on two 200,000-lb. proving rings as they carry one corner of the three-span continuous-truss bridge over the Cape Cod Canal at Bourne, Mass.

The earliest use of elastic properties of materials as load-measuring devices will probably never be known with certainty. One of the early uses doubtless was when Captain Eads, about 1870, devised a calibrated steel rod, to check the accuracy of two testing machines that he had had built. A few years later A. H. Emery built for the National Bureau of Standards a measuring device which more nearly approached the present-day elastic loop. He took two steel plates and joined their ends by flexure plates (these are well-known features of the testing machines and weighing scales built to his designs), to form a rectangular structure., with the flexure plates vertical. This device was placed so as to cause the load to be measured to pass along its vertical axis, and deflections were measured micrometrically. After the relation between load and deflection had been determined by calibration of the device under known loads, it was used to calibrate testing machines and for other similar purposes. Elliptical or oval elastic lops have been used in foreign countries, but it was not until about ten years ago that a circular elastic loop was developed and patented by H. L. Whittemore and the late S. N. Petrenko, of the Bureau of Standards.

A proving ring is essentially a load-weighing device, although it uses neither lever-and-poise nor pendulum —common devices for the measurement of the attraction of the earth for a mass. Essentially, it is a spring, and when loaded axially it assumes a slightly elliptical form, the long axis horizontal. In order, then, to effect accurate measurement of load, it is necessary only to find a means of measurement of this deflection.

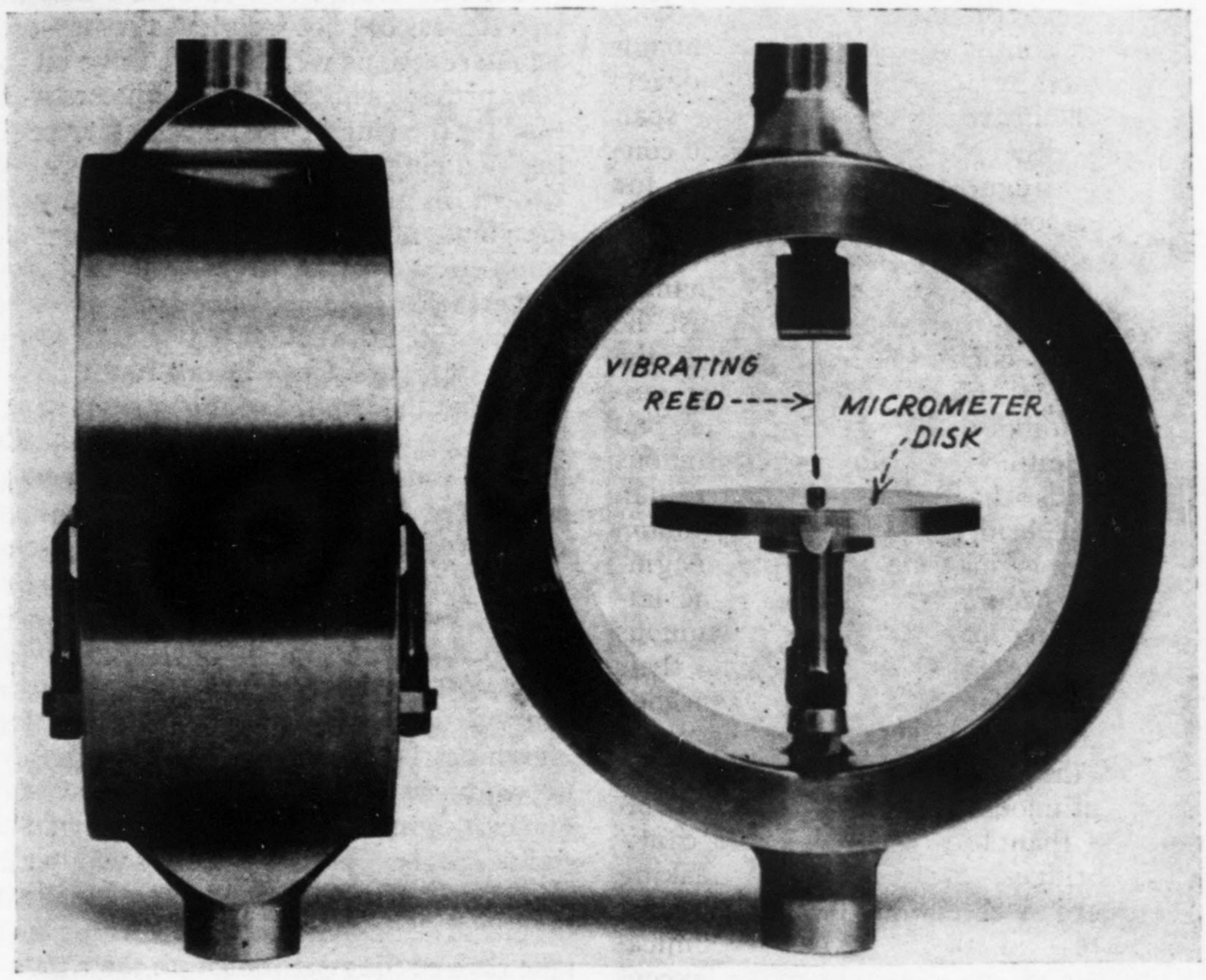


FIG. 2—TYPICAL PROVING RING, showing vibrating reed and micrometer disk. By turning the disk until it makes contact with the end of the vibrating reed, an unusually accurate measure of the diameter of the ring is obtained. The difference in diameter between a no-load reading and a reading under load furnishes a figure directly convertible into pounds by means of a calibration equation.

This problem has been solved by the use of the vibrating reed—a thin spring-steel strip loaded at the end by a small mass — and a carefully constructed micrometer screw with vernier (Fig. 2). A disk with rounded central projection anvil is mounted on the micrometer screw. The periphery of this disk is divided accurately into 100 (or 200, depending on size) divisions. The index mark is on the vertical standard, which is close by but not touching the disk.

In operating one of these proving rings the reed is set in vibration by pushing it sidewise and releasing. The disk is turned (the anvil rising) until the anvil just contacts the mass on the end of the vibrating reed. The quality of contact is judged by one (or all) of three criteria: the tone of the vibrating reed, the time of decay of the vibration, and the "feel" of the vibrations in the ring structure.

On good authority it is said that the sensitivity of these indicating devices is such that a change of 1 lb. at a load of 100,000 lb. is detectible. Guaranteed accuracies are 1/10 of 1 per cent and calibration is made at the National Bureau of Standards, Washington, D. C. On the average, a deflection of 0.060 in. is allowed for full load.

Rings up to 300,000-lb. capacity have been built by the Morehouse Machine Co., York, Pa. The Bureau of Standards has three rings of such capacity, while the Baldwin-Southwark Corp. and associated companies have five rings of 200,000-lb. capacity each. A 200,000-lb. ring requires about 22 in. in vertical height and about 13 in. in transverse dimension.

This then is the instrument—a circular ring of high-strength steel, whose change in diameter under load is measured by means of a vibrating-reed indicator and a micrometer disk, this change then being translated into pounds by a calibration equation—which has been adapted to use in weighing the reactions of continuous-truss bridges.

Weighing a bascule

The initial use (so far as is known) of these devices in the determination of bridge reactions was in April, 1933, when the unbalance of a 90-ft. single-leaf bascule bridge was determined by the insertion of two 200,000-lb. rings between the outer end of the leaf and the pier upon which it rested. This determination was made for the American Bridge Co. at the suggestion of David S. Fine, assistant engineer of the erecting department. Readings were taken with the end of the leaf at various heights above the pier and also progressively, as the bridge was lifted and as it was lowered from position to position. Counterweight was added as indicated by the proving-ring determinations, and an excellent balance of the bascule leaf was obtained.

Weighing continuous bridges

The first use of proving rings for weighing continuous-truss reactions was on the Little Bay Bridge in New Hampshire. This structure consists of a channel crossing in a three-span continuous layout (Fig. 3), flanked by

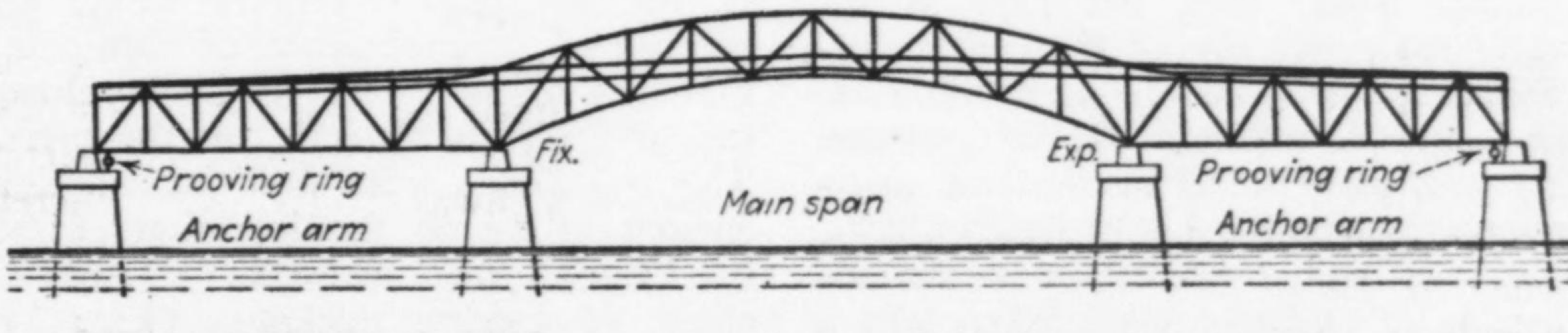


FIG. 3—THREE-SPAN continuous-truss bridge with proving rings located so as to measure the reactions at both ends. Outline of Little Bay Bridge is shown. The Cape Cod bridges are similar except that the main span is a high arch with the roadway suspended beneath.

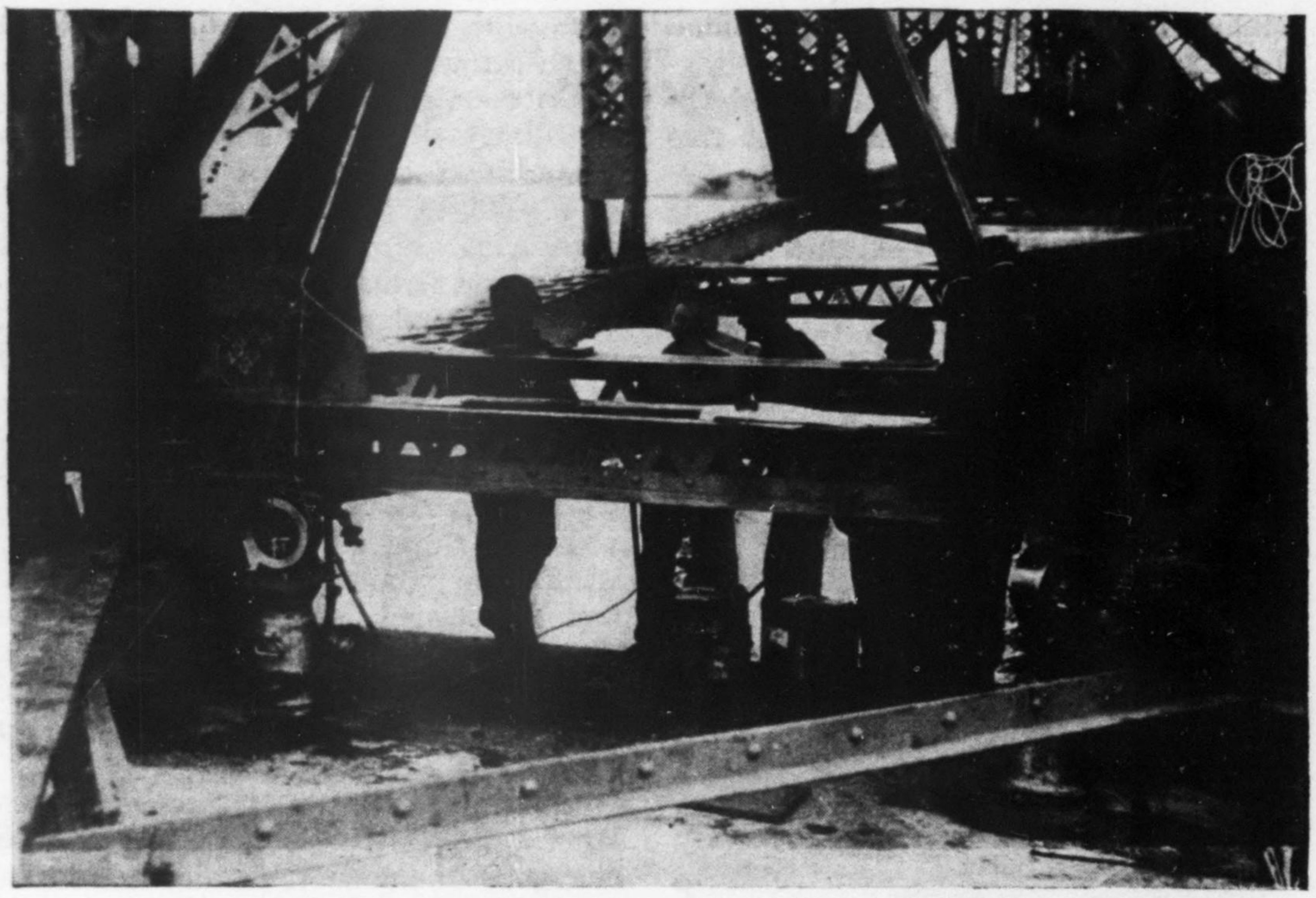


FIG. 4—WEIGHING THE REACTION at one end of a two-span continuous truss on the Little Bay Bridge in New Hampshire. Note that proving rings are set on jacks.

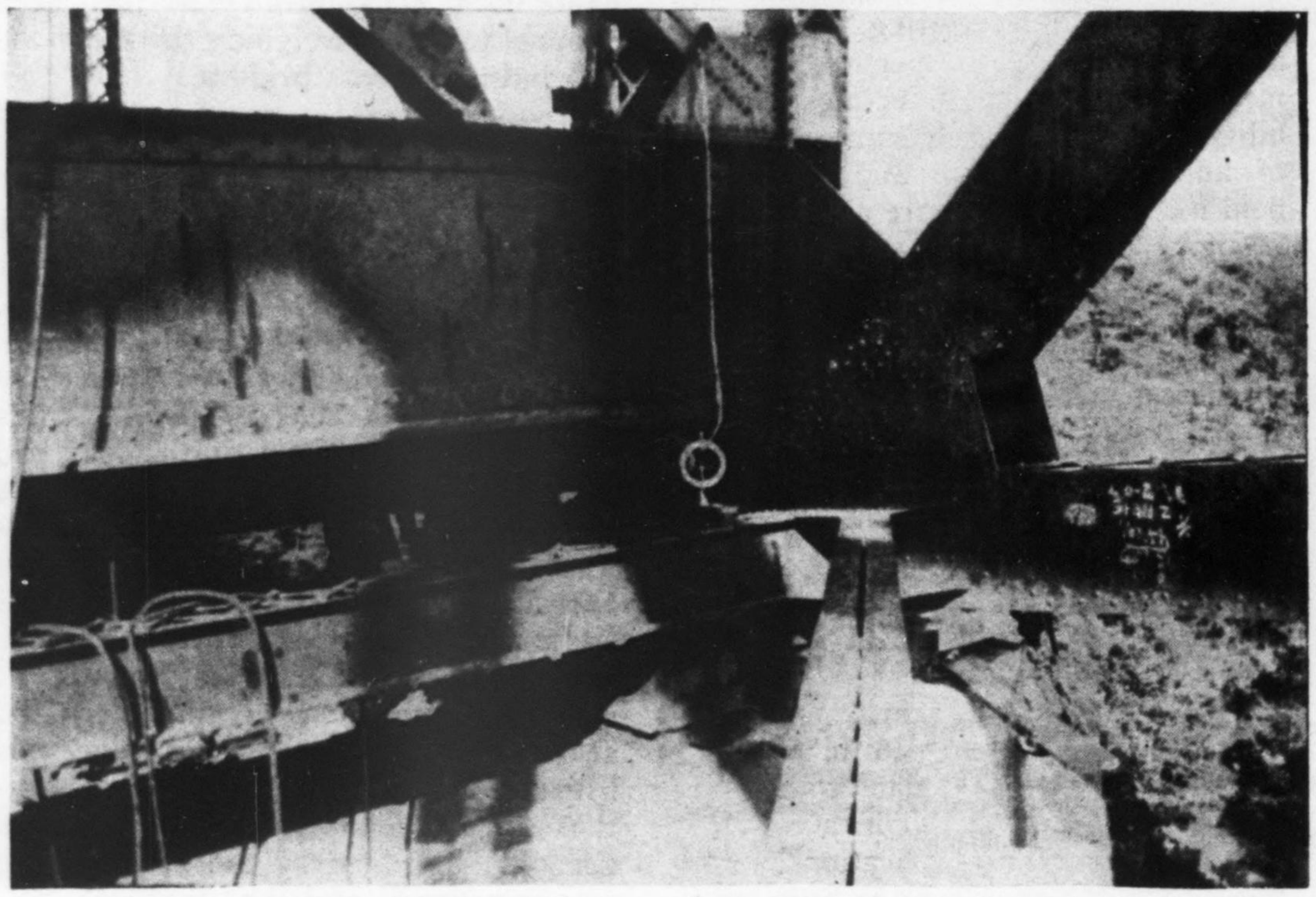


FIG. 5—WEIGHING SET-UP on one of the Cape Cod bridges, showing special reinforced girder installed to provide a jacking reaction and to furnish the means for transferring the bridge load to the proving rings. A rope is attached to the proving ring as a precaution against its falling from the pier.

three deck truss spans on either side; two spans of each of these three span groups of deck trusses are also continuous. The principal use of the proving rings was on the main three-span layout made up of a 275-ft. center span and 200-ft. side spans. However, in June, 1934, while the bridge was being erected, floods took out part of the falsework under one of the continuous-truss side spans, making it advisable to check the reactions. These reaction determinations were made with proving rings mounted on hydraulic jacks. The truss shoes were set at various elevations, readings were taken and a curve of reactions was plotted. Since the desired reactions were known from previous calculations, it was easy to determine the required elevation at each pier, and by the use of packing pieces to bring the trusses to these elevations.

The reaction determinations on the main three-span continuous truss were made a few weeks later. In this case the rings were not placed on the jacks, as they were during the determination of the side-span reactions (Fig. 4), because of inconvenience of manipulation, but instead were set on auxiliary supports resting directly on the piers. The truss ends were first jacked high enough to permit the insertion of the two rings, each mounted on several plates of definite thickness (Fig. 1). The jacks were then exhausted, the truss ends settling on the rings so that the entire reaction passed through them. At that stage the continuous-truss portion of the bridge was supported on proving rings at the outer ends of both anchor spans, on fixed shoes at one main pier and on rollers at the other main pier.

Deflection readings were then taken on the proving ring at each corner. Converting these readings into reactions, the four values were compared, and if not in close agreement among themselves and with the theoretical value, adjustments in the height of the truss shoes were made and the reactions were again determined. Telephone communication was provided between the observers at the opposite ends of the span, to assist in this work.

On this bridge it was found that the two trusses did not weigh the same—that is, the reactions were not the same on the four proving rings. Two engineers were reading the rings (one on each pier reading two rings), and the results were not known in pounds of load but only in divisions of deflection, until after the completion of the series of readings. Nevertheless, the evidence from one pier corroborated the evidence from the other. This discrepancy may be due to the possibility that the lengths of the various members in the trusses are not identical, and that the ends of one truss, in consequence, may have been at different elevations (with respect to the elevations of the supports at the two middle piers) than the ends of the other truss.

In using proving rings for weighing operations, certain preparations and precautions are required. Not only for weighing but also for possible future adjustments of the span ends, jacking girders should be provided between the end verticals. At the span ends where the reactions are to be weighed, these girders should be designed of sufficient strength to take care of the reactions with the trusses supported either on the hydraulic jacks or on the proving rings. With through trusses, end floor beams may be used in place of the jacking girders.

Inasmuch as the final elevation of the ends of the trusses will probably vary somewhat from the designed elevations, provisions have to be made in the design and specifications to permit the adjustment of the floor construction after the continuous truss ends are set at the heights determined upon to give proper reactions. Finally, if the ends to be jacked rest upon abutments, the floors, fence and curbs of the latter should not be completed until the jacking has been done; if other spans adjoin the continuous spans, jacking girders or other devices should be provided to permit the adjustment of the heights of these spans.

Cape Cod Canal bridges

The next use of the proving rings was on the Cape Cod Canal bridges. These two structures, located at Bourne and Sagamore, Mass., are identical so far as their continuous portions are concerned. Similar to Fig. 3, except that the center span has a much more pronounced arch, these bridges have a center-span length of 616 ft. and flanking spans of 396 ft.

The reactions on the Bourne bridge were determined in October, 1934, and on the Sagamore bridge in February, 1935. Due to the magnitude of the pier reactions on these structures (on the order of 500,000 lb. at each end), it was necessary to weigh one end of the bridge at a time, since there were only four 200,000-lb. proving rings available. Thus the operation was somewhat different than on the Little Bay bridge, where the reactions at both ends were determined simultaneously.

At Cape Cod the rings were set on a pier in pairs just inside the trusses. Then, as the truss ends were jacked to a designated height, the truss ends on the pier at the other end of the bridge were elevated correspondingly, although there was no weighing apparatus at that end. After a complete determination on one pier, the four rings were moved to the other pier, at the opposite end of the bridge, and the process was repeated, the levels on the pier where determination had been made being varied to correspond to those previously attained when the weighing was being done.

Altogether, this method provides the bridge engineer for the first time with a method of real accuracy for the determination (and variation as desired) of pier reactions on continuous-truss spans. While there is no thought that this method shall lead to any marked increase in the adoption of continuous-truss bridges over other types, it does provide means of the elimination of several of the uncertain variables in the construction of this type of bridge.

Current Research Activities of the A.S.T.M.

COMMITTEES of the American Society for Testing Materials are continually carrying on research programs, to provide the new information upon which future standardization may be based. This research work is reviewed annually by the committee on research, and last year's summary included 114 research projects of which the following are particularly interesting to the civil engineering and construction field. Progress reports on many of them will be presented at the annual meeting of the society in Detroit, June 24-28.

Committee A-1 is beginning work on the development of inspection methods and recommended practices for surface imperfections in structural steel. It is also assisting in an investigation of the effect of phosphorus and sulphur in steel and in a study of the yield point of structural steel.

Committee A-2 is studying the quality of wrought iron, with particular emphasis on the influence of raw material and processes of manufacture. The optimum amount of manganese is being given special study.

Committee A-3 is studying the correlation of properties of iron castings and test bars and is also investigating various methods of impact testing of cast iron.

Research projects under Committee A-5 include atmospheric corrosion tests of uncoated steel and iron sheets, of galvanized sheets, and of metallic-coated hardware, structural shapes, tubular goods, wire and wire products; total-immersion tests on uncoated sheets; accelerated corrosion tests on metallic-coated products; and embrittlement studies of hot-dipped galvanized structural steel.

Committee C-1 has the following research projects under way: compressive-strength test of cement mortars; particle size and specific surface of cement; and volume change and soundness of portland cement.

Committee C-9 has the following projects under way: studies of designing and proportioning of concrete, of concrete aggregates, of deleterious substances in concrete, curing, workability, admixtures, elastic properties, and permeability. Conditions affecting the durability of concrete in structures, methods of analyzing concrete and of making field tests for concrete are also being investigated.

The committee on paint and paint materials (D-1) is making accelerated tests for protective coatings, studying anticorrosive and anti-fouling paints, and investigating the physical properties of paint materials.

New methods of testing the soundness of lime and the plasticity of hydrated lime are being studied by Committee C-7.

In the field of bituminous materials, Committee D-4 is investigating methods of separating cutback asphalts and of testing bituminous emulsions. Committee D-8 is investigating accelerated weathering tests of bituminous roofing materials and studying tests of bituminous joint compounds for sewer pipe.

In the field of timber, Committee D-7 is giving consideration to methods for making two types of moisture determinations, the first an accurate method for laboratory use and the second a method sufficiently accurate and practical for field use.

Leakage in Long Pipe Line Found Small After Two Years' Service

AFTER being in service for two years the 25-mile 36-in. cast-iron pipe line from Ashland to Lincoln, Neb., was tested last fall in accordance with the contract specifications. The original test made directly after completion indicated a remarkably tight job, and the recent test shows that the leakage is still far below that specified as allowable. The original test made by the contractor (*ENR*, April 20, 1933, p. 487) gave an average leakage of 11.18 gal. per day per mile per inch diameter of pipe. The final test, made in October, 1934, by the city averaged 25.2 in.-gal.

The second test was conducted in the same general manner and with the same equipment as used by the contractor in the first test. As the line was purely a supply main, there are no cross-connections and no water leaving the line between Lincoln and Ashland. The main was tested in four sections between valves. The leakage was determined by measuring the drawdown in the water level in a large tank from which the test water was taken. The figures in the accompanying table in both cases represent the total leakage for the section tested, including any leakage from blowoff valves, air-relief valves and large valves at each end of the section.

ASHLAND-LINCOLN
LEAKAGE TESTS ON PIPE LINE
(25 miles long, 36-in. diameter)

Sect.	Length, Ft.	Leakage in Gal. per 24 Hr. per Mile per Inch Diameter 1932	1934
A	51,200	10.16	17.8
B	29,700	15.95	52.2
C	31,600	6.02	11.1
D	28,100	14.83	26.4
Average		11.18	25.2

The specifications provided that the leakage should not exceed 100 in.-gal. during both tests. "Water Works Practice," the manual of the American Water Works Association, sets up a standard figure of 200 in.-gal.

In section B of the line (see table) there were valves which it is believed were not closed absolutely tight, inasmuch as manipulation of them closed off part of a larger leakage showing during the preliminary tests.

The original pipe was laid and tests carried out under the direction of D. L. Erickson, city engineer. Abel and Dobson were the contractors.