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Design of the Detroit River Suspension Bridge

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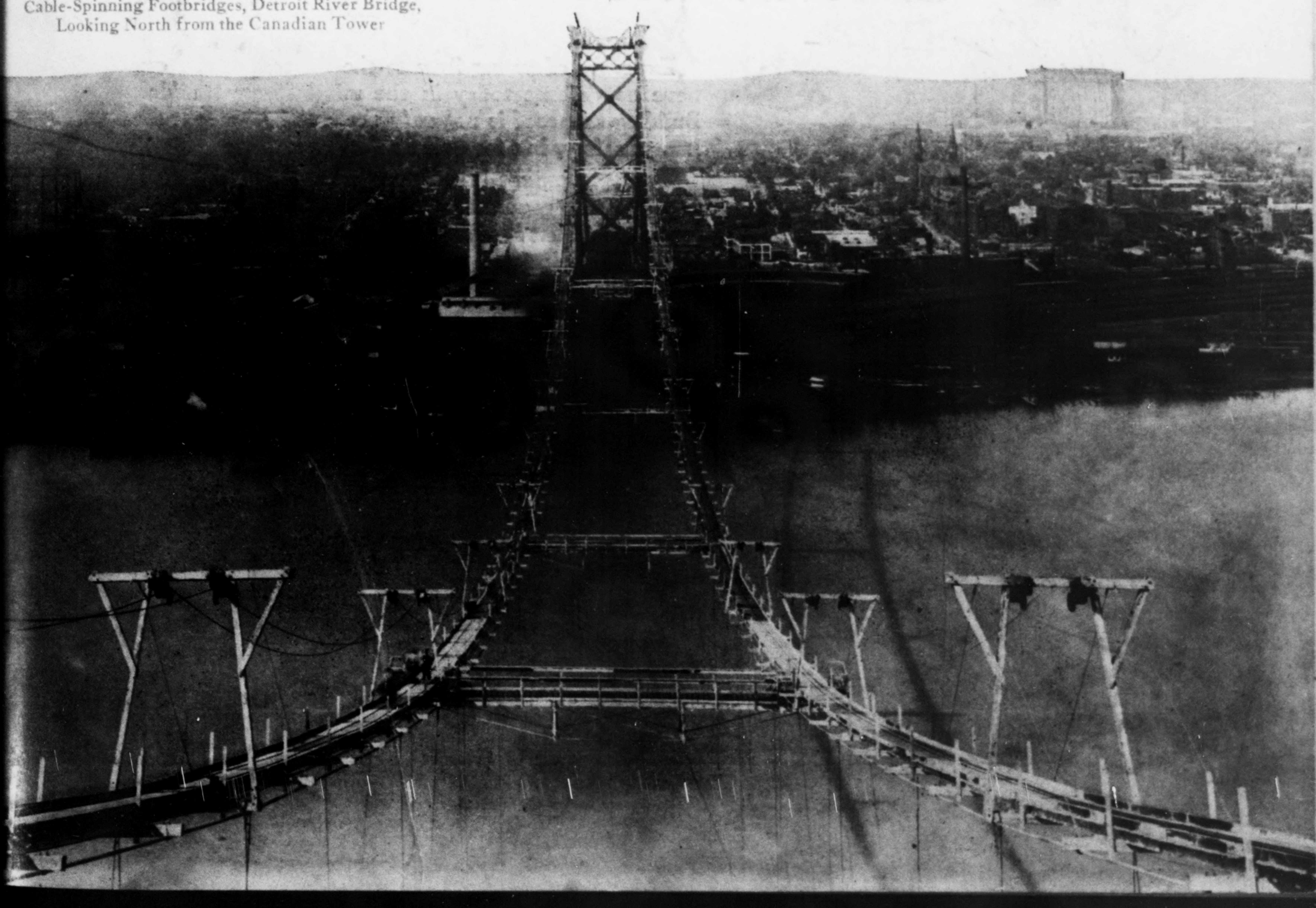
Cable-Spinning Footbridges, Detroit River Bridge, Looking North from the Canadian Tower



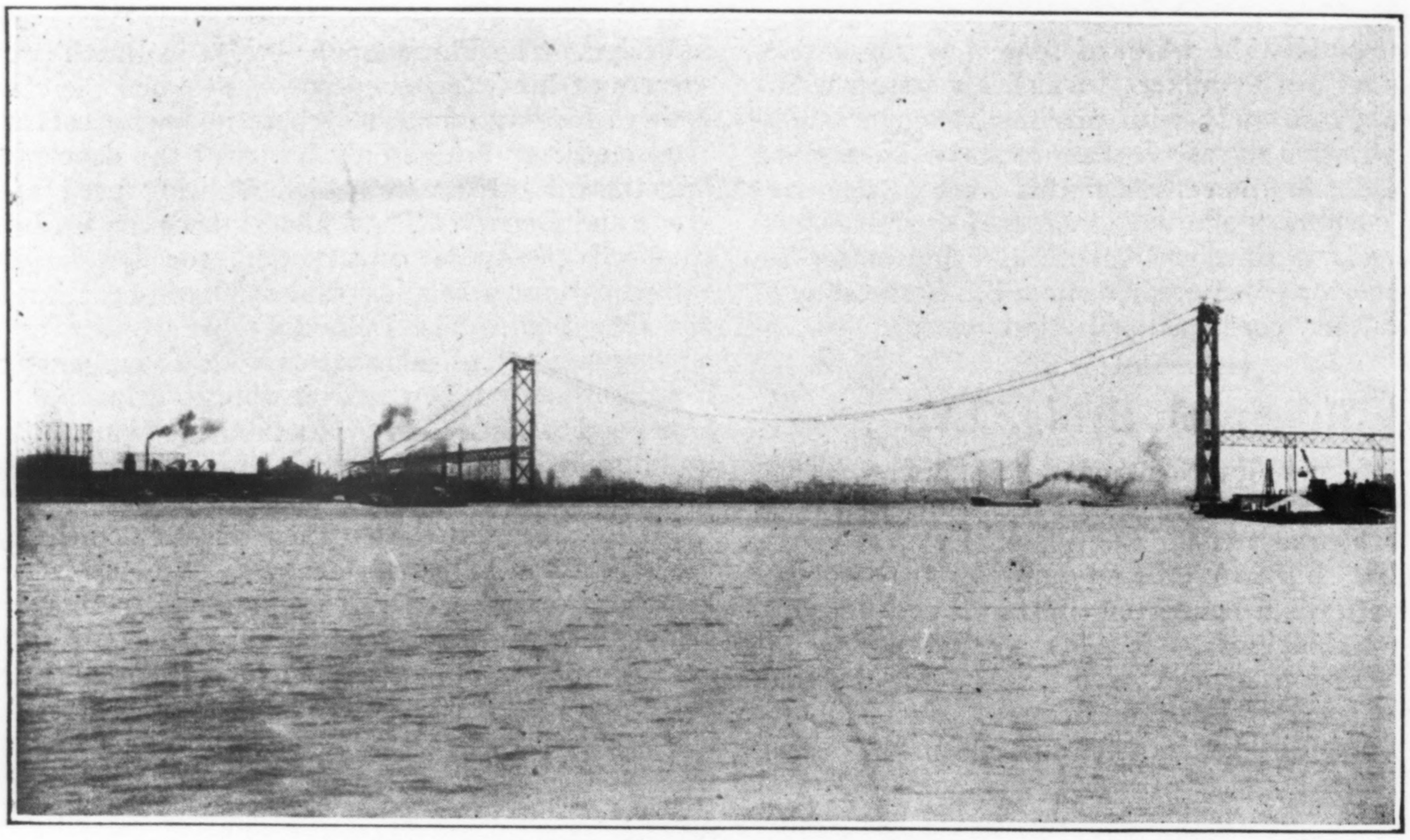


Fig. 1—International Suspension Bridge. Detroit in the Background

Design of Great International Suspension Bridge Over Detroit River

Unusual Anchorages—Unloaded Backstays on 1,850-Ft. Span—Towers, Lacking Full-Height Stability Before Placing Cables, Are Braced by Approach

By Jonathan Jones
Assistant Chief Engineer, McClintic-Marshall Company, Pittsburgh, Pa.

The Detroit River toll bridge now under construction between Detroit and the Canadian shore has passed through a number of stages in both its financial and its technical phases. Originally it was promoted by the American Transit Company, of which Charles Evan Fowler was chief engineer, which determined on the present location and had complete design plans made for a combined railway and highway structure. Finding itself unable to finance the project, this company finally turned it over in 1923 to the Detroit International Bridge Company and the Canadian Transit Company, the interests which are now building the bridge. The original location was accepted as the best available, but all consideration of railway tracks was eliminated and numerous changes in the design were made, particularly in the anchorages, the main towers and the roadway layout. The bridge upon completion will be known as the "Ambassador bridge." Subcontracts for the main piers and anchorage substructures were signed on Sept. 20, 1927, one month and three days after the general bridge construction contracts went into effect. The main piers were ready to receive steel on Feb. 16 and April 2 on the United States and Canadian sides, respectively. This article covers the design considerations of the structure as it is now being built. Numerous special and difficult problems have arisen in the construction, particularly in the foundation work, and these will be treated in a separate article or articles. —Editor.

The International Bridge being constructed between Detroit and the "border cities" of Canada is approximately 7,400 ft. long between abutments, and beyond the abutments lie two terminals or plazas each roughly equivalent to a city block in area, so that the distance traveled from entrance to exit is approximately 9,000 ft. The main span, of wire cable suspension type with unloaded backstays, is 1,850 ft. center to center of main piers, which is 100 ft. longer than the Philadelphia-Camden span. The new bridge will be the longest suspension structure and the longest highway bridge in existence until the completion of the Hudson River bridge in 1931. Features of the project are the unusual and costly anchorages, the slenderness of the main towers, the attachment of gussets to floor beams in the shop to facilitate erection and the provision for picking the unloaded backstay cable up to nearly straight during erection.

The roadway is 47 ft. wide and runs generally north and south. There is a single 8-ft. sidewalk along the west side. The 47-ft. roadway affords five traffic lanes and will therefore accommodate three lanes in one direction and two in the other, of the maximum type of automotive vehicle likely to develop. Provision will be made for so directing traffic that three lanes will always be available in the direction of heaviest movement.

According to government requirements, the clearance over recorded high water in the river will be 135 ft. near the shore and 152 ft. for a short distance in the center, making provision for the passage of vessels 20 ft. higher than any that are now plying on the lakes. The center clearance was required because of a similar clearance under the St. Lawrence bridge at Quebec. The maximum approach grade is 5 per cent for a distance of approximately 1,000 ft.

Main Piers—The Canadian main pier is located in 8 to 15 ft. of water, immediately within the harbor line, while the United States main pier is more than 70 ft. inside the harbor line in order to avoid interference with an existing slip and track. Conditions at the two sites are substantially alike, blue clay predominating to about half the depth, underlain by a deep layer of sand and about 12 ft. of hardpan or clay and boulders, limestone bedrock being finally reached at about 105 ft. depth below river level. Core borings were taken 20 ft. (at the anchorage sites 70 ft.) into this rock to prove its soundness.

Each pier carries about 20,000 tons and consists of two hollow, cylindrical, concrete caissons with 10-ft. walls and 18-ft. shaft, making the exterior diameter 38 ft., or about one-third the height. The caissons forming a pair are capped off below water and joined with a concrete strut above which two solid concrete cylinders 29 ft. in diameter complete the pier as shown in Fig. 3. Reinforcement is generous, but no stresses in steel are an element of the design. The Canadian pier is granite faced where exposed to ice action.

Anchorages—Because of the depth to rock and the difficult material to be penetrated (substantially the same as above recited for the main piers) the anchorages are necessarily an item of great cost, so much so that they are one of the primary reasons for adopting unloaded backstay cables to deliver the cable pulls to the anchor grillages reasonably near the ground, rather than loaded backstays which would have made this delivery much higher in the air, whatever expedient might have been adopted to take the forces thence to rock.

Earlier designs contemplated an articulated form of anchorage, in which the pull of each backstay cable would have been resolved, near the ground line, into two components: (1) A rear or tension component carried by eyebars in an inclined concrete tunnel to rock and thence into the rock for direct anchorage without resort to weight; and (2) a forward or compression component carried by a reinforced-concrete shaft (more probably a steel column with concrete protection) similarly carried to a bearing on bedrock. The apparent economy of this design lay in the spread thus obtained between the rear and the forward leg, which reduced the intensity of the stress in each, and in the elimination of great mass as a factor in stability. However, this design was abandoned after the core borings revealed that the bedrock was seamy and carried water under a head of as much as 110 ft., and after it was realized that the effectual exclusion of this water from the anchorage steel involved uncertainties.

FIG. 2—THE DETROIT RIVER BRIDGE CLOSES A GAP IN A NEW EAST-WEST HIGHWAY THROUGH CANADA

The tentative design then resolved itself into a vertical force to be supplied by mass concrete with some assistance from the approach superstructure, and a forward or compression component carried by the steel and concrete strut which would necessarily be of practically twice the originally intended size. On developing this principle in detail, it proved difficult to provide in this inclined strut for the fluctuating line of stress due to changes in live-load stress in the cable. Furthermore, however economical in material this inclined strut might be, it had no precedent in construction, and the net opinion was that it would pay in speed of completion to use more material in a more usual type of construction.

In the plan issued for subcontract bids, therefore, the four inclined struts (one at each end of each cable) are replaced by rectangular caissons, which could be conceived of as the same inclined struts plus sufficient excess material to give the rectangular form and allow vertical instead of inclined sinking. The vertical weight is supplied by mass concrete kept, for economy, above the groundwater line. Sufficient weight is included in the substructure subcontract to permit erection of the cables at the earliest possible moment. The remainder of the mass concrete, all of which will be above street level, is being developed under architectural guidance and will be placed after the cable-stringing operation is completed and the equipment therefor removed and before the suspended structure weight demands it.

Each anchorage substructure, then, comprises two elements: (1) A pair of vertical caissons centered under the cables $22\frac{1}{2}$ ft. in transverse width, 100 ft. in length parallel to the bridge cables and sunk to and into bedrock at depths of about 100 ft. below river level: (2) a cross-

COMPARISON WITH OTHER SUSPENSION BRIDGES

Bridge	Brooklyn	Williams-burg	Man-hattan	Bear Moun-tain	Philadelphia-Camden	Detroit Interna-tional
Year built	1882	1903	1907	1924	1926	1927
Main span, ft	1,595	1,600	1,470	1,632	1,750	1,850
Side spans carried by cables or columns	Cables	Columns	Cables	Columns	Cables	Columns
Number of cables	4	4	4	2	2	2
Wires per cable	5,458	7,696	9,472	7,252	18,666	7,622*
Roadway width, ft	2@17	2@20	1@35 1@23	1@38	1@57	1@47
Transit tracks	2	6	8	0	4	0
Sidewalks and width, ft.	1@15	2@17	2@12	2@5	2@10	1@8
Height of main towers, ft.	272	302	291	350	397'	363
Mid-span height over water, ft	135	135	135	'153	135	152

*First use of heat-treated wire with higher elastic limits.

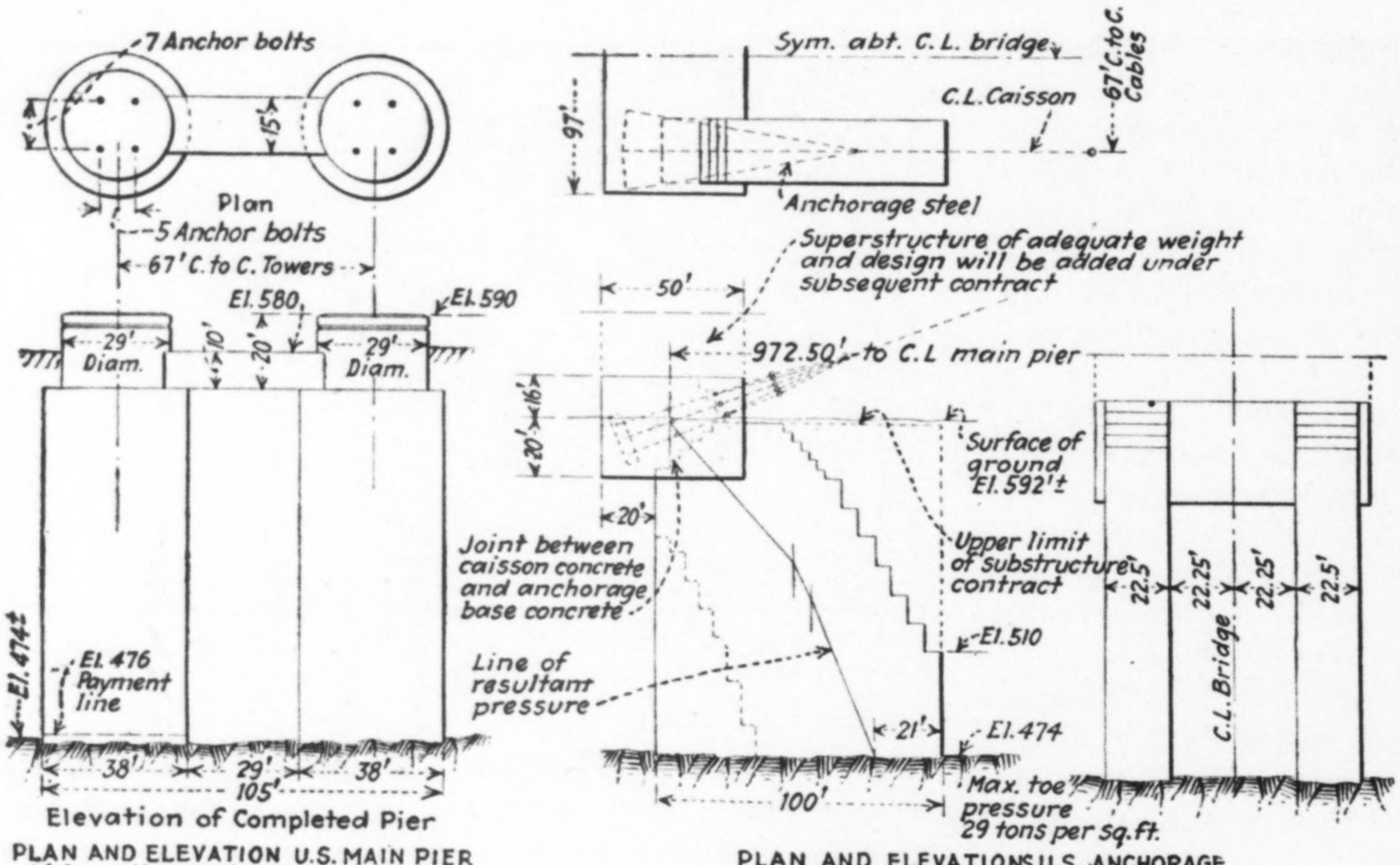


FIG. 3—DETAILS OF MAIN PIERS AND ANCHORAGES

block of concrete 50 ft. from front to rear, 36 ft. deep and 97 ft. transversely, with provision in it for embedment of the anchorage eyebars and grillage, placed above and spanning transversely across the rear ends of the pair of caissons. In this design no account whatever is taken of resistance from the surrounding ground or of stress in the caisson reinforcement, both of these being reserve additions to the stability.

Approaches—Due to the use of unloaded backstay cables, the distance from main piers to anchorages is traversed by deck spans. On the Canadian side this distance is 798 ft., being the economical distance with equal cable angles at the top of the main towers, and is divided into five 152-ft. spans with a 38-ft. bracing tower. On the United States side, interference from the engine houses of the Pere Marquette, Wabash and Pennsylvania railroads forces the anchorage north to a distance of 953 ft., which interval is divided with reference to tracks and other interferences into six spans

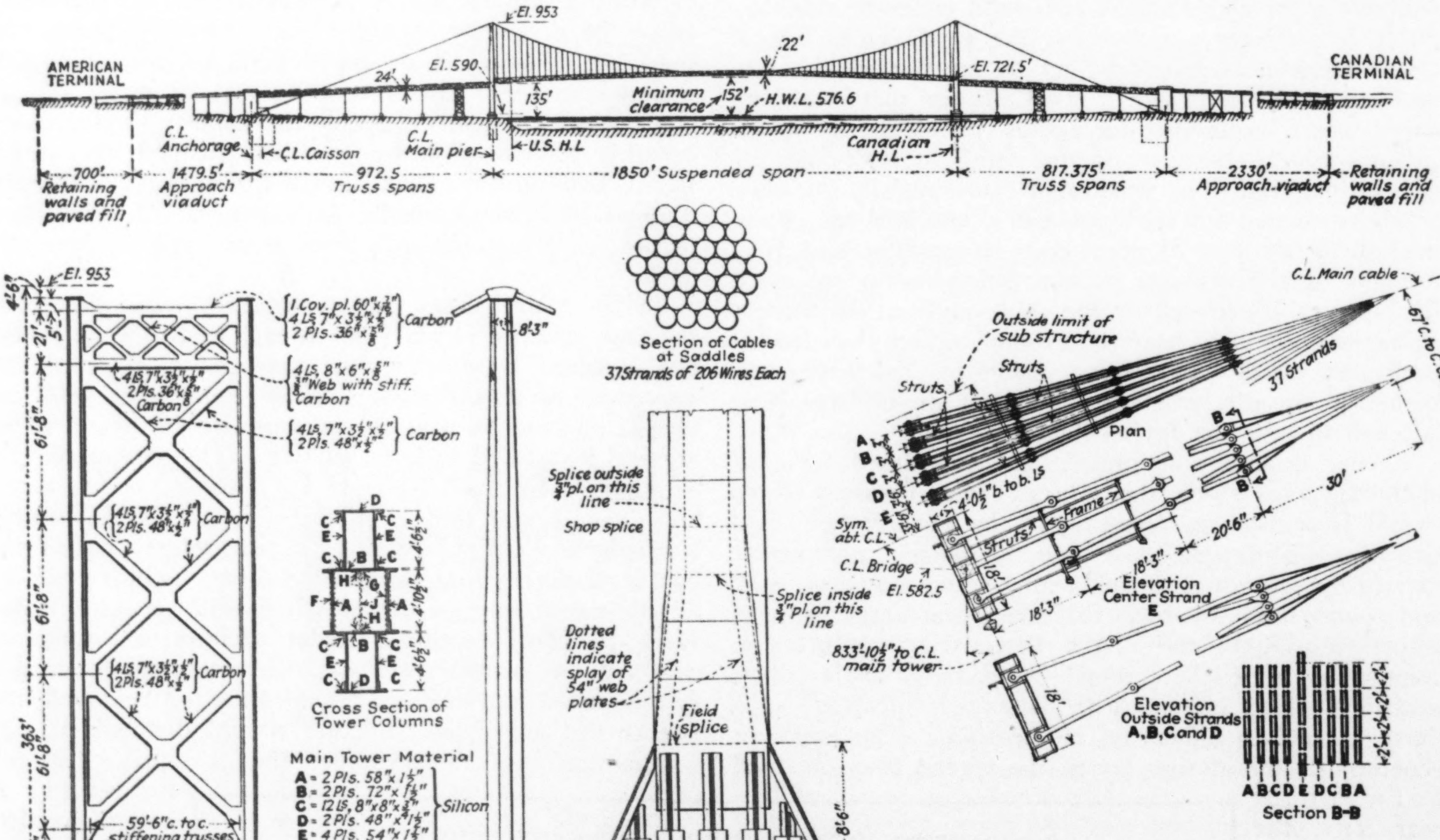


FIG. 4—ELEVATION AND DETAILS OF 1,850-FT. SPAN SUSPENSION BRIDGE OVER DETROIT RIVER

varying from 121 ft. to 196 ft., with a 26-ft. bracing tower.

From the anchorages to the respective terminals the typical construction is a viaduct comprising four rows of plate girders with corresponding four-column bents. On the United States side the steel bents will be replaced at two boulevard crossings by tall piers of mass step-back concrete recalling the architecture of the anchorage superstructure.

In the approaches, instead of longitudinal stringers as used on the main span, transverse floor joists are used, spaced quite uniformly 6 ft. on centers except where curves make this impossible. These joists are considerably cantilevered to reduce the necessary width of the supporting structure. Throughout the viaduct approaches the joists are supported on four lines of plate girders, spaced 14 ft. center to center, or 42 ft. between outer girders. Throughout the truss spans, the inner girders are replaced by 27-in. Carnegie beam stringers, fabricated in two-panel lengths; the outer pair of girders are correspondingly replaced by trusses, the spacing of which increases to 45 ft. Throughout the truss spans and the approach viaducts the number of braced towers is kept to a practical minimum for appearance sake; and the intermediate bents are designed not to rock but to flex with changes in temperature.

Due to long stretches of 5 per cent grade on the United States truss spans and approach, it has been decided to use smooth-dressed granite block from the United States main tower to the terminal, while on the Canadian side the lesser rate of grade permits the use of asphalt.

Main Span—Comparison was made between the suspension and cantilever types, the arch being wholly impracticable. Due to the cost of the anchorages, the difference indicated in favor of the suspension type was less than would be normal for so great a span, though it should be admitted that the factors of uncertainty in such preliminary estimates undoubtedly favored the cantilever and that the actual comparative economy of the suspension is greater than was then computed. The factors of hazard and time, especially the latter, also pointed definitely to the suspension type.

Certain limitations as to size and type of structure are placed upon the contractor, but the contract is in general broad in recognizing that the general contractor is also the designing engineer; and it affords latitude

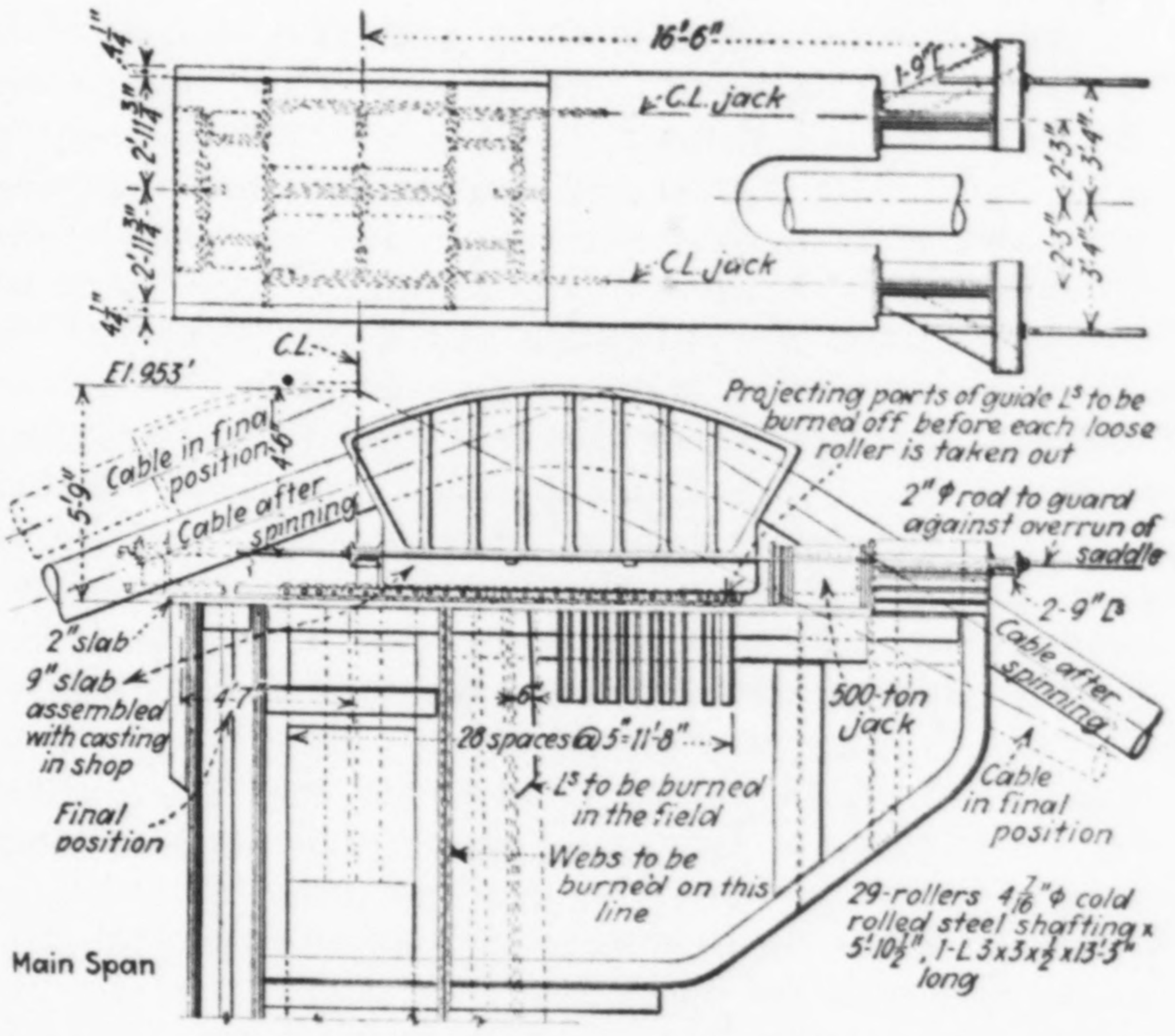


FIG. 6—JACKING ARRANGEMENT USED IN MOVING SADDLE CASTINGS AS MAIN SPAN LOAD IS ADDED

Brackets are temporary for carrying the pairs of 500-ton jacks. The saddles are shifted a distance of 63 in. riverward during erection of the suspended span.

for changes as the details are worked out, such changes being subject to review by the owner's consultant.

LOAD AND STRESS SPECIFICATIONS

Roadway Live Load:
For main span floor stringers, a 24-ton truck
For main span floor beams and suspenders, eight 21-ton trucks
For approaches, series of trucks four abreast, as follows:

If one row	24 tons each
If two rows	21 tons each
If three rows	18 tons each
If four rows	15 tons each
If five or more rows	12 tons each, or 100 lb. per sq.ft.

Total live load for main span cables, towers, stiffening trusses and anchorages:
Normal ....1,650 lb. per lin.ft. (about 30 lb. sq.ft.)
Congested ....3,300 lb. per lin.ft. (about 60 lb. sq.ft.)
Impact Addition—30 per cent on floor slabs, joists and stringers
Loading combinations for main span cables, towers, stiffening trusses and anchorages:
DW = dead and 40-lb. wind
DNWT = dead and normal live and 20-lb. wind and 60 deg. temperature
DCT = dead and congested live and 60 deg. temperature
Stresses = Carbon steel on an 18,000-lb. basis and silicon steel on a 24,000-lb. basis.

Main Towers—Each main tower is composed of two columns 67 ft. apart with crossbracing and a height from

FIG. 5—SETTING BASE STEEL ON THE UNITED STATES MAIN PIER

The left-hand view shows the placing of one of the base sections, while on right the first main section of the tower weighing 54 tons is being raised. Note on far pier in left-hand view the pneumatic machine used to prepare pier for base slab.

masonry to cables of 363 ft. A column comprises three nearly square cells of silicon steel. All angles are 8x8x$\frac{3}{4}$ in. and all plates are $1\frac{1}{2}$ in. in two thicknesses of $\frac{3}{4}$ in. each. The central cell is continuous in dimensions from base slab to saddle casting. Its longitudinal webs are 58 in. and transverse webs 72 in. To the latter are attached all transverse bracing and floor supports. The side cells have 48-in. transverse cover plates, being narrower transversely than the central cell to which they rivet.

The longitudinal webs of the side cells are 54 in. wide throughout about the lower two-thirds of their height, above which they taper to 18 in. under the saddle castings. All studies preceding the final design contemplated a similar taper in the lower third, but the consultant for the purchaser pointed out that by omitting this lower taper and carrying the normal shaft material straight down without change, base castings could be dispensed with and a semi-fixed base in structural steel substituted with the maximum fiber stress under flexure within permissible limits. The term "semi-fixed" is here used because the proportions of the base would not permit of erecting the shafts to their entire height without exterior support, as was the case with the flared-base towers of the Manhattan, Philadelphia-Camden, Poughkeepsie and Mount Hope bridges. The lack of full-height stability (before placing of cables) is in this case no objection, since the deck-truss side spans, with supporting piers, afford a simple means of temporarily staying against longitudinal forces.

It is to be doubted, however, whether the adoption of this type of base makes any particular contribution to the previous discussion of fixed vs. hinged tower for suspension bridges. It is not a sufficiently general case. The long straight backstays, which must have full sag while spinning and must be picked up to near straightness by the erection of the suspended span, compel an initial setting of the saddle casting 63 in. shoreward from the column center, with heavy brackets to support the load thus eccentrically and to provide reaction for the pairs of 500-ton jacks which will be used to roll the saddles over the column centers as the main span load is added. All this is very expensive; but with towers of the hinged-base type the saddles would have to be similarly set off column centers, as was done at Florianapolis, or else the towers would have to be leaned back out of plumb by the same 63 in. and the problem of supporting their enormous weight in that position would also involve expensive steel supports and jacking devices as well as additional strength in intermediate pier foundations. In the absence of a completely detailed plan for these operations it is not practicable to prove, but is the belief of the contractor, that the adopted type is best suited to the immediate case.

Contact between column and masonry is through a 4-in. machined steel slab, in two pieces, each $6\frac{1}{2}$x18 ft. The base proper is $8\frac{1}{2}$ ft. high, tapered four ways to contract the area covered from $11\frac{1}{2}$x$16\frac{1}{2}$ ft. on the base slab to 6x$14\frac{1}{2}$ ft. under the first column shaft. It is riveted up in vertical cells, the principal webs of which bear on the column webs. There are no lug angles or riveted attachments to the base slab, but a vertical boot around the whole engages the anchor bolts. The base is divided for shipping purposes into five transverse slices, of which the central one weighs 24 tons. After erection and riveting, these bases were immediately filled with concrete.

The lowest column section is special in that the 48-in. transverse cover plates of the two side cells, together with their attaching angles, flare out so that when the base is reached they equal the 72-in. width of the central cell. Above this section, the column remains in large part typical save for floor and truss connections. Horizontal diaphragms are spaced 7 ft. 3 in. apart throughout. Horizontal splices in the two side cells are staggered with those in the central cell. There are seven lifts to each shaft, the heaviest section weighing 58 tons.

The top lift of the side cells is milled short of the center cell and horizontal filler plates are provided under the cap plate. These fillers were planed in Detroit, after all the shafts were erected, so that minute errors in milling the various shaft sections to length are compensated for.

The top lift is quite special in having built into it the heavy brackets for eccentric support of the saddle casting on rollers. These excess portions of shaft will be burned off after they have performed their function, and the slots through the 72-in. transverse web will then be welded up. The saddle castings are standard in type, but a special provision required is the 9-in. slab provided under them to give space for travel of the jack plungers.

The tower transverse bracing is of carbon steel in boxed two-channel sections about 48x58 in., 48 in. being the depth of the channel webs. The top and bottom struts are deepened and are in effect lattice trusses. The usual transverse strut of the roadway portal is omitted in order to avoid any feeling of constricted headroom; and for consistency of appearance the transverse struts are omitted from the X-panels above this.

FIG. 7—ERECTING THE UNITED STATES MAIN TOWER
The erection rig is a stiff-leg derrick on a climbing underframe mounted on the river face of the tower. On the United States side a locomotive crane erected the first sections of the tower and the derrick and underframe, while on the Canadian side the initial erection was done from a stiff-leg derrick on pile clusters in the river.

FIG. 8—UNITED STATES TOWER COMPLETE WITH FOOTWALK CABLES IN PLACE

The second tower was completely erected $11\frac{1}{2}$ months after the contract went into effect and the footwalk ropes were hoisted to place on Aug. 8.

Main Cables—Wire was selected for the cables without detailed comparison with eyebars, because of the higher cost of the anchorage in the latter case. The specification adopted for the cable wire was a virtual duplicate of that used for the Philadelphia-Camden bridge, except that basic steel was admitted on equal terms with acid. Before award of a subcontract for the wire, however, the American Cable Company came forward with evidence that it could go into adequate production of a heat-treated wire with a minimum yield point of 190,000 instead of the specified 144,000 and a minimum tensile strength of 220,000 as against 215,000 specified. All the properties of this wire were found suitable, and it was permitted to compete with the cold-drawn wire on the basis of a working stress increased from 76,000 to 84,000. With this advantage, the best offer received for the cable wire in place was on the basis of the heat-treated wire; this is now in satisfactory production and shipments to the bridge site have begun. Each cable comprises 37 strands of 206 No. 6 galvanized wires. Had it been known three months earlier that the new wire would be available, the design might more economically have been based on 19 instead of 37 strands. Under present conditions the west or sidewalk cable will not be so highly stressed as the east cable, on which the design is based; but in all parts of the bridge this difference is neglected to care for any possible future change of pavement layout.

The suspender arrangement is usual and the strand shoes, suspender sockets, cable bands and splay castings are of cast steel, of types which have become practically standardized on the last few important suspension bridges.

Suspended Span—The design finally adopted for the stiffening trusses used a panel length of 24 ft. $2\frac{3}{8}$ in., and a truss depth of 22 ft., omitting overhead bracing, and staying the top chords by floorbeam brackets. After careful comparison of several different types of floors, a 6-in. concrete slab with $4\frac{1}{2}$-in. welded truss reinforcement and a $2\frac{1}{2}$-in. covering of sheet asphalt was adopted.

The stringers, spaced 4 ft. 8 in. centers, are figured as of simple span, though they will be fabricated in two-panel lengths. They are supported on instead of framed into the floorbeams to eliminate dangerous deformations in the framed connections during passage of the erection traveler.

The floorbeams are single-web plate girders of carbon steel. Their end connections are quite unusual, in that the bottom chord gussets of the stiffening trusses are to be built with the floorbeams at the shop. This will make for simplicity in erection. At its first pass the traveler will hang the floorbeams from the suspender ropes, hoist the bottom chord into the space between gussets and hang it on a few bolts, set the laterals and enough stringers to run on, and proceed. The cable distortions, which are bound to be severe in this stage, will thus be no obstacle to speedy completion of the first pass.

In the floorbeam design it is assumed that, of the live loads indicated, the excess stops at the stiffening truss and only so much travels out to the suspenders as represents a one-panel quota of the intensity assumed for all panels loaded. To provide, however, for possible misadjustments of suspender shims, by which one or more suspenders might be receiving too much load, the total suspender pull is increased 10 per cent for the floorbeam design.

The stiffening truss chords are of silicon steel and the webs of carbon steel. The permissible unit stresses are increased one-third over those for other parts of the structure, and are accordingly based on 32,000 for silicon and 24,000 for carbon steel. All stresses are worked out for the load combinations in the accompanying table by the "deflection theory," using in the calculations the actual sizes of members that are now being fabricated. Top and bottom chord sections are increased by wind stress throughout the central half of the span and by congested live loading (without wind) in the end quarters.

Chord areas (silicon) vary from 60 to 75 sq.in. gross for the top, and from 47 to 70 sq.in. gross for the bottom. Web diagonals are uniformly two 15-in. channels at 33.9 lb., except in the five panels at each end, where they are progressively stepped up to equality with the top chord section; this is for consistency with the overload assumed as a precaution against misadjustments in the rocker which provides the truss support on the tower. Bottom chords are spliced at main panel points near the centers of main gussets, being fabricated as straight members approximately 48 ft. long.

Concreting of the floor is to proceed both ways from the center of the span, and during concreting the riveting of chords and diagonals is to proceed as the dead load radius of curvature is locally approximated. At least one joint in eight is to be left unriveted until concreting is completed and the placing of the asphalt pavement is under way. This will preserve as nearly as practicable the distribution of dead load between truss and cable assumed in the design.

Organization—The McClintic-Marshall Company, Pittsburgh, is engineer for the design of the bridge as well as the general contractor for its entire construction.

As general contractor, the McClintic-Marshall Company has already awarded the following subcontracts: core borings, Pennsylvania Drilling Company, Pittsburgh; wrecking, United States approach, Fulton Lumber & Wrecking Company, Detroit; wrecking, Canadian approach, General Wrecking Company, Detroit; main piers and anchorage substructures, the Foundation Company, New York; Canadian approach piers and terminal walls, Merlo, Merlo & Ray, Ford City, Ont.; Canadian approach steelwork, Canadian Bridge Company, Ltd., Walkerville, Ont.; United States auxiliary piers, engine house group, H. H. Esselstyn, Inc., Detroit; United States approach piers, Walbridge Aldinger Company, Detroit; United States terminal walls, H. H. Esselstyn, Inc., Detroit; main span cables, suspenders and hand rope, Keystone State Corporation, Philadelphia.

The subcontractor last named has in turn placed the order for the wire with the American Cable Company, which is producing it at the plant of its subsidiary, the Page Steel & Wire Company, Monessen, Pa. This subcontractor has engaged Robinson & Steinman, New York City, as consulting engineers on cable erection. Except as above noted, the McClintic-Marshall Company is fabricating all steelwork at its Rankin, Pa., shop and is erecting it with its own forces. Robert MacMinn, succeeding E. J. Paulus, engineer of construction, is in entire charge of operations at the site, and R. G. Cone is resident engineer for the bridge company's consultant.

The McClintic-Marshall Company has retained Moran, Maurice & Proctor, New York City, as consulting engineers on main foundations, Leon S. Moisseiff as consulting engineer on the main span suspended structure and Monsarrat & Pratley, Montreal, to give all important elements of the design an independent review. On all architectural matters Smith, Hinchman & Grylls, of Detroit, are consulted. For the Detroit International Bridge Company, Modjeski & Chase are acting as consultants for both design and construction.

Air-Activation Gaining at Birmingham

The decision to install a 12-Imp. m.g.d. activated-sludge plant with air diffusion through porous plates at the sewage-works of the Birmingham, Tame and Rea Drainage District, England, taken in conjunction with the history of activated-sludge treatment at these works, shows that diffused air is gaining ground over mechanical means of activation at these works. According to the London *Surveyor* of Aug. 10, bids for such an installation have been accepted by H. C. Whitehead, engineer of the district, following sanction of the project by the Ministry of Health. The earlier history of activated-sludge practice at Birmingham is thus summarized by the *Surveyor:* "Starting in 1922 with a plant operated wholly on mechanical lines, and working through a combination of 'spiroflow' and 'Simplex' methods of treatment, the usefulness of the plant has within the last eighteen months been extended by the addition of a diffused air unit for reactivation of the sludge. It has also been demonstrated that the addition of a number of diffusers to the 'spiroflow' flocculation plant effects a further improvement and increases the effective capacity of the plant. . . . A number of diffusers sufficient to give a run of about 1,000 ft. are being added . . . and with the addition of these it is hoped to increase the flow treated [by the existing installation] from 8 to 12 Imp. m.g.d." It should be noted that the sewage of this district goes to sprinkling filters for final treatment.

Fast Concrete Roadbuilding Under Rush Orders

Ten-Mile Link in Fifty-Mile Galveston-Houston 20-Ft. Concrete Road Paved at Rate of 1,170 Ft. a Day on Fresh Fill—Methods of Compacting

By A. J. Wise
County Engineer, Harris County, Houston, Tex.

A NOTABLE example of rapid roadbuilding is afforded by the construction early this spring of the 10.4-mile link necessary to complete the 20-ft. all-concrete highway from Galveston to Houston, Tex. The haste to build the road was prompted by the fact that the Democratic convention was to be held in Houston. It was desired to provide quick and comfortable access by motor car to the convention city to that part of the convention crowd which had arranged to live in Galveston. It happened fortunately that all

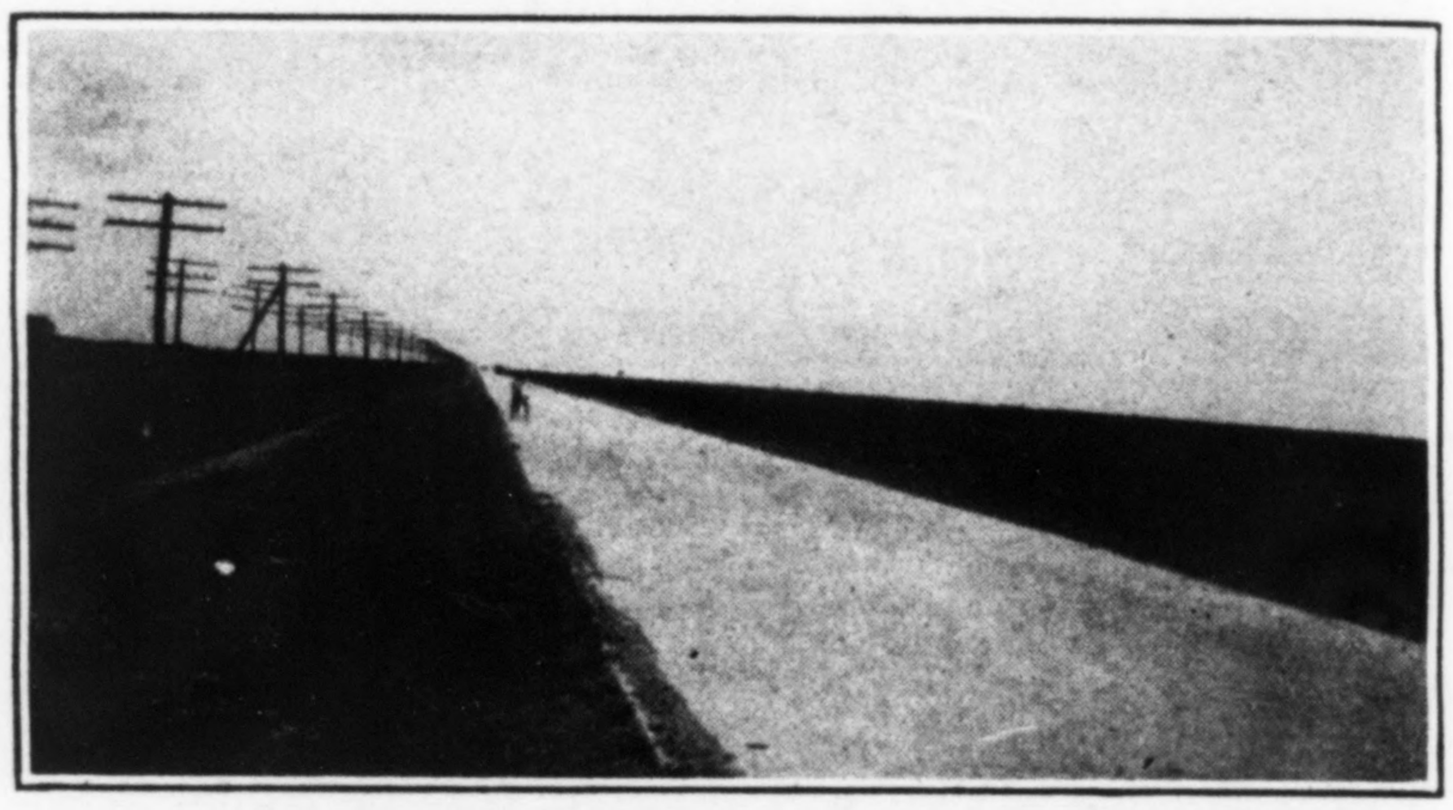


FIG. 1—COMPLETED ROAD OVER SWAMP KNOWN AS ELLINGTON FIELD GAP
Note the posts set for the 3-mile guard rail at this point. A wire-mesh guard rail was employed.

the plans for construction had been previously completed and approved by the State Highway Commission and the Bureau of Public Roads.

Bids for the construction of the road were received on Jan. 30, 1928, and the contract was awarded to A. A. Davis & Company of Kansas City, Mo., at its bid price of $367,286. The contract required the work to be started by Feb. 22 and the pavement to be completed in 75 working days. The contractor gained fifteen days by starting his camp and preparations for grading on Feb. 6. Exactly one month later, on March 6, his forces began to pour concrete pavement. The 10.4 miles of pavement slab was finished at 10 a.m. May 19, about twenty days ahead of the contract time. In 30 days more the shoulders were completed. The finished road in view of its speedy completion challenges admiration.

Paving on Fresh Fill—It is believed that this is the first time in Texas roadbuilding, if not in roadbuilding anywhere, that paving immediately following fresh fill and new bridge work has been undertaken on so large a scale. The methods followed, it is thought, although never before employed in Texas, are in keeping with sound engineering practices and will serve as a precedent for future similar operations. Of course several years of observation will be required to demonstrate conclusively what amount of settlement there will be, but there is confidence that it will not be injurious.

Grading Problems—The grading of the new road presented an unusual set of problems. To begin with, the

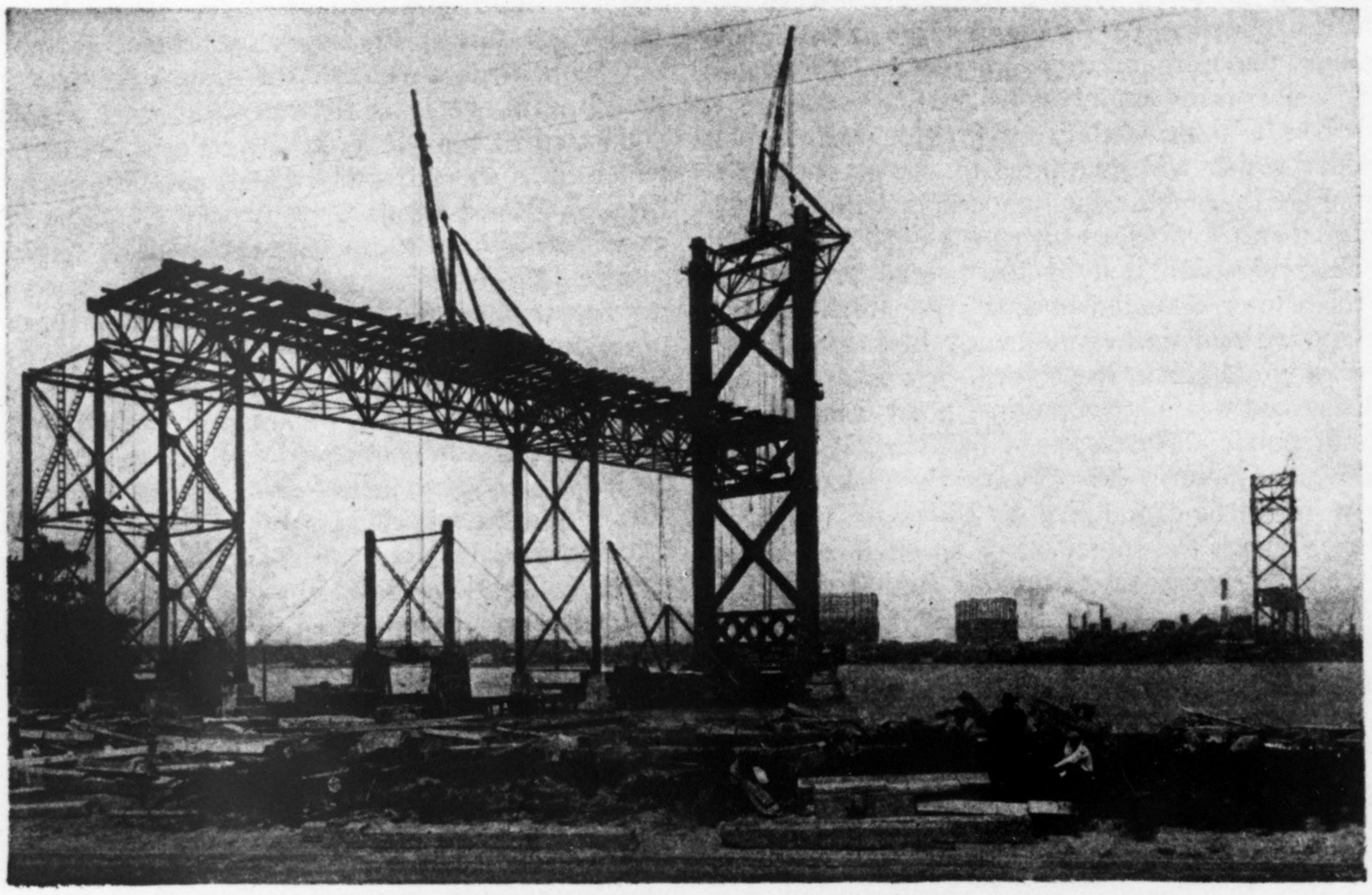


Fig. 1—Detroit River Bridge Towers Under Erection. View From Canadian Side.

Suspension Bridge Tower Erection at Detroit by Creeper Traveler

Sections up to 60 Tons in Weight Handled—Erection Procedure—Adjacent Approach Trusses Also Placed by Creeper Derrick to Give the Tower Erection Stability

By Lloyd L. Martin

Erection Engineer, McClintic-Marshall Company

The Detroit River suspension bridge, with a span of 1,850 ft. is second in length only to the Hudson River bridge, under construction in New York City. A design article on the Detroit River bridge was published in *Engineering News-Record*, Sept. 27, 1928, p. 461, and that article should be referred to for a complete understanding of the make-up of the towers. Briefly, each tower is composed of two columns 67 ft. apart and connected by the necessary bracing. Each tower column is made up of three cells designated by Mr. Martin as the center or main section and the wing sections. His article, which follows, gives a detailed account of the erection of these interesting structures. —Editor.

The erection of the 363-ft. towers for the Ambassador bridge over the Detroit River between Detroit and the Canadian border cities in Ontario involved somewhat greater problems than other suspension bridge towers recently erected because the average weights of the column sections handled were greater and because the tower was comparatively slender and lacked stability for its full height against wind and erection forces. The heaviest piece handled, the top center section, weighed 60 tons, and there were several other sections weighing between 55 and 60 tons. As these heavy pieces were successfully handled, it is believed an economy resulted from this erection arrangement which decreased the number of field splices and rivets and the number of units to be erected. There were only 23 field rivets per ton of steel.

The erection equipment consisted of a steel stiff-leg derrick carried on a special underframe which was raised progressively up the river face of the towers. Two of these tower creepers were used and work was carried on simultaneously on both the Canadian and United States towers. The derricks were designed with the idea of their being useful equipment on other work.

Setting the Bottom Tower Sections—The bottom lifts of the towers had to be placed before the creeper underframe could be erected. The United States tower was located on land, while the Canadian tower was placed some little distance from shore. These respective locations made necessary different erection arrangements for these bottom tower sections.

The United States tower was located in the Pere Marquette railroad yards and was therefore accessible with locomotive trains. The bases, first lift of columns, bracing and creeper were erected with two 40-ton cranes working from a track located close to the pier. The center bottom section, 47 ft. long and weighing 54 tons, was set up on the base with the two cranes, using 55-ft. booms. This operation involved turning the piece on end from the cars and passing it between the cranes to erect it on the base. The cranes during this operation were working to their maximum capacity and drift. On account of the length of the section, brackets were attached just above the center of gravity for attaching the crane hooks. These provided the needed drift and acted somewhat as a balance beam. The erection of the creeper itself presented no special difficulty. A crane with a 120-ft. boom was used to erect the derrick members on top of the underframe in its first position. The remainder of the tower steel was erected by the creeper picking the sections direct from cars on the track in front of the pier.

The Canadian tower was located about 225 ft. out from the bank of the river. There was no derrick boat available in Detroit with sufficient capacity and length of boom to erect the first lift of the tower and the creeper. Therefore one of the available derrick boats was used to erect the derrick temporarily on pile clusters on the shore side of the pier in the same relative location to the tower as it would be when on the river side and a part of the creeper. The derrick was counterweighted with beams from the approach span (Fig. 3) and in this position erected the first lift of the tower and the creeper underframe. All steel was delivered by barge from a storage yard on the United States side of the river.

Following the erection of the bottom sections of the tower, the derrick was erected on top of the creeper underframe. The boom was jumped to the bottom strut of the tower and set in a steel foot block. Using one leg of the tower as a mast, it then transferred the derrick mast, sills and stiff-legs to the creeper, after which the

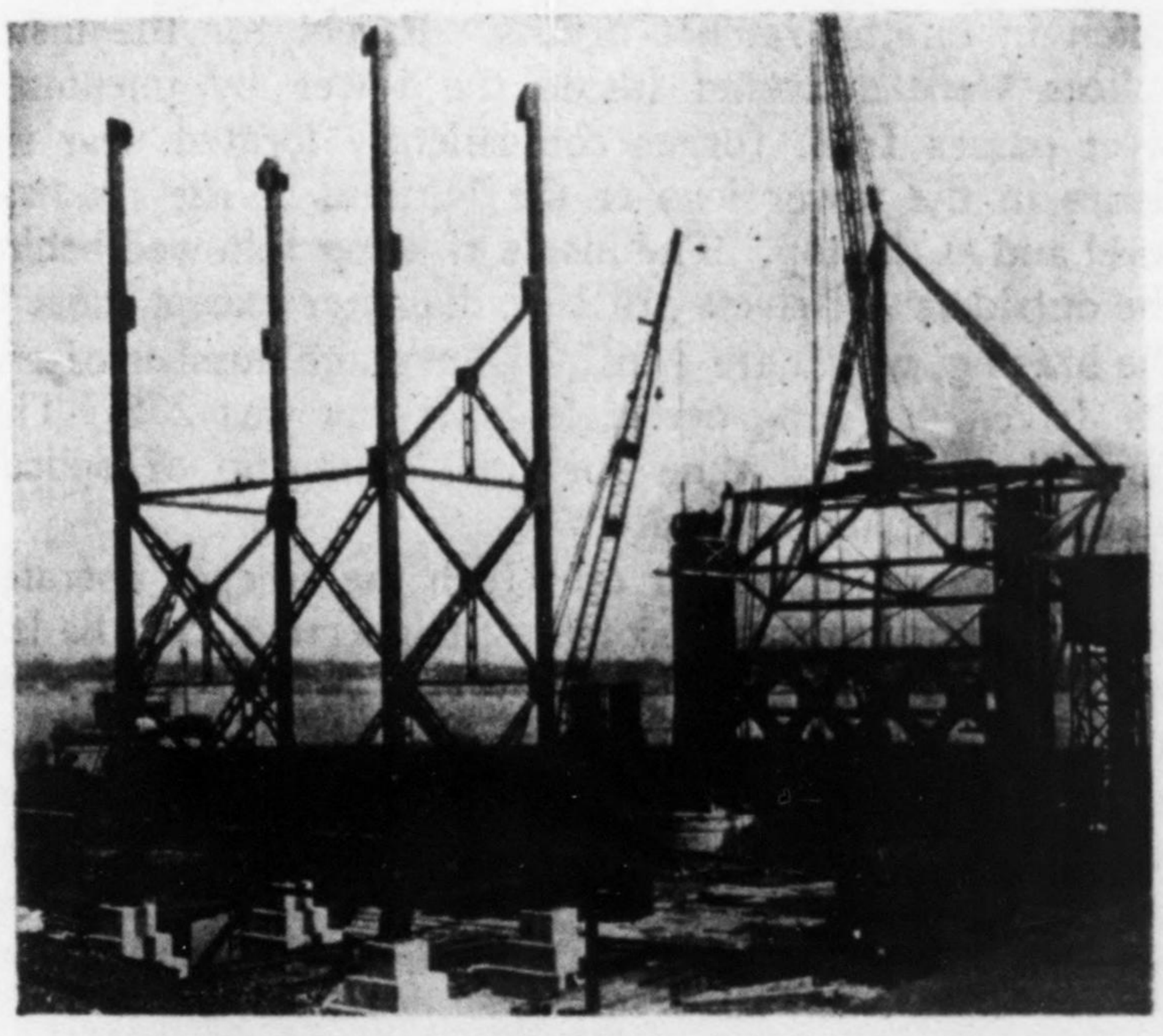


FIG. 2—CREEPER TRAVELER IN FIRST POSITION ON U. S. TOWER
Approach tower bent being erected in foreground.

boom was jumped from the tower strut to its position on the creeper. When the tower was erected and the creeper lowered, it was dismantled by again transferring the boom to the foot block on the bottom strut and reeving up with the roadway strut for the boom falls connection.

The Creeper Traveler—The derrick of the creeper traveler had a sill spread of 67 ft., equal to the distance between the tower legs, and carried a 90-ft. boom having a live-load capacity of 65 tons at a 42-ft. radius. In addition there was a 10-ft. jib on the boom with a 15-ton capacity. The mast of the derrick was set on the center line of the bridge and 15 ft. from the transverse center of the tower legs to allow the tower bracing to be erected with a steep boom, the minimum radius being 13 ft. The details of the creeper traveler are shown in Fig. 4.

The underframe was designed to carry all derrick reaction with the boom in any position ahead of the transverse center line of the tower legs—that is, in an arc of nearly 180 deg. in front of the mast. Only loads were picked behind this line which would give equivalent reactions. The total weight of the underframe was 48 tons and the weight of the derrick 42 tons, so that the weight of the creeper to be raised, including rigging, was about 100 tons.

The creeper was raised to its successive positions by two sets of twelve-part $\frac{7}{8}$-in. wire line tackles, the lower

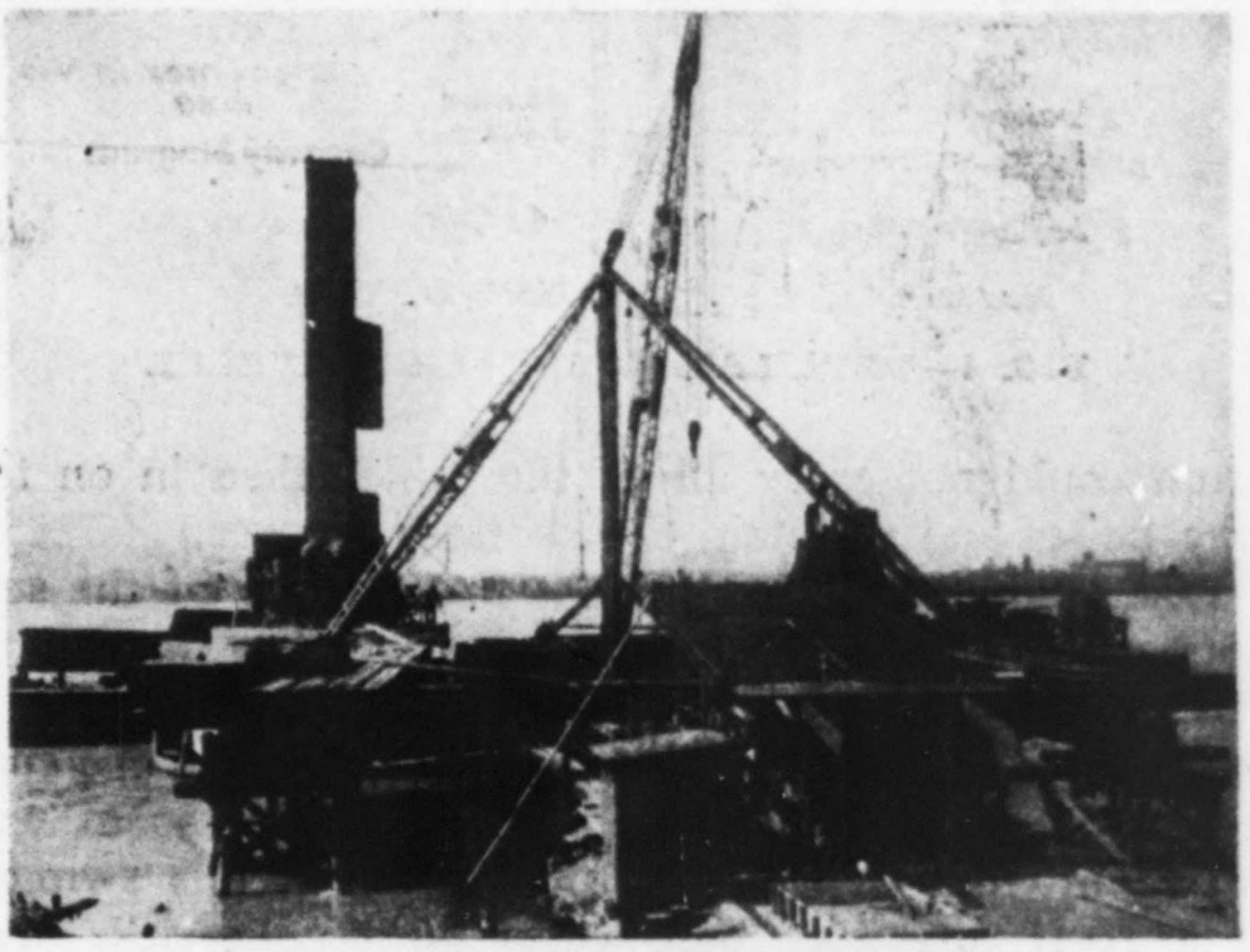


FIG. 3—ERECTING FIRST LIFT OF CANADIAN TOWER
Derrick from creeper set up on piling for this purpose and later transferred to underframe on tower.

blocks being built into the bottom of the underframe and the upper blocks attached to a temporary bracket on the top of last column erected.

When jumping the traveler, the top blocks and brackets had only to be raised with the creeper boom from the deck of the creeper to the top of the next tower section. When the creeper was being jumped, the overturning forces were resisted by rollers at the bottom against the river side of the columns, and by rollers at the underframe deck engaging the outstanding angle legs and cover plates of the wing column section. Sliding side guides were also provided both at the deck and at the bottom rollers. The jumping operation from the time the erection of one lift of tower was completed until the creeper was ready to continue erection was about eight hours.

Two three-drum 135-hp. electric hoists were used to operate the creeper, both of these being placed at the bottom of the tower on steel platforms attached to the tower bases. A steel sheave stick attached to the bottom strut of the tower between the hoists received all of the derrick lines and diverted them in the proper directions to the six drums. Additional sheaves were placed on each tower leg just above the engine platform for the lifting tackle leads. One hoist operated the boom falls, one end of the main falls and the auxiliary falls. The other hoist operated the becket end of the main falls, the elevator cage for taking workmen up and down and one jumping falls. The other jumping falls was placed on the auxiliary falls drum of the first hoist, so that both hoists were used in the jumping operation. Both hoists also were used to raise the heavy sections, since one

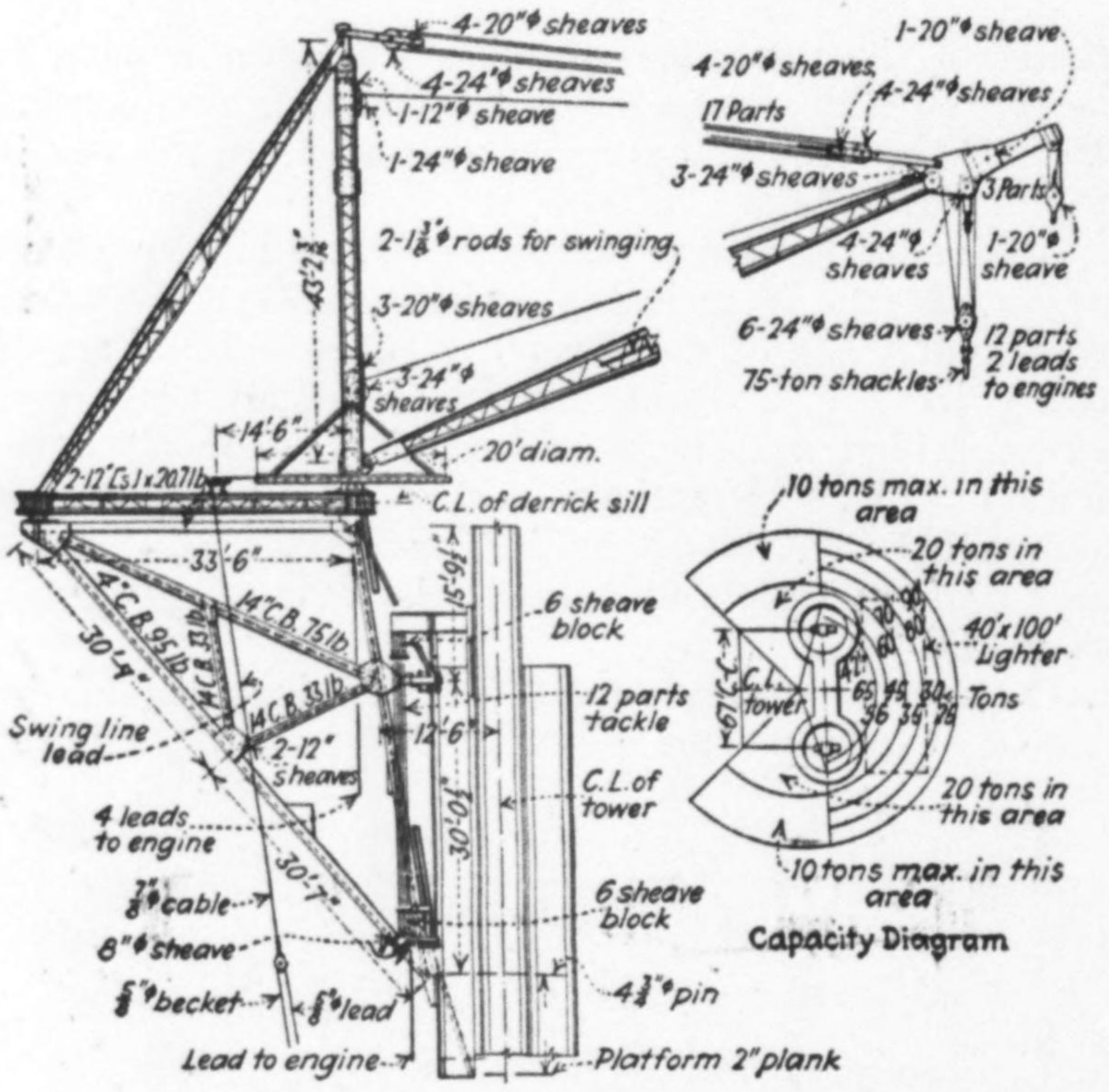


FIG. 4—DETAILS OF CREEPER TRAVELER

drum could not carry all of the cable taken in on the long hoists and maintain the necessary power.

One engine was used for the final setting of the sections after they had been raised, inasmuch as this engine had main, boom and auxiliary falls lines. In addition, the engineer operating this engine also operated the controller of an adjacent swing engine. The boom was swung by four-part tackles running from this swinger to a 20-ft. diameter bull wheel on the derrick. All signals were given by an electric bell system from the deck of the creeper, a separate bell being used for each drum.

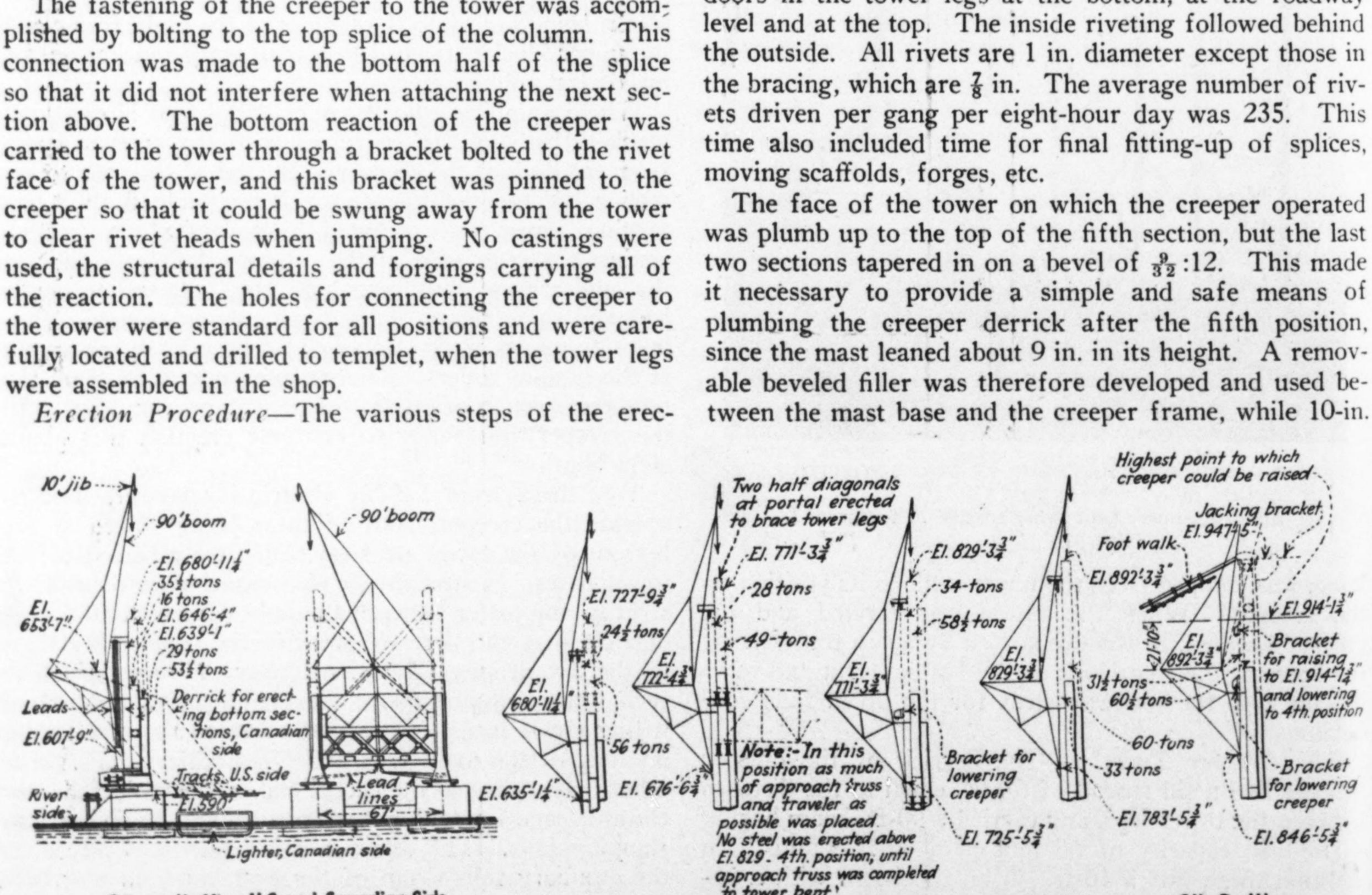
The fastening of the creeper to the tower was accomplished by bolting to the top splice of the column. This connection was made to the bottom half of the splice so that it did not interfere when attaching the next section above. The bottom reaction of the creeper was carried to the tower through a bracket bolted to the rivet face of the tower, and this bracket was pinned to the creeper so that it could be swung away from the tower to clear rivet heads when jumping. No castings were used, the structural details and forgings carrying all of the reaction. The holes for connecting the creeper to the tower were standard for all positions and were carefully located and drilled to templet, when the tower legs were assembled in the shop.

Erection Procedure—The various steps of the erection procedure are shown in Fig. 5. In raising the tower sections, a device was used which bolted to the splice plates and connected to the main fall block swivel with pins and links to permit the up-ending of the sections. The same lifting device was used for the bracing with hitch plates to attach it to the bracing members.

The splice plates were shop riveted to the top ends of the center sections of the towers, but the splice angles were left loose. On the wing sections, however, the splice plates were loosely bolted to allow a horizontal movement of the section being erected so that it would clear the rivet heads on the webs of the center section of th tower which extended above the wing section. The connectors worked from a scaffold built around the tower leg and permanently fastened to the creeper. This platform may be seen in Fig. 2. Generally it was jumped with the creeper, although the portion which fouled the bracing was removed when necessary.

No difficulty was experienced in placing the milled joints in contact. All dirt and excess paint were carefully cleaned from between the splice plates, while in setting a section it was lowered nearly to place and then picked up about 2 in. so that any dirt or paint which had been scraped between the milled joints could be blown out with an air hose. The fitting-up was usually done from the erecting scaffold before the creeper was jumped; riveting followed two splices behind the erection. The riveters worked from steel-frame cages with wooden floors and roofs hung from U-bolts in the splices above by ½-in. wire lines attached to small hand-ratchet drums on the cages. These cages were jumped with the creeper boom when the 150 ft. of ½-in. cable was all taken in on the ratchet drums. Rivets for the inside splices were delivered inside the tower by pneumatic rivet passes from forges conveniently located near the doors in the tower legs at the bottom, at the roadway level and at the top. The inside riveting followed behind the outside. All rivets are 1 in. diameter except those in the bracing, which are ⅞ in. The average number of rivets driven per gang per eight-hour day was 235. This time also included time for final fitting-up of splices, moving scaffolds, forges, etc.

The face of the tower on which the creeper operated was plumb up to the top of the fifth section, but the last two sections tapered in on a bevel of $\frac{9}{32}$:12. This made it necessary to provide a simple and safe means of plumbing the creeper derrick after the fifth position, since the mast leaned about 9 in. in its height. A removable beveled filler was therefore developed and used between the mast base and the creeper frame, while 10-in.



IG. 5—ERECTION PROCEDURE SHOWING VARIOUS POSITIONS OF CREEPER TRAVELER ON THE TOWER

I-beam stools were inserted under the rear end of the derrick sills, capable of being removed and replaced with beveled plates to allow the necessary lowering of the sills. The operation was as follows: The boom was landed on top of the fifth column section to relieve the load at the mast base, which was then jacked up and the changes were made in the beveled sills under it. Then by swinging the boom in line with one sill and then the other and manipulating the four 2-in. tie-down bolts at the end of the sills, the stools were removed and the sills lowered onto the beveled plates.

The lifting tackle lines were made long enough to lower the creeper in two jumps, the top blocks of the tackle fastening to special brackets bolted to the face of the tower legs. The boom was removed with the creeper in the highest position and lowered first, since it could not be carried high enough to clear the tower bracing and because to swing to the side toward one stiff-leg would have caused unequal reactions on the roller guides and lifting tackles and a greater overturning reaction than was provided for.

Tower Stability During Erection—The towers when standing alone and erected to their full height were not able safely to resist wind loads and creeper reactions. Therefore at the roadway level it was necessary to stop tower erection and erect the approach spans as far as the first approach tower bents, these to take the wind reaction from the tower through the truss system to the ground. In Fig. 7 the creeper derrick is shown erecting one of the 37-ton trusses on the United States side. Each of these trusses had a span of 121 ft. and was assembled and completely riveted on the ground. The creeper derrick erected these trusses with about a 76-ft. reach. The tower bent was erected with a locomotive crane.

On the Canadian side, it was necessary to erect two

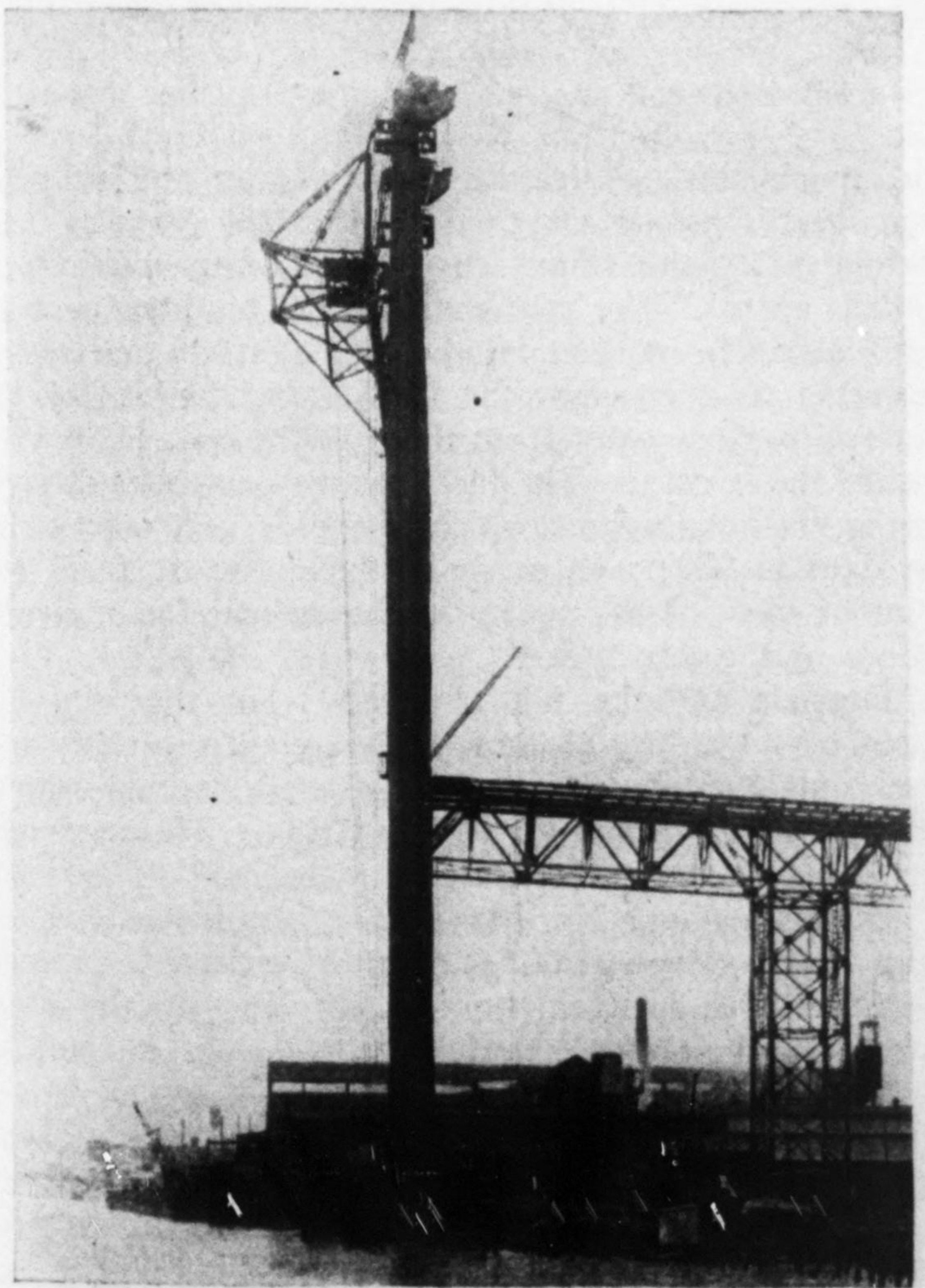


FIG. 6—CREEPER IN HIGHEST POSITION ON COMPLETED TOWER—U. S. SIDE

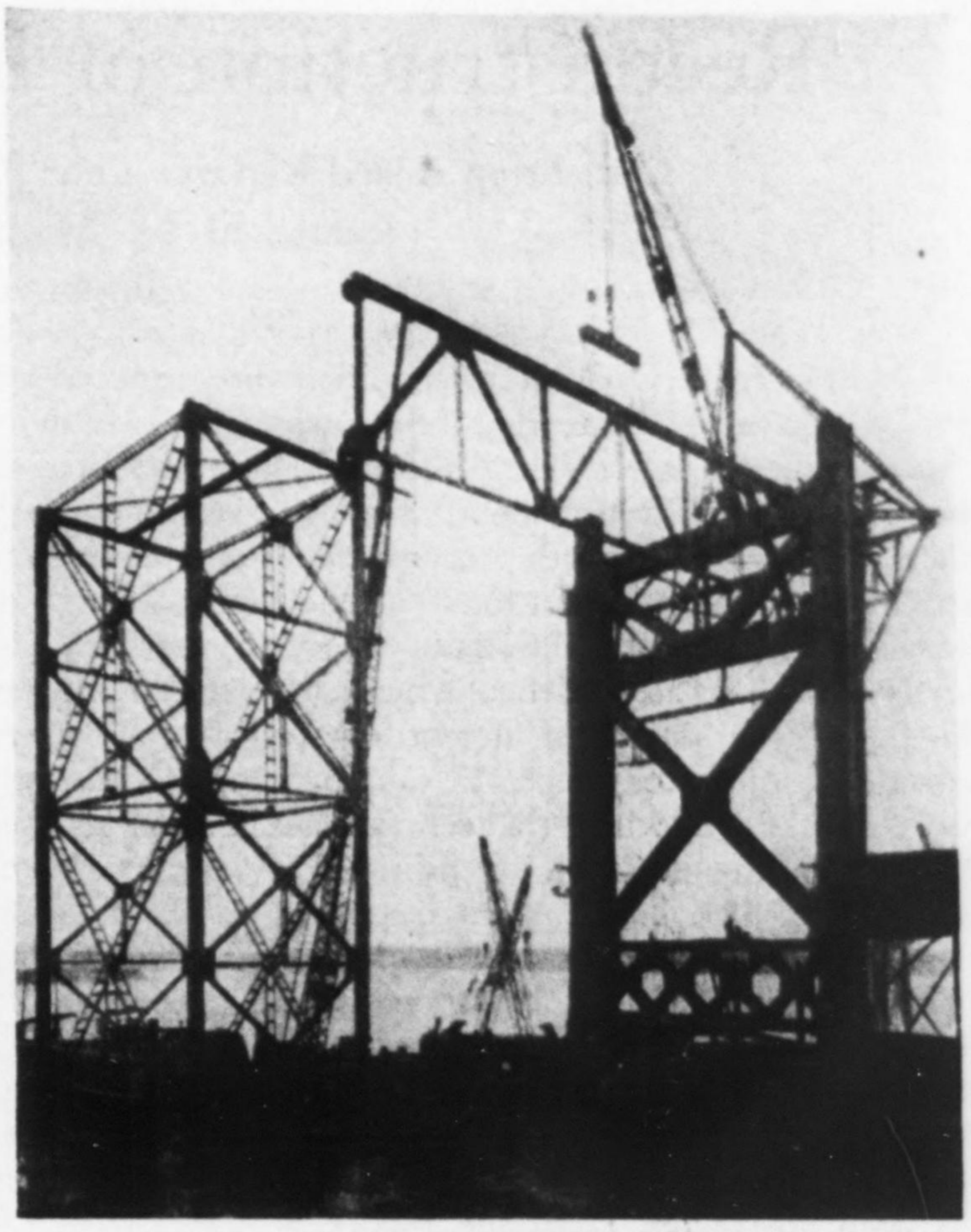


FIG. 7—ERECTING APPROACH TRUSSES TO ASSURE FULL-HEIGHT STABILITY OF TOWER

152-ft. truss spans in order to reach the first tower bent, as shown in Fig. 1. The complete 152-ft. truss exceeded the capacity and reach of the creeper so that temporary steel falsework was erected at the center of the span and the half-trusses erected in units by the creeper. The approach span traveler was then erected on this half-span and proceeded with the truss erection toward the tower bent. The steel used for the temporary falsework was the bent from the span ahead of the one being erected.

Administration—The erection work was carried out by the McClintic-Marshall Company, Pittsburgh, as engineer and contractor for the complete bridge. G. A. Caffall was manager of erection and the writer was erection engineer. A. Toohey was foreman in charge of the Canadian tower and G. A. McClain was foreman in charge of the United States tower. The erection of the United States tower was started March 31, 1928, and was completed, including erection of cable saddles and rollers ready for cable spinning, July 14. The Canadian tower was started April 30 and completed July 27. This time includes the period of approach truss erection when erection of the tower was suspended. There is approximately 2,265 tons of steel in each tower. The record tonnage erected in eight hours was 185, this being four fifth-story sections.

Long Concrete Runway Feature of New Airport

At the Grand Central Air Terminal now nearing completion at Los Angeles, Calif., a concrete takeoff runway 72 ft. wide and 3,000 ft. long has been laid. On either side of the concrete strip is a 10 ft. width of asphalt paving placed as a shoulder, giving the runway a total width of 92 ft. Another important and unusual feature of this new airport is the fact that special emphasis is to be placed on the elimination of dust.

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 meas-

FIG. 1—MOUNT HOPE BRIDGE JUST PRIOR TO DISMANTLING

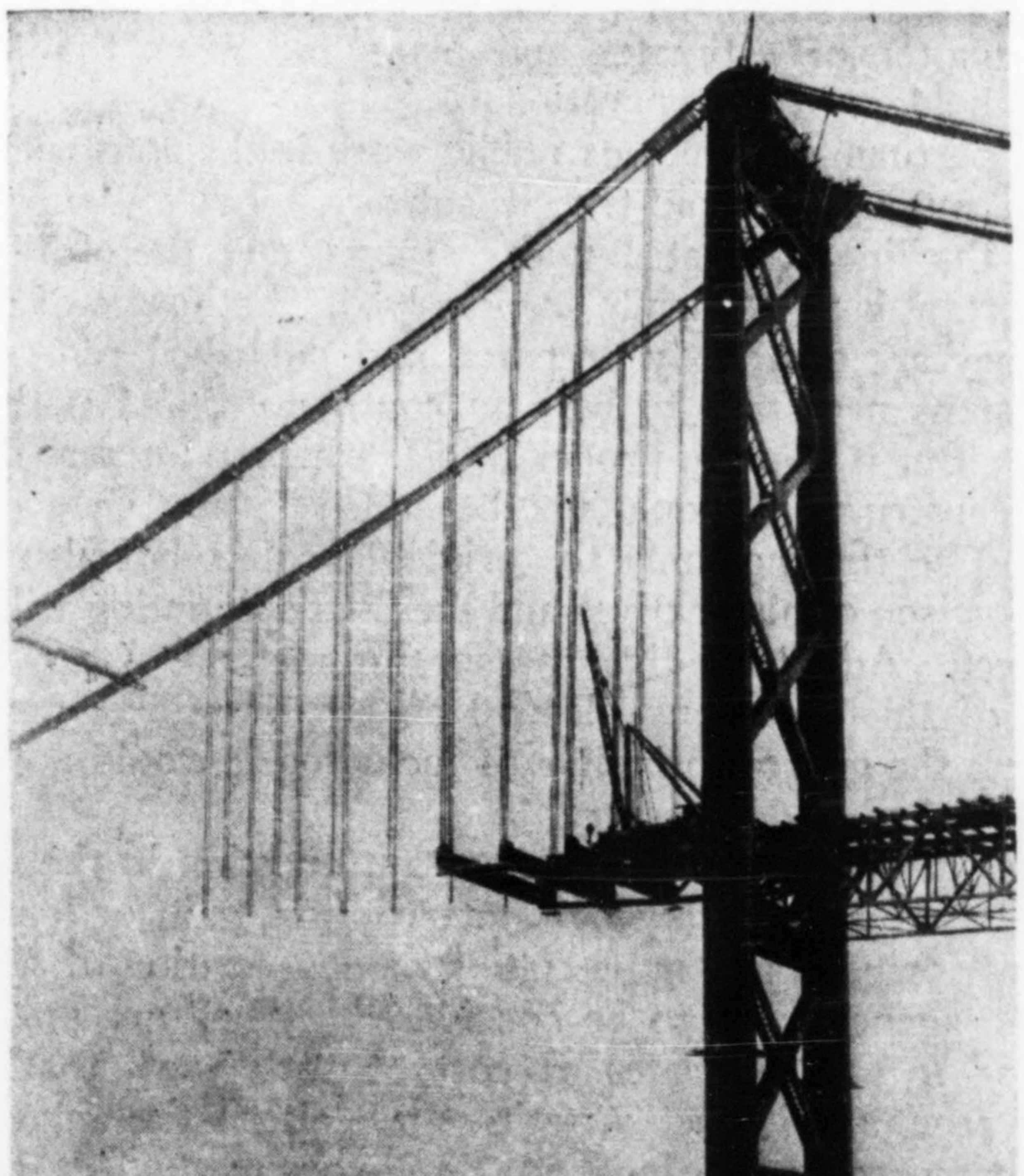


FIG. 2—AMBASSADOR BRIDGE BEFORE DISMANTLING

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

ures 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.

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

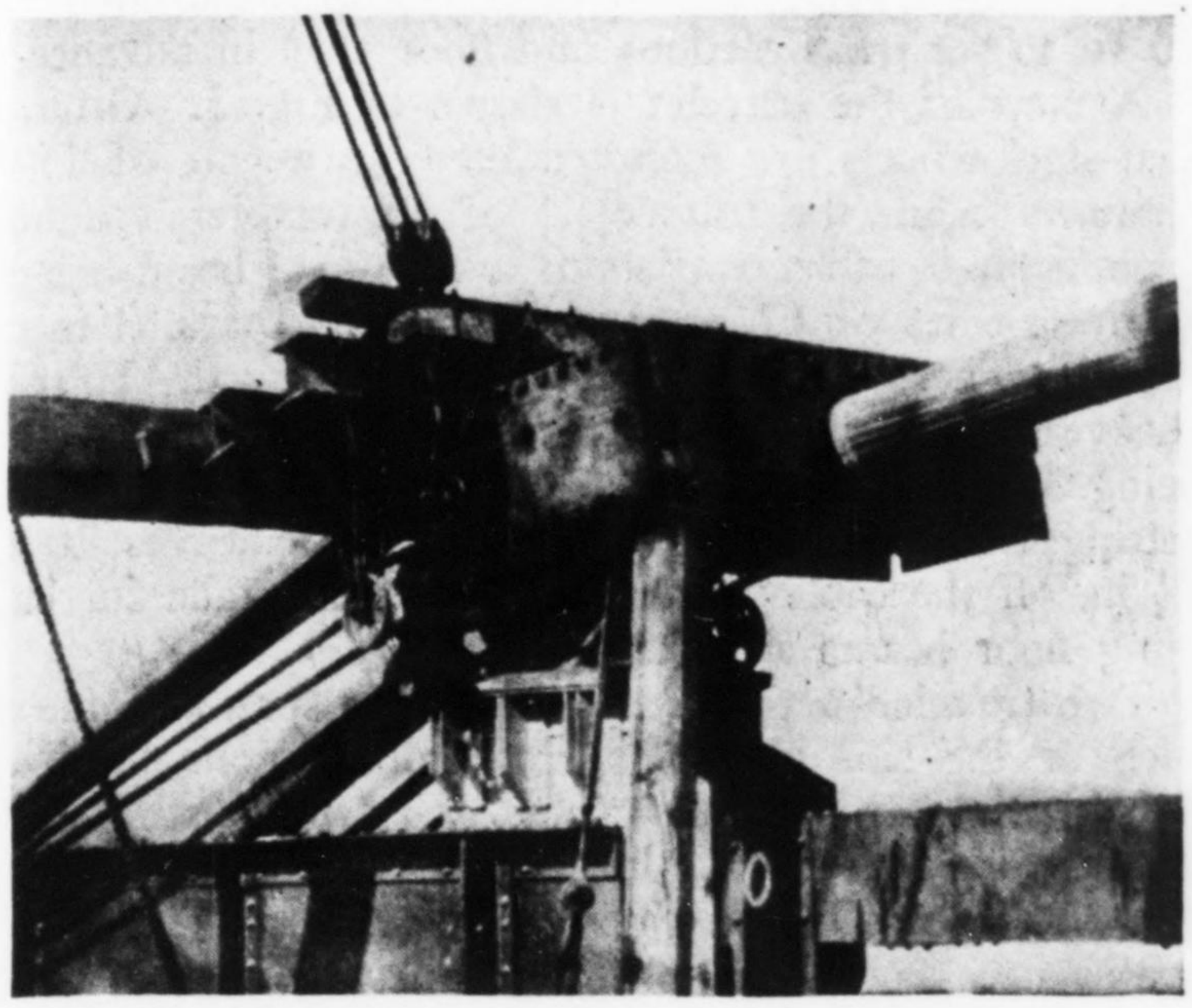


FIG. 3—HOLDBACK YOKE IN FRONT OF CABLE-BENT SADDLE, MOUNT HOPE

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,

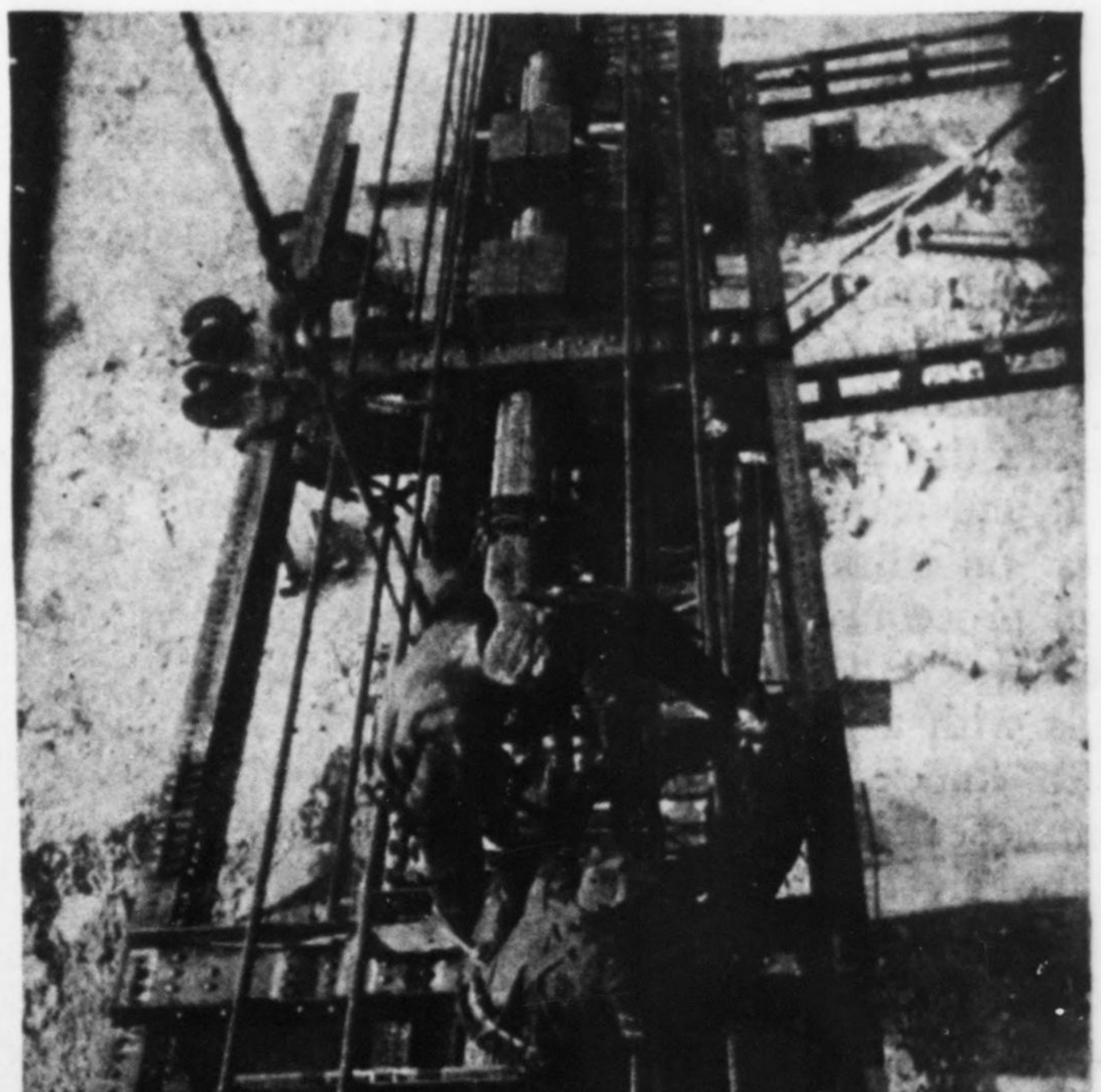


FIG. 4—REMOVING THE EQUIPMENT USED TO INSTALL BACKSTAY HOLDBACKS, MOUNT HOPE

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 holdback rope, which has been cut off, can be seen in bottom yoke just inside of right-hand side angles.

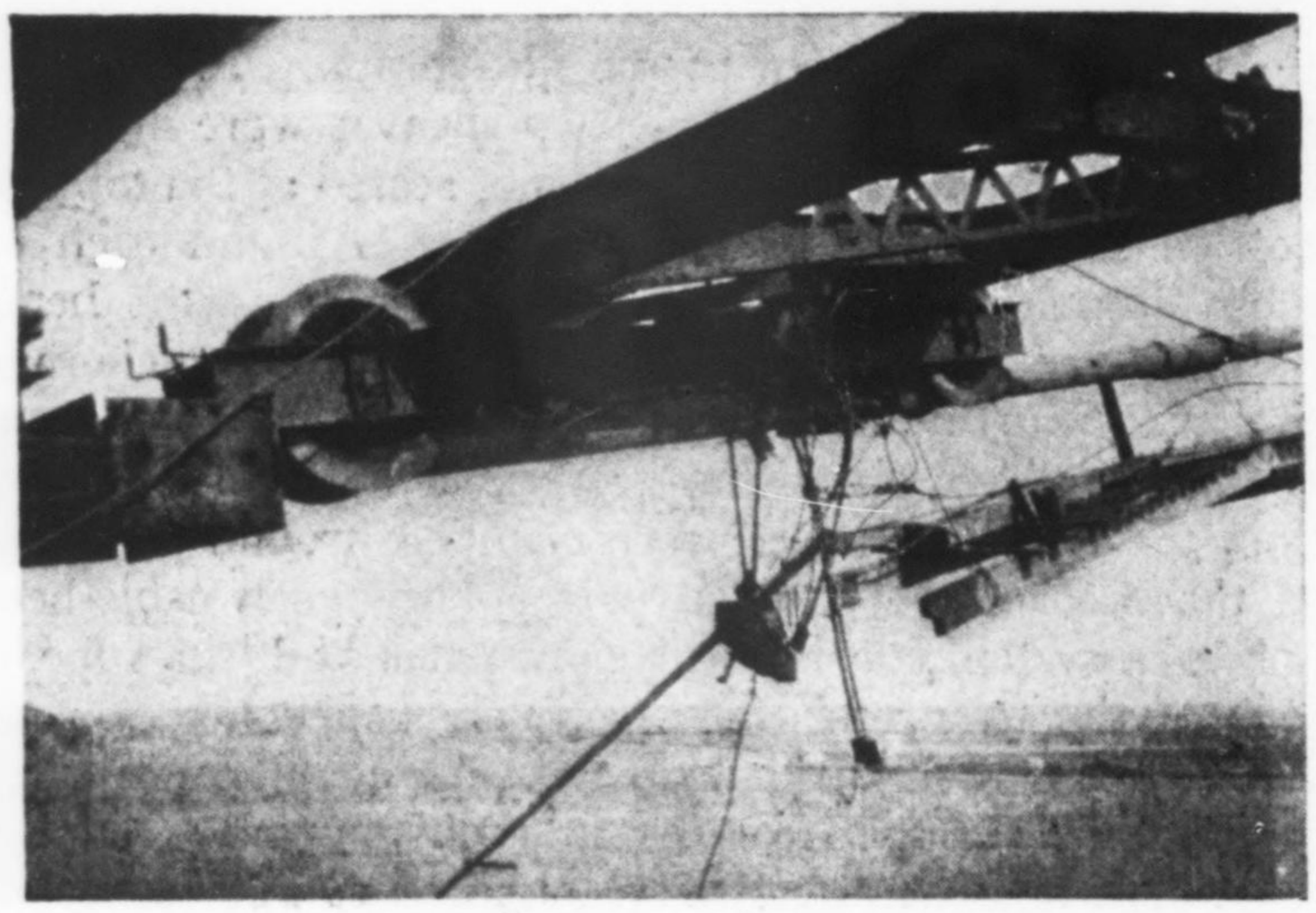


FIG. 5—TRAVELERS AT MOUNT HOPE DESIGNED TO RUN ON CABLES

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.

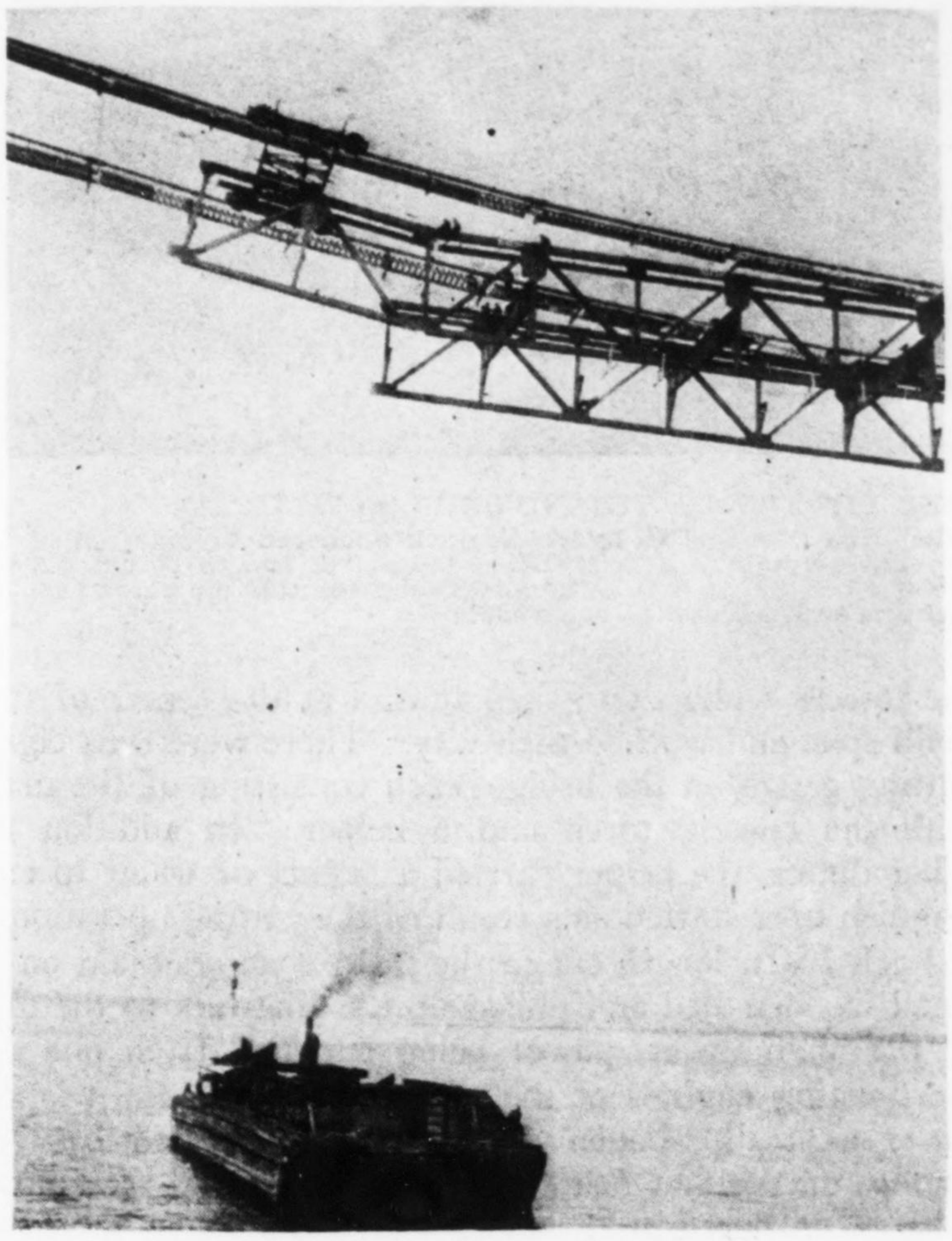


FIG. 7—TRAVELER LIFTING TRUSS SECTION FREE OF SUSPENDERS, MOUNT HOPE

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.

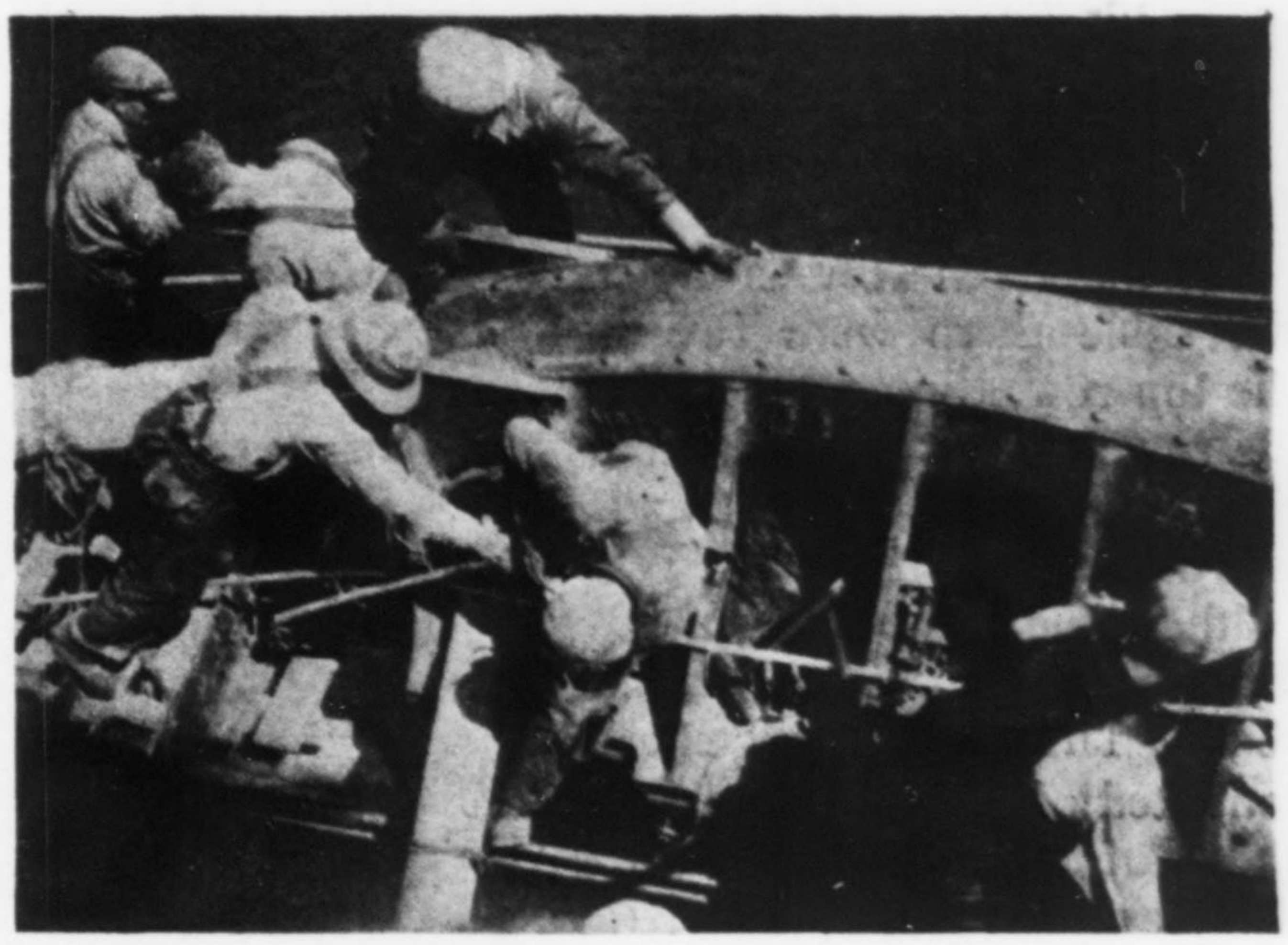


FIG. 8—JACKING THE MAIN TOWER SADDLE AT MOUNT HOPE

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.

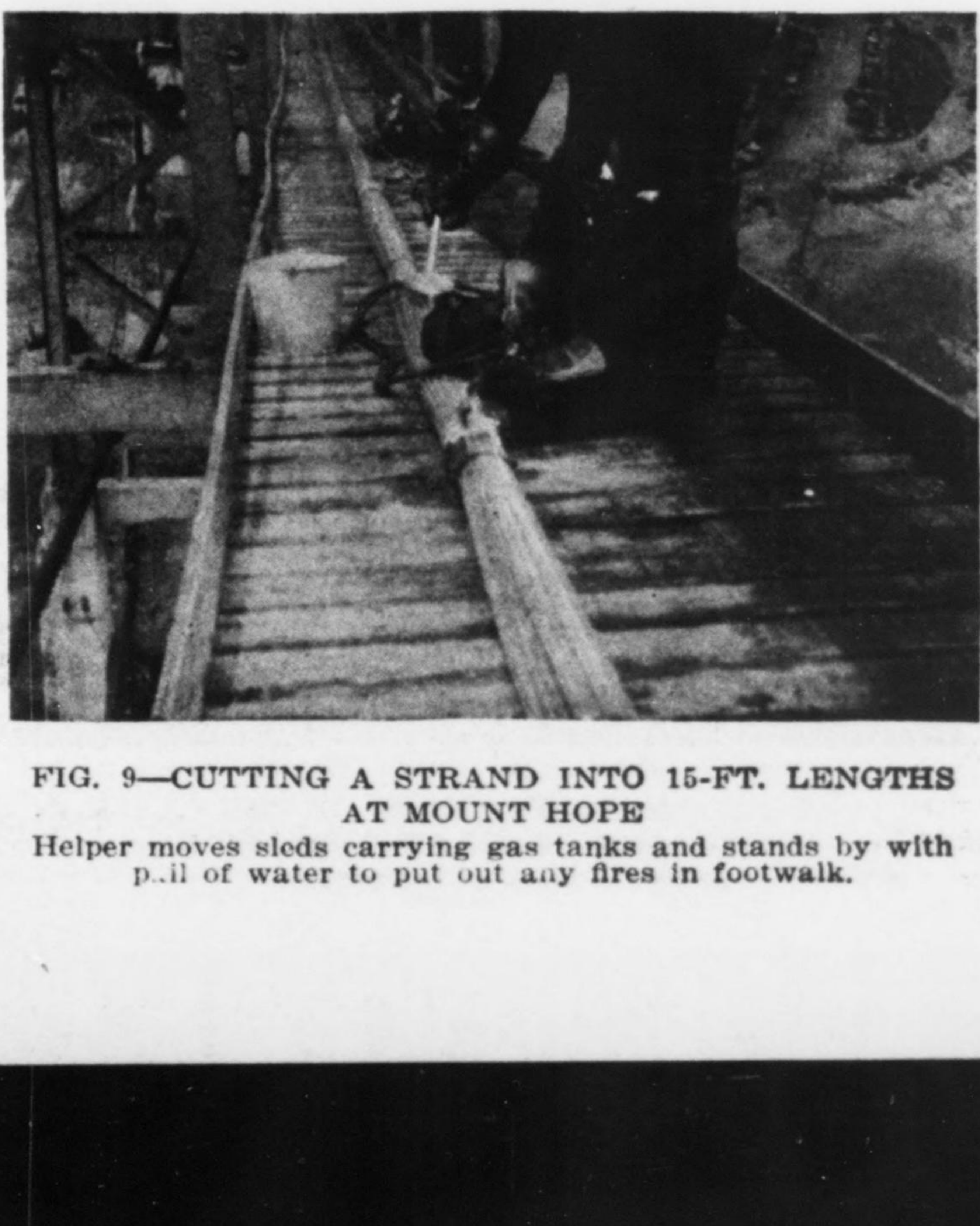


FIG. 9—CUTTING A STRAND INTO 15-FT. LENGTHS AT MOUNT HOPE

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.

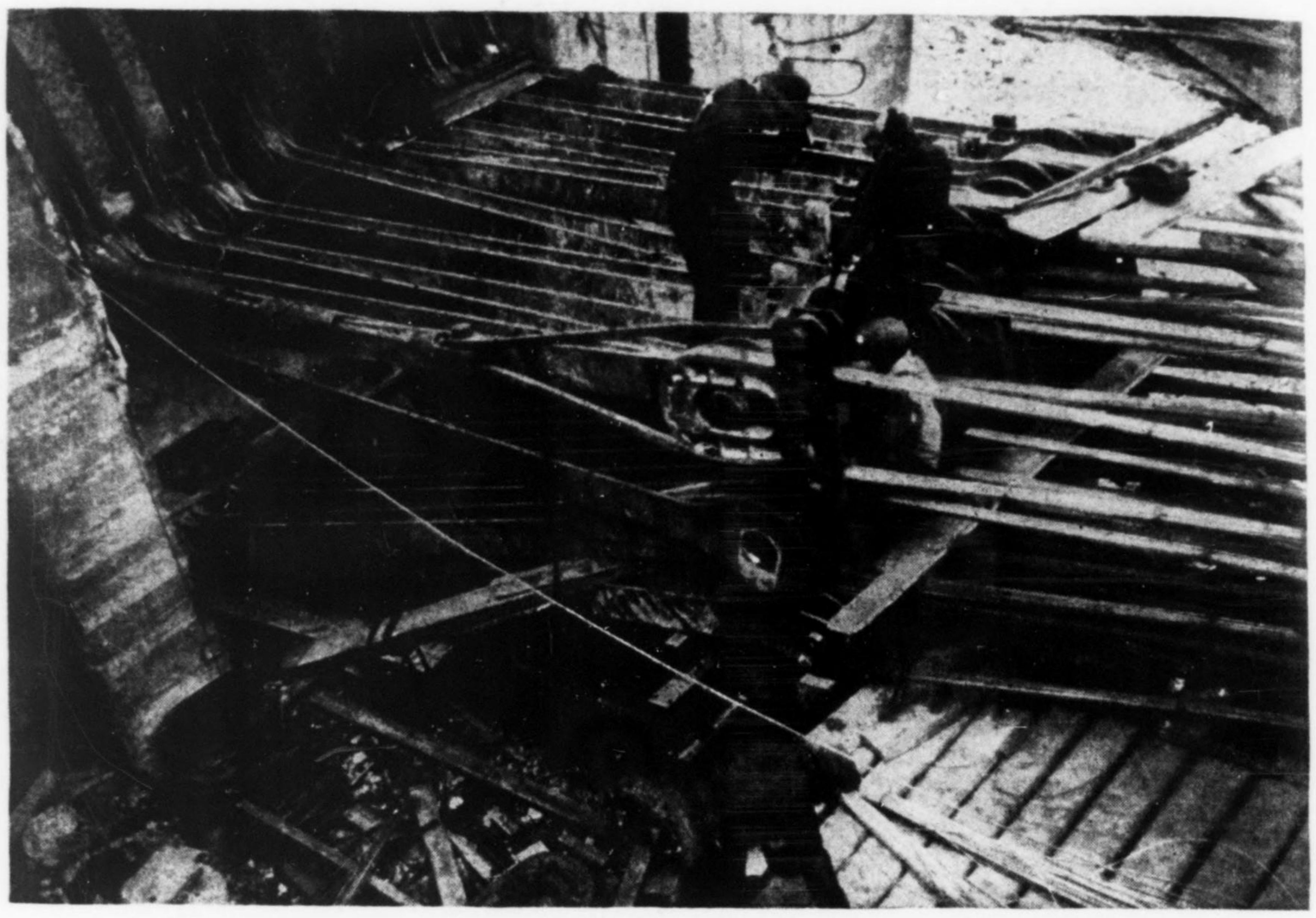


FIG. 11—REMOVING PIN FROM A STRAND SHOE AT DETROIT
Note yoke ahead of strand shoe and attached to hydraulic jack anchored to rear pin of anchorage eyebar. Note also concrete adhering to eyebars; 150 cu.yd. had to be cut out before the dismantling work could begin. At bottom in foreground the take-up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

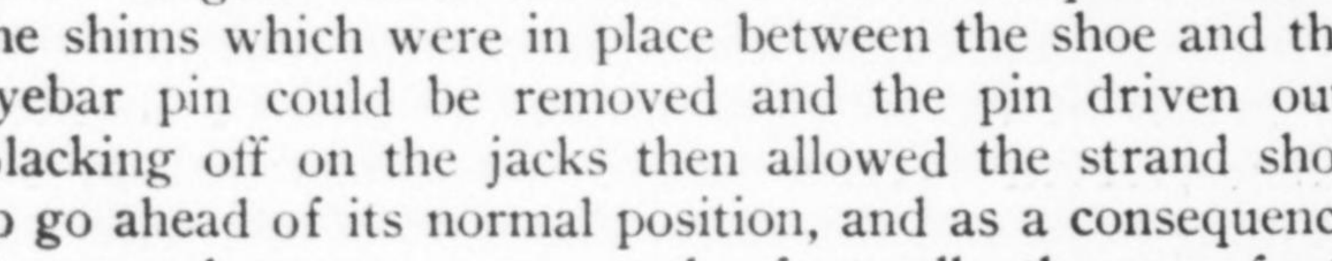


FIG. 10—A STRAND JACKED FREE OF THE CABLE IN THE MAIN SPAIN AT DETROIT
Cross footwalk in foreground.

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 floor-beams, 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

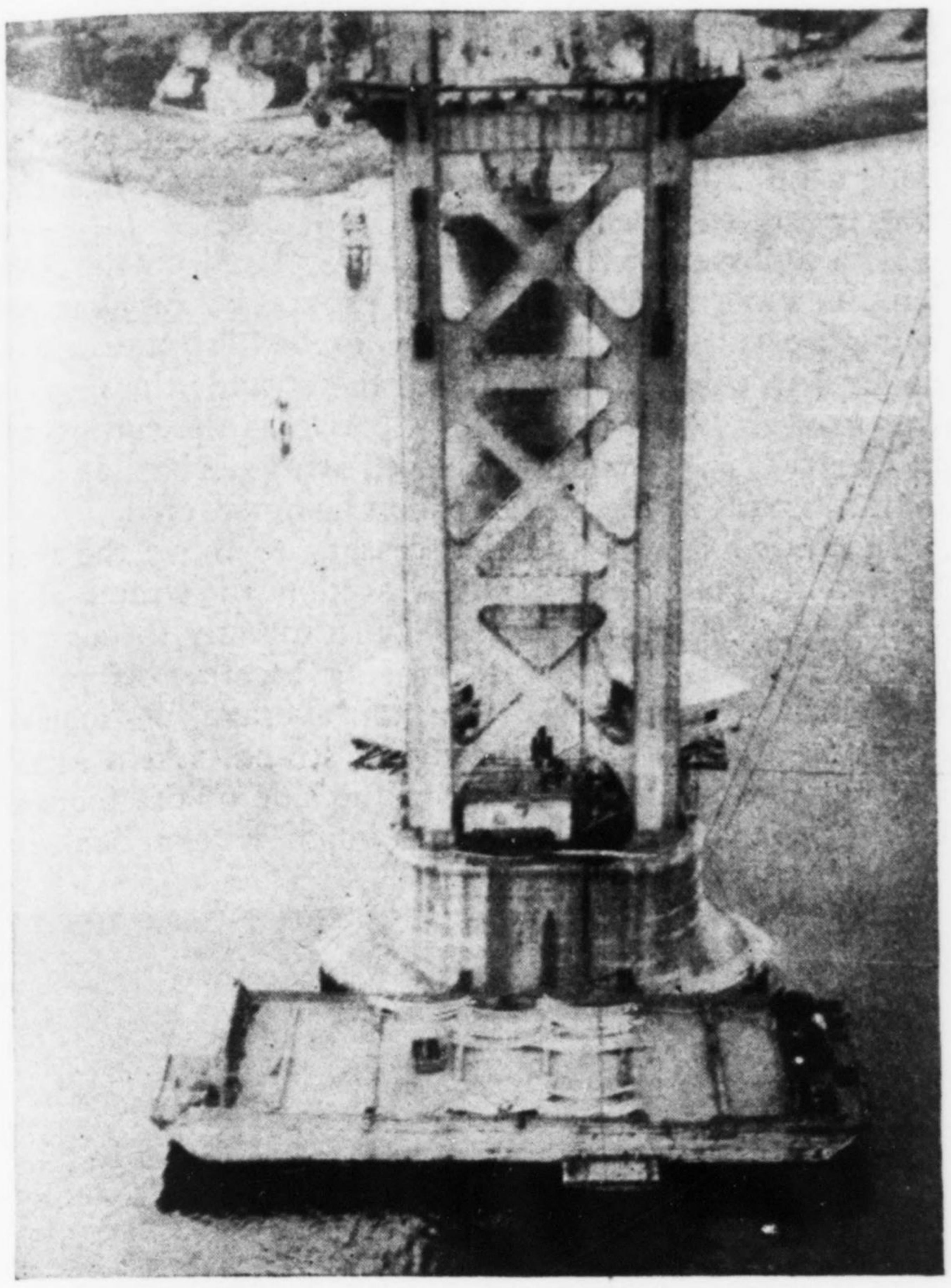


FIG. 12—SEVERED SECTIONS OF CABLE ON BARGE AT BASE OF MAIN PIER, MOUNT HOPE
Note houses on far side of tower inclosing hoists which operate the travelers.

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

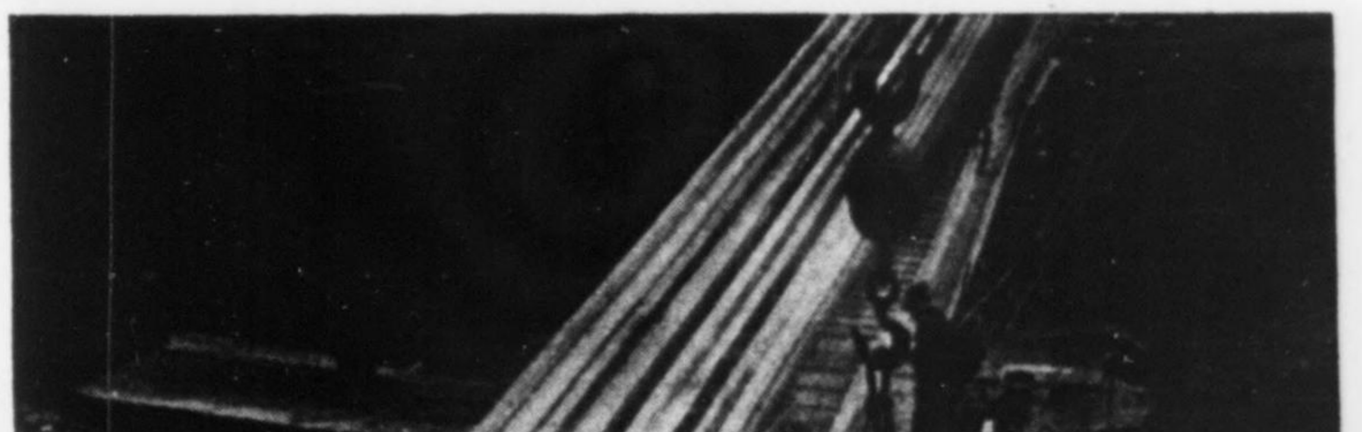


FIG. 13—CUTTING A BACKSTAY STRAND INTO 30-FT. SECTIONS AT DETROIT ANCHORAGE
Note widened footwalk and men placing wire seizings on strand. As a section is cut it is pushed off the footwalk to the "boneyard" below. Men on ground are ready to hook a half-dozen sections to boom falls for transfer to waiting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

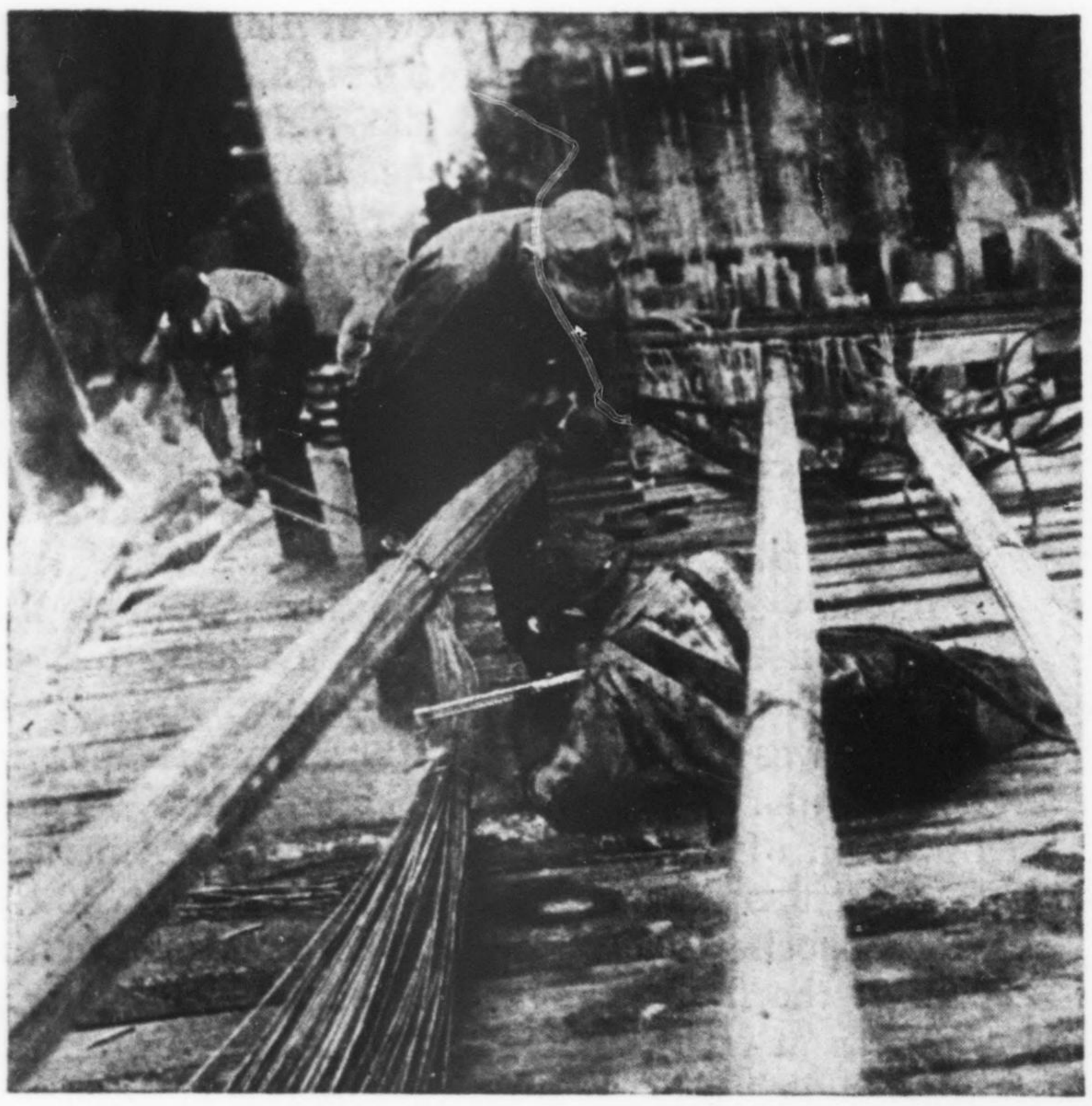


FIG. 14—CUTTING OPERATION AT DETROIT WEST ANCHORAGE
Note that the cut is being made between seizings about 6 in. apart.

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

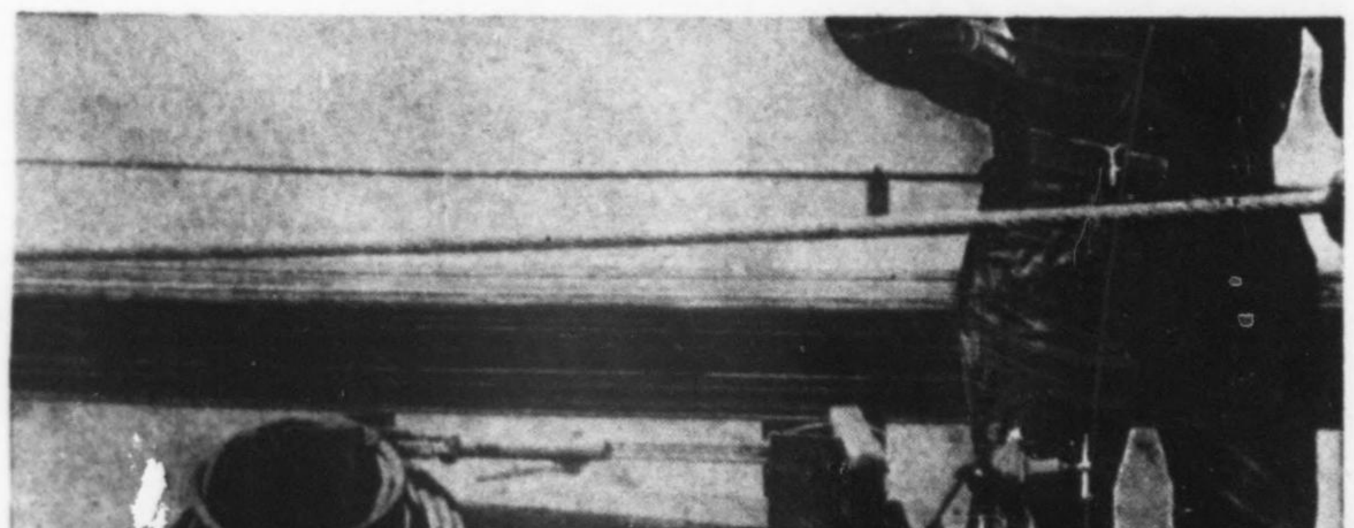


FIG. 15—FLAME-CUTTING OUTFIT USED ON MAIN SPAN AT DETROIT

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.

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

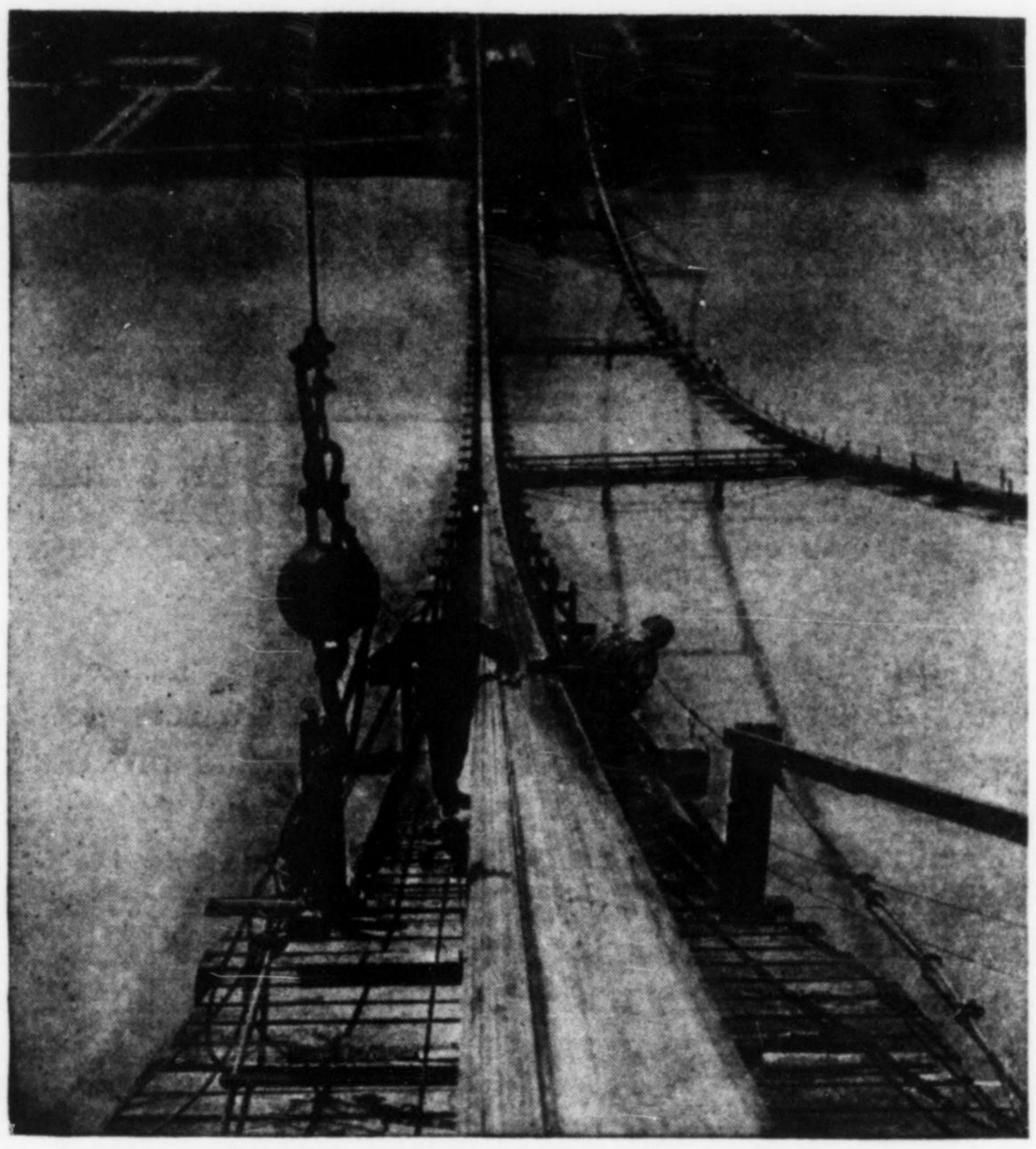


FIG. 16—TWO 30-FT. LENGTHS OF CABLE STRAND COMING UP FROM DETROIT MAIN SPAN ON SLED
Wire lowered at main tower to gondola cars in railroad yard below.

away. About two minutes was consumed in disposing of each 30-ft. section.

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

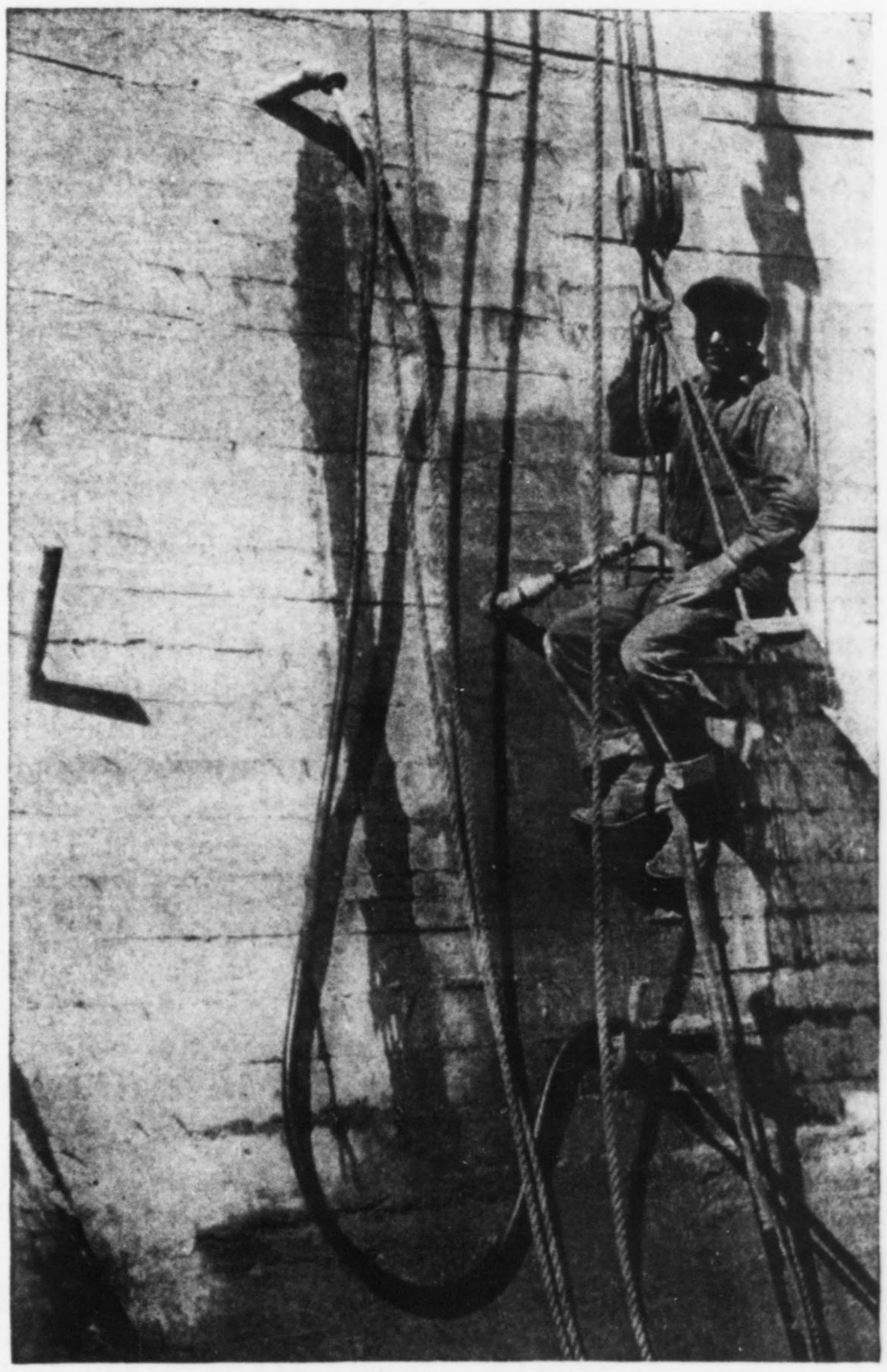
Arch Dam 110-Ft. High Contains Only 970 Cu.Yd. —Reinforced for Temperature, Doweled to Abutments and Grouted

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.

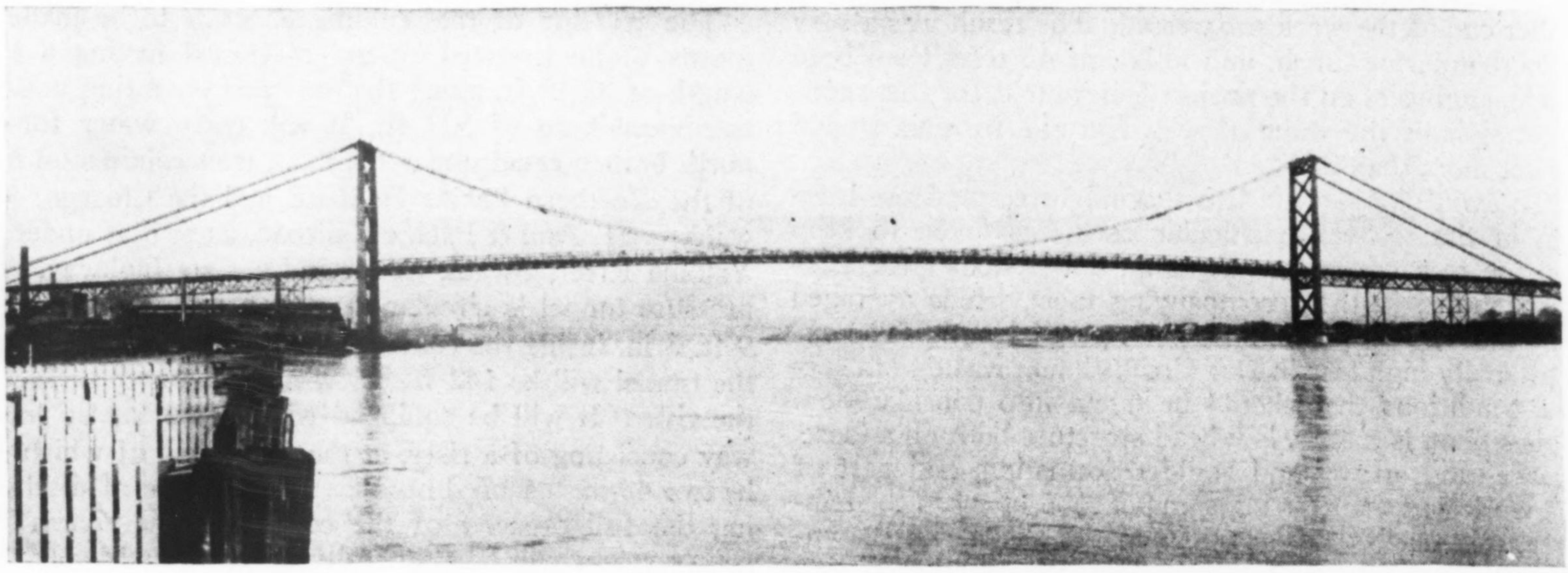
To provide a good seat for the arch without restricting free arch movement, the abutments were stepped to make the thrust normal to the rock surface and were cut away to eliminate contact with the arches on either face. Possible separation of the concrete from the rock at the abutment extrados was counteracted 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



The Ambassador Bridge Crossing the Detroit River in an 1,850-Ft. Suspended Span

Detroit Suspension Bridge Opening Nine Months Ahead of Schedule

Remarkable Record Made in Dismantling and Re-erecting Structure With 1,850-Ft. Main Span—Some Changes in Design Necessary in the Interest of Speed

When the decision was made last April to dismantle the Ambassador suspension bridge in Detroit and install new cables, the structure was well along toward completion and about one year ahead of schedule. So rapid has been the progress made in dismantling and re-erecting the structure that it will be opened as this is published, still nine months ahead of the scheduled opening date in August, 1930. This remarkable record is matched only by the similar work carried out on the Mount Hope suspension bridge in Rhode Island, in charge of the same contractors. Some of the methods used at Detroit are here set down as an indication of what a construction organization can accomplish when faced with the necessity.

The Ambassador bridge is a private toll structure connecting the city of Detroit with Windsor and the other border cities of Canada. It has a central span of 1,850 ft. and unloaded backstays. There are two cables, each consisting of 37 strands of 206 No. 6 galvanized wires. The roadway is 47 ft. wide, and there is a single 8-ft. sidewalk along the west side. Towers of unusual slenderness are utilized, and a departure of note in the fabrication is the attachment of gussets to the floorbeams in the shop, in order to facilitate erection. The design of the bridge was described in *Engineering News-Record*, Sept. 27, 1928, p. 460. Other articles have appeared, as follows: foundation construction, Dec. 6, 1928, p. 830; tower erection, Jan. 31, 1929, p. 186; decision to replace cables, April 4, 1929, p. 564.

At the time it was decided to take down the original cables, erection of the steelwork for the suspended span had been started and the first pass of the travelers was well under way. Steelwork was substantially in place for the American approach, and was completed on the Canadian approach. Subcontracts had been awarded for the paving of these portions of the work, as well as for paving the main span when it should be completed. Subcontracts also had been awarded for the construction of the two anchorage superstructures, concrete masses about 50x90 ft. in plan, rising up to the bridge deck level. The portions of these which incased the anchorage eyebars had been poured.

With the decision to remove the cables, work on the anchorage superstructures was stopped, the formwork taken down and stored and the eyebar incasement cut out, as described in *Engineering News-Record*, Oct. 10, 1929, p. 562. At the same time it was decided to concentrate on the approach paving, so that nothing would remain on completion of the new cables except steel erection and paving of the main span.

In order to carry this out, a change in the design of the anchorage superstructure was necessary. As originally designed, these superstructures were masonry piers which received the approach girder span on the one side and the deck truss span on the other side. Such a superstructure could not be built until the cables were spun, and if it were left uncompleted, it would be necessary to leave the truss span hanging as a cantilever and to leave the girder span on the ground. This meant that there could be no concrete deck on either of these two spans. In recalculating the time for completion it was found absolutely necessary to fill in this gap of pavement in order that the hauling of paving material across it to the main span could be commenced at once. Therefore, while the cables were being spun, steel bents were designed, fabricated and erected on the anchorage superstructure to receive the girder spans and the truss spans. With these in place, the pavement was completed.

In re-erecting the suspended span steel every effort was made to economize the time of the raising gang, and all materials possible were framed together on the ground or on the barge. The original plan of suspended-span design by which the floorbeams instead of the trusses were directly attached to the suspenders proved of great value in speeding up the work. Between Sept. 6 and Sept. 27 about 4,000 tons of structural steel was erected.

Erection of the suspended-span steel was effected by means of deck travelers, which at the first pass from the towers to the center of the span hung the floorbeams which carried the stiffening truss bottom-chord gussets and then raised the bottom chords, bottom laterals and enough stringers to run on. On completion of this pass a day was devoted to backing the travelers empty to the towers. On the second pass the travelers placed the web verticals, web diagonals and most of the top chords of the stiffening trusses, as well as the sidewalk framing and sidewalk railings. On the final pass, backing from the center to the towers, the trusses were completed by inserting the remaining floor steel, erecting the roadway side railing and miscellaneous details.

By advancing the erection of the sidewalk railing and framing, it was possible to erect immediately on this framing a wooden roadway for transporting form lumber, reinforcement and concrete. All materials were hauled on this roadway by means of light gasoline tractors, and as soon as the travelers started back from the center on the final pass, it was possible to commence setting floor forms at mid-span, following the travelers to the towers. Thus, with the last steel being placed on Sept. 27, it was possible to commence pouring the roadway slab Oct. 7, and to complete it Oct. 15. In the eight days, 1,850 lin.ft. of 6-in. slab, 47 ft. wide, was placed. While this concrete was curing, preparatory to placing the asphalt surface, the road forms were removed and the sidewalk was poured.

While erection of the suspended-span steel has been under way, the anchorage superstructures have been carried up and are substantially complete. Final wrapping and painting of cables was commenced upon completion of the concrete slabs, and although these will not be completed when the bridge is opened to traffic, they will be completed shortly thereafter.

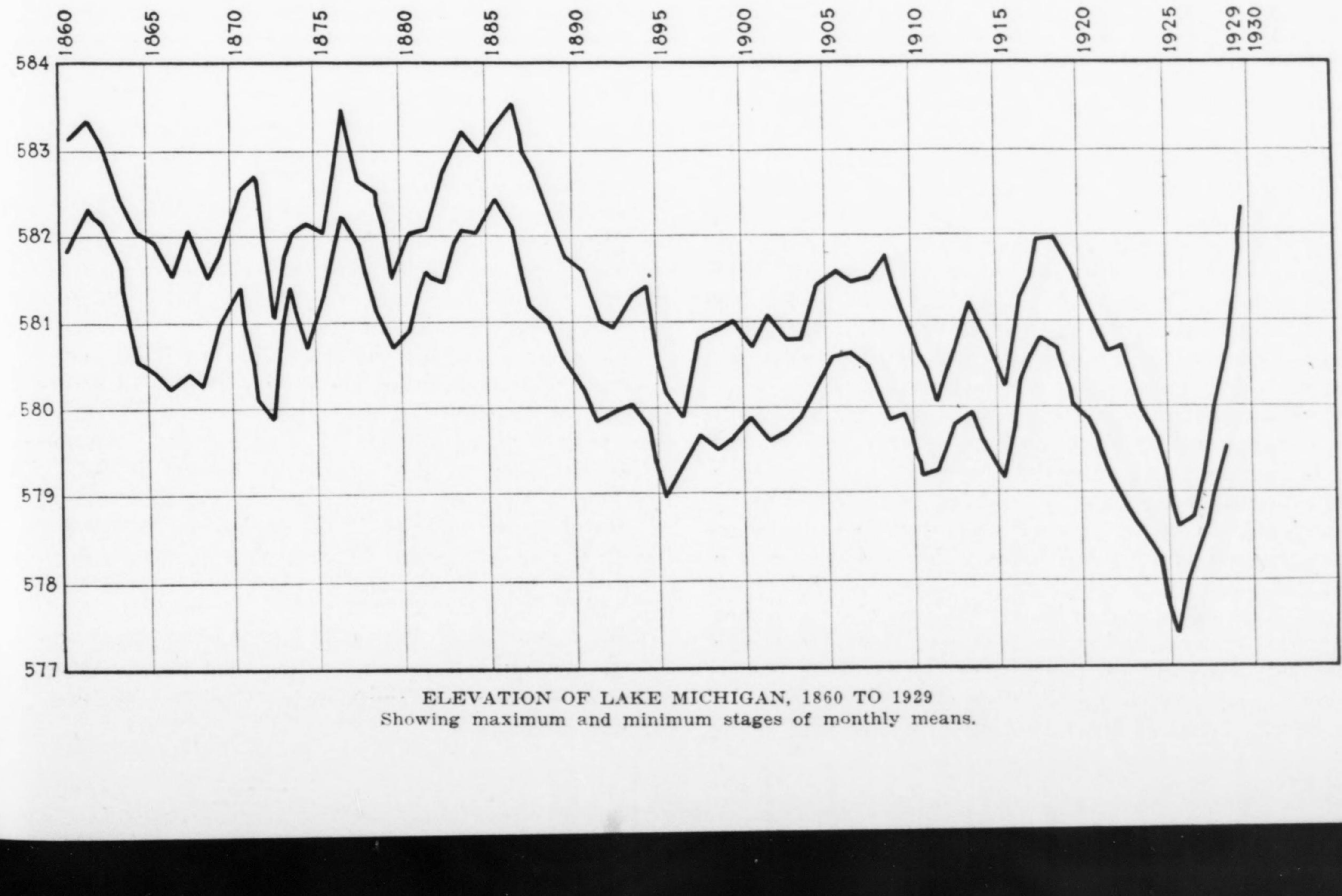
According to Jonathan Jones, chief engineer of the McClintic-Marshall Company, designer and erector of the bridge, complete credit goes to the field forces for this remarkable record. Robert MacMinn, assistant chief engineer and L. L. Martin, erection engineer, McClintic-Marshall Company, deserve credit for the way in which the operations were scheduled and pushed, and L. N. Gross, of the Keystone State Corporation, for the excellent record made in dismantling and re-erecting the cables. Mr. Jones also acknowledges the co-operation of the W. E. Wood Company, of Detroit, subcontractor for paving slabs and anchorage superstructures, and of the Zipp-Beckmeyer Company, of Huntington, W. Va., subcontractor for the electric lighting system.

Lake Levels and Damage to Shore Property at Chicago

Waterfront Development and Disregard of High-Water Stages Contribute to Damage by Recent Storms

In local accounts of the recent extensive storm damage to lakefront property, public and private, at Chicago, much stress is laid on the present high stage of Lake Michigan as a major cause. But a paper read recently before the Western Society of Engineers by W. G. Potter, drainage engineer of the Illinois State Waterways Division, shows that this lake elevation has been exceeded in previous years and that the destructive effects of the waves during the past few weeks may be attributed largely to the remarkable improvement developments along the lakefront during recent years, combined with a general tendency to insufficient consideration of the fact that high-water stages in the lake are of periodical recurrence. In regard to development and shore protection, this paper, of which an abstract follows, deals largely with private property. But it is evident that park and other authorities controlling public property along the lakefront must give more attention to the protection as well as to the development of such property. —EDITOR.

FOR the last few years Lake Michigan has been on a rampage. Protective piers, jetties and bulkheads have been destroyed. Concrete breakwaters have been ruined. Roads and boulevards have been undermined and put out



ELEVATION OF LAKE MICHIGAN, 1860 TO 1929
Showing maximum and minimum stages of monthly means.