placed a hydrostatic test of 170 lb. per sq.in. at zero level is to be kept on the line for 24 hours, and the leakage measured.

The backfill in the trenches will consist of crushed rock enough to cover the pipe. The method of placing the curved section on the west end is shown in the accompanying sketch, the concrete having been placed to resist any tendency to straighten out the curve under pressure.

On the barge the pipe-laying crew consisted of a superintendent, foreman, two engineers, ten pipe men and two laborers. There were two crews, each working 12 hours and the 228 lengths in the submarine section were placed in 30 days.

Where the pipe was lowered from the trestle this was accomplished by means of 1½-in. threaded rods extending from the trestle down to bands around the pipe sections, two bands being used on each 12-ft. length. Tar paper was used to prevent the bands from damaging the protective coating on the pipe. At the top the threaded rod passed through a cap timber carrying a washer and nut. On the latter a wrench with 4-ft. handle was provided for hand operation and when the pipes were to be lowered a crew of 40 men manned these wrenches and uniformly lowered the pipe so as to avoid deflection of joints.

The Bay crossing division was constructed under the general direction of M. M. O'Shaughnessy, city engineer, N. A. Eckart, principal assistant. R. P. McIntosh was designing engineer on structures and Leslie Stocker on submarine pipe. C. R. Rankin was resident engineer on the Bay crossing division. The pipe was manufactured by the United States Cast Iron Pipe Co., and contract for placing the pipe and also for building the bridge piers and the large caisson was carried out by Healy-Tibbitts Construction Co. The bridge spans were fabricated, delivered and placed by the United States Steel Products Co.

How Land Development Is Financed In Various Countries

LONG-TIME land loans are provided in government land settlement work in many foreign countries, according to data given by Prof. W. W. Long, of Clemson Agricultural College, Clemson, S. C., at the conference on reclamation and land settlement held at Washington, Dec. 14. The time allowed for principal repayment in these various countries ranges from 33 to 75 years. Interest is charged in all cases.

REPAYMENT TERMS OF GOVERNMENT LAND SETTLEMENT ABROAD

Countries	Rate of Interest, Per Cent	Time Given to Pay for Land or for Repaying Loan
Denmark	3 to 4	65 years
Italy	2.5	50 years
Holland	4.7	....
Norway	3.5 to buy land and 4 to owners	....
Hungary	4	50 years
Austria	4 to 4.5	54½ years
Russia	4.5 principal and interest	55½ years
Germany	3.5 to 4	56½ years
France	4 to 4.5	75 years
England	4	50 years
Ireland	3.5	68 years
Belgium	4.5	30 years
Switzerland	4.5	57 years
New Zealand	4	36½ years
Victoria, Australia	4.5	36⅓ years
New South Wales	3 to 5	30 to 40 years
Other Australian States	4 to 5	30 to 40 years
British and German South Africa	4	....
Chile	4	33 years
Argentina	4	....
British Columbia	1 per cent more than the interest on state bonds; 5 per cent at present	36½ years

In comparison, it is to be noted that the United States land settlement policy as laid down in the federal reclamation law originally called for repayment of principal without interest in 10 years, but was amended before payments had been made to provide for no-interest repayment of principal in 20 years. The latter arrangement is equivalent to 1½ per cent interest per year and amortization of the principal in 20 years, and represents about the same annual burden as the longest-period foreign system, but continued for only a fraction of the length of time. Under the amended reclamation law of 1924, which authorizes repayment at the rate of 5 per cent of the gross crop return, direct comparison with the foreign systems is not possible. It has been estimated that on a number of projects the new repayment method would result in paying off the principal (without interest) in periods of 50 to 100 years, which means that annual payments would be ¾ to 1½ per cent of the principal, without interest.

Large Cantilever Planned for Mount Hope Toll Bridge

Span of 1,200 Ft. Recommended for Rhode Island Bridge—Trusses to Have Three Chords—Tolls Will Repay Cost Rapidly

REPORTING on the results of an eight months' study of a possible toll bridge to connect the island of Rhode Island with the mainland at Bristol, crossing the entrance to Mount Hope Bay, a state commission has just reported to the legislature that the traffic amply warrants the construction of such a bridge and that a cantilever structure with main span of 1,200 ft. should be built. The design proposed for this cantilever is of novel character, both in its general proportions and outline and in the detail construction advocated by the commission. It is proposed that for economy in floor construction the cantilever structure shall consist of four lines of top chord, but that there should preferably be only two bottom chords, each connected with its pair of top chords by two inclined web systems. The cross-section of the bridge would thus show two structures of triangular cross-section.

The following main features of the recommendations are abstracted from the report:

Location—The natural location of the bridge is from a projecting point of the mainland a short distance south of Bristol to the northerly point of the island, north of Portsmouth. The strait here is about 3,000 ft. wide, shore to shore. Both shores are rocky, and the subaqueous topography as well as borings showed that the whole width of the channel is in rock with only a few feet of earth cover. At the north or mainland end the rock is granite, while the southerly half of the crossing and the outcrops on the island are slate. The shores rise rapidly to a height of more than 100 ft. above water level. The channel depth is something more than 80 ft. on a width of some 300 ft., but the bottom rises rapidly enough so that piers 1,200 ft. apart would reach rock at about 20 and 30 ft. depth respectively.

There is much shipping through the strait, but the necessary provision for masted vessels is readily made in view of the shore configuration. The scheme provides clearance heights under the suspended span ranging from 137 ft. to 152 ft.

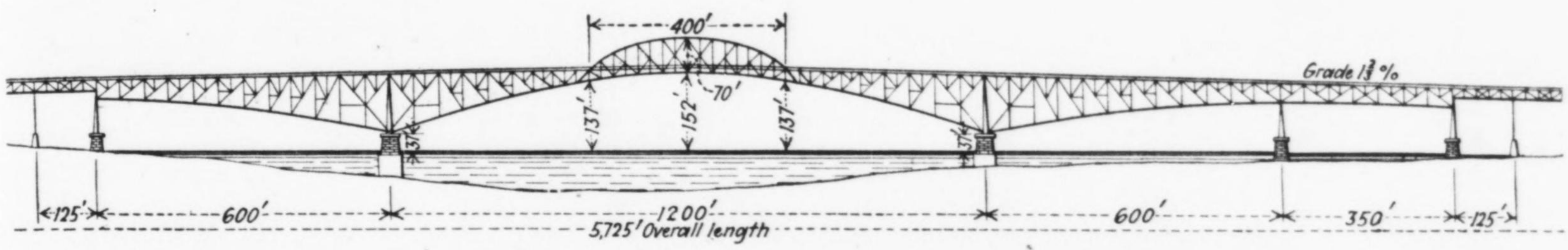
Type of Structure—Three types of structure, suspension, cantilever and arch, were compared. The most economically proportioned suspension bridge, says the report, would involve an unnecessarily long central span and the construction of massive anchorages, only one of which would be on the shore, resulting in a total cost greater than for the cantilever type. A series of braced spandrel steel arches was considered, but the many foundations would increase the cost extensively. It was concluded that a cantilever bridge with 1,200-ft. main span would be the best adapted to the location. With 600-ft. anchor arms and a 350-ft. prolongation of the south anchor arm beyond its primary anchorage, the main structure becomes 2,750 ft. long. There would be approaches totaling 2,975 ft. in length, consisting of plate-girder spans 50 to 100 ft. long and deck truss spans of 125 ft., making the total length of the structure 5,725 ft. between abutments.

As shown by the adjoining elevation of the recommended design, the cantilever has unusual outline. The lower chord of the main span is laid out in the form of a flat segmental arch appearing to spring from the main piers. The suspended span is of crescent form and rises above the roadway, whereas the cantilever and anchor arms are kept below the roadway and have horizontal top chords. The lower-chord line of the anchor arm is made similar to the line of the half main span. The suspended span is 450 ft. long and the cantilever arms each 375 ft.

Triangular Truss Construction—Deck construction is adopted in order to reduce the weight of the floor framing.

> A study has been made utilizing in these deck cantilever trusses four relatively light trusses side by side with a 7-in. slab of reinforced concrete for a floor carried between the upper chords. Light trussed floorbeams extending between the top chords at frequent intervals will give support to this floor slab and form part of the system of lateral bracing. The top plates in the chords themselves are to be protected by a thinner concrete slab. It is suggested that the lower chords of each pair of these four trusses be combined into a single member of correspondingly larger and more efficient section, possibly hexagonal in shape. The various members of the web system could converge to these lower chords forming two triangular shaped units as part of the swaybracing system. On the contrary, at the main piers the vertical posts and the principal diagonals might well flare outward, forming a tower-like structure with greater stability, at the same time distributing the loading to several points on the masonry of the pier. This adaptation of the trusses eliminates the cost of a separate tower between the pier and the grade of the road, as well as materially reducing the size of the pier base that would be required for stability if the structure lay wholly above the plane of the roadway. Connection between the members will be somewhat complicated, but may be a development of bent steel plates, pressed steel forms, light steel castings or a combination of all.

The deck is to consist of a 40-ft. roadway and two 7-ft. sidewalks bracketed from the main floor system and separated from the roadway by a concrete collision curb high enough to keep any wheel from mounting the sidewalk grade. This width, the commission says, will easily take care of traffic for some time to come. Provision can be made to enable the bridge to be strengthened later on and a lower floor to be added, to carry either vehicles or railway tracks. "The V-shaped design of the truss system is particularly adapted to carrying railroad tracks below the floor at a minimum of expense, if provision is made for adding plates to take care of the increased loading. The factor of safety, however, may take care of this extra loading without any such additional construction, if future designers decide this procedure to be advisable."



WEST ELEVATION OF PROPOSED MOUNT HOPE BAY CANTILEVER BRIDGE BETWEEN MAINLAND AND ISLAND OF RHODE ISLAND

Traffic and Amortization by Tolls—There is one bridge connecting the island with the mainland, Tiverton Stone Bridge, crossing the Sakonnet River east of Portsmouth. The traffic over this bridge (1,800,000 vehicles in 1925) is taken to be a close indication of the traffic which would use Mount Hope bridge were it now in service. The commission therefore estimates that in 1925 the Mount Hope bridge would have carried at least 1,000,000 cars. Projecting this estimate along a line asymptotic to the curve of Rhode Island motor vehicle registration (roughly 100,000 in 1925), the expected Mount Hope bridge traffic is set by the commission as 2,000,000 in 1934, 3,000,000 per year in 1941, etc., figures which represent rates of increase of 8 per cent per year between 1926 and 1935, 6 per cent between 1936 and 1940, and 4 per cent per year between 1941 and 1945, reaching 3,500,000 vehicles per year in 1945. These rates of increase are very much lower than those over Stone Bridge and other focal points of traffic in the vicinity.

With this heavy traffic, the cost of the structure (estimated at $6,000,000, including land damages, engineering, and interest during construction) can be amortized on quite moderate toll rates within periods ranging from 10 to 20 years. An average toll rate of 50c. per vehicle would retire the bonds in nine years, besides paying interest and operating costs (operation and maintenance estimated at $25,000 to $30,000 per year). A 30c. rate would retire the bonds in 14 years. A rate of 50c. for the first 5 years and 25c. thereafter would retire the bonds in 12 years.

Herbert W. Smith is chairman and Arthur A. Sherman secretary of the Mount Hope Toll Bridge Commission. Daniel O. Cargill is engineer, succeeding Clarence L. Hussey, who died two months ago.

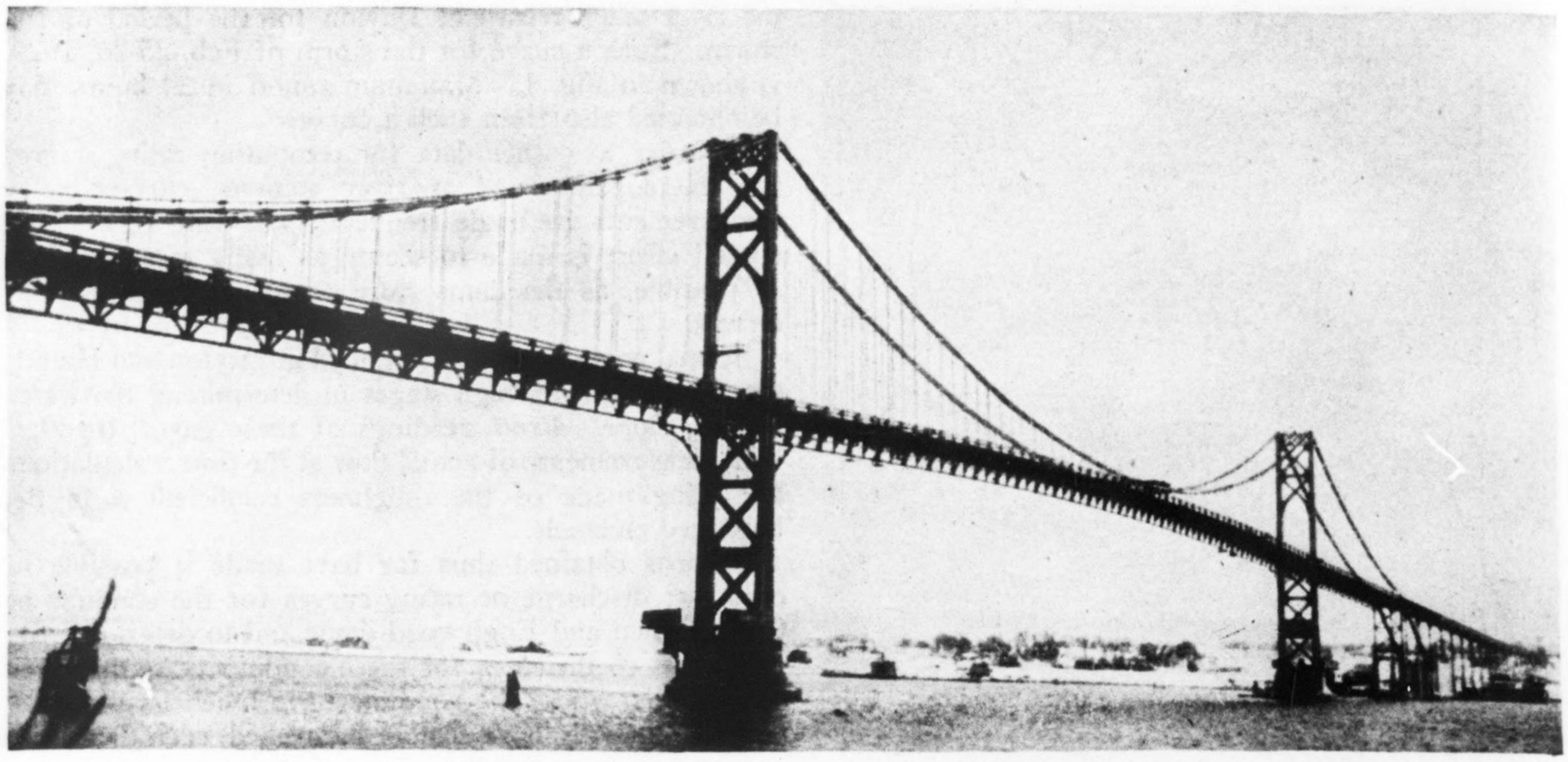


FIG. 1—THE MOUNT HOPE BRIDGE ON FEB. 23 JUST AFTER DISCOVERY OF BROKEN WIRES
Preparations were under way for placing concrete floor slab.

Technical Aspects of Cable Wire Breakages on the Mount Hope Suspension Bridge

Breaks Localized in Strand Shoes—Progressive Breaking Noted in Spite of Load Reduction—Cable Spinning Methods—Characteristics of Wire and Its Manufacture—Temporary Repair Measures

WIRE breakages in the cables of the Mount Hope and the Detroit suspension bridges which resulted in decisions to replace the cables on both bridges with new ones (*Engineering News-Record* March 28, p. 516, and April 4, p. 564) represent the first serious failure of bridge suspension systems recorded in modern times. As noted in our previous issues, the wire was new and untried in suspension structures, so that its failure has no bearing on the safety of suspension bridges now in place or under construction. However, because of the constant and continuing desire to develop new materials with high stress characteristics, of which this wire was one example, it is quite necessary to review the circumstances surrounding these wire breakages in detail. This article covers certain technical aspects of the situation at Mount Hope, including the discovery and location of wire breakages and the methods of repair, a review of the wire specification, including the origin of its acceptance and some of the preliminary test studies, a description of the cable-spinning methods used and a résumé of the wire-manufacturing process.

Description of the Bridge—The Mount Hope bridge is a private toll highway bridge of the suspension type located across an arm of Narragansett Bay in Rhode Island some 15 miles south of Providence. It runs in a north-south direction with the towns of Bristol and Portsmouth near its north and south ends respectively. It has a 1,200-ft. main span with 135-ft. channel clearance and 504-ft. side spans. From cable bents placed at the ends of the side spans, the cables slope down sharply to the anchorages. The length of the bridge from anchorage to anchorage is 2,648 ft. The structural steel towers are 284 ft. above mean low water and are of the fixed-base flexible type.

There are two 11-in. wire cables 34 ft. apart, each consisting of seven strands of 350 wires (No. 6, heat treated and galvanized) having a theoretical diameter of 0.196 in. and providing a total cross-section per cable of 74 sq.in. Actually the wires ran somewhat under the theoretical cross-section and several more than the 350 wires were included in most of the strands. The seven strands are arranged in three tiers, two strands on the top, three strands in the center and two strands in the bottom tier (Fig. 5, center).

The wire specification was originally drafted for cold-drawn wire but was subsequently changed to admit a new type of wire hardened by heating and quenching after the wire drawing. The specification stipulated that check analyses of the finished or semi-finished wire should not show in excess of 0.935 per cent carbon, 0.05 per cent phosphorus and 0.05 per cent sulphur. The diameter of the wire before galvanizing was specified as 0.192 in. with 0.003 in. tolerance, and after galvanizing as not more than 0.005 in. larger in diameter than the bright wire. The specified bend tests required that the wire bend continuously around a mandrel 2 in. in diameter without developing cracks in the galvanizing visible to the naked eye, and that it bend continuously around a mandrel of four diameters without sign of fracture. The ultimate strength was placed at 220,000 lb. per sq.in. minimum on the gross cross-section, the elongation at 4 per cent in 10 in., the reduction in area at 30 per cent and the yield point at 190,000 lb. per sq.in., the yield point being defined as the point where the elongation of the specimen in a 10-in. gage length is 0.75 per cent. The test for yield point was required to be made on samples taken from 10 per cent of the coils manufactured.

The heat-treated wire was first adopted for use on the Detroit bridge following studies and tests made by its engineers. Later, after work had been started on the Mount Hope bridge and following special recommendations, the new wire was substituted for the cold-drawn wire originally provided for in the specifications. It is understood that the manufacturer planned to subject to a magnetic analysis about 2 ft. of every 30 ft. of the wire manufactured, and to safeguard the manufacture in every way possible. In spite of these assurances and the promise of increased efficiency, the engineers for the Mount Hope bridge declined to dispense with any of the original number of wires in the cables, as is understood to have been done on the Detroit bridge, electing rather to give to the Mount Hope bridge a 10 per cent greater efficiency without additional cost.

Construction of the bridge was begun in the early spring of 1928 and rapid progress had been made toward meeting the opening date of July 1, 1929. The cables were spun in September and October, 1928. The main and side-span stiffening trusses and the floor system were completed Jan. 15, 1929. An article on the design of the Mount Hope bridge appeared in *Engineering News-Record* April 12, 1928, p. 585.

Early in February, at the time of the discovery of the first breaks in the wire, the bridge was completed to the extent shown in Fig. 1. Forms and reinforcing steel in considerable quantities were in place for pouring the concrete floor and the cables were stressed to about 32,000 lb. per sq.in.; their ultimate working stress was to be 80,000 lb. per sq.in.

Discovery of Breaks—During the handling and spinning of the cables nothing unusual occurred. Three broken wires were noticed in January, six on Feb. 8 and eleven additional on Feb. 19. All of these breaks were at the strand shoes (as were later breaks) at or close to the point of tangency of the wire to the curve of the back of the shoe; they did not affect any one strand but were scattered among the 28 shoes of both anchorages.

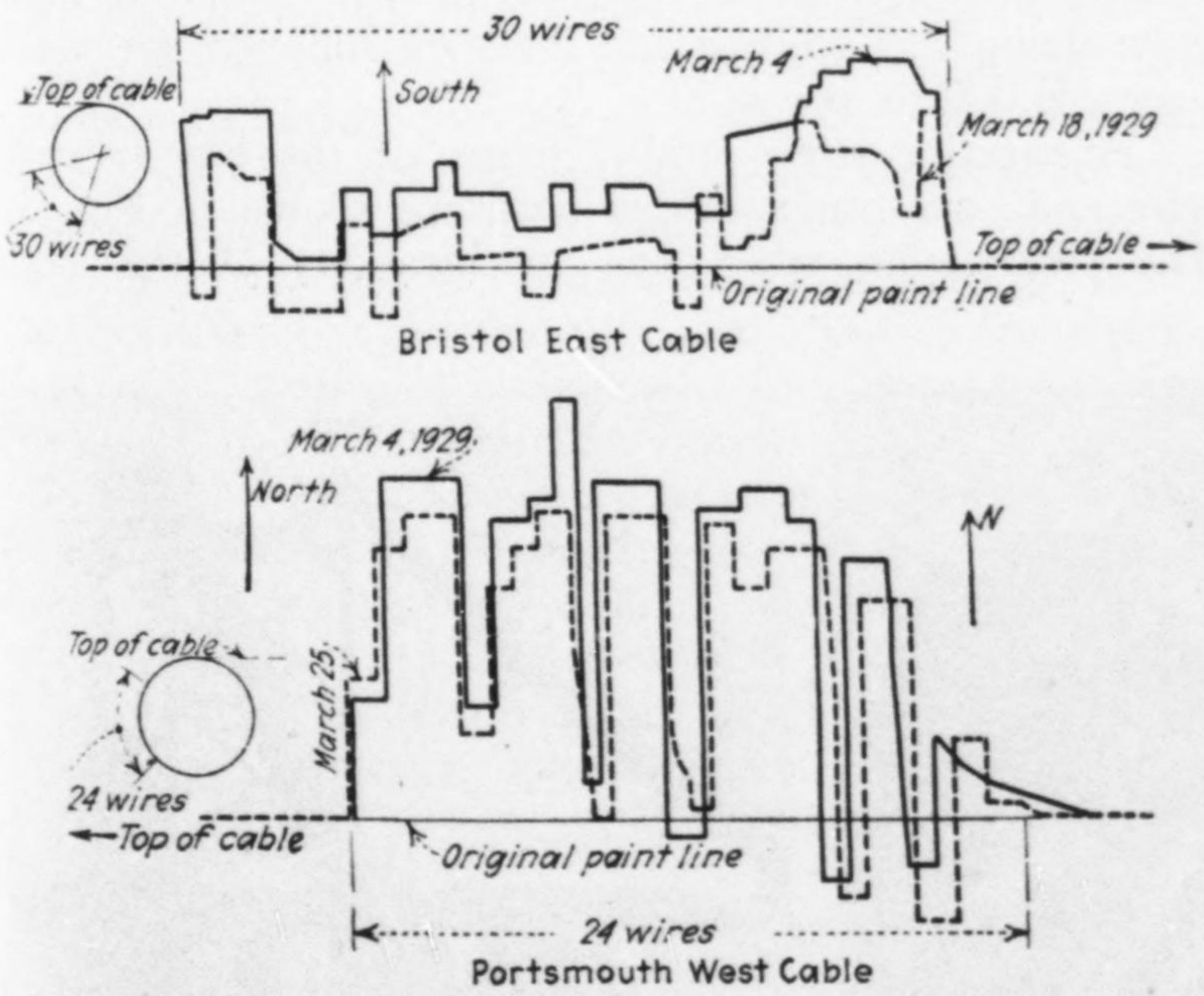


FIG. 2—CABLE SLIPPAGE DIAGRAMS FOR POINTS OF GREATEST FAILURE

Solid lines represent total recorded slip from a paint line applied on Feb. 23 just above the splay casting. Dotted lines show position of wires after hairpins were applied and slipped wires pulled back. Diagram for Portsmouth West cable probably shows all slippage, while at Bristol East anchorage considerable slippage had already occurred when the paint band was applied.

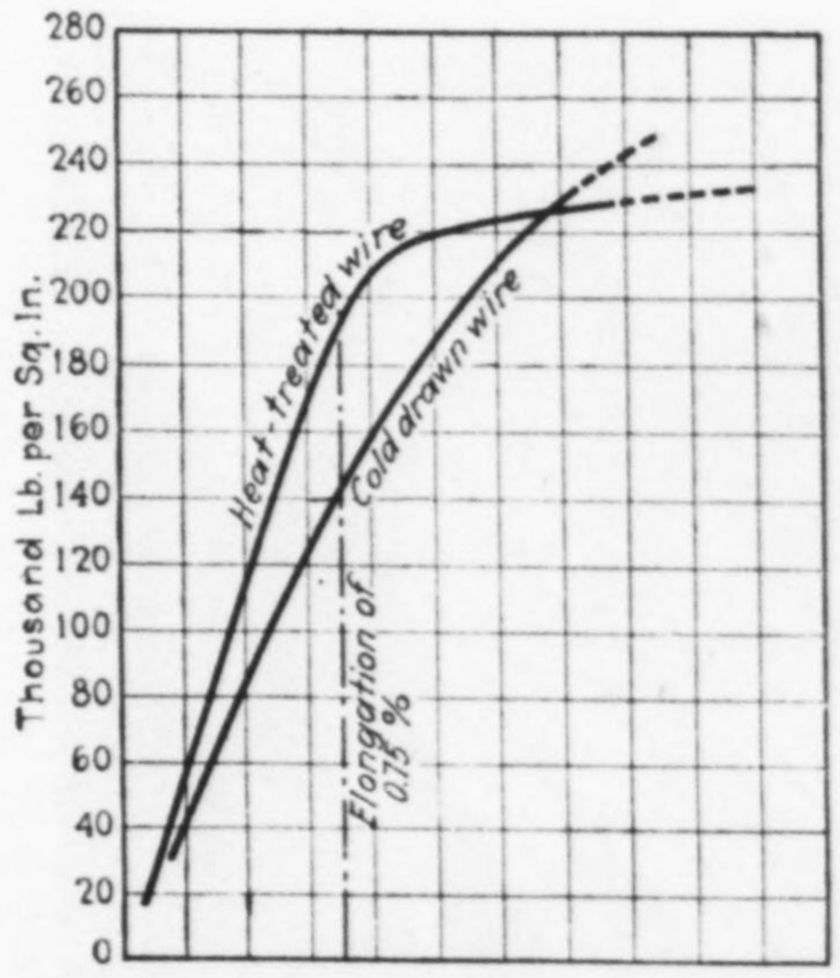


FIG. 3—COMPARATIVE SHAPE OF STRESS-STRAIN CURVES OF COLD-DRAWN AND HEAT-TREATED CABLE WIRE

Note abrupt change in the slope of the curve for the heat-treated wire at about 85 per cent of ultimate strength.

No particular anxiety appears to have been felt at this time, as the placing of floor forms and reinforcing steel was continued several days longer. But on Feb. 22, a holiday when work was shut down, 65 broken wires were discovered in the Bristol anchorage in the east strand of the bottom row of the east cable. On Saturday, Feb. 23, a more careful examination of the same strand was made, which showed that about 75 per cent of the wires in that strand were broken near the shoe. No additional breaks were found in any of the other anchorages.

Work Stopped—Work was immediately stopped. In order to detect any slippage occurring, on Feb. 25 bands of red paint were placed around the cables just ahead of the splay castings and also on each strand just behind the splay castings. It is probable that by this time considerable wire slippage had occurred on the bad strand at the Bristol East anchorage, but it is believed that at Portsmouth the paint band was in place before any breaks occurred. The maximum slippage subsequently measured at the Bristol end was about 1½ in., while that at the Portsmouth end was 3½ in. The slippage diagram for the Portsmouth West cable shown in Fig. 2 is a fairly accurate picture of the total slippage occurring in this cable. A similar slippage diagram is shown for the Bristol East cable. These diagrams cover a group of 25 to 30 wires on the outside circumference and are thus merely indicative of what may have happened within the strand. The painted lines on the Portsmouth East cable and the Bristol West cable showed that no appreciable slippage occurred there and the absence of breaks in these anchorages verified the fact.

On Feb. 27, ten broken wires were located at the Portsmouth West anchorage, all in one strand, and at the Portsmouth East anchorage one new break was located in addition to one previous break. Following a careful inspection on Feb. 28, made by pulling each wire with a hook inserted into the different strands, 77 additional broken wires were found at Portsmouth West. Additional broken wires were discovered on March 1, and further ones on March 2. On March 11, 90 tons of reinforcing steel was removed from the main span and on March 3 as much snow as possible was removed.

On March 4 a check on the damaged strand in the Bristol East anchorage showed that a total of 140 wires were broken. On the same day the final count in the Portsmouth West anchorage showed that 130 wires were broken, thus affecting 260 wires out of 360 in the strand (east strand of middle tier).

On March 9 numerous other breaks were discovered at the Portsmouth end. In Portsmouth East sixteen breaks were discovered in one strand and fifteen breaks in another strand. In the Portsmouth West anchorage eight new breaks were found.

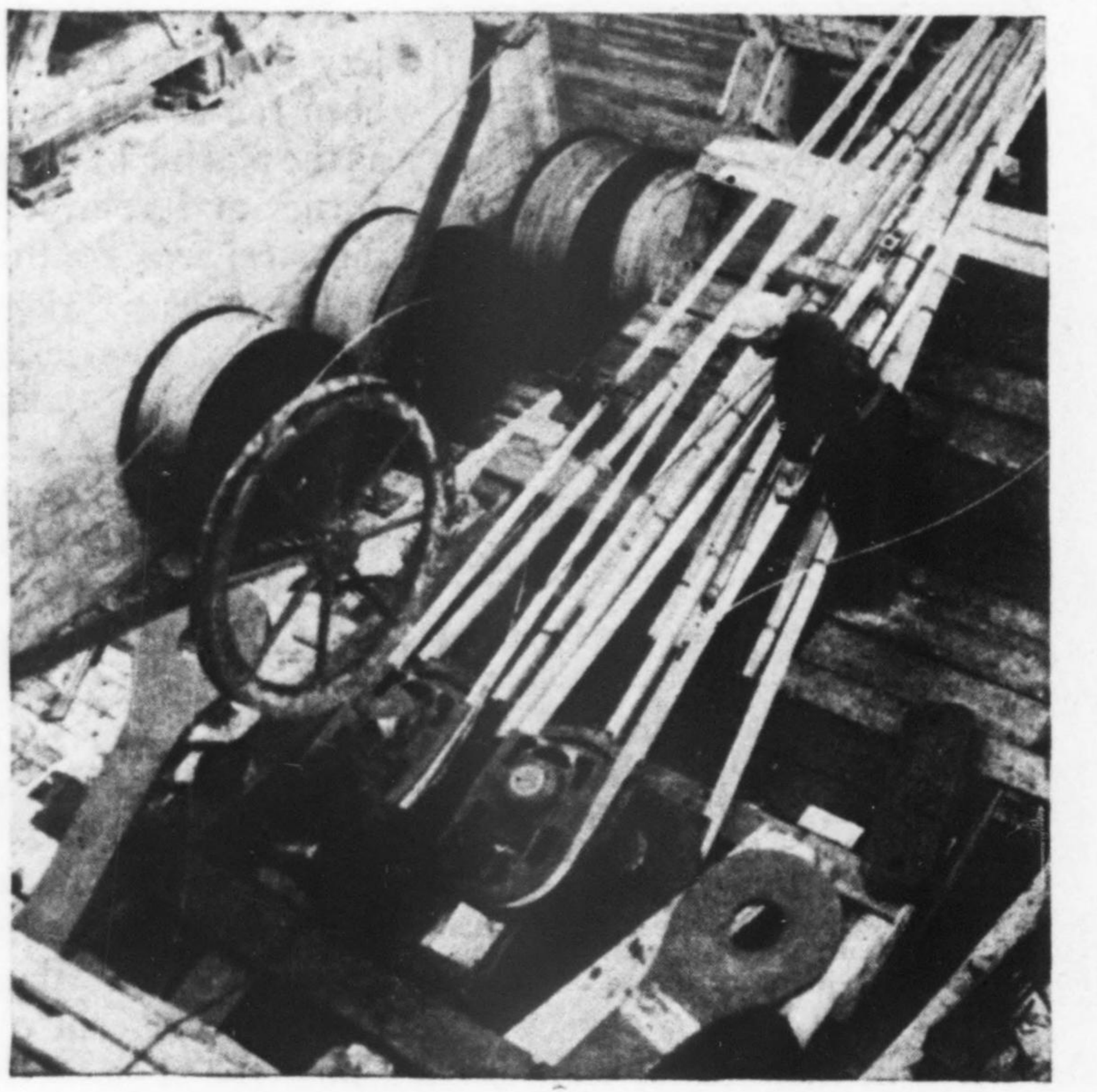


FIG. 4—SPINNING CABLE IN THE BRISTOL WEST ANCHORAGE
Note pulley sheave on lower left of strand shoe around which each wire was preformed so that it would lie flat around the shoe. Traveling wheel is ready to leave anchorage with a bight of wire to be carried to the opposite side.

On March 13 an examination of the outside surface of the cables for their complete length was made and one break was found in the west cable of the Bristol backstay about 10 ft. from the face of the cable bent saddle. This is believed to have been an old break made during cable squeezing.

Repairs—Repair work was immediately begun following the discovery of the first breaks. Precautionary measures consisted in removing the reinforcing steel, 90 tons, and later the floor forms from the bridge, also in painting the red bands around the cables to keep a check on cable slippages. Splicing the breaks began soon after Feb. 23, and by March 3 a number of wires had been repaired. This work was done by inserting "hairpins" around the shoes, splicing them to the ends of the broken wires on each side. These hairpins were made of cold-drawn wire in about 20-ft. lengths and were applied as fast as possible by a crew of men working 24 hours a day. Hairpins had been attached to all broken wires by March 15 and strain-gage readings on some of them showed a stress of 1,200 lb. per wire. In addition to the actual repair of the breaks, temporary hold-back cables were placed on both ends of the bridge between the cable bents and the anchorage eyebars.

In the foregoing account of the cable wire breakages several significant points are to be noted. In the first place, the majority of the breaks occurred in two strands, one in the east cable and one in the west cable. In the east cable the breaks were localized in the Bristol anchorage very near the tangent point on the strand shoe. In the west cable the breaks were localized in the Portsmouth anchorage, but also near the tangent point. In the Bristol anchorage about 75 per cent of the total number of wires in the strand were broken, while in the Portsmouth anchorage it is stated that 130 wires out of a total of 180 in a half-strand were found broken.

Another point to keep in mind is that on March 9, two weeks after the first breaks were discovered, eight additional breaks were found in the Portsmouth West anchorage and 31 breaks in the Portsmouth East anchorage; in the latter only two breaks had been discovered previously. Since careful inspection had been made of all cables daily since the first breaks were discovered, these additional breaks were evidently new ones. In the two-week interval no additional loads had been placed on the bridge—in fact, some 90 tons of reinforcing steel had been removed and in addition the snow load had either been removed or had melted.

Characteristics of Cable Wire—Utilization of heat-treated wire in the Mount Hope bridge, following its adoption for use in the Detroit bridge, marks the introduction of this type of wire to suspension bridge service. The ultimate strength of the heat-treated wire (about 220,000 lb per sq.in.) is not appreciably above that of wire which has been used heretofore. The cold-drawn wire in the Manhattan bridge has an ultimate strength of 210,000 lb. per sq.in.; the specifications for the Philadelphia-Camden bridge raised the requirement to 215,000 lb. per sq.in. On the Fort Lee bridge, now under construction, the ultimate strength is designated as 220,000 minimum and 225,000 average. The wire of these bridges is galvanized; that of the Williamsburg bridge is bare or bright. On the Portsmouth (Ohio) bridge galvanizing was omitted and the wire supplied averaged over 230,000 lb. per sq.in.

Comparative stress-strain curves of the heat-treated wire and cold-drawn bridge wire are shown in Fig. 3. The yield point, taken (by specification) as the load

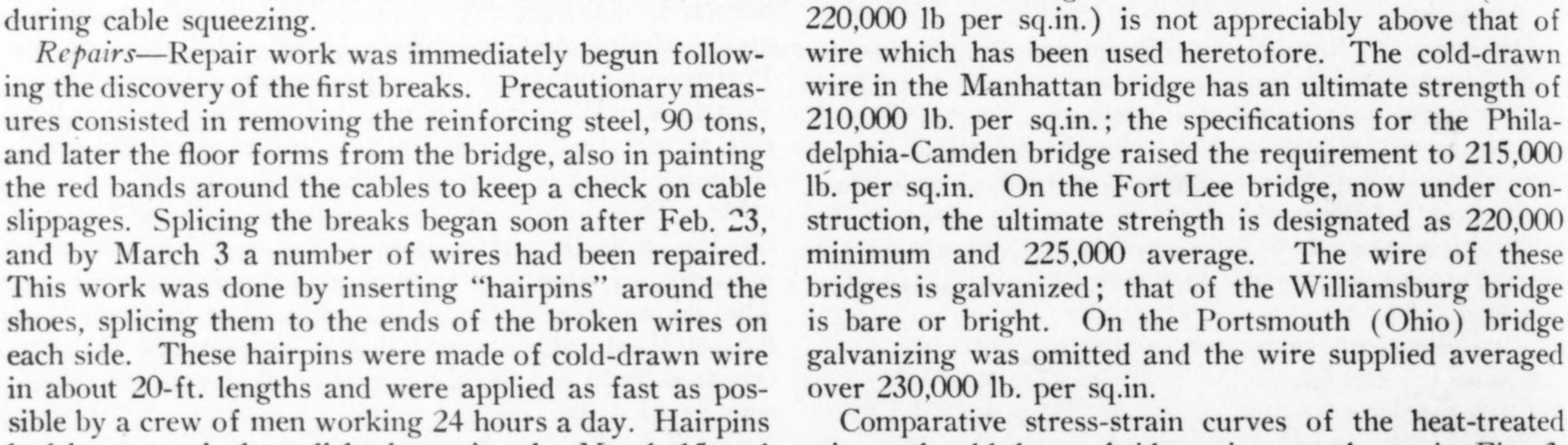


FIG. 5—THREE VIEWS OF THE CABLE-SPINNING OPERATIONS ON THE MOUNT HOPE BRIDGE
At left—Arrangement of equipment in the Bristol East anchorage showing reels of wire in reel frames; extra reels in pit at right. Center—Adjusting wires for sag. Note arrangement of strand shoes in three tiers. Right—Traveling wheel running on endless hauling cable and bringing a bight of wire from the far shore.

necessary to cause 0.75 per cent elongation, is about 190,000 lb. per sq.in. for the heat-treated wire and about 145,000 lb. per sq.in. for the cold-drawn wire. Tests are also reported to have demonstrated that the heat-treated wire has the high modulus of elasticity of 29,000,000, as compared with about 25,000,000 for cold-drawn wire. The manufacturers claim for the heat-treated wire a reduction in area of between 30 and 50 per cent.

Because of these qualities several advantages have been claimed for the heat-treated wire. In the first place, because of its high elastic limit, it is not given a permanent set when wound on 5-ft. blocks for shipment; conversely, when unwound it will lie straight on the ground. This feature was pointed out as contributing to ease of erection and as insurance against internal stress when the wire was laid up in the cable. Another claimed advantage was that the increase in strength below the yield point makes possible the use of higher working stresses than have heretofore been used. Also, because of the increase in the modulus of elasticity, these higher stresses could be used without increasing the deflection.

Cable Spinning—Cable spinning began on Sept. 10, 1928, and was completed on Oct. 25, 1928. The method used included several important innovations: (1) spinning directly in the permanent saddles, both on the main towers and the cable bents, rather than in temporary saddles from which the completed strands must be hoisted into place; (2) installing the anchorage eyebars in a flat rather than in a vertical position as on previous bridges; since cables are always spun with the shoes in a horizontal position the above practice eliminated the necessity for turning the shoes 90 deg. to attach them to the eyebars as would be necessary if the eyebars were vertical; and (3) preforming the wires as they passed around the strand shoe by bending them sharply around a small diameter sheave.

The equipment for cable spinning did not differ appreciably from that used on previous suspension structures. The footbridges were hung so as to give a continuous walk from anchorage to anchorage 5 ft. below the main cable curve. An endless hauling cable of wire rope was installed passing around large horizontal wheels at each anchorage, the wheel at the Bristol anchorage being driven by a 50-hp. motor and the one at the Portsmouth anchorage serving as an idler. Wooden bents fitted with sheaves and located at 250-ft. intervals along the footwalk supported the hauling cable. Two traveling sheaves or wheels were attached to the hauling cable at such a distance apart that they would pass in the center of the main span and each reach the side opposite from which it started at the same time.

Two reel frames holding two reels each were mounted on each anchorage. Each traveling wheel on the hauling rope carried a bight of cable wire from one reel across the footbridge, spinning two wires in each strand, and then returned with a bight of wire from the reel on the opposite anchorage, spinning two wires in a second strand. In this way two strands of each main cable were spun simultaneously.

The traveling wheel when coming into a strand shoe released the wire to the shoe as soon as the line end was clamped. A man stationed at the shoe then bent the free end of the wire around a 9½-in. diameter sheave (visible on top of the shoe in Figs. 4 and 5). This bending gave the wires a permanent set, and since the bending was done in a complete circle of half the diameter of the 19-in. shoe, the wire should upon release exactly conform to the lower half of the strand shoe. The lengths

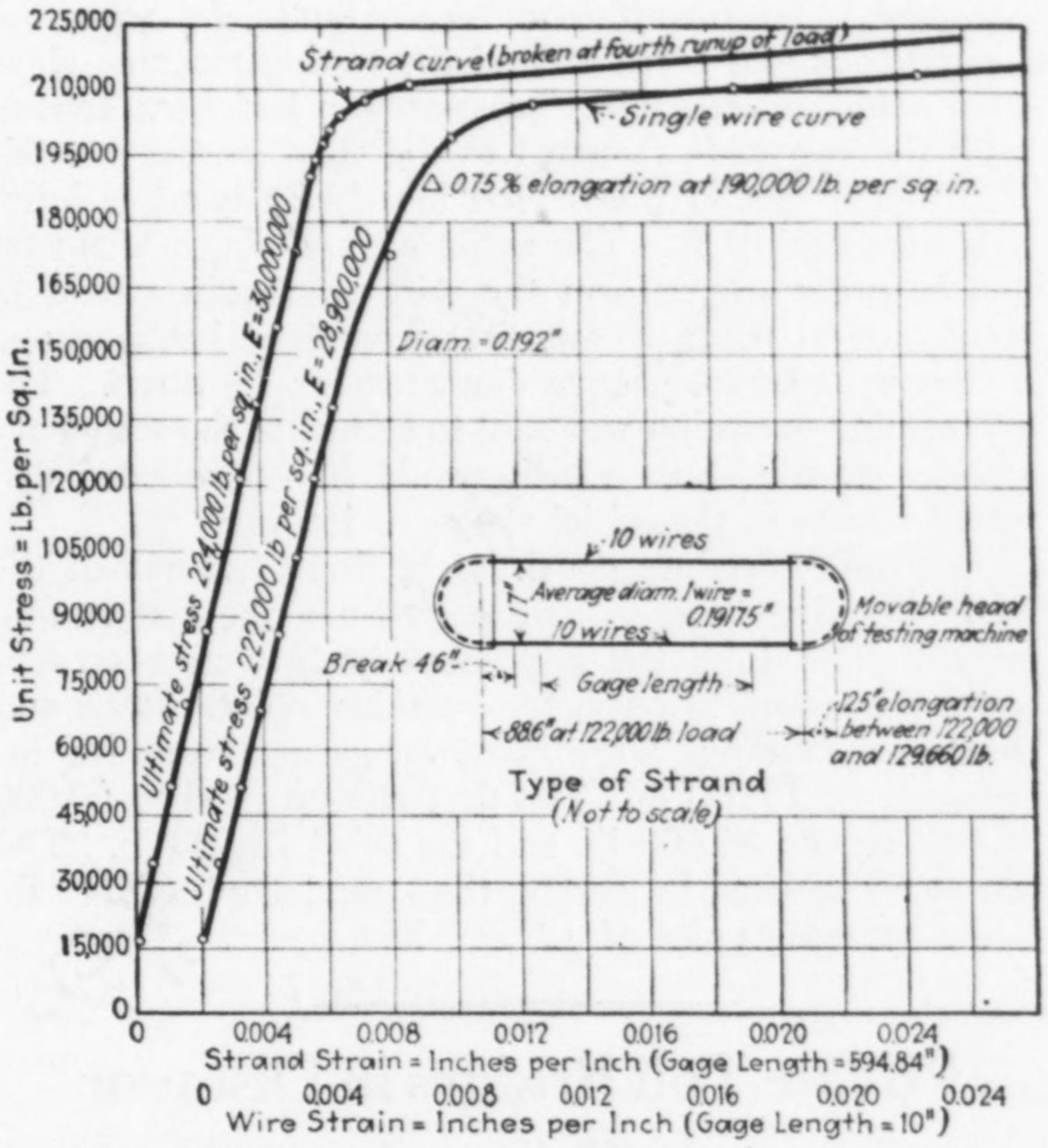


FIG. 6—STRESS-STRAIN DIAGRAMS OF A TWENTY-WIRE STRAND AND OF AN INDIVIDUAL WIRE (FROM PRELIMINARY SHOP TESTS)

of the strands were regulated by wedges in the strand shoes moved by hydraulic jacks. Tension during spinning was controlled by hand brakes on the wire reels.

Wire Manufacturing—In the manufacturing process, which is said to be continuous, the wire after drawing is passed through a lead bath and oil quenched, the galvanizing process following as an integral step in the manufacture. It is understood that the cold wire is first subjected to a normalizing process in which it is heated in lead to a temperature well above the critical range and then quenched in another lead bath at a temperature well below the critical range. The wire is then again heated in a lead bath to the proper temperature for hardening, quenched in an oil tank and as a final heating operation is tempered in another lead bath to remove the oil. Following this the wire is washed, cleaned and galvanized in the usual way and wound upon blocks 5 ft. in diameter.

It is understood that any length of wire required may be manufactured, since electric welds after being subjected to the heat treatment have shown a strength and ductility approximating that of the rest of the wire.

Test Results on the Wire—Data on some of the tests which were made prior to the adoption of the wire for the bridge are available. All show uniformly high results.

During the development of the wire magnetic analyses were made, utilizing an oscillograph for recording the results, and unusual uniformity of the wire was noted. The coils of wire which showed good magnetic analysis were later tested in 3-ft. lengths and showed uniform properties of tensile strength, elongation and reduction in area. In addition to these tests, long lengths of the wire were wound into close spirals and no brittle spots were disclosed in wire which showed a good magnetic analysis.

In Fig. 6 are shown the stress-strain diagrams of a strand containing twenty parallel wires of No. 6 gage and of a single wire from the same coil, the data originating in a shop test made in May, 1928, to determine the modulus of elasticity. The wire in the strand was approximately 2,000 ft. long and was cut from a coil which had been rejected because of a rough coat of galvanizing. To manufacture the strand, the wire was looped back and forth around the testing machine shoes until a loop of ten wires cross-section had been formed with the two ends fastened to the shoe on the movable head. The strand was bound loosely by hand at intervals of about 10 ft. The shoes were 17 in. in diameter and in order to preform the wire so that it would fit snugly about them, it was first wound by hand around a sheave of $6\frac{3}{4}$-in. groove diameter for $1\frac{1}{4}$ turns. The preforming operation was said to eliminate the secondary stresses at the shoes which would have produced distorted results in the initial stages of loading.

The test procedure consisted in three run-ups of the load, dropping back to zero each time, the last run-up being followed by the additional loading necessary to cause the strand to reach the breaking point. Load and elongation readings were made on each run-up for increments of 10,000 lb. The first run-up reached 60,000 lb., the second 80,000 lb. and the third 79,980 lb. The last step consisted in loading the strand from 80,140 lb. to the breaking point of 129,660 lb.

Private Toll Bridges in Disfavor in California

THE BUILDING of private highway toll bridges in California should be prohibited, according to a recommendation by the California State Highway Commission after a study of such structures throughout the state extending over a two-year period. This investigation, which was authorized by the 1927 state legislature, resulted in the recent publication of a 140-page report presenting several general conclusions supported by detailed statistical data on the financing and construction of toll bridges by private capital. The report recommends that the Highway Commission or the director of public works be empowered by law to acquire by purchase or by condemnation all privately owned toll bridges in the state.

The general finding is made that the cost and resulting toll charges in the case of the private toll bridges are higher than they would be for similar state-built bridges and that in addition there is a serious lack of conformity to the plans for a state highway system. Anticipated profits from promotion fees, contracts and tolls based on expected future traffic often inspire private promotion, it is charged, at locations where traffic does not justify a bridge. However, the conclusion is also drawn that private interests are apt to recognize the necessity of bridges at strategic points where public officials fail or are slow to see the need.

As a substitute for private toll bridges the report recommends that the state or county be empowered to finance and build toll bridges by the issuance of bonds payable through the income from such structures. Regarding the acquisition of existing privately owned bridges the report finds that condemnation affords the only means possible because of the intricate problem of fixing equitable value. The estimated cost of acquiring the present five toll bridges and one toll road in California is $20,156,300 based on the cost of the projects to the toll companies. The cost of reproducing similar bridges by the state is estimated to total $16,215,900.

More definite statistical data of the report in regard to the actual bridges studied are given in the following:

Promotion and organization are found to constitute a major expense item of the privately owned toll bridges. These costs in the case of the Carquinez and Antioch bridges, owned and operated by the American Toll Bridge Company, were found to total $1,156,776, as compared with an estimated promotion and estimated organization expense of $153,500 under state construction and finance. Similar expense items in connection with the San Mateo-Hayward bridge, including money and stock allotment, indicate a total of at least $785,670, compared with an estimated charge of $160,000 in the case of state construction for the same bridge.

The financing charge for the privately owned toll bridges studied is also declared excessive, particularly in regard to stock bonuses and interest charged on bonds. Financing of the Carquinez and Antioch bridges included a stock bonus of 500,000 shares and a $673,853 item for bond discounts. Investigations indicate that all of the 120,000 shares of common stock of the San Mateo-Hayward bridge had been issued to those directly interested in the project for services rendered without actual capital investment of money.

The interest paid for bond financing was found to average 7.7 and 8.7 per cent for the two largest bridge companies, including bond discounts and an estimated return on stock bonuses. In comparison, the interest rates on capital for state-built structures are estimated to be $4\frac{1}{2}$ per cent for state bonds sold at par or 6 per cent if bonds were secured by the income of the bridge built under public control.

The cost of constructing privately owned toll bridges was estimated to be from 10 to 25 per cent higher than for constructing similar public structures, principally due to lack of competitive bidding on the contracts. As a direct result of high expenses and cost the rate of tolls on private bridges is higher than would be necessary to operate and amortize the cost of state-built structures.

The public owns and operates 95 per cent of California's roads and bridges through state or county agencies. "It should not be necessary nor should private capital be allowed to pick out advantageous points on the highway system and build toll roads or bridges that will take profit that would otherwise tend to lessen the average cost of highway service on the entire public highway system. . . . The present enormous investment by the public in state and county highways is being capitalized by private toll bridge companies."

The steps necessary to make possible complete public control of toll bridges, primarily of a legal nature, are outlined in the following: The existing laws governing the issuance of franchises for toll bridges by delegating authority to counties are obsolete, inconsistent with the present idea of a state highway system and should be changed. The right to grant franchises either should be vested in the California Highway Commission or be made with the approval of that body, and the rates of toll should be under the jurisdiction of the California Railroad Commission.

Power should be given by law to the Highway Commission or to the director of public works to "locate, design, construct and operate" toll bridges and finance the same by issuing income bonds having as their sole security the income from tolls. The entire income from tolls, after deducting necessary expenses, should be used to amortize the cost of the structure, after which the bridge would be made free.

The investigation was made under the direction of C. H. Purcell, state highway engineer, and Charles E. Andrew, bridge engineer of the division of highways.

Dismantling Two Long Suspension Bridges

With No Precedents as Guides, the Contractor Removed Structural Members and Wire Cables on Mount Hope and Detroit Bridges in Record Time—Cable Was Separated Into Strands and Flame-Cut Into Sections for Convenient Handling

IN MARCH of this year engineering circles were shocked to learn of serious breakages in the cable wires of two important highway suspension bridges then under construction. News of the breakages was followed by a cessation of work on both structures and later by the decision to dismantle the bridges and replace the suspension systems with satisfactory wire. Interesting operations of unprecedented kind were involved in the dismantling. Both structures utilized the same kind of wire, a heat-treated product of high elastic limit, heretofore untried in suspension bridge service. The Mount Hope bridge, near Providence, R. I., which had a center span of 1,200 ft., was within about twelve months of completion, requiring only the placing of the concrete floor slab. The Ambassador bridge at Detroit, whose suspended span of 1,850 ft. ranked second only to the 3,500-ft. span of the Fort Lee Hudson River bridge under construction in New York City, was not so near completion, although its cables were finished and about one-third of its stiffening truss and floor steel was in place. The major wire breaks on both bridges occurred in the anchorages near the tangent point of the strand shoes, although numerous breaks were subsequently discovered under the cable bands in the main span at Detroit. No satisfactory explanation of the wire breakages has yet been found. Many specimens of single wires and of strands were furnished the Bureau of Standards from the Mount Hope bridge, and metallurgical and physical research investigators are at work studying the problem.

The same contractor was erecting both structures, so that when the decision came to replace the cable wire, both dismantling operations were speedily got under way. This article describes the methods evolved by this contractor on two pieces of work for which there were no precedents available as guides and yet which required the utmost dispatch in their execution.

Complete details of the bridges and the cable breakages have previously been given by articles in *Engineering News-Record* as follows: design of the Mount Hope bridge, April 12, 1928, p. 585; design of the Ambassador bridge, Sept. 27, 1928, p. 461; tower erection, Ambassador bridge, Jan. 31, 1929, p. 186; first news of wire breakages, March 28, 1929, p. 516; discontinuance of work at Detroit, April 4, 1929, p. 564; and analysis of breaks at Mount Hope, April 11, 1929, p. 602. —EDITOR.

DISMANTLING operations on the Mount Hope and Ambassador suspension bridges, near Providence, R. I., and Detroit, Mich., respectively, occasioned by wire breakages in the cables, have been completed and reconstruction work is now under way on both bridges. This article covers the methods used in dismantling the two structures, and in the case of the Mount Hope bridge considers the precautionary measures taken to safeguard the structure after the first breaks were discovered. Precautionary measures were not considered necessary at Detroit, since the load being carried was only about one-third of that at Mount Hope.

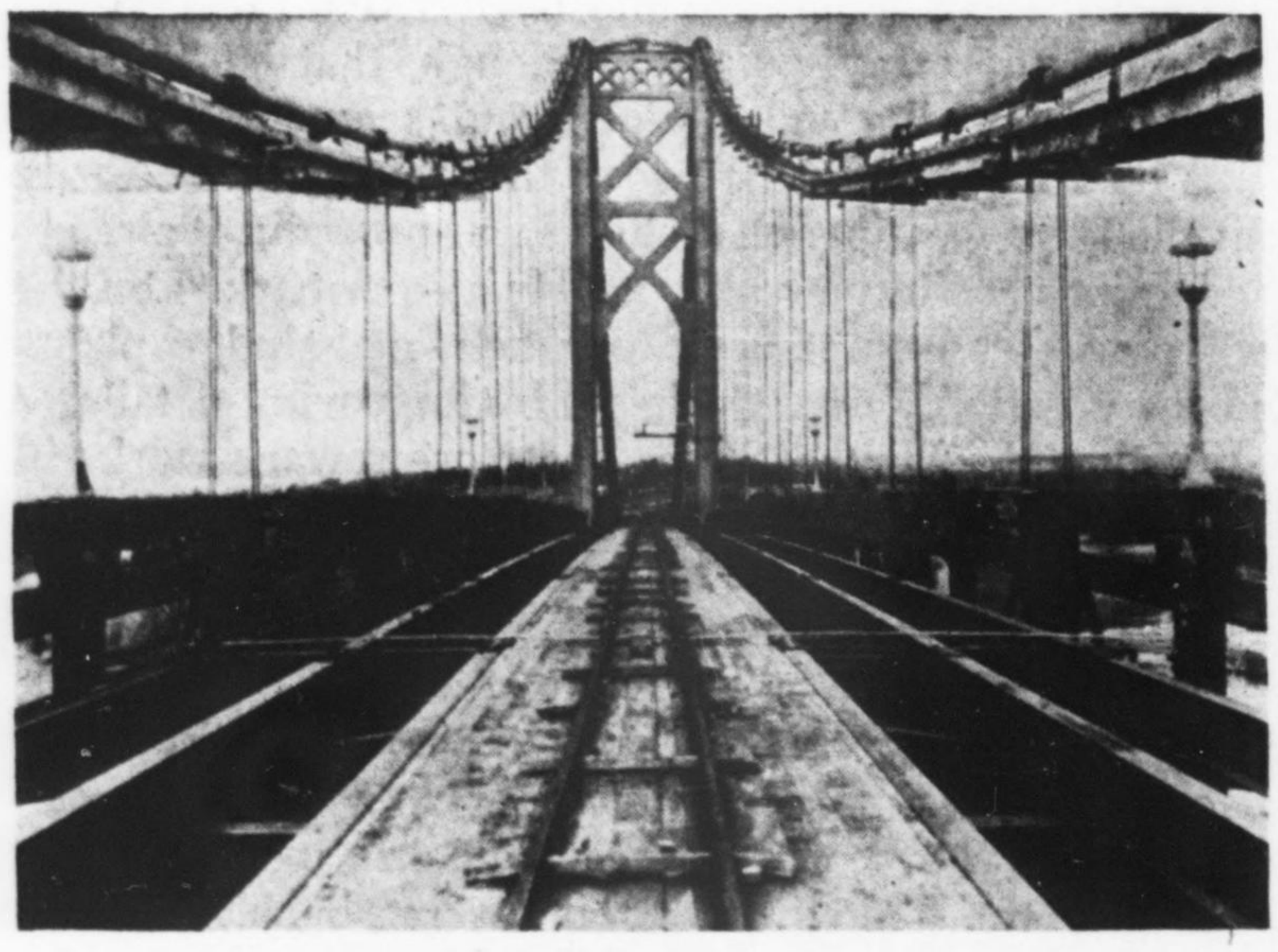


FIG. 1—MOUNT HOPE BRIDGE JUST PRIOR TO DISMANTLING

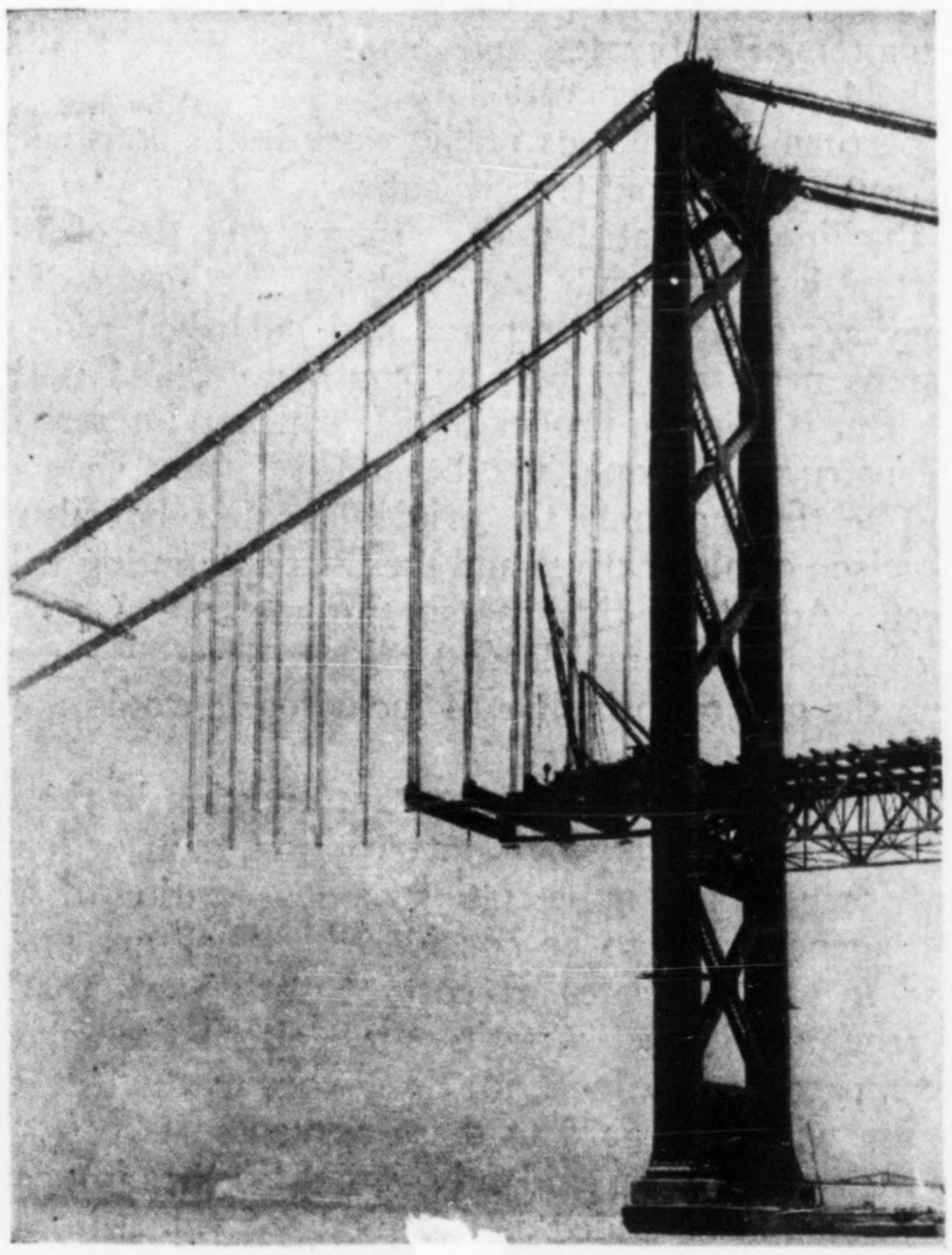


FIG. 2—AMBASSADOR BRIDGE BEFORE DISMANTLING

Four of fourteen panels of floor steel that had been erected before dismantling began are still in place.

Although the same contractor carried out both jobs, there are marked differences in the dismantling procedure. The work was new, and the designs of the two bridges differed. At Mount Hope, the stiffening trusses were erected complete in two-panel lengths, whereas at Detroit a design was used which resulted in the erection of the bottom chords of the stiffening trusses along with the floorbeams, which were hung from the suspenders, as the initial operation. The separate wire strands in the Mount Hope cable were heavier than the strands at Detroit; the anchorage eyebars were installed with flat side horizontal at Mount Hope and vertical at Detroit; the anchorage was concreted in at Detroit, whereas it was

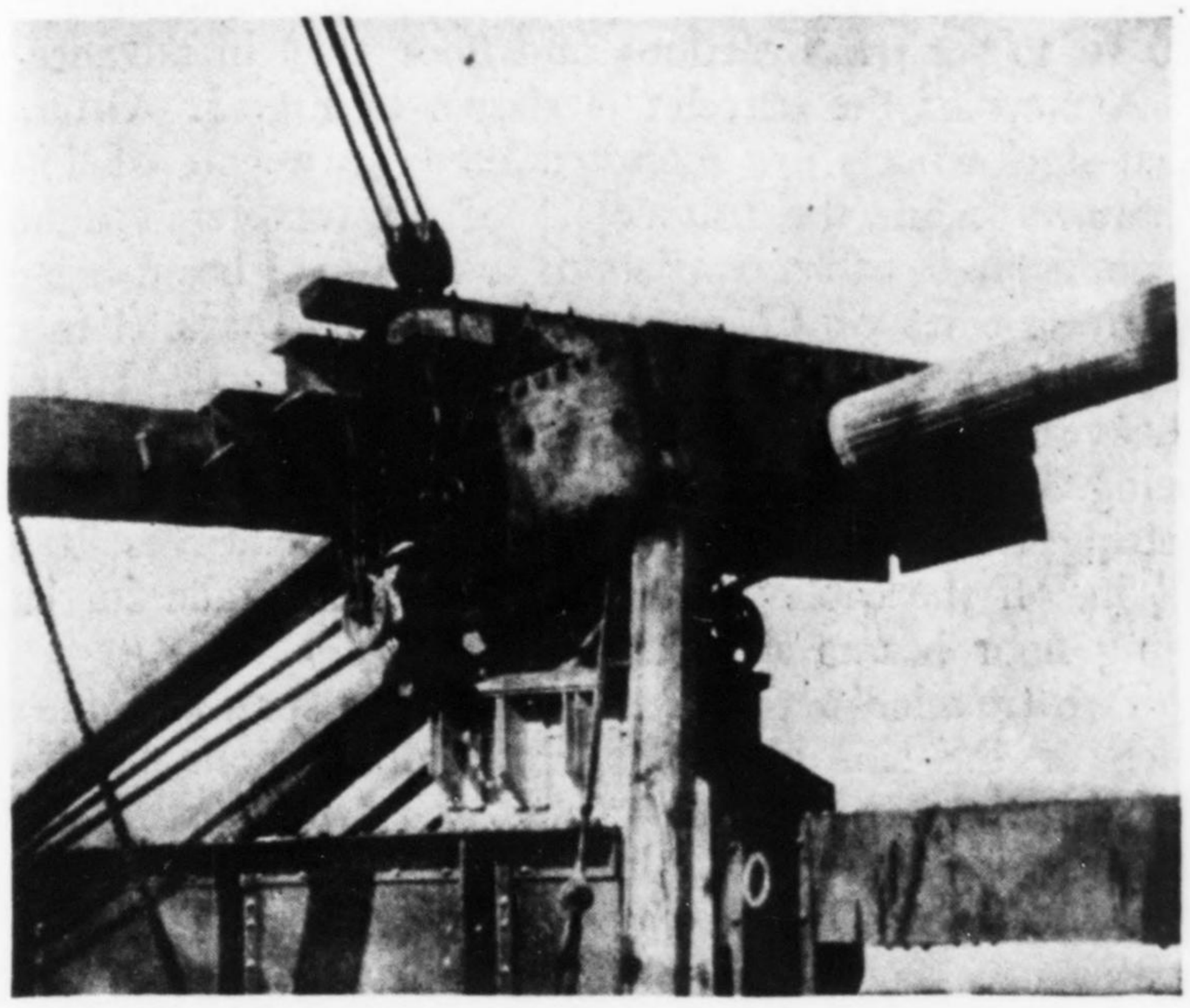


FIG. 3—HOLDBACK YOKE IN FRONT OF CABLE-BENT SADDLE, MOUNT HOPE

Four holdback ropes are visible and four similar ones are attached at the far side of the yoke.

to be sand-filled at Mount Hope; and there were several other major differences.

Mount Hope Bridge

The Mount Hope bridge is located about 15 miles southeast of Providence, R. I., and spans the strait connecting Mount Hope and Narragansett bays. It is a highway toll crossing of wire-cable suspension design with a 1,200-ft. suspended main span and 504-ft. suspended side spans. Cable-bent towers are used at the ends of the side spans, the backstay portion of the cable between these towers and the anchorages covering a horizontal distance of 220 ft. The main towers, of structural steel, are 284 ft. high and carry two 11-in. cables 34 ft. apart, each cable consisting of seven strands of 350 No. 6 wires. The stiffening trusses are 18 ft. deep, discontinuous at the towers, and fabricated in two-panel lengths totaling approximately 38 ft. Floorbeams placed about 19 ft. on centers rest on kneebraces, while the stringers and transverse slab beams are erected on top of the floorbeams.

At the time of the discovery of the wire breaks, the bridge was substantially complete with the exception of the concrete floor slab. Forms and reinforcing steel in considerable quantities were in place and the cables were stressed to about 32,000 lb. per sq.in. (the intended maximum working stress was to be 80,000 lb. per sq.in.). Because of this comparatively heavy load and because the cable wires were continuing to break at intervals, precautionary measures were immediately devised to relieve the strain on the cables. At the same time, since the breaks were localized near the strand shoes in the anchorages, hairpins of new wire were spliced to the broken ends.

Precautionary Measures—Two systems of holdbacks were devised to relieve the strain on the cables. One system was applied just ahead of the cable-bent castings on the side spans; the other was placed on the backstays and consisted of yokes bearing against cable clamps attached with each yoke tied back to the anchorage eyebars.

The yoke of the holdback system at the cable-bent tower is shown in Fig. 3. It was made up of two I-beams placed flat transversely to the cable, one above and one below the cable and inclosed into a box shape by the bolted plates shown in the illustration. In this box and on each side of the cable two sheaves were mounted in a flat position on pins which were seated at the top and bottom in the webs of the I-beams. Around each of these four sheaves $1\frac{3}{4}$-in. plow-steel ropes with hemp centers were passed, running down to the anchorage eyebars. Fortunately these eyebars had not been concreted in. Also, since they were installed in a flat position, the holdback ropes were clamped around the head of the bottom eyebar of a pair. Thus each cable-bent holdback system consisted of eight parts of $1\frac{3}{4}$-in. rope at each of the four anchorages. Since each holdback rope was fitted with a turnbuckle with an effective strength of 25 tons, the load carried in each anchorage by the cable-bent holdback system was limited to 200 tons.

The structural-steel yokes of the backstay holdback system were tied to the anchorages by $1\frac{3}{4}$-in. ropes with steel centers and plow-steel strands and were designed to remove all of the remaining load from the bridge cable, should this prove necessary; it was estimated that twelve yokes, each connected to the cable by two ropes stressed to 50 tons, would completely remove all load from the main cables. Although only one of the twelve proposed yokes was actually applied, the method of application was interesting enough to record.

Briefly, this involved placing the yokes around the backstay, connecting them to the anchorage by two-part cables, stressing these cables by means of hydraulic jacks and holding the yokes in a stressed position by heavy cast-steel cable clamps. The apparatus necessary to effect this is shown in Fig. 4. It consisted of three yokes, the upper two of which were for erection purposes only. The top and bottom yokes were connected by angles into a rectangular steel frame; the middle yoke was loose,

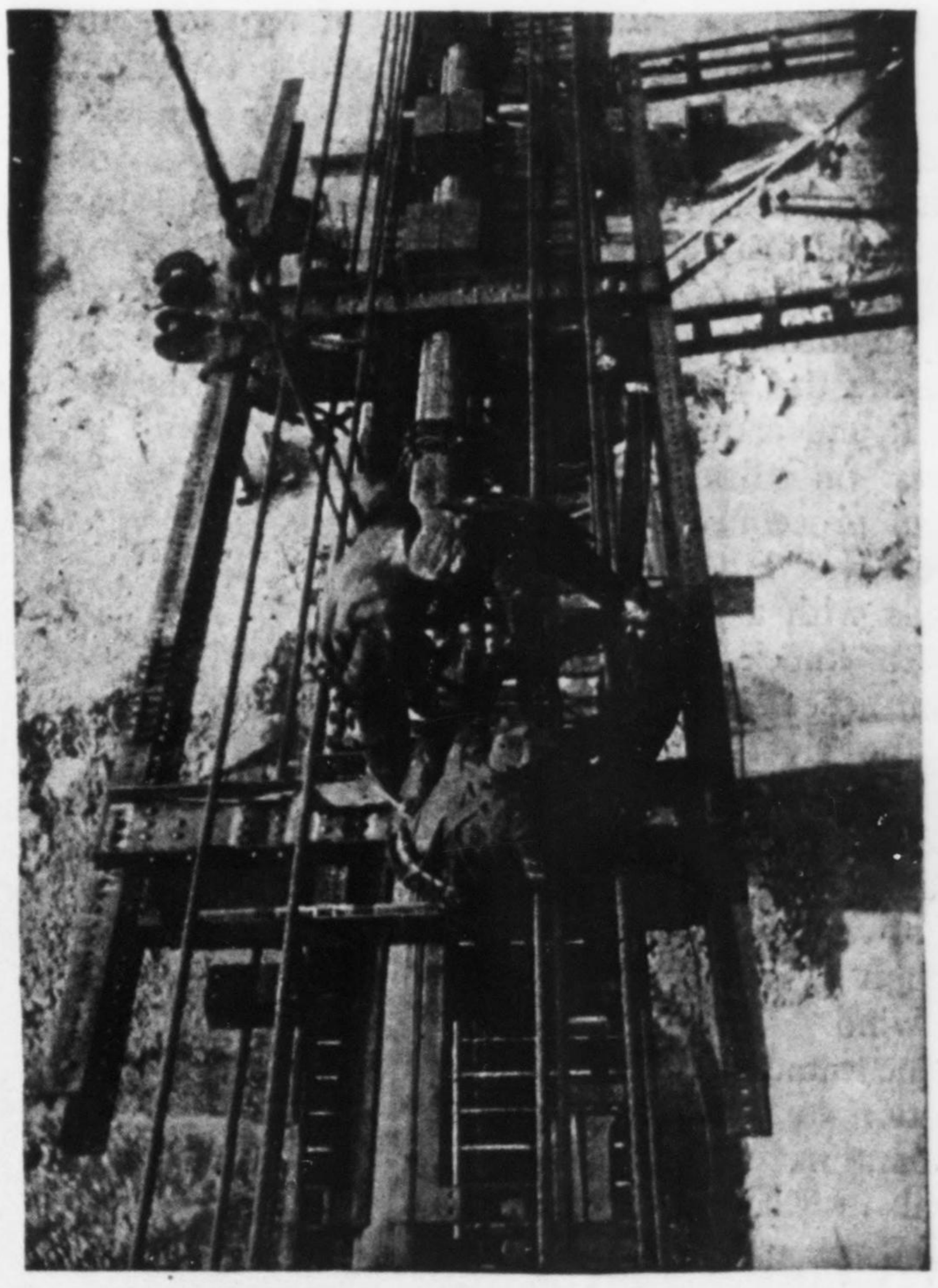


FIG. 4—REMOVING THE EQUIPMENT USED TO INSTALL BACKSTAY HOLDBACKS, MOUNT HOPE

Yoke upon which men are working is ordinarily not attached to side angles. Top yoke has been removed. Bottom yoke rests against cable clamp; end of hold-back rope, which has been cut off, can be seen in bottom yoke just inside of right-hand side angles.

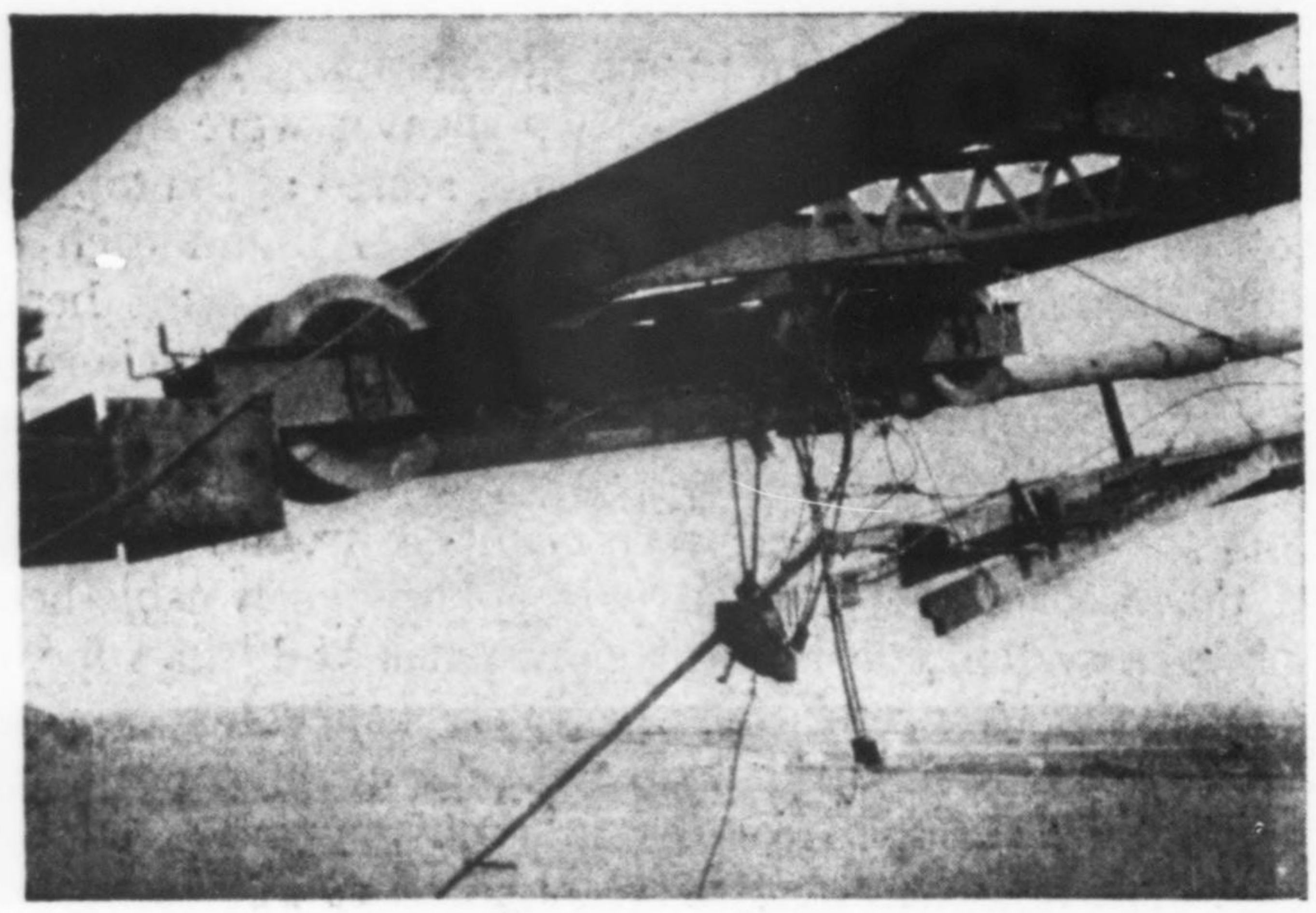


FIG. 5—TRAVELERS AT MOUNT HOPE DESIGNED TO RUN ON CABLES

Sheave shown used for setting truss sections. Another sheave just out of picture at the right carried falls for handling floorbeams and stringers. Cable bent holdback yoke at left.

but rested against a cable clamp as shown. This middle yoke, being fixed in position by the cable clamp, served as a reaction girder for hydraulic jacks set between it and the upper yoke. As these jacks were run out, they moved the top yoke and therefore the rectangular frame which included the bottom yoke up the cable, stressing the holdback ropes attached to the latter. These jacks, being run up to a gage load of 50 tons, placed a corresponding load on the holdback ropes. A second cable-clamp was then applied below the bottom yoke, holding it in the stressed position and permitting the release of the jacks and the removal of the two top yokes and the side angles of the rectangular frame. Thus one set of holdbacks was in position and the upper two yokes were free to repeat the operation. Instead of removing the upper two yokes, it would have been possible to leave them in position and merely extend the side angles of the rectangular frame down to another bottom yoke. By this method these 100-ton holdbacks could have been increased to twelve on each backstay cable.

Dismantling Travelers—With only one of the backstay holdbacks in place, further failures in the cable wire ceased; the contractor then discontinued the preventive work and concentrated all efforts on dismantling. The usual procedure for removing rivets was either to cut off the heads with a pneumatic chisel or to knock them off with blunt-nosed tools. The shafts of the rivets could then be punched from the holes and replaced with bolts. The smallest possible number of bolts was used in general only enough to steady the steel in the wind.

The same travelers used in erection were available for the dismantling. They were unusual in that they were designed to run on the cables rather than on the deck, the ordinary deck traveler being considered too heavy for the comparatively light cables of the bridge. Also, the floor design of the structure was such that a deck traveler would have had to boom out about 40 ft. to set truss sections and floor steel in advance.

A view of the traveler is shown in Fig. 5. Grooved cast-steel wheels are mounted between a pair of 15-in. channels, while the framework of the traveler, spanning from cable to cable, consists of two 18-in. I-beams spaced by four pairs of 12-in. channels running parallel to the cables. These 12-in. channel frames carry the hoisting sheaves, those shown in the accompanying illustration being 3 ft. off the center line of the cable and used for setting truss sections. Another set of sheaves 10 ft. $7\frac{1}{2}$ in. off the cable center line carries the falls for handling floor beams and stringers.

Two travelers were used on the main span and one on each of the side spans. Each required four lines, an uphaul cable, a downhaul cable and two hoisting falls. Two three-drum electric hoists placed on the pier at the base of each main tower furnished twelve of the sixteen drums necessary to supply these lines. The four other drums were obtained by placing two-drum electric hoists on the approaches to operate the uphaul and downhaul cables of the side-span travelers. All uphaul cables were three-part lines; the downhaul cables on the center-span travelers were two-part lines and on the side-span travelers single lines.

Stiffening Trusses and Floor Steel—Removal of structural steel was practically the reverse of erection. Since the steel had not yet received its first field coat of paint, it was necessary only to freshen up the match marks on the truss sections. On the other hand, new match marks were given to floorbeams, stringers, laterals and plates.

The stringers and laterals, as well as the floorbeams at stiffening-truss splices (alternate floorbeams), were removed on the first pass of the travelers, followed by the stiffening trusses and the remainder of the floorbeams on the second pass. For the first operation, the side-span travelers started at the cable-bent towers, whereas the main span travelers started at the center of the span, all working toward the main towers. When all had arrived at the main towers, there remained the stiffening trusses and the alternate floorbeams at the suspenders to dismantle; this steel was lowered as all of the travelers worked away from the main towers. By

FIG. 6—DISMANTLING TRAVELERS WORKING AWAY FROM MAIN TOWERS, MOUNT HOPE

Note that side spans are down except for one panel. Before the last ten panels in main span could be removed, the main tower saddles had to be jacked shoreward 10 in. to maintain the system in balance.

this procedure the suspension system was kept balanced at all times. Also, the steel was piled in the storage yard in the order in which it was to be re-erected.

The truss sections were removed in two-panel lengths (38 ft. long) as they had been erected. They had been connected to the suspenders by sockets attached to the gusset plates at the panel points so that the dismantling operation involved lifting the trusses a short distance to disengage the sockets and then lowering away to the barge below.

Before the last ten pairs of truss sections at the center of the main span could be removed (all the side-span trusses having been previously taken down as shown in Fig. 6) the saddles on the towers had to be jacked back 10 in. toward the anchorages—the reverse of the operation performed during erection. Graphite had been spread on the base plate when the saddles were erected, so that two 25-ton screwjacks easily moved them the required distance (Fig. 8). After these ten sections of stiffening trusses had been lowered, jacking was continued for $10\frac{1}{4}$ in. more in order to restore the towers to a plumb position.

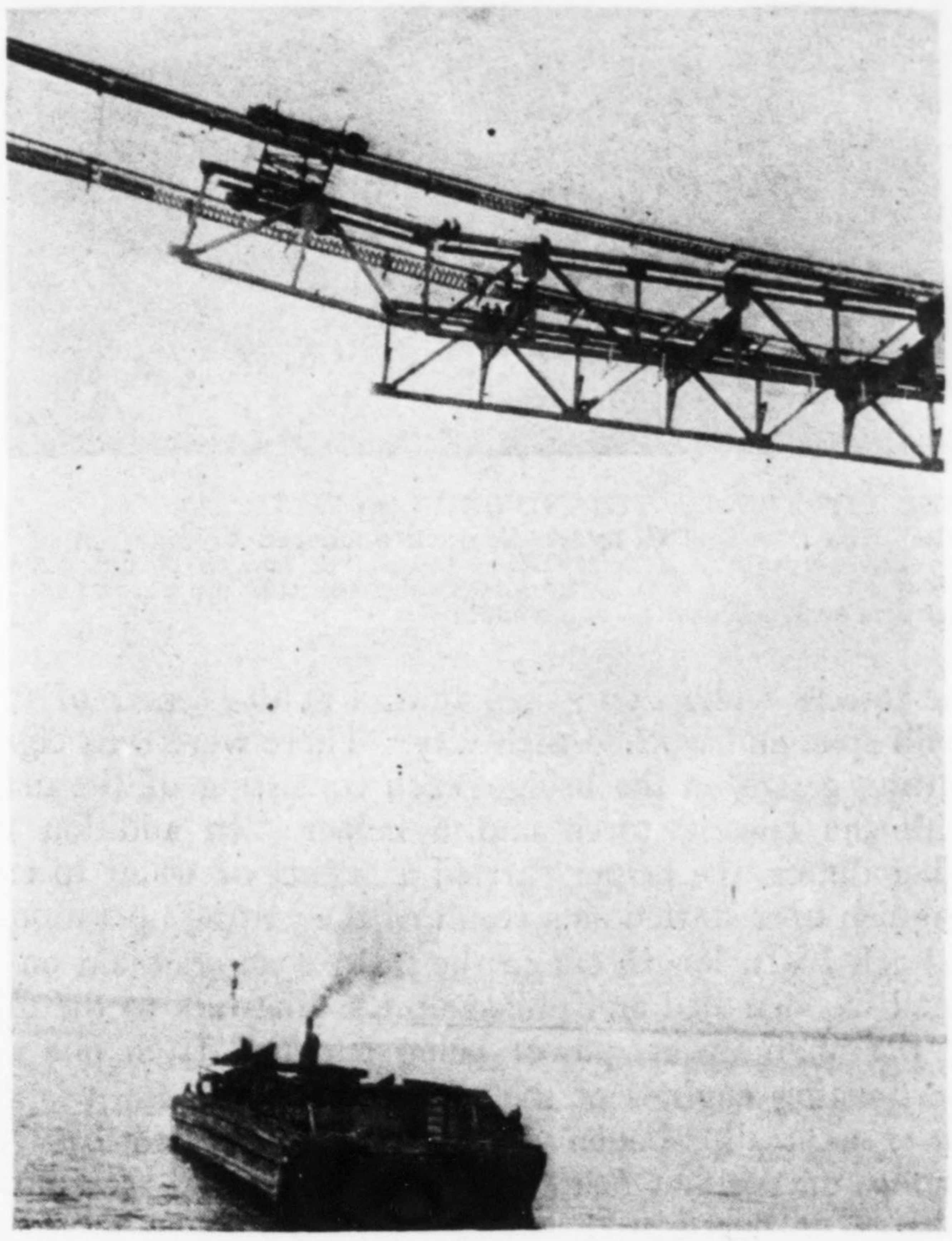


FIG. 7—TRAVELER LIFTING TRUSS SECTION FREE OF SUSPENDERS, MOUNT HOPE

Note alternate floorbeams at suspender points. Floorbeams at splice points were lowered on first pass of traveler. Truss section has just been deposited on barge.

The third dismantling operation consisted in removing the suspenders and cable bands. The number of each suspender was stamped on the socket with a letter to indicate whether it was the long or the short member of a pair; suspenders were dropped to barges and stored on the Portsmouth shore. Cable bands were dragged up the footwalk to the towers and lowered to the piers for for storage.

New Footwalks—Before actual work could commence on cable removal new footwalk cables had to be erected, since the old footwalk cables had been cut up into suspender ropes and the footwalks hung from the bridge cables by wire rope lashings at 20-ft. intervals. The new footwalks cables, two for each walk and 2 in. in diameter, were pulled across the footwalk from the Bristol end, fastened to the original footwalk cable anchorages at the Portsmouth end and pushed off the walk so that they hung below it. Then, adjusting from the Bristol ends, these ropes were tightened until they picked up the footwalk and relieved the strain on the lashings. The final operation consisted in making the ropes fast to the Bristol anchorages.

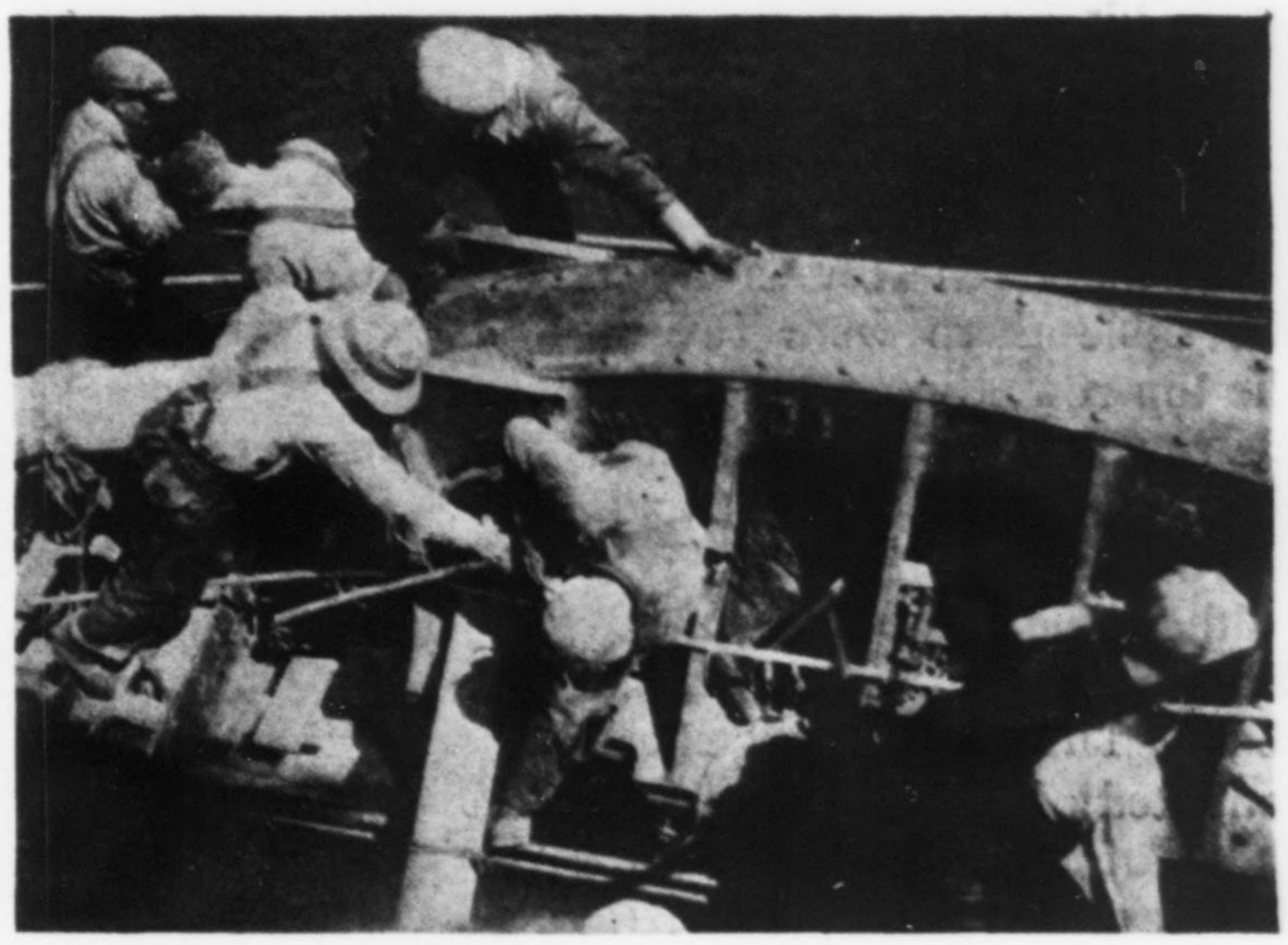


FIG. 8—JACKING THE MAIN TOWER SADDLE AT MOUNT HOPE

Cable Removal—Since there was no necessity for removing the cable wire intact, the most rapid means of dismantling was adopted—namely, separating the cable into strands and cutting the strands to convenient lengths by means of oxyacetylene torch. A total of fourteen strands, seven to each cable, was dismantled in six working days and the entire 700 tons of old wire was transferred to a large packet barge and towed away.

At the time the cable dismantling began the two cables were in their original unloaded position and bound only by the three turns of seizing wire which had been put on at 3-ft. intervals after the squeezing. Upon removal of these permanent seizings each 11-in. cable became essentially seven strands, each 4 in. in diameter and about 2,820 ft. long from strand shoe to strand shoe. Each wire weighed about 1 lb. to every 10 ft., hence the weight of the 4-in. strand of 350 wires was approximately 35 lb. per foot. It was decided that a weight of about 500 lb.

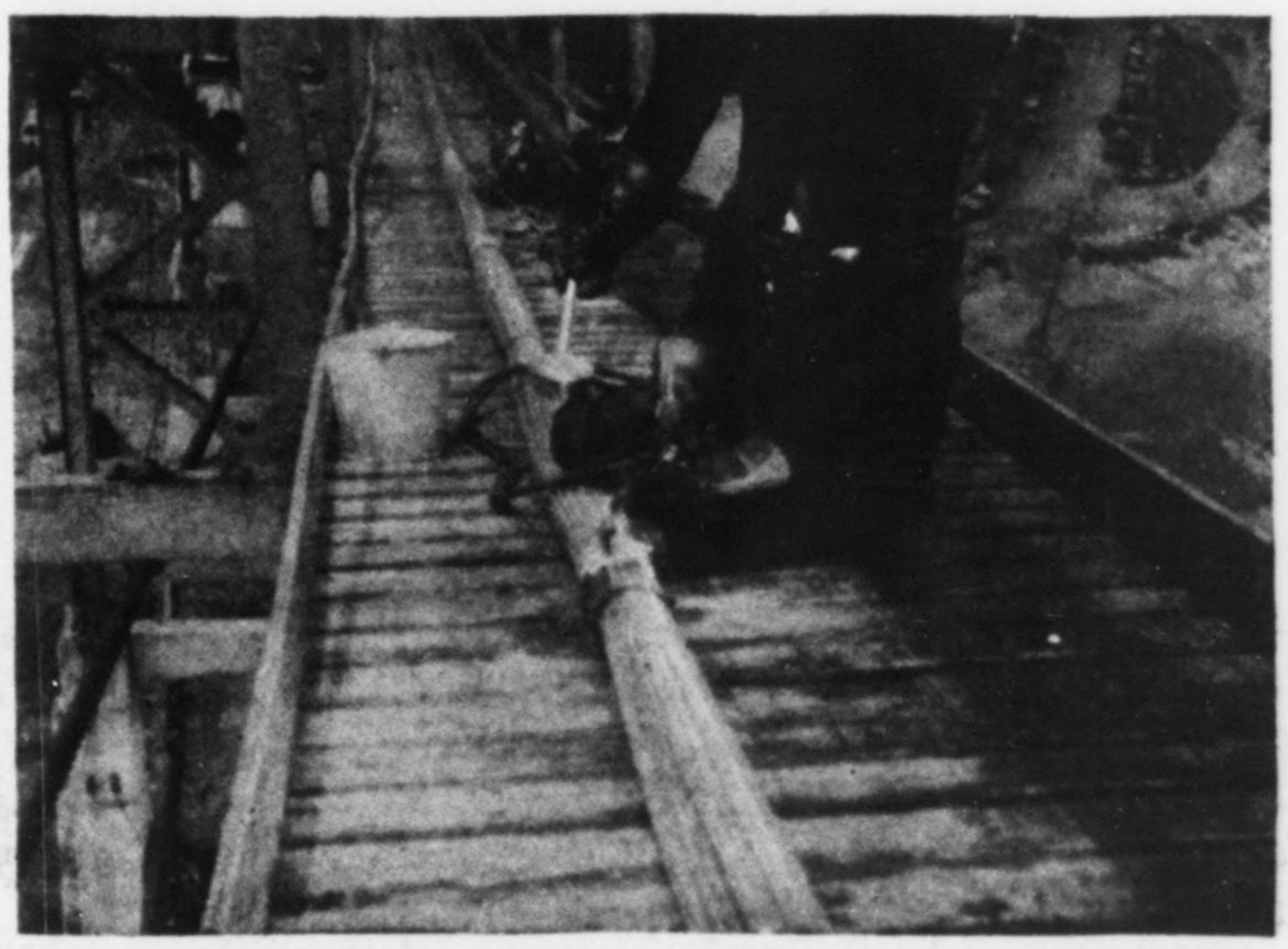


FIG. 9—CUTTING A STRAND INTO 15-FT. LENGTHS AT MOUNT HOPE

Helper moves sleds carrying gas tanks and stands by with pail of water to put out any fires in footwalk.

could be economically handled, and the length of such a section, 15 ft., was found not to be unwieldy.

As the seven strands lay in the cable there were two in the top row, three in the middle row and two in the bottom row. One of the top strands was first raised from the rest of the cable by applying a 50-ton pulling-jack to the strand shoe. The jack arrangement consisted of a yoke placed just ahead of the shoe attached to the jack which lay parallel to the anchorage eyebars, its lower end connecting to a large hook which passed around the pin at the lower end of one of the eyebars in the first link of the anchorage.

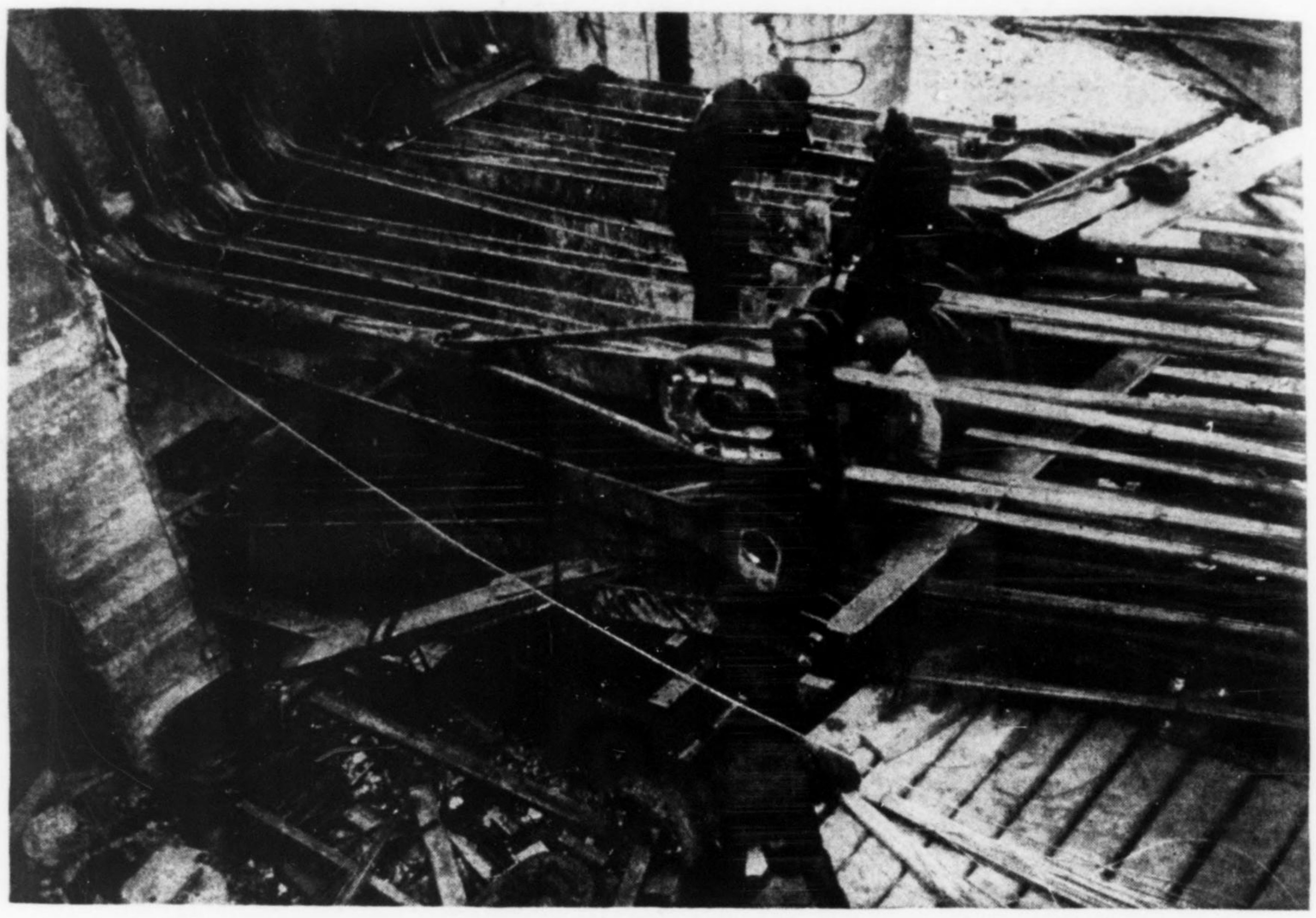


FIG. 11—REMOVING PIN FROM A STRAND SHOE AT DETROIT
Note yoke ahead of strand shoe and attached to hydraulic jack anchored to rear pin of anchorage eyebar. Note also concrete adhering to eyebars; 150 cu.yd. had to be cut out before the dismantling work could begin. At bottom in foreground the take-up screws on the footwalk anchorage are visible.

By pulling a few inches on the jacks at both ends of the strand the sag in both the main and the side spans of that particular strand was lessened and the strand was raised above the six others. In this position two light wire seizings were applied 6 in. apart every 15 ft., followed by a third seizing placed at the center of each 15-ft. length. With the strand shoe thus pulled back, the shims which were in place between the shoe and the eyebar pin could be removed and the pin driven out. Slacking off on the jacks then allowed the strand shoe to go ahead of its normal position, and as a consequence the strand came to rest on the footwalk, the new footwalk cable having been designed for the additional load of 35 lb. per foot. Then, since most of the load was on the footwalk, the strand was removed from the cable-bent saddles by man power and from the main tower saddles by a small jinniwink.

With the strand lying on the footwalk, four gangs with ordinary acetylene torches cut the strand into 15-ft. lengths, burning between the two seizings 6 in. apart. One gang started at each anchorage and worked toward the towers while two gangs started at the center of the main span and worked each way. There were thus eight cutting gangs on the bridge, each consisting of the man with the cutting torch and a helper. In addition to other duties, the helper carried a bucket of water to extinguish fires started as a result of the cutting operations.

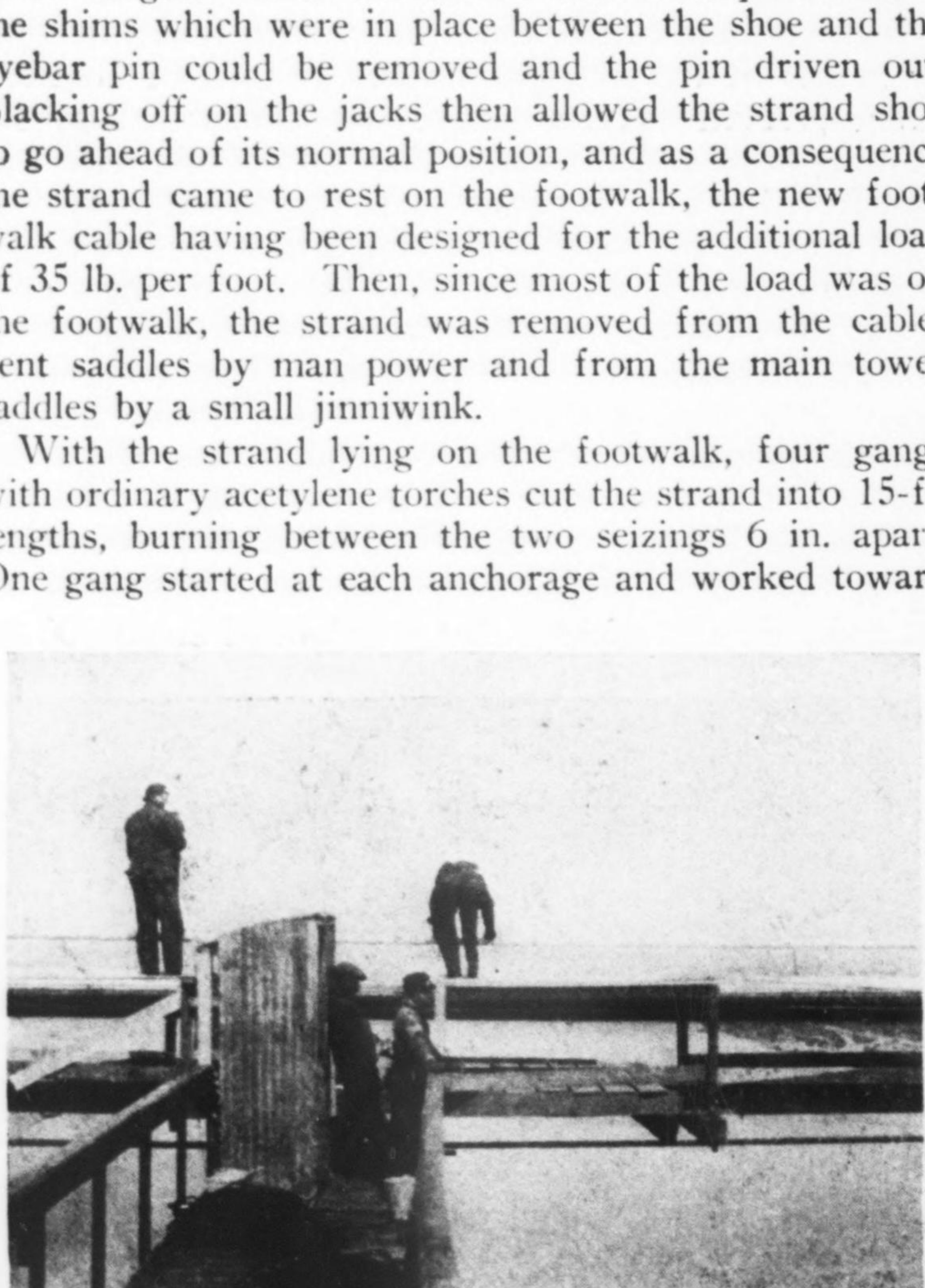


FIG. 10—A STRAND JACKED FREE OF THE CABLE IN THE MAIN SPAIN AT DETROIT
Cross footwalk in foreground.

Each 15-ft. length cut in the main span was laid on a small wooden sled and pulled up the footwalk to the top of the main tower, power being supplied from one of the hoisting engines at the tower base. On top of the tower a small wooden derrick (also operated by the engines on the pier below) lowered the strand section to a barge anchored at the pier.

In the side spans the burned sections were placed on sleds and dragged down the walk by hand to a point near the anchorage, where they were lowered.

At the time of commencing cable dismantling a tentative schedule was arranged which allowed five minutes to cut a length, five minutes to move the sled, ten minutes to drag the length to the towers and ten minutes to lower. In actual practice it required from four to five minutes for each cut, and three cuts were made before moving the tanks once. The tentative schedule was followed very closely.

The Ambassador Bridge at Detroit

The Ambassador bridge, over the Detroit River, joining the city of Detroit to the border cities of Canada, has a main suspended span of 1,850 ft. and unloaded backstays. Each main tower, built of steel, is 363 ft. high above the masonry pier. Stiffening trusses with a panel length of 24 ft. $2\frac{3}{8}$ in. and a depth of 22 ft. are used. Stringers spaced 4 ft. 8 in. on centers and fabricated in two-panel lengths rest on top of the floorbeams, long stringers being adopted to eliminate danger of deformation in the connections during passage of the erection traveler. The floorbeams are single-web plate girders, to the ends of which the bottom-chord gussets of the stiffening trusses are attached in the shop. By virtue of this arrangement the traveler on its first pass

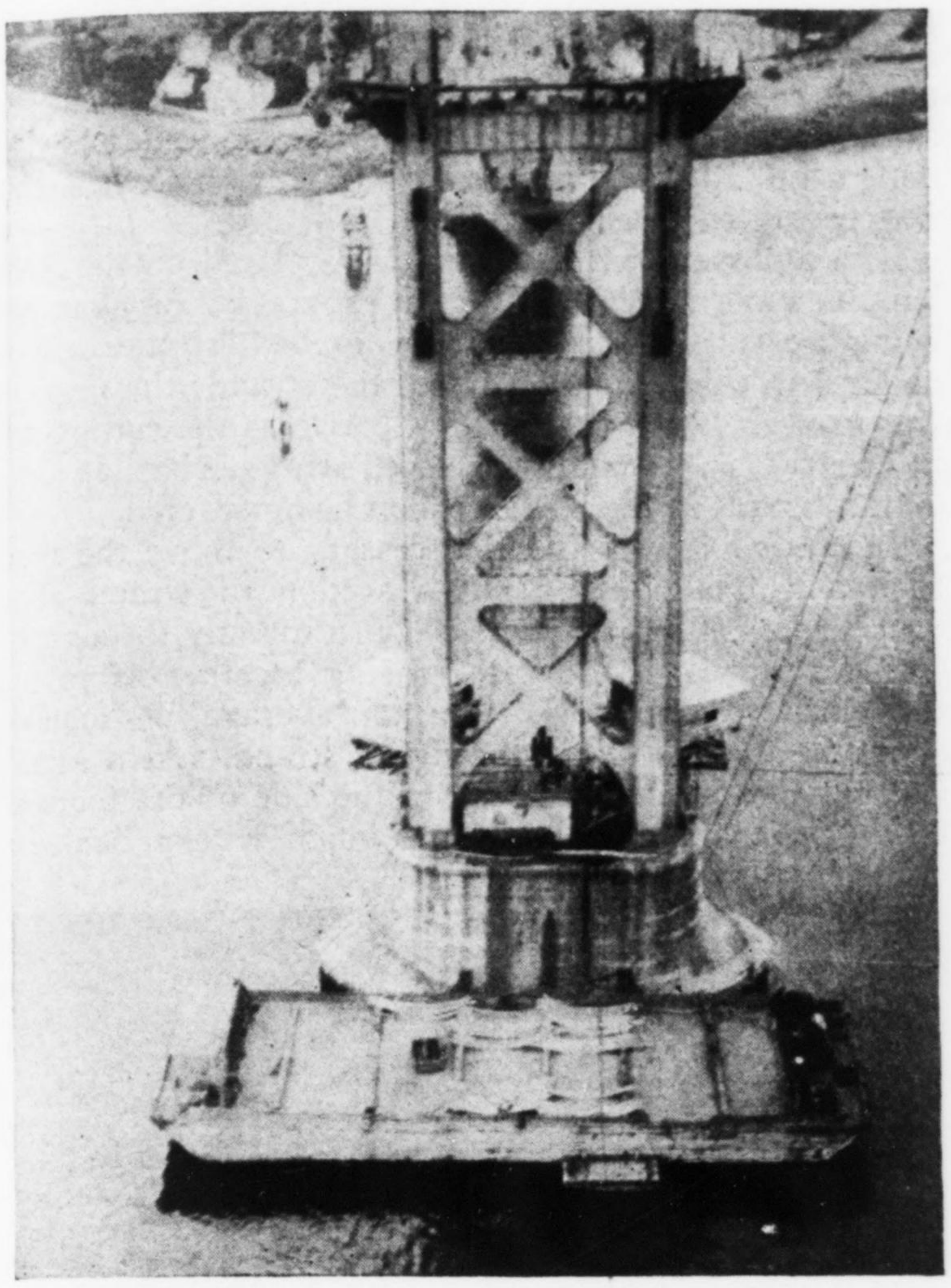


FIG. 12—SEVERED SECTIONS OF CABLE ON BARGE AT BASE OF MAIN PIER, MOUNT HOPE
Note houses on far side of tower inclosing hoists which operate the travelers.

hung the floorbeams from the suspenders, hoisted the bottom chord into the space between the gussets, set the laterals and enough stringers to run on, and proceeded. The bottom chords are spliced at main panel points and fabricated as straight members approximately 48 ft. long. Each of the two cables of the bridge comprises 37 strands of 206 No. 6 galvanized wires, of the same size and manufacture as those used on the Mount Hope bridge.

The Detroit bridge was not so far advanced toward completion as was the Mount Hope bridge when work was ordered stopped on March 1. According to estimates, the cables were stressed to about 10,000 lb. per sq.in. Precautionary measures such as used at Mount Hope were therefore not considered necessary, and dismantling work was immediately begun.

Dismantling Structural Steel—Fourteen panels of structural steel similar to those shown in Fig. 2 and extending outward approximately 340 ft. from each main tower were in place when dismantling work began. This steel comprised floorbeams, bottom chords of stiffening trusses and four lines of stringers. No riveting had been done, hence removal of the structural steel was easily accomplished. The procedure was the exact reverse of erection, the deck travelers lowering the stringers, bottom chords and floorbeams in sequence to barges in the river below. Structural steel dismantling began on April 2, and was completed on April 10.

Cable Wire Removal—The anchorages had been completely concreted in, so that before any steps could be taken to release the cable strands the first link of the eyebar anchorage chain had to be exposed. This involved cutting away 150 cu.yd. of concrete in each of the four anchorages, a task accomplished by the use of pneumatic hammer drills. A view of the east Detroit anchorage with the concrete removed is shown in Fig. 11.

As at Mount Hope, the footwalks were lashed to the cable, the footwalk cables having been cut up into suspenders. Two $1\frac{1}{2}$-in. diameter wire ropes were therefore erected to support each side of each footwalk. This operation was unusual because in order to take up the great stretch in these new ropes and to lift them sufficiently to carry the footwalks, ratchets and turnbuckles were necessary over the tower saddles, while at the anchorages the ropes were strained by steel cable falls and hydraulic jacks. Take-up screws fitted with 7 ft. of thread were provided in all anchorages and it was necessary to use about half of the available take-up before the footwalks were properly swung.

The cable was separated into its various strands by operations similar to those used at Mount Hope. Yokes placed in front of the strand shoes in both the American and the Canadian anchorages were connected to 40-ton jacks, the opposite ends of which were hooked over the rear pins of the first anchorage eyebars (Fig. 11). A 42-in. movement was available in the jacks, and after the shoes were pulled back, permitting removal of the pins, jacking was continued until the strand shoes had moved backward approximately 2 ft. in each anchorage. This jacking raised the strand about 1 ft. above the rest of the cable in the main span (Fig. 12) and from 2 to 3 ft. above in the backstays. As the jack was eased off the strand was laid on the footwalk for the greater part of

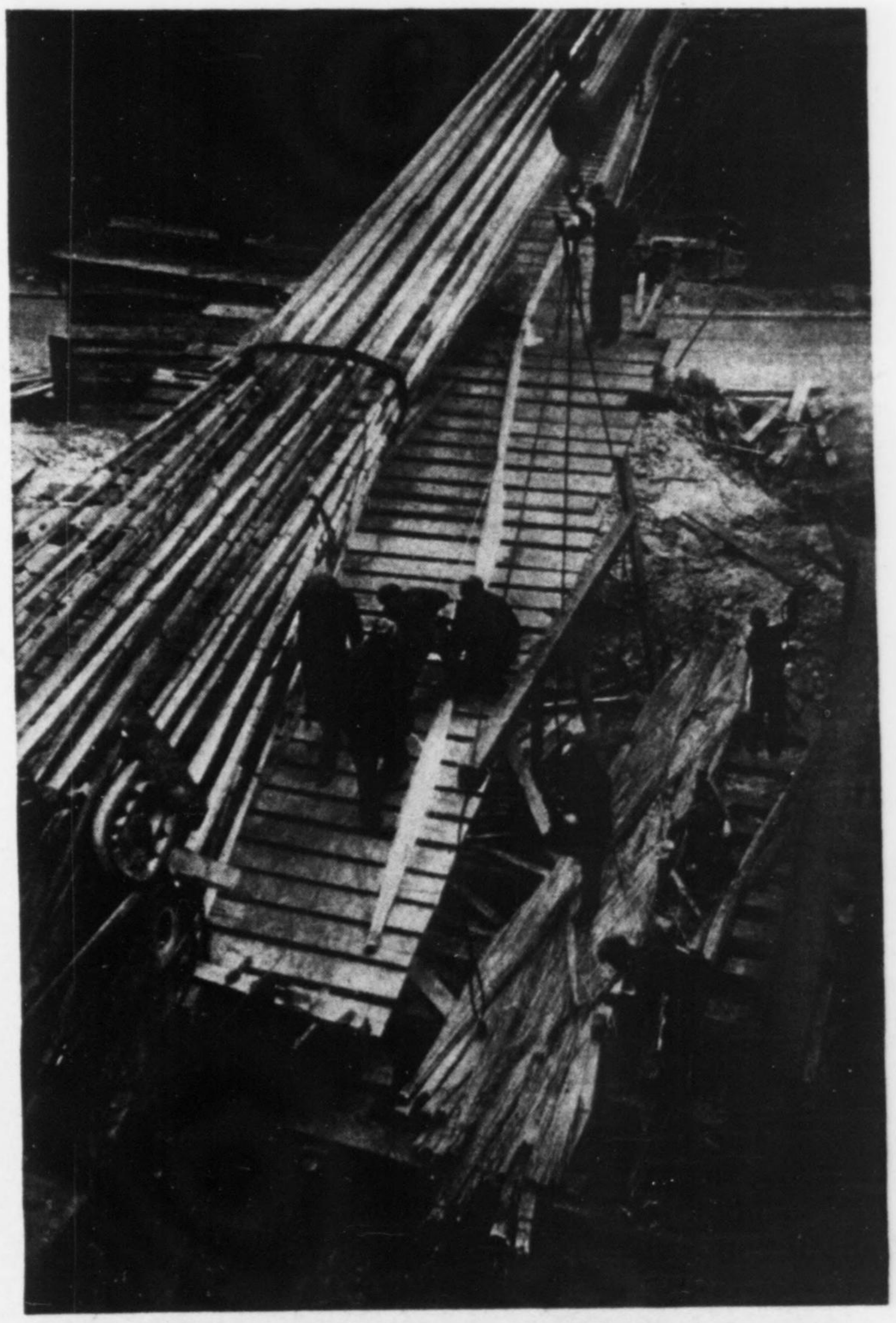


FIG. 13—CUTTING A BACKSTAY STRAND INTO 30-FT. SECTIONS AT DETROIT ANCHORAGE
Note widened footwalk and men placing wire seizings on strand. As a section is cut it is pushed off the footwalk to the "boneyard" below. Men on ground are ready to hook a half-dozen sections to boom falls for transfer to waiting trucks.

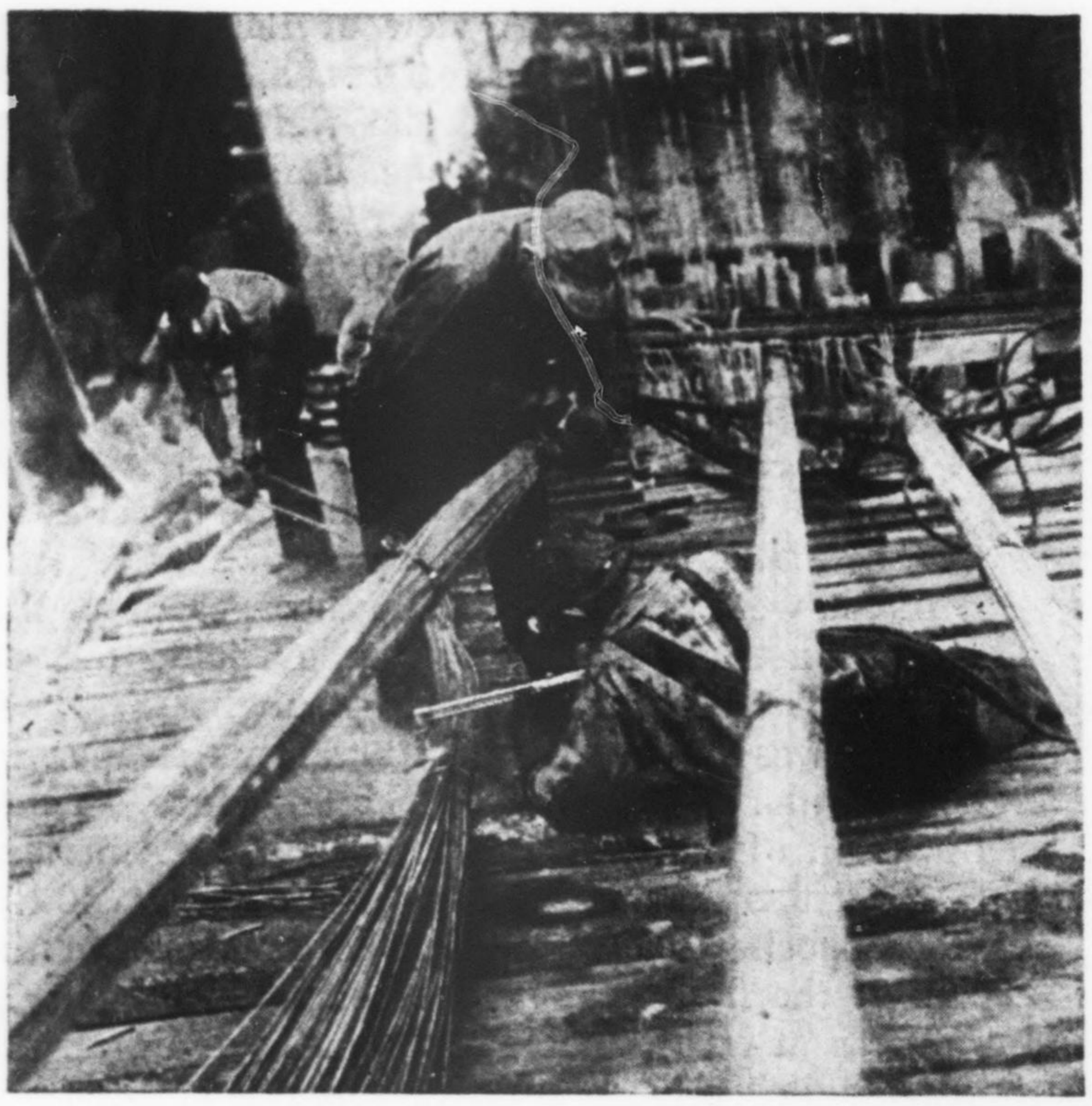


FIG. 14—CUTTING OPERATION AT DETROIT WEST ANCHORAGE
Note that the cut is being made between seizings about 6 in. apart.

its length, although still passing through the cable-bent and main tower saddles.

Flame-cutting was begun by cutting out a 12-ft. section, 6 ft. on either side of the main tower saddles. Rope hitches were placed around the upper ends of the backstays and of the main span strand thus formed and cables running to one of the drums of the two two-drum hoists installed on the top of the main tower held these strand sections from sliding down the footwalks. After the backstay strand was lifted manually from the cable-bent saddles, the strand was lying on the footwalk from anchorage to anchorage.

The work of flame-cutting the strand into 30-ft. lengths was carried out simultaneously in the main span and in the backstays. The first cut in the backstays was made about half way between the anchorage and the main tower. The top end of the bottom section thus formed was lashed to a sled which was held by cables running up the footwalk to the hoist drum on the top of the main tower. The second cut was made at the splay point in the anchorage, removing the hairpin and permitting the bottom section of the backstay to lie free on the footwalk. The cutting gang was stationed near the bottom of the backstay where the footwalk had been widened to 20 ft. and severed 30-ft. lengths as the strand was permitted to slide past.

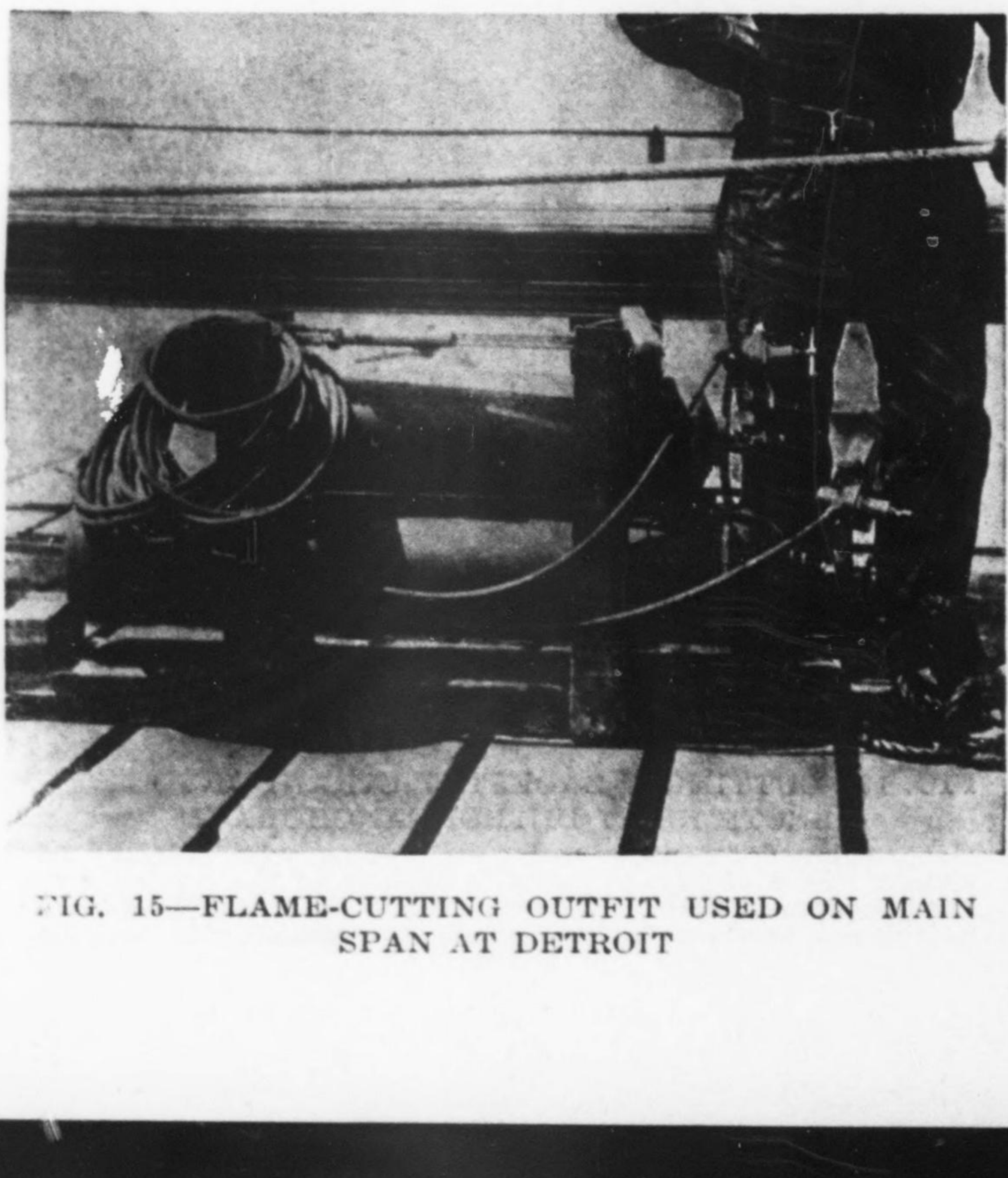


FIG. 15—FLAME-CUTTING OUTFIT USED ON MAIN SPAN AT DETROIT

The necessary wire seizings were placed on the 30-ft. lengths while the cutting was in progress. As a cut was being made two seizings were applied by a workman about 4 to 6 in. apart 30 ft. up the strand; the next cut was made between the seizings. Also as the cutting was in progress two other men placed wire seizings at about 6-ft. intervals in the 30-ft. section being severed.

In cutting it was found necessary to move the torch back and forth, taking out a section of strand about $\frac{3}{4}$ in. wide. Attempts made to burn directly through the strand resulted in the wires fusing together as fast as the cutting progressed. On an average, it required about 50 seconds to cut through a strand. As a section was cut free it was pushed over the side of the footwalk to a "boneyard" below, from which trucks hauled it away. About two minutes was consumed in disposing of each 30-ft. section.

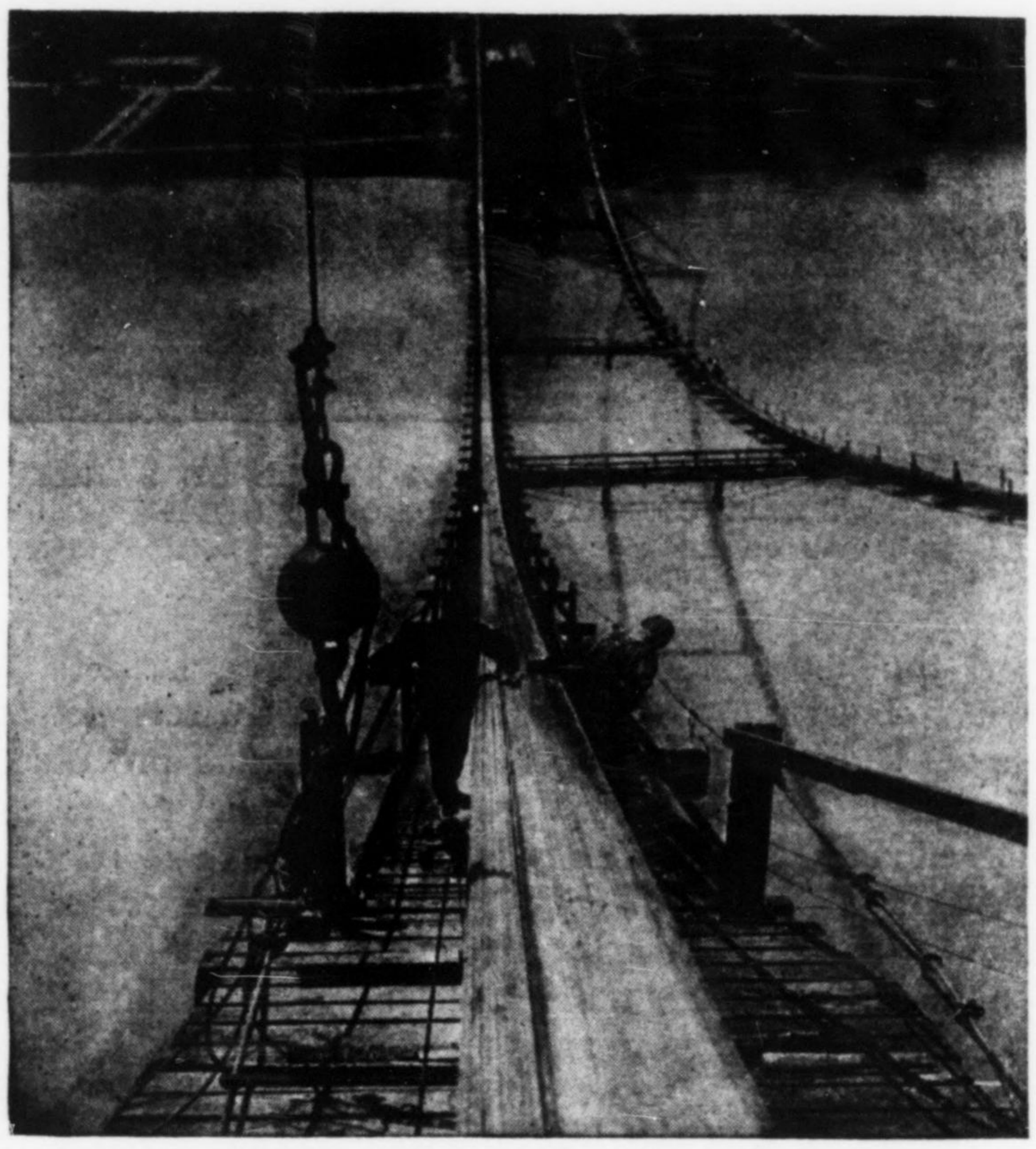


FIG. 16—TWO 30-FT. LENGTHS OF CABLE STRAND COMING UP FROM DETROIT MAIN SPAN ON SLED
Wire lowered at main tower to gondola cars in railroad yard below.

As the backstay strand became shorter, the friction between the strand and the footwalk became effective to such an extent that the strand would not slide down of its own weight. It was then necessary to attach a cable to its lower end, this cable running through two snatch-blocks to a hoist in a shed on the ground in front of the anchorage.

As soon as the lower half of the backstay was cut into 30-ft. lengths, the upper half was permitted to slide down the footwalk until in proper position for the cutting gang, when the operations described above were repeated.

On the main span the cutting gang was mobile, making its cuts at 30-ft. intervals marked along the footwalk. The cut sections in the main span were hauled up the footwalk two at a time on a pair of improvised wood sleds, as shown in Fig. 16, power being supplied by one of the hoist drums on top of the tower. The derricks on the tower transferred the strands from the sleds to storage below. On the American side they were dropped directly into gondola cars, whereas on the Canadian side they were placed on the push cars of a construction railway and hauled to the anchorage, where they were transferred to trucks together with the strand sections cut from the Canadian backstays.

Eight cutting crews were employed in dismantling the cable. On the main span the oxygen and acetylene tanks of the cutting outfits were placed on sleds (Fig. 15), and each cutting crew consisted of a torch operator, a helper and a man with water to put out any fires.

The principal difficulty in the main span was occasioned by broken wires found at the suspender points. Some of these wires became loose and entangled, and others in one strand were so tightly squeezed to another strand that they had to be burned separately.

Administration—Dismantling operations on both structures were effected by the same organizations which had previously erected the materials. The structural steel was disassembled and taken down by the McClintic-Marshall Company, Pittsburgh, Pa., which was in each case the general contractor. The resupporting of the footbridges and the removal of the cables was carried out by the Keystone State Corporation, Philadelphia, Pa., subcontractor for supplying and erecting the cable wire. The methods and organization for the latter were developed by L. N. Gross, superintendent. W. G. Brenneke is resident engineer at Mount Hope for Robinson & Steinman, consulting engineers to the Mount Hope Bridge Company, and R. G. Cone is resident engineer at Detroit for Modjeski & Chase, consultants to the Detroit International Bridge Company.

Small Arch Dam Design Governed by Concrete Economy Need

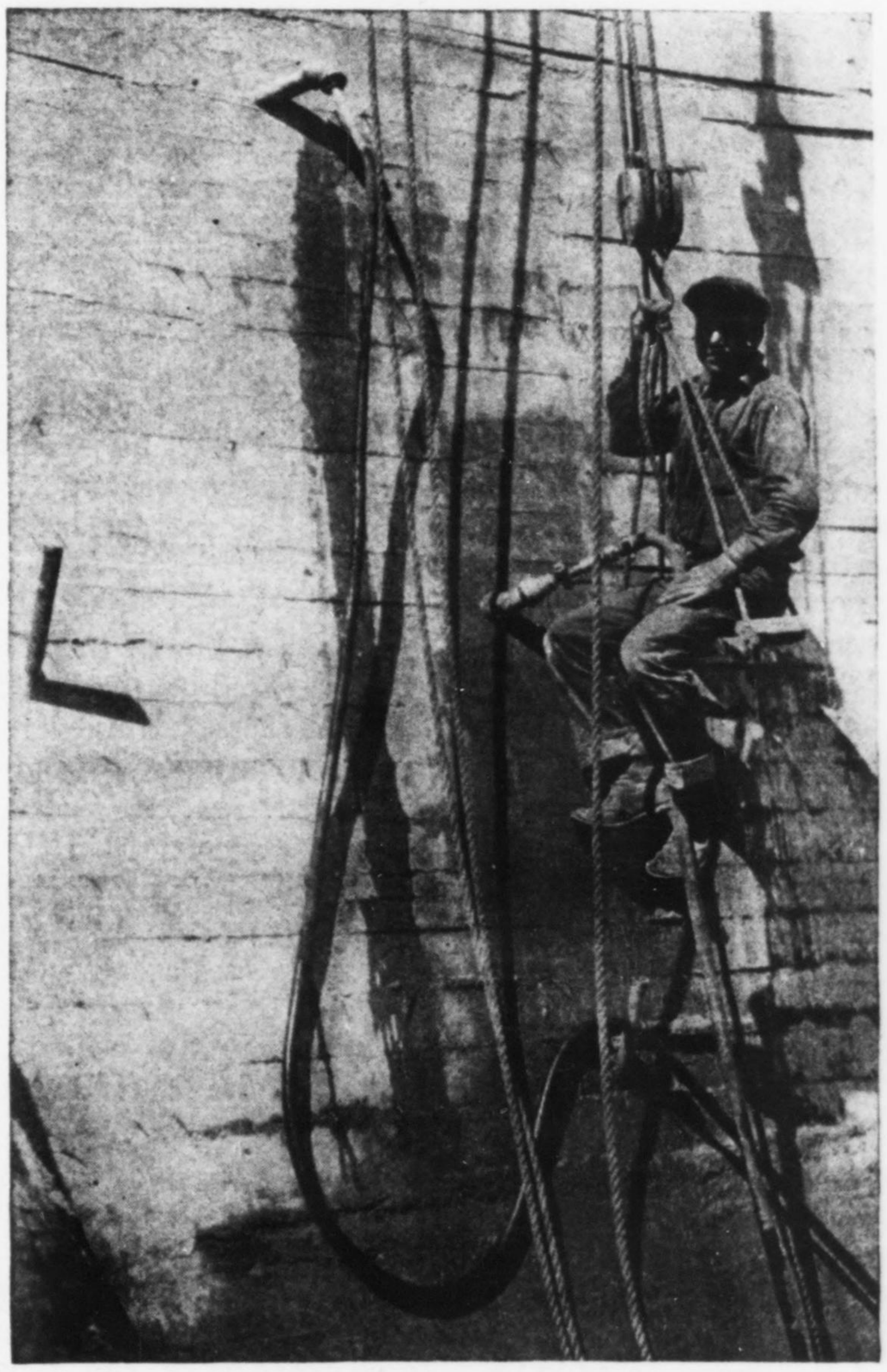
Arch Dam 110-Ft. High Contains Only 970 Cu.Yd. —Reinforced for Temperature, Doweled to Abutments and Grouted

By James Girand, Jr.
Civil Engineer, Phoenix, Ariz.

REFINEMENTS of design and construction that usually are found only in much larger structures were recently used in a small variable radium arch dam near Safford, Ariz., containing 970 cu.yd. of concrete. Limited funds available for increasing the water supply of the town required rigid concrete economy to be used in developing a 153-acre-ft. reservoir site. The narrow gorge, 110 ft. deep and about 130 ft. wide, on Frye Creek afforded a site favorable to an arch dam of thin section aided by bedrock of diorite exposed over the whole contact area that yielded diamond drill cores showing an average strength of about 21,000 lb. per sq.in.

The dam was designed according to the Cain formula, with maximum compressive stresses of 450 lb. per sq.in. Tension was eliminated by keeping the central angles at about 130 deg. for all elevations, which gave radii varying from 14 ft. at the base to 80 ft. near the top. The narrowness of the site and the design for favorable stresses to reduce the concrete yardage required that the lower arches be moved upstream slightly. Full advantage of this hollowing effect could not be taken, because of the necessity of retaining an old rubble dam which the contractor claimed as flood protection. The crown cross-section meeting these requirements has a thickness of 2 ft. at the top and 3 ft. near the base.

To provide a good seat for the arch without restricting free arch movement, the abutments were stepped to make the thrust normal to the rock surface and were cut away to eliminate contact with the arches on either face. Possible separation of the concrete from the rock at the abutment extrados was counteracted by placing 1-in. square steel dowels, 10 ft. long, on 2-ft. centers set in drill holes 30 to 40 in. deep and grouted. Reinforcing steel consisting of $\frac{1}{2}$-in. square bars on 24-in. centers



GROUTING AT ABUTMENTS
Washing out holes with 250-lb. water pressure before applying grout.

both horizontally and vertically was placed on both faces of the dam to assist in preventing shrinkage cracks during setting and to relieve temperature stresses.

Sand and gravel aggregate, developed in the reservoir site, was put through a crushing and screening plant producing two sizes of aggregate—sand up to $\frac{3}{8}$ in. and coarse aggregate with a $2\frac{1}{2}$-in. maximum size. A typical mix was 1:2.0:3.8, and the average cement content for the job was 1.62 bbl. per cubic yard of concrete. The slump varied from 4 to 5 in. and the probable water-cement ratio was 6 gal. per sack. The crude plant arrangement made concrete control difficult and the water-cement ratio was controlled from the appearance of the concrete reaching the forms rather than by equipment at the mixer. With daily pours in 10-ft. lifts, the water in the concrete had a tendency to rise to the surface