# Rope-Strand Cables Used in New Bridge at Portland, Oregon

**St. Johns Bridge, With a 1,207-Ft. Suspended Span Over Willamette River, Sets New Record for Rope-Strand Cables—Extensive Architectural Studies Result in Use of Unique Batter-Leg Towers in Which Pointed Arches Accentuate the Pleasing Appearance**

By D. B. Steinman
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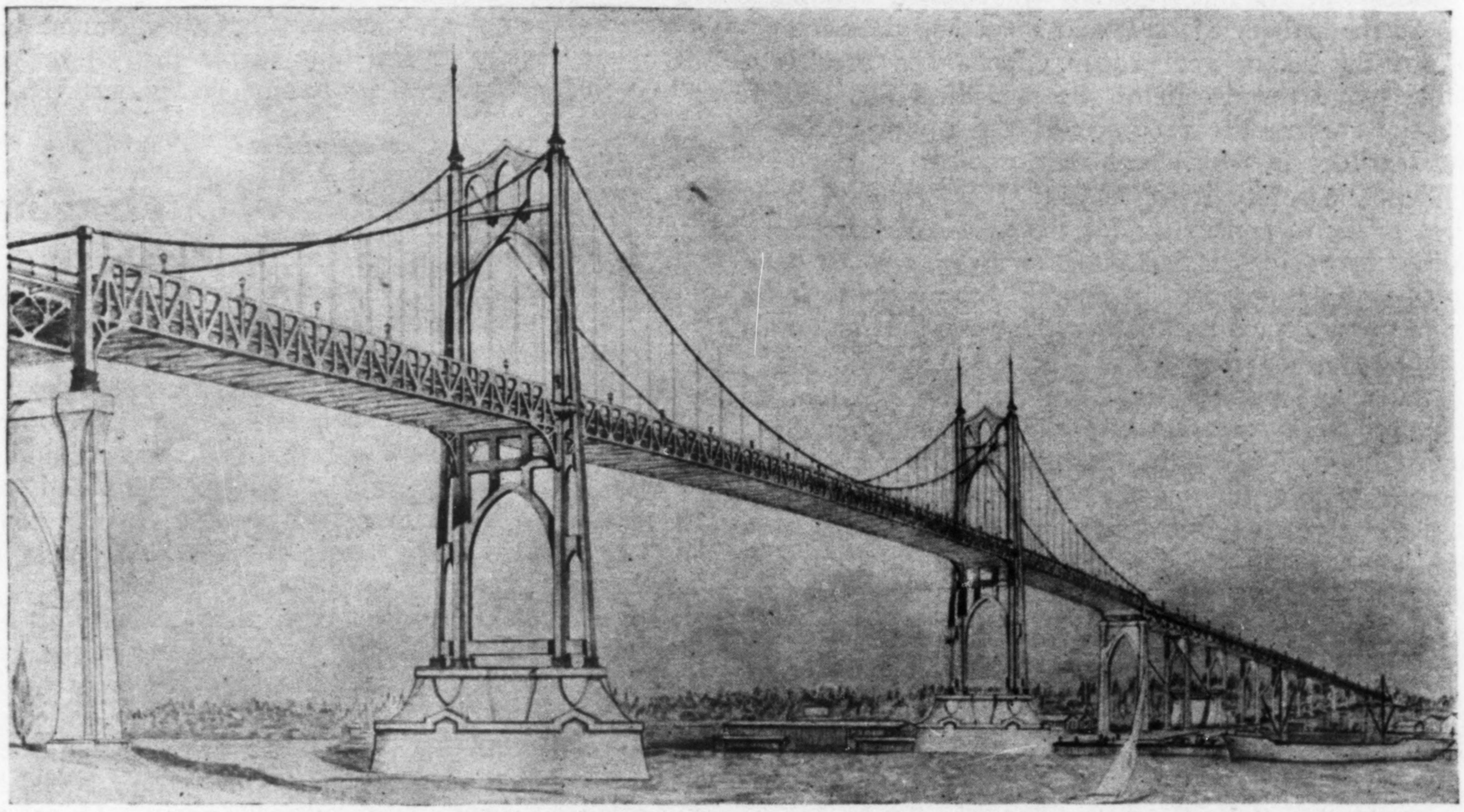


FIG. 1—NEW ST. JOHNS BRIDGE AT PORTLAND, ORE., AN ARTIST'S CONCEPTION
Longest span structure utilizing rope-strand cables. Notable also for architectural design featuring substitution of batter-legs for diagonal bracing in the main towers.

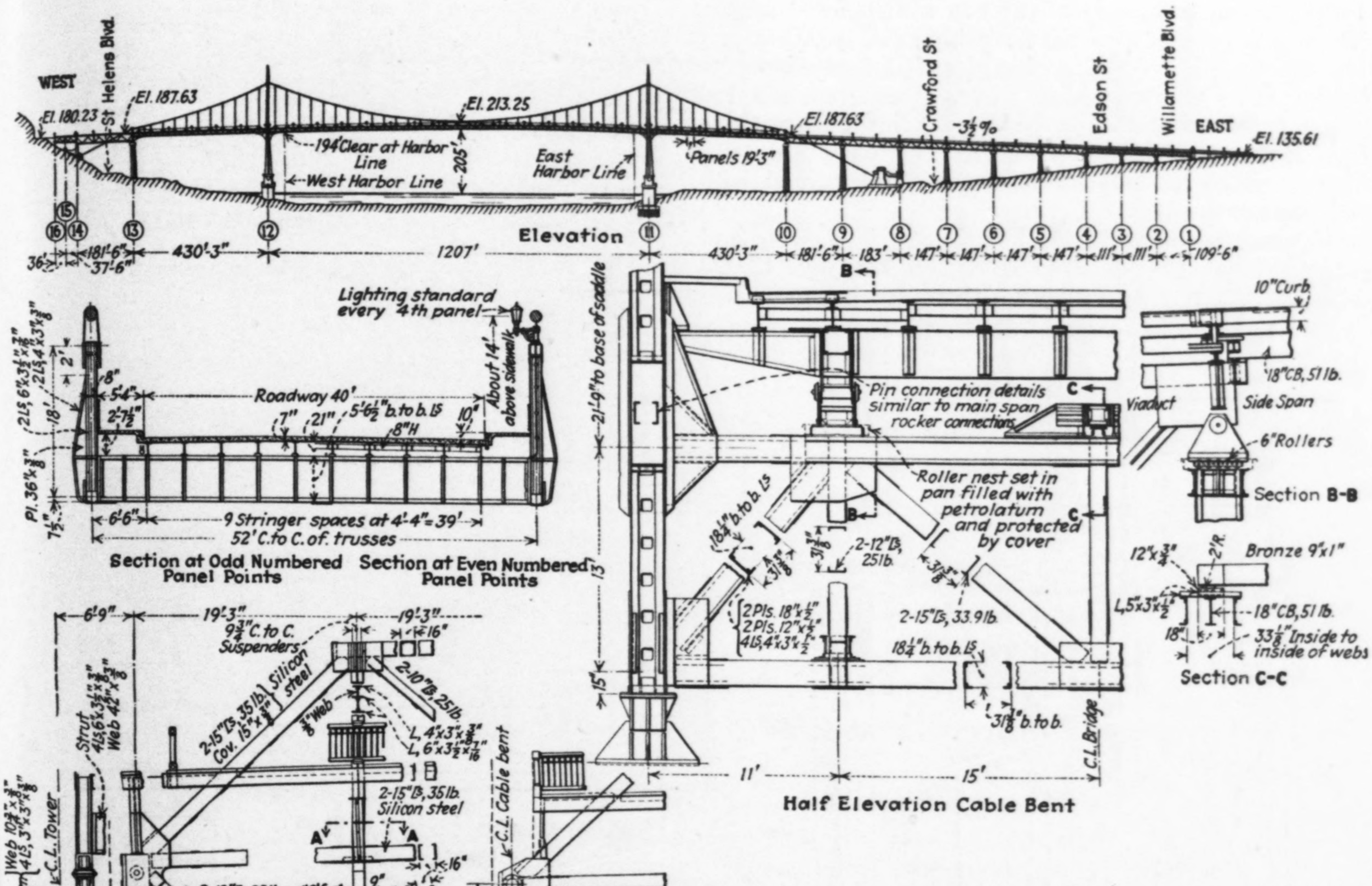
NOTABLE innovations in suspension bridge practice have been incorporated in the design of the St. Johns bridge, now under construction over the Willamette River at Portland, Ore. With a main suspended span of 1,207 ft., it represents the longest bridge upon which rope-strand cables have yet been used, and at the same time it is the longest suspension structure of any type west of Detroit. An underclearance of 205 ft. above navigable water constitutes an additional record. Unusually complete architectural studies accompanied its design. The use of batterleg towers, presenting a pleasing appearance by virtue of the elimination of the conventional diagonal bracing and the use of pointed arch outlines above and below the roadway are features worthy of note. This same motif of pointed arches is extended to the main masonry piers and to the approach piers, where the vaulted opening in the tallest of these piers exceeds a height of 150 ft. Duplication of truss span lengths in the approaches, combining the masonry of the anchorages with the masonry of the adjacent approach span piers, and the use of a special anchorage layout in which silicon steel plates form the anchor chains are other features of the structure.

The bridge is being built by Multnomah County to span the Willamette River between two sections of Portland—the St. Johns district on the east bank and the Linnton district on the west bank. Seven bridges now span the Willamette at Portland; the St. Johns bridge will be the most northerly crossing and nearest the point where the river flows into the Columbia River. The new bridge will afford a connection between the Upper Columbia River Highway east of Portland and the Lower Columbia River Highway from Portland to Astoria.

Alternative designs and estimates for cantilever and suspension designs, respectively, at four different locations were prepared. The comparative estimates revealed a saving of $640,000 by using a suspension design at the adopted location, a saving which would permit the capacity of the structure to be increased from three to four lanes. In addition a large balance of the authorized appropriation would be left unexpended. This suspension design was finally adopted and is now under construction.

In the St. Johns bridge, the desire to secure a beautiful public structure was a governing consideration. The selection of the suspension type, with its naturally graceful cable curves, was the first step in that direction. The development of the design of the high steel towers, to secure outlines in steel that would express the harmonious combination of beauty and strength, led to the adoption of pointed arches as the dominant motif, and this style of architecture also was carried out in the design of the tall reinforced-concrete piers of the viaduct approach.

Special designs for the ornamental balustrades, the



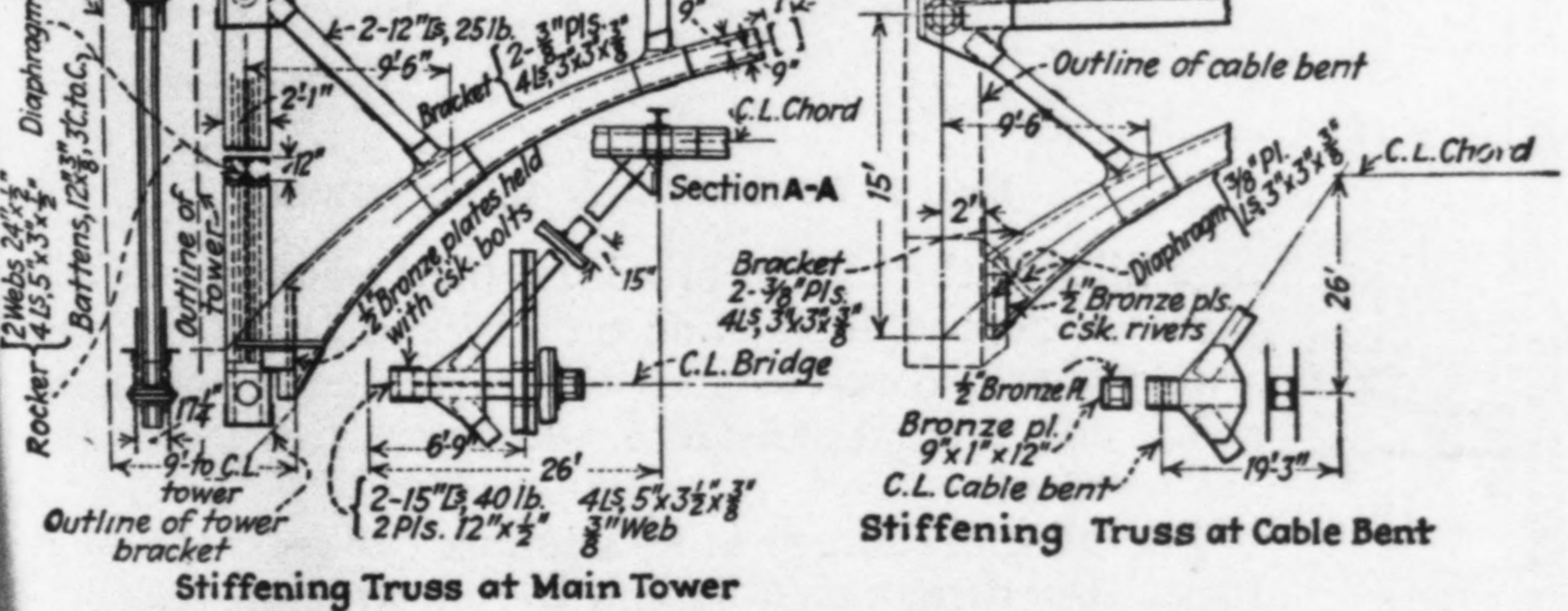




FIG. 2—PRINCIPAL DETAILS OF ST. JOHNS BRIDGE
Including stiffening truss connections at main and cable bent towers.

lighting standards and other details were developed to harmonize with the main towers. Details of the abutment pylons and parapets, the viaduct piers, the massive main piers and the anchorages were modeled to the graceful curves and contrasting reliefs of the governing style. A special attempt was made to develop each material in a manner appropriate to its accepted characteristics. It is sincerely hoped by the designers that the St. Johns bridge, by virtue of the substitution of buttressed columns and arched outlines for the conventional utilitarian X-bracing, of the gracefulness and rhythm of its approach spans supported on an arcade of concrete piers of ascending height, and of the harmonious development of the architectural features in an attempt to present a pleasing and impressive view of the structure from all possible angles, will establish a new mark in artistic bridge design.

*Principal Dimensions*—The main span of the St. Johns bridge clears the harbor lines with a length of 1,207 ft. center to center of main piers and is flanked by two suspended side spans each 430 ft. 3 in. long. The cable sag is 120.7 ft., one-tenth of the span. In order to meet navigation requirements established by the port of Portland, the underclearance is 205 ft. at the center of the main span.

The east viaduct consists of two 180-ft. spans, four 144-ft. spans and three 108-ft. spans. In the west viaduct there is one 180-ft. span and two 36-ft. spans, making the total length of the bridge, from abutment to abutment, 3,606½ ft. A U-type east abutment, carrying concrete roadway on fill, adds 227 ft. more to the length of the construction. The west abutment marks the junction with the west approach, which involves about 1 mile of side-hill road construction. Accordingly the work embraces about 4,000 ft. of bridge construction and 5,000 ft. of approach road construction.

*Foundations*—Preliminary wash borings, made at the four different locations contemplated for the proposed bridge, formed the basis for preliminary designs and comparative estimates. After the site was selected, final confirmatory core borings were made at the locations of the piers and anchorages. On the east bank the borings revealed only layers of clay and sand, which necessitated a substructure design with spread footings and with pile foundations under a number of piers. On the west bank the borings showed rock at El. —11 to —23 for the west main pier, and rock under 4 to 8 ft. of clay for the viaduct piers.

All piers are of reinforced concrete. The east main pier has its foundation at El. —50, on wood piles driven under water into hard sand. It contains 17,000 cu.yd. of reinforced concrete and in order to reduce its weight, 72-in. wood stave pipes are placed vertically in the body of the pier so as to leave hollow cylindrical shafts. The west main pier is founded on rock at El. —25.

The shafts of the viaduct piers are of tapering cruciform section, connected at the top with pointed arches. These piers ascend in height reaching El. +160 at pier No. 10. It is believed that this pier, 163 ft. from top to bottom and containing 3,700 cu.yd. of concrete and 150 tons of reinforcing steel, is the largest reinforced-concrete rigid-frame pier yet constructed. The piers are reinforced with structural steel frames supplemented with rod reinforcement.

The east anchorage is 115 ft. long, 91 ft. wide and extends from El. +12 to +82. It contains 13,000 cu.yd. of concrete. In addition there are hollow chambers filled with sand, to provide a total weight of 30,000 tons. A stepped foundation on a spread footing is used with pedestal concrete piles to provide additional horizontal and vertical resistance. About half of these piles are battered.

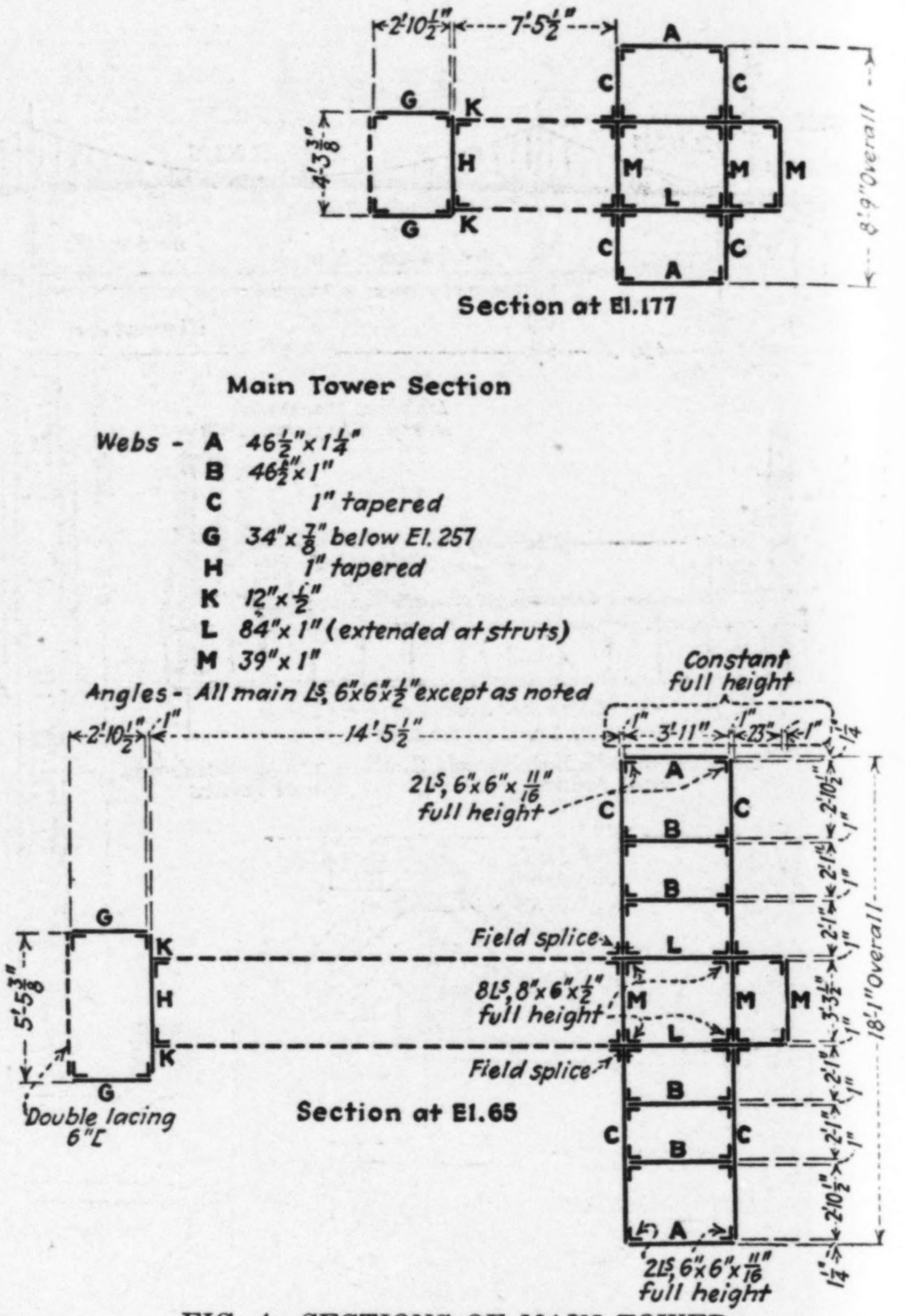



FIG. 4—SECTIONS OF MAIN TOWER

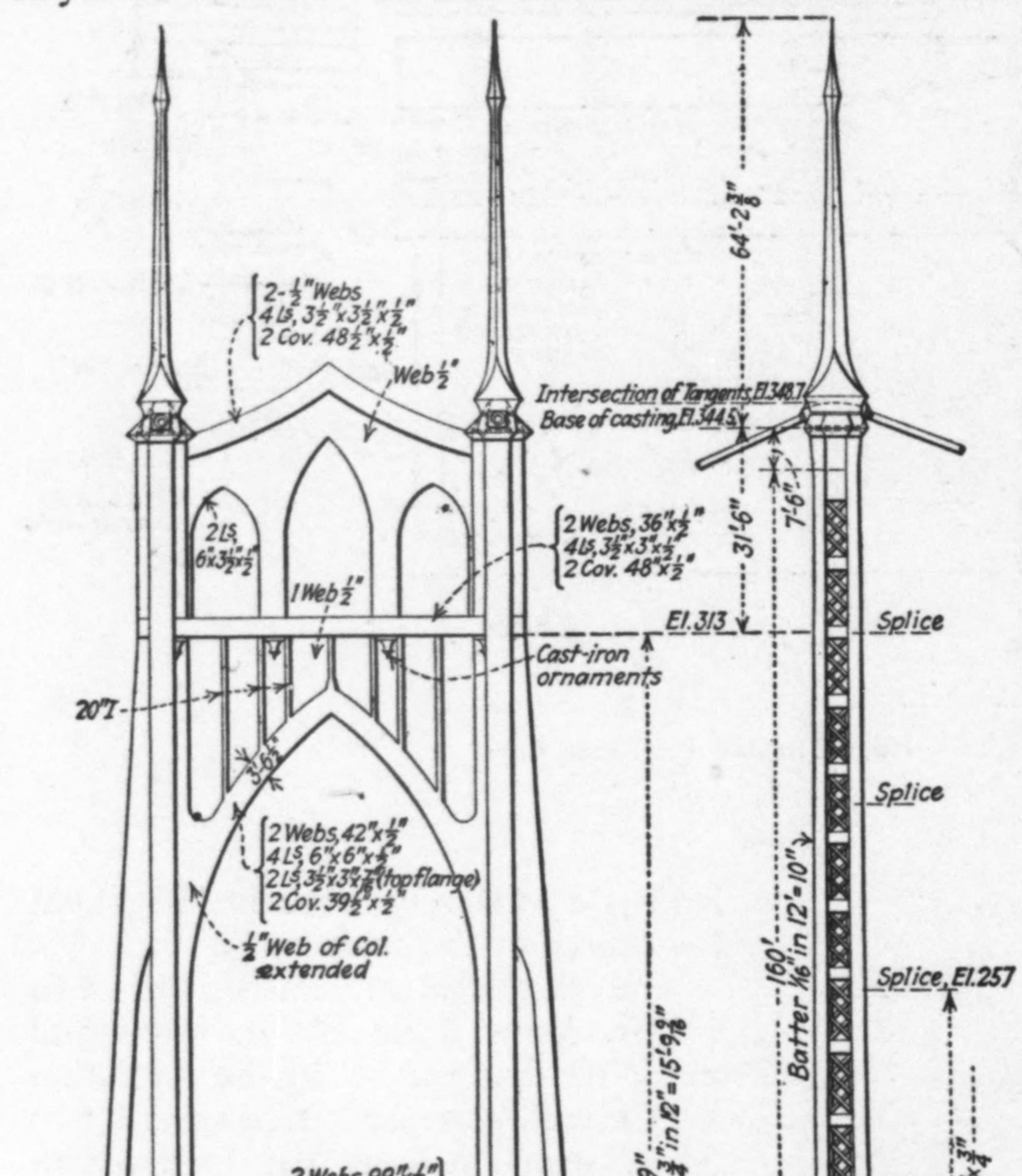



FIG. 3—BATTER-LEG MAIN TOWERS ON ST. JOHNS BRIDGE

Vertical leg for direct loads and batter-legs in lieu of diagonal bracing, combined with pointed arch portal outlines, are distinguishing factors in architectural design of the bridge

The west anchorage is in a tunnel cut from the solid rock. It is surmounted by an exterior structure supporting viaduct piers Nos. 14 and 15.

The anchorage steel is shown in Fig. 7. The anchor chains, connecting to the anchor girders, are formed of plates of silicon steel, with reinforcing pin-plates, as a substitute for the usual eyebar construction.

*Main Towers and Cable Bents*—The main towers comprising one of the unusual features of the structure, are of medium-carbon steel, of the semi-flexible type, fixed at the base. Each tower consists of two main vertical columns, 52 ft. center to center, supplemented by outside batter-legs. The combination of vertical legs for direct load with batter-legs for bracing produces a highly satisfying solution of the esthetic problem of tower design for suspension bridges. The effectiveness of the pointed arches in the towers of the Mount Hope bridge, near Providence, R. I., prompted the adoption and further development of this motif in the design of the St. Johns bridge, where similar arches are utilized both above and below the roadway as shown in Figs. 1 and 3.

The tower legs are of cruciform section, with box divisions. The main column section tapers from a width of 18 ft. at the base to 7 ft. 4 in. under the saddles, with a constant breadth of 4 ft. 1 in. The total height of the towers, including the finials, is 408.7 ft. above mean low water.

Following the mathematical analysis of the deflections

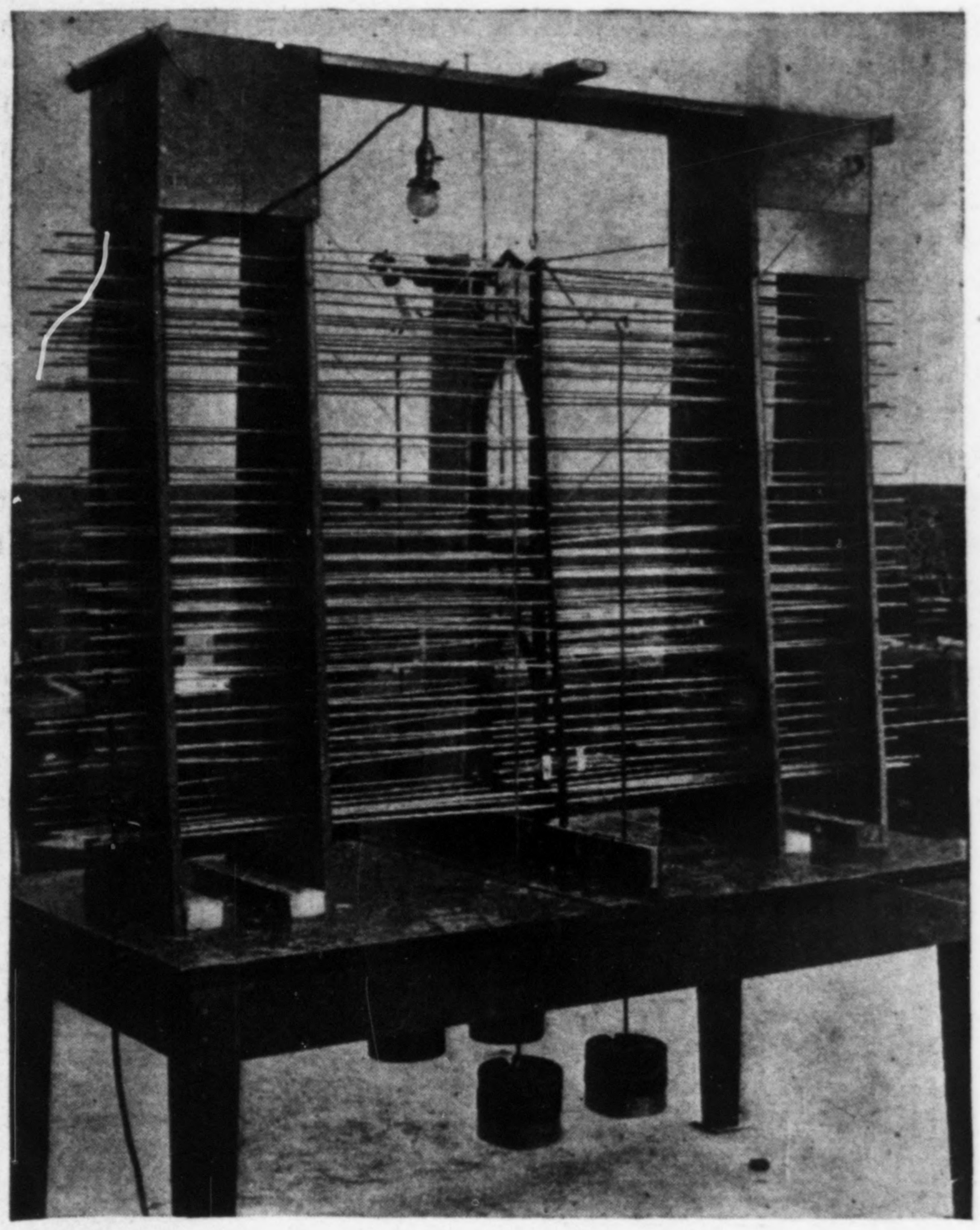


FIG. 5—MODEL ANALYSIS SET-UP FOR ST. JOHNS BRIDGE TOWERS
Note application of cable loads to top of celluloid tower model by weights hanging beneath table. Transverse loads applied simultaneously with vertical loads gave deflections that indicated stresses below computed values.

and stresses in the main towers, check tests were made on an elastic model (Fig. 5) built up of sheets of celluloid to represent the variations of cross-section and moments of inertia. Scale forces were applied to represent the simultaneous action of maximum vertical and transverse loads on the tower, precision strain gages being used to read the stresses. This work was done by the engineering staff for the bridge under the supervision of Prof. George E. Beggs, in the engineering laboratory at Princeton University.

The model test results were distinctly reassuring, being in all cases lower than the computed stresses and strains. The indicated maximum transverse deflection of the tower top, referred to the scale of the actual structure, was $2\frac{3}{8}$ in. from 30-lb. wind load combined with the effect of the resultant eccentricity of the maximum vertical reactions from cable and truss loads. The maximum indicated fiber stress, after adding the computed stress from unbalanced cable pull, was only 14,000 lb. per sq.in. The model tests showed that the tower was generously proportioned for any combination of loadings to which it might be subjected.

The cable bents are of the rocker type, but are provided with wing plates at the base to hold them during erection, these plates to be burned off after the cables are in place. The connection between the cable bent columns and the side span stiffening trusses is made by means of 6-in. pins in lubricated bronze bushings. A transverse truss in the cable bent supports the roller bearings for the 180 ft. approach span and the center-line sliding bearing for the lateral truss of the suspended side span.

*Twisted Strand Cables*—Following the successful application in 1928 of twisted strands for the cables of the 950-ft. span Grand'Mere bridge in Quebec (*Engineering News-Record,* Nov. 18, 1929, p. 841), the specifications for the St. Johns bridge invited bids on twisted-strand construction as an alternative to the parallel wire cable design. The two alternative bids of the John A. Roebling's Sons Company were lower than any other bids received for the cables and showed a saving of $42,000 by adopting the twisted-strand design. Moreover, the proposal announced an expected saving of two months in the time of completion of the bridge, this saving in time being made possible by dispensing with the con-

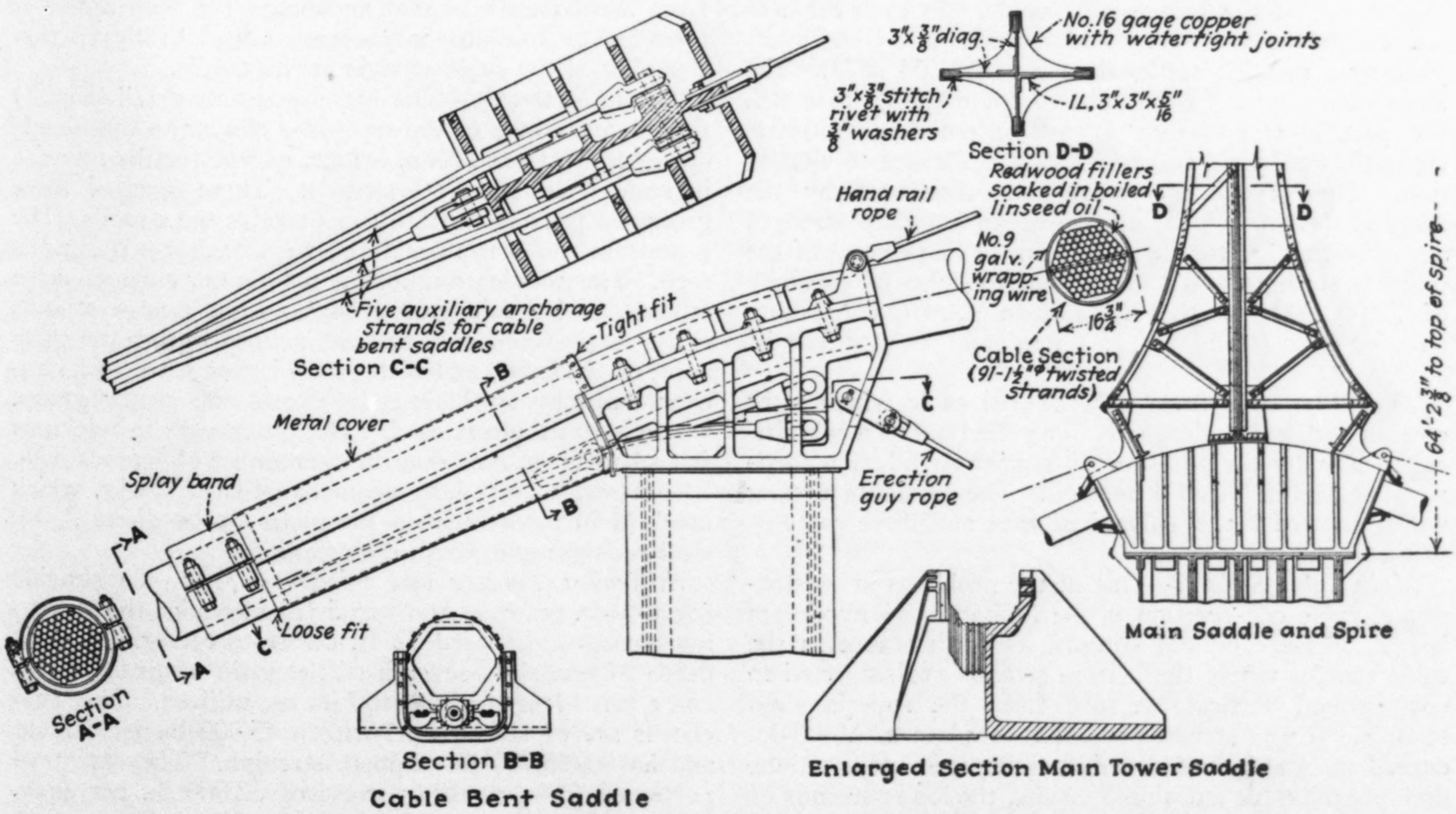



FIG. 6—SADDLE DETAILS FOR ROPE STRAND CABLE SUSPENSION BRIDGE
Note use of auxiliary ropes to anchor cable bent saddle rather than depending upon friction clamping of cable in saddle.

struction of footbridges and by shortening the time required for cable stringing.

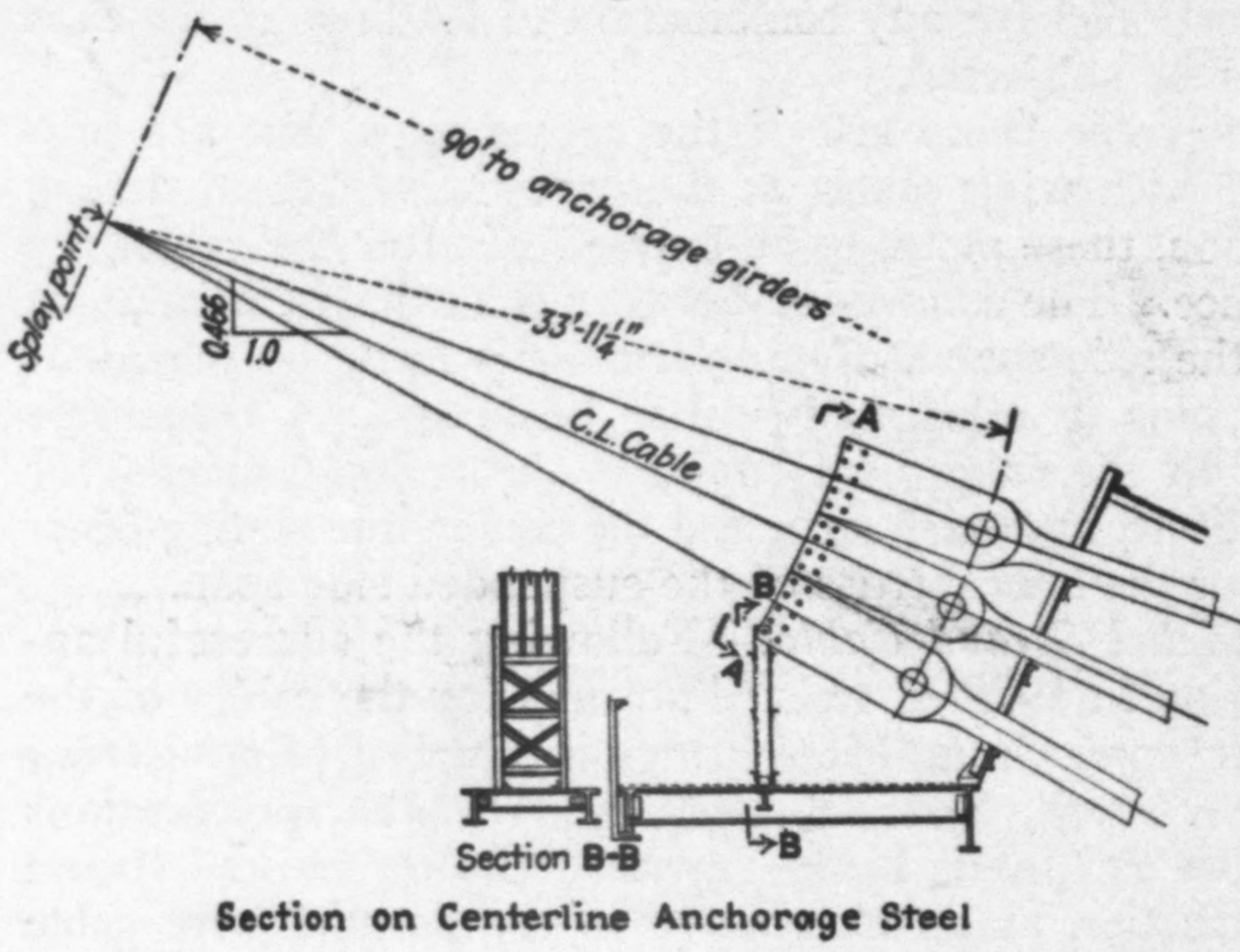
Under the specifications the successful bidder on the twisted-strand construction absorbs the cost of the slight strengthening of towers and anchorages to carry the increased cable weight and also assumes any extra cost of modification of details of saddles and anchorage connections. In the St. Johns bridge the required strengthening of the main sections of the towers and anchor chains amounted to less than 2 per cent.

The parallel wire design provided for each cable to consist of nineteen strands of 184 No. 6 galvanized wires each, yielding a sectional area of 105.47 sq.in. The specified physical properties were 170,000 lb. yield point, 225,000 lb. ultimate strength and 27,000,000 lb. modulus of elasticity. The cable diameter was specified as 12¾ in., but the use of twisted-strand construction increased this to 16¾ in. before wrapping. An incidental advantage accruing from the increase in cable size was an improved appearance of the bridge as a whole. In each cable as adopted, there are 91 twisted strands of 1½-in. diam., yielding a metallic sectional area of 121.64 sq.in. The specification requires the twisted-strand design to match the parallel-wire design in guaranteed total ultimate strength, total yield point and total resistance to elongation. These requirements are more than met by the adopted design, which guarantees an ultimate strength per strand of 135 tons, a yield point of 75 per cent of the ultimate strength and a guaranteed modulus of elasticity of 22,000,000. Actual tests have shown a breaking strength per strand of 150 tons and a modulus of 25,000,000.

The 91 strands form a hexagonal cable which after coating with red lead paste is to be filled out to a cylindrical section by the addition of segmental fillers of redwood soaked in boiled linseed oil. The outside wrapping will consist of No. 9 galvanized wire and three coats of paint.

*Main Cable Saddles*—One of the problems in twisted-strand cable construction is the limitation of maximum lateral pressure on the strands, as for instance in the cable saddles where the bottom strands are subjected to concentrated vertical pressure from the superimposed strands. It was at first proposed to place a 30x4½-in. curved steel slab horizontally across each main saddle, dividing the cable and thus relieving the lower strands of the pressure from the strands above. This construction was not ideal, however, since the resulting vertical spread of the cables at the saddles was undesirable because of adjustment and stress considerations.

To aid in a solution of the problem, tests were made at the Trenton plant of the John A. Roebling's Sons Company to determine the effect of transverse pressure on the physical properties of the strands under tension. Previously published tests have shown that cable wires are unaffected in strength and yield point by pressures of 3,300 lb. per linear inch of wire. The tests performed for the St. Johns bridge showed that even higher pressures did not affect the physical properties of twisted strands, although they did produce visible deformation of section of the wires in the outer layers. It was decided to limit the maximum pressures to 3,300 lb. per linear inch. This was accomplished by rearranging the strands in the saddle, so as to have eight strands in the bottom layer instead of six and so as to reduce the number of layers from eleven to ten. In addition, the radius of the saddle curve was increased slightly. As a further precaution, the strands of adjacent horizontal layers were designed to have opposite lay, so as to obtain "valley" contact rather than "crossing" contact between the contiguous wires.



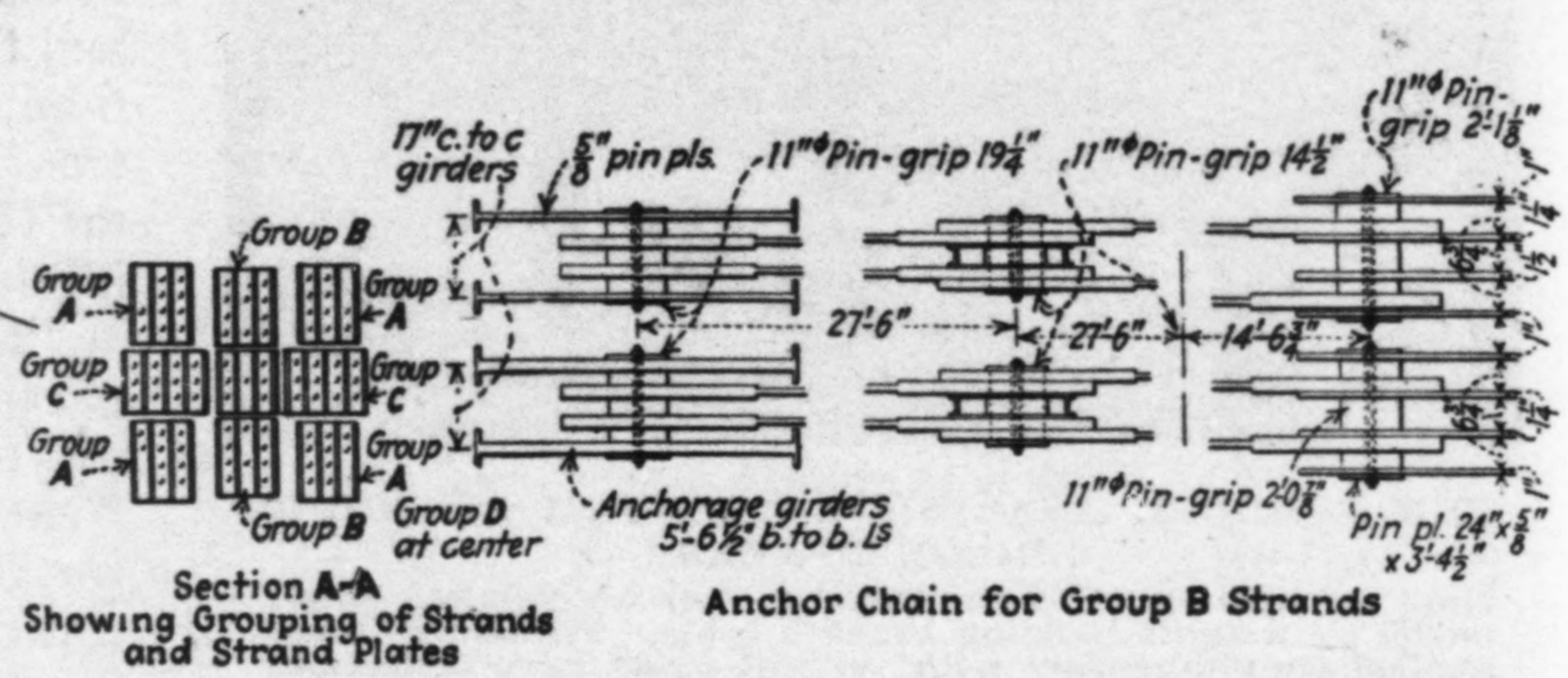




FIG. 7—ANCHORAGE STEEL FOR ROPE-STRAND CABLES
Silicon steel plate chains are used in place of conventional eyebars.

As shown in Fig. 5, the cable bent saddle is a departure from usual design in that anchorage for each saddle is provided by five auxiliary ropes instead of by friction clamping of the cable strands in the saddle.

*Strand Assembly*—The strand assembly detail (Fig. 7) for the anchorage of the strands is similar to that developed for the Grand'Mere bridge, except for the increase in scale from 37 to 91 strands. Three tiers of three groups of parallel vertical plates receive the strands. The groups in the bottom and top tiers anchor ten strands each. The middle group of the middle tier anchors seven strands, and the outside groups anchor twelve strands each. The groups of plates are inclined to face the splay point of the cable, and each group is connected to its anchor chain by an 11-in. pin, except the center group, where a 10-in. pin is used. The plates vary in thickness from 1 to 1½ in. according to the number of strands held. The strand sockets bear against cast-steel blocks, which are held in place between the plates by two long 2¼-in. steel rods going through each group.

*Stiffening Trusses and Viaduct Spans*—To provide for a 40-ft. roadway and two 5-ft. sidewalks, the stiffening trusses are spaced 52 ft. center to center. A truss depth of one sixty-seventh of the main span, or 18 ft., and a panel length of 19 ft. 3 in. are utilized. The truss chords are of silicon steel with a 45,000 lb. yield point and an 80/95,000 lb. ultimate strength. They are proportioned for a working stress of 32,000 lb. per sq.in. The maximum working stresses in the medium-carbon steel members are 26,000 lb. for the stiffening truss diagonals and 18,000 lb. for the hangers, floor system and bracing.

Only two different sections, two 15-in. channels of 35 and 40 lb. respectively, are used for the upper chord, the heavier section being required only from the one-sixth to the five-twelfths point of the main span. The lower chord sections also are two 15-in. channels, of 35, 40, 45, and 50 lb. weights respectively.

The truss splices are laid out for fabrication and erection of the trusses in two-panel units, 38½ ft. long. Expansion movement for the main and side spans is provided by supporting the spans on rocker posts at the main towers, as shown in Fig. 2, and these rocker posts are covered by the end brackets extending under two truss panels each side of each tower. The connections at the top and bottom of the rocker posts consist of 6-in. pins in bronze bushings, provided with grease-cup lubrication.

The layout of the approach viaducts was planned for maximum duplication of span and panel lengths, together with special consideration to esthetic requirements. It provides duplication of 180-ft., 144-ft. and 108-ft. spans, with truss depths of 18 ft. for the two former span lengths and 9 ft. for the 108-ft. spans. All panel lengths are 9 ft. These viaduct trusses are deck type, 30 ft. center, the roadway and sidewalks being cantilevered out to obtain the total required width of 50 ft.

*Contract Costs and Quantities*—The work is divided into seven contracts, which were awarded to the respective lowest bidders as follows:

| | |
|---|---|
| (1) Substructure: Gilpin Construction Company, Portland | $1,026,897.00 |
| (2) Fabrication and Erection of Bridge Steel: Wallace Bridge & Structural Steel Co., Seattle | 986,445.80 |
| (3) Furnishing and Erection of Cables: John A. Roebling's Sons Company, Trenton, N. J. | 472,200.00 |
| (4) West Approach: La Pointe Construction Company, Portland | 267,603.40 |
| (5) Fabrication and Erection of Viaduct Steel: U. S. Steel Products Company, San Francisco | 290,000.00 |
| (6) Concrete Deck: Lindstrom & Feigenson, Portland | 146,060.00 |
| (7) Electrical Work: National Electric Company, Portland | 33,000.00 |

The total contract cost was $3,222,206.20, which was $572,000 below the estimate of the engineers. The authorized bond issue for the bridge was $4,250,000, so that allowing for cost of right-of-way, engineering and contingencies, it is estimated that more than half a million dollars of the appropriation will remain unexpended.

*Personnel*—The St. Johns bridge was designed by Robinson & Steinman, consulting engineers, New York City, and is being constructed under their direction, with R. Boblow as resident engineer. The work is being done under the authority of the board of county commissioners of Multnomah County, consisting of Clay S. Morse, chairman, Grant Phegley and Fred W. German. The highway department of the county is represented by George W. Buck, county roadmaster and M. E. Reed, county bridge engineer.

# Reinforced-Concrete Columns With Cast-Iron Cores

### Reduced Size for Given Strength Is Advantage in Large Buildings—Solid and Hollow Cores—Use and Tests

VARIOUS inquiries received as to the nature and service of the Emperger type of reinforced-concrete column, used in a number of buildings, including the McGraw-Hill Building, Chicago (*Engineering News-Record,* July 25, 1929, p. 129), are answered by the following compilation of authoritative data.

In this type of reinforced-concrete column, as generally used, the center is a solid cast-iron bar or core, while near the exterior is a steel spiral attached to vertical bars. The spiral represents about 1 to 2 per cent of the cross-sectional area of column, the vertical bars about 1 to 4 per cent and the cast-iron core from 5 to 30 per cent in different designs. The main advantage claimed over ordinary reinforced-concrete columns is a material reduction in size, due to the substitution of one cast-iron member for several steel members. In tall buildings this reduction in size amounts to 8 or 10 in. in diameter of column, with the same or even less cost per linear foot, besides enabling more stories to be built than with ordinary reinforced-concrete without reaching excessive column sizes. The advantage in reducing the size of column is not so much the saving in floor space but rather: (1) the saving of space in congested service areas, as between elevators and around stairways, and (2) architectural value or appearance of the smaller column. A method for the design of columns with cast-iron cores was given in *Engineering News-Record,* March 4, 1920, p. 459.

It has been suggested that a structural-steel core, as sometimes used, would give greater strength. But tests to destruction are reported to have shown the contrary, and in tests made by the Austrian Concrete Committee, the cast-iron cores embedded in hooped concrete carried twice as much as structural steel in similar concrete. Further, the cast-iron core has a lower price and can be obtained usually in less time than structural steel of heavy section. The castings are made purposely with a rough surface to effect good bond with the concrete. In Chicago, however, the ruling or special ordinance under which the Emperger columns are built does not give the concrete any credit when structural steel cores are used.

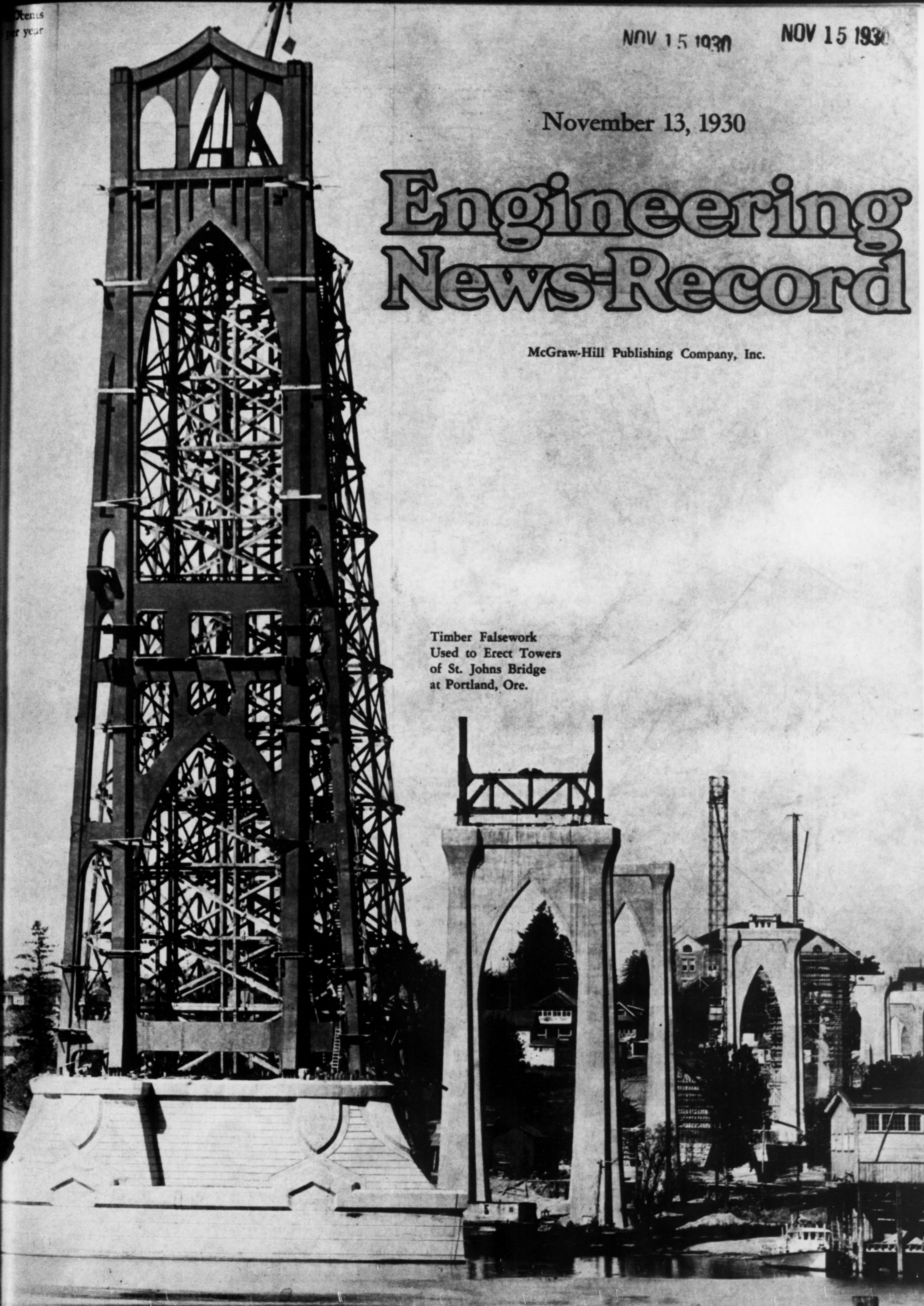
Some inquiries assumed the cast-iron-core columns to be cast-iron columns fireproofed with concrete, and asked how the reinforced-concrete beams are connected to the cast-iron. In their design the columns are treated as reinforced-concrete columns, to which the concrete beams are connected in the usual way, as was shown in the description of the McGraw-Hill Building. There is no attachment of the horizontal reinforcement to the cast-iron core.

In designing the McGraw-Hill Building, the columns were made of uniform section (24 and 26 in. square) from basement to roof for economy, convenience and rapidity in construction. Since the cost of formwork in tall buildings amounts to about 40 per cent of the entire cost of the structural frame or skeleton, a decided saving in forms was accomplished. The variation in carrying capacity was obtained by reducing the cast-iron cores gradually from 12¾ to 3 in. in diameter. For the upper floors, where cores were not needed, standard hooped concrete construction was used. It was estimated by the designer that with the ordinary type of steel reinforcement the McGraw-Hill Building column section required would have been 34 and 36 in. square. In the 30-story Trustees Building, now under construction in Chicago,

November 13, 1930

# Engineering News-Record

McGraw-Hill Publishing Company, Inc.



Timber Falsework
Used to Erect Towers
of St. Johns Bridge
at Portland, Ore.

# Timber Falsework Used to Erect Suspension Bridge Towers

THE STEEL towers for the St. Johns suspension bridge over the Willamette River at Portland, Ore., have been erected by the use of derricks resting on tall timber falsework. This is a novel if not unprecedented method for modern suspension-bridge construction, and this article, based upon information supplied by R. Bob-low, resident engineer for Robinson & Steinman, engineers on the bridge, describes the falsework and the method of erecting the towers. An article describing the bridge, which is unique in that it uses rope-strand cables, was published in *Engineering News-Record,* Feb. 13, 1930, p. 272.

The timber falsework extends to El. +300, referred to the zero datum of mean low water. Each falsework tower rests on 56 piles approximately 100 ft. long and contains 252,000 ft. b.m. of Douglas fir timber, exclusive of piling. The main posts are 12x14-in., the girts 8x10-in. and the diagonal braces 4x12-in. timbers. Connections are bolted and spiked, and steel straps are used for splicing the vertical posts. The falsework is 100x98 ft. in plan at the base and 40x49 ft. at the top.

The falsework was erected by means of a guy derrick acting in the form of a creeper traveler inside the frame. The wood bents were assembled on the ground in 40-ft. sections and lifted into position by the guy derrick.

Field erection has been carried on by means of a stiff-leg derrick on top of the falsework. The boom of this derrick is unusual, being a 28-in. diameter stick of fir 80 ft. long. The mast is directly supported on the double bent which bears on the shelf of the main pier at El. 30. The upward kick of the back legs of the derrick is taken by two sand boxes, each weighing 36 tons and hung from 2½-in. diameter rods.

The bottom section of each of the tower legs was placed upon 28 steel wedges, which in turn rested upon steel plates embedded in the concrete. Final plumbing of the tower was not done until the steel had been erected up to the peak of the first arch, at approximately El. 150. Steel erection for the east tower was begun on May 23 and completed on June 18. Riveting was begun on June 9 and completed Aug. 8. There are approximately 40,000 field rivets in each tower. The heaviest lifting piece is the bottom section, which weighs 34 tons. The greatest amount of steel erected in one day was 131 tons for the west tower. For this tower, steel erection was begun on June 20 and completed on July 8. The erection crews worked two shifts totaling fifteen hours, five days per week. It is stated that the erection, with the use of the timber falsework towers, was particularly easy and free because of the great amount of drift available at practically all elevations. The timber falsework was erected by J. H. Pomeroy & Co., of Portland, subcontractors for erection of all main-span steelwork.

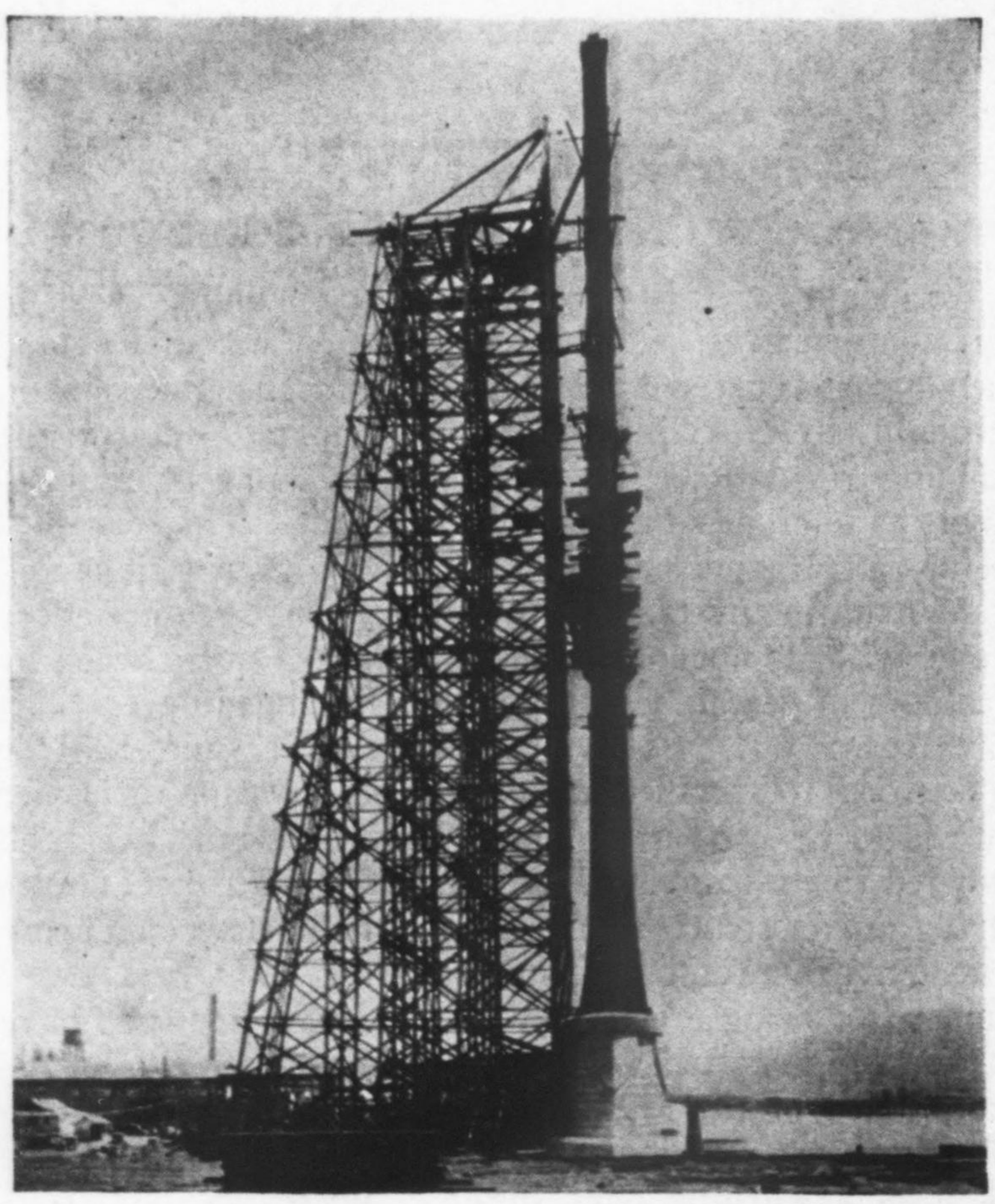


FIG. 1—TIMBER FALSEWORK USED TO ERECT STEEL TOWERS OF ST. JOHNS SUSPENSION BRIDGE

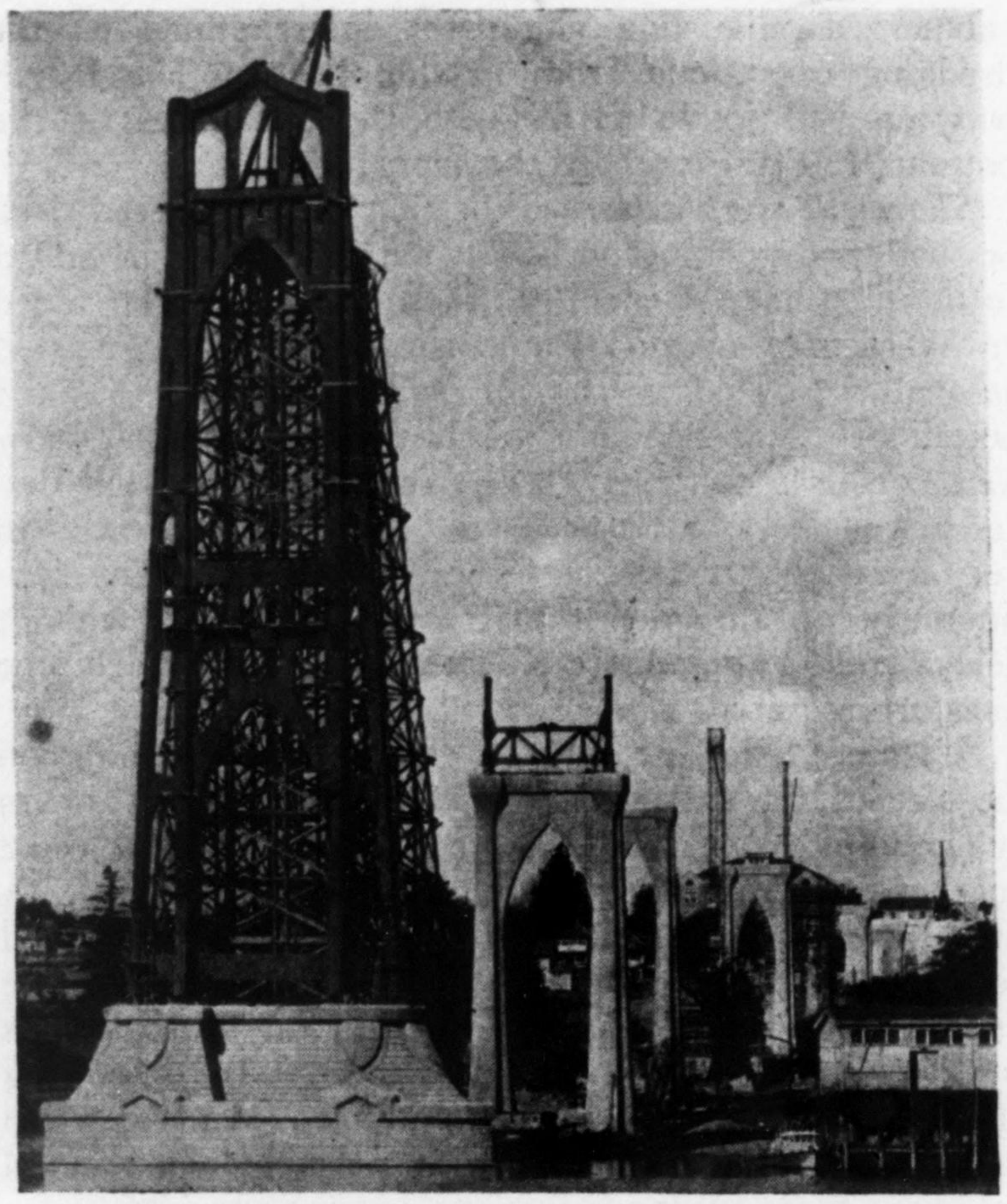
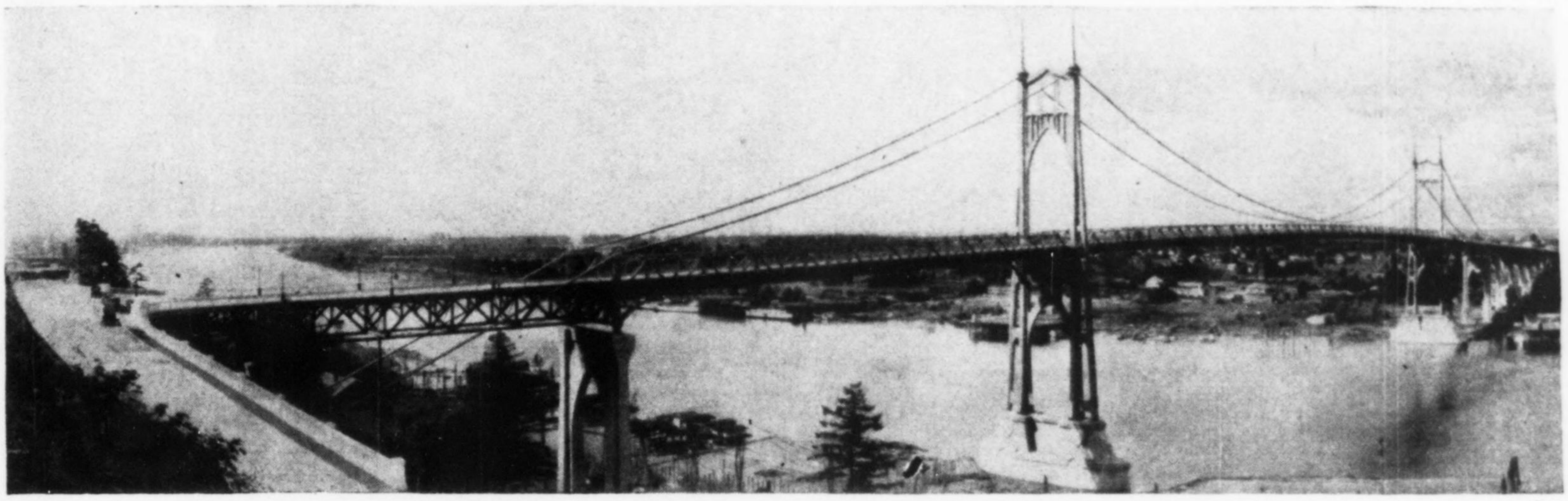


FIG. 2—COMPLETED EAST TOWER WITH CABLE BENT AND APPROACH PIERS IN BACKGROUND

## Use of Steel Railway Ties in Argentina

Although there are some steel crossties used by a few of the railway systems in Argentina, the large majority of ties are of wood. Wooden ties are required by law to be made from native woods, and they are usually cut from red quebracho, white quebracho or alagarrobo. Several railway lines, both state and private, have used steel crossties, but as these become too old for further use they are replaced by wooden ties. Until recently about 5 per cent of the trackage of the Central Argentine Ry. was supported on Livesey iron pot crossties, but these have now been practically all replaced. Some steel trough ties are still in use on the Entre Rios, Argentine North Eastern, Central Northern Argentine and the Central del Chubut railways, but the tendency seems to be to replace such ties by wooden ones whenever the opportunity presents itself.

# Stringing Rope-Strand Cables Features St. Johns Bridge Construction



•Fig. 1—St. Johns bridge, Portland, Ore., establishes a record span of 1,207 ft. for rope-strand cable type

**Cables for 1,207-ft. span at Portland, Ore., erected in 23 days without footwalks—Advantages of stranded design discussed—Erection system of endless hauling ropes described—Foundation and approach construction outlined**

By R. BOBLOW

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DISTINGUISHED by the largest rope-strand cables ever erected and thoroughness of architectural treatment, the St. Johns bridge at Portland, Ore., has the further distinction of being the first modern suspension bridge of major size ever built on the Pacific Coast. The structure was built by Multnomah County and spans the Willamette River about 7 miles northwest of the business district, connecting the section of the city known as St. Johns with Linnton. In accordance with federal requirements, a clearance of 205 ft. at the center of the main span is provided, and a 203.6-ft. clearance for a channel width of 440 ft. The project cost $4,000,000 and was opened to traffic June 13, 1931.

The structure is a typical stiffened suspension bridge with suspended side spans, having two-hinged stiffening trusses, fixed main towers, and cable bents on rocker castings for the purpose of deflecting the cables to permit favorable locations and economical design of the anchorages. The main span is 1,207 ft. long and is flanked by two 430-ft. side spans. Spaced 52 ft. apart, the two cables are $16\frac{3}{4}$ in. in diameter and support stiffening trusses of conventional Warren type 18 ft. deep, or the equivalent of 1/67 the length of the main span. The roadway is 40 ft. wide between curbs, providing for four lanes of traffic; in addition there are two 5-ft. sidewalks. The roadway deck and sidewalks are of reinforced-concrete slab construction. Design features of the structure were described in detail in *Engineering News-Record*, Feb. 13, 1930, p. 272.

The St. Johns bridge exceeds the Grand 'Mere bridge, where rope-strand cables for a long span were first introduced, both in span and size of cables. Further, erection of the latter structure occurred during the winter season in Quebec, and the frozen surface of the river was utilized for stringing the cables (*Engineering News-Record*, Nov. 18, 1929, p. 841). Thus the St. Johns bridge provided the first real opportunity in working out problems of erection procedure for rope-strand cables under conditions that may be considered as normal.

*Approaches*—The approaches to the bridge, 1,511 ft. long on the east side and 255 ft. on the west, are made up of a series of steel deck-truss spans having lengths of 108, 144, and 180 ft. A concrete U abutment, 227 ft. in length, is used at the east end of the bridge to provide the necessary flaring out of the roadway to connections with existing streets. The west approach leads into the side of a hill at a point about 120 ft. above a main highway, which necessitated the construction of a two-branch road cut in the hillside and dropping on a 5 per cent grade both ways from the bridge terminal to junctions with the road.

Erection of the approach trusses was by means of a jinniwink deck traveler equipped with a 50-ft. boom of $10\frac{1}{2}$-ton capacity and operating on the roadway stringers. For the 108-ft. spans a single bent of falsework was used, midway between the piers, the trusses being assembled on the ground and picked up in half-span units. For the longer spans a tower bent of falsework was used (Fig. 2). The falsework was shifted as erection proceeded by means of a trolley on a 1-in.-diameter cableway spanning the length of the approach. The viaduct steel contract involved the fabrication and erection of 1,943 tons of structural steel.

The viaduct spans are supported on tall slender viaduct piers forming a very impressive progression of increasing heights, their tops varying from 22 to 150 ft. above the ground. The two tapering shafts of each pier rise from a heavily reinforced concrete base and

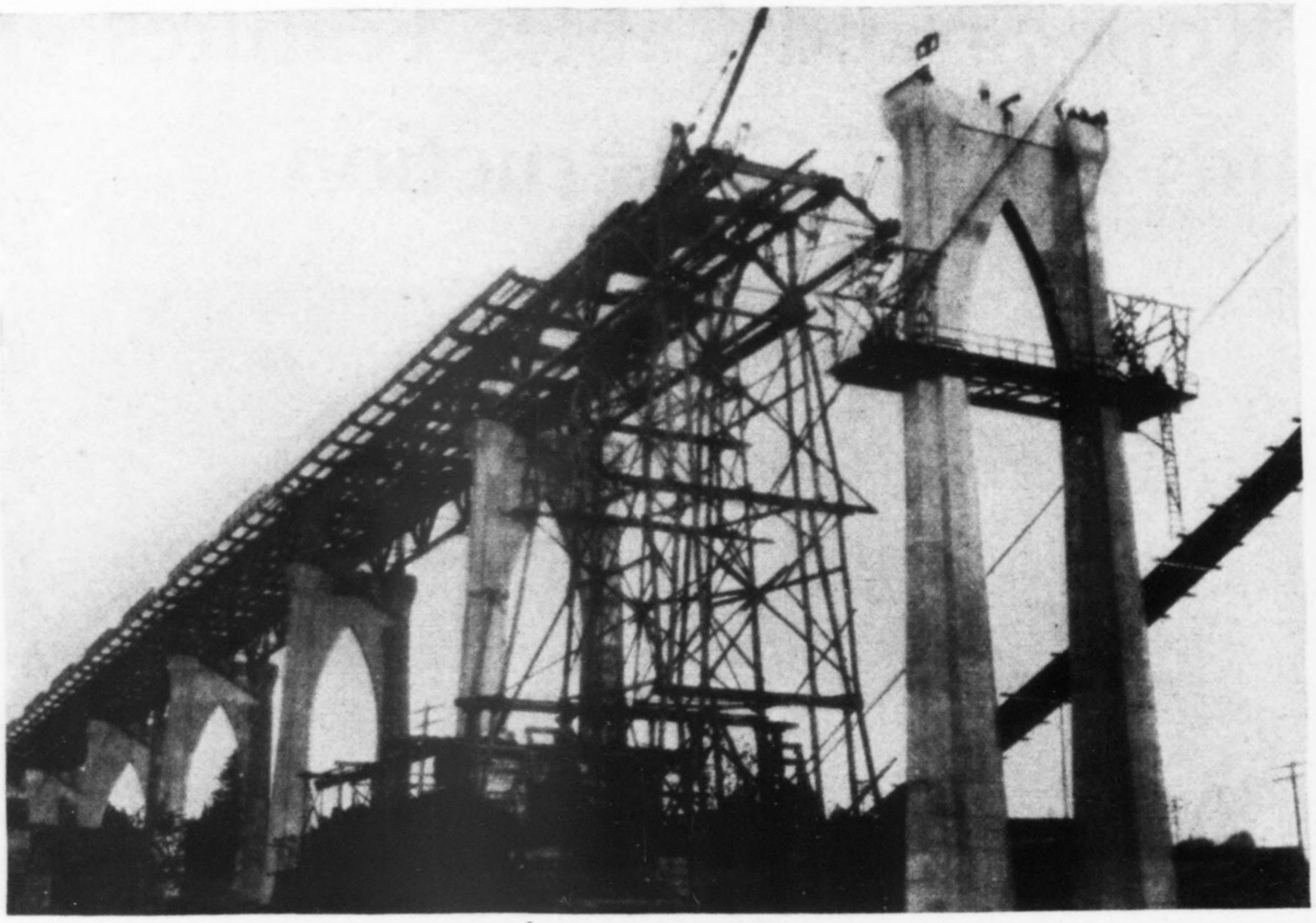

**Fig. 2—Viaduct approach on pointed arch piers**
Timber tower-erection bent was shifted by cableway.

On the west side the anchorage steel for each cable is embedded in concrete, filling a tunnel excavated for a distance of 80 ft. into dense basaltic rock and tapering from a 14 x 14-ft. section at the portal to a 24 x 27-ft. section at the rear. Incidentally, about 40,000 cu.yd. of concrete used in the viaduct piers, anchorages and roadway deck was manufactured and delivered by mixer trucks.

*Main Towers*—The two main steel towers of the bridge offer a variety of interesting features. Their design embodies a combination of vertical and battered main posts combining the structural advantages of both types. The conventional crisscross bracing is supplanted by lofty pointed arches, one above and the other below the roadway level, thus retaining the architectural impression created in the viaduct piers. The tower rises 290 ft. above the concrete pier, weighs 1,250 tons and is of the flexible type, being fixed at the base to the concrete pier by means of $2\frac{1}{2}$-in. anchor bolts. Each tower was erected by means of a wooden stiff-leg derrick resting on top of a 300-ft. timber tower erected on the shore side of each main pier. This feature of construc-

converge to form an impressive arch. These piers are reinforced by means of rigid structural-steel frames set 4 in. from the face of the concrete. They were poured (Fig. 3) by means of steel concreting towers in vertical lifts of 20 ft. Forms were fabricated in suitable panels and hoisted by a boom supported on the concreting tower. The rigidity of the structural-steel frame contributed greatly toward the perfection of shape and line attained in the construction of the viaduct piers, since they provided a practically inflexible support for the forms.

*Main Piers and Anchorages*—The two main river piers extend to an elevation of 60 ft. above mean low water. The east pier rests upon 1,068 wood piles driven into a hard formation of black sand by an air-driven hammer, set in 90-ft. telescopic leads to permit the hammer to follow the pile under water to refusal. The base of this pier is 50 ft. below low water. The west main pier is keyed into solid rock at El. —21. Both main piers were constructed by open-caisson method, using seals poured by tremie. An unusual design feature for the main piers is the use of 72-in. wood-stave pipes on 9-ft. centers throughout the piers for the purpose of saving concrete.

The topography of the site required two types of anchorages: the more common gravity type on the east side and the more economical tunnel type on the west side. Each anchorage is required to resist a cable pull of 8,500 tons. The east anchorage is an outstanding structure 115 x 91 ft. in plan and 50 ft. high, containing 12,500 cu.yd. of concrete. Heavily reinforced concrete walls within the anchorage divide it into chambers, some of which are filled with sand and others left open to permit inspection of the cable connections. Including the weight and loads of the viaduct pier, which rests upon the rear of the anchorage, the total weight available for resisting the pull of the cables amounts to 29,000 tons. This anchorage rests on 516 concrete pedestal piles; 192 were driven on a batter of 1 to 4, approximating the slope of the resultant force acting on the anchorage.

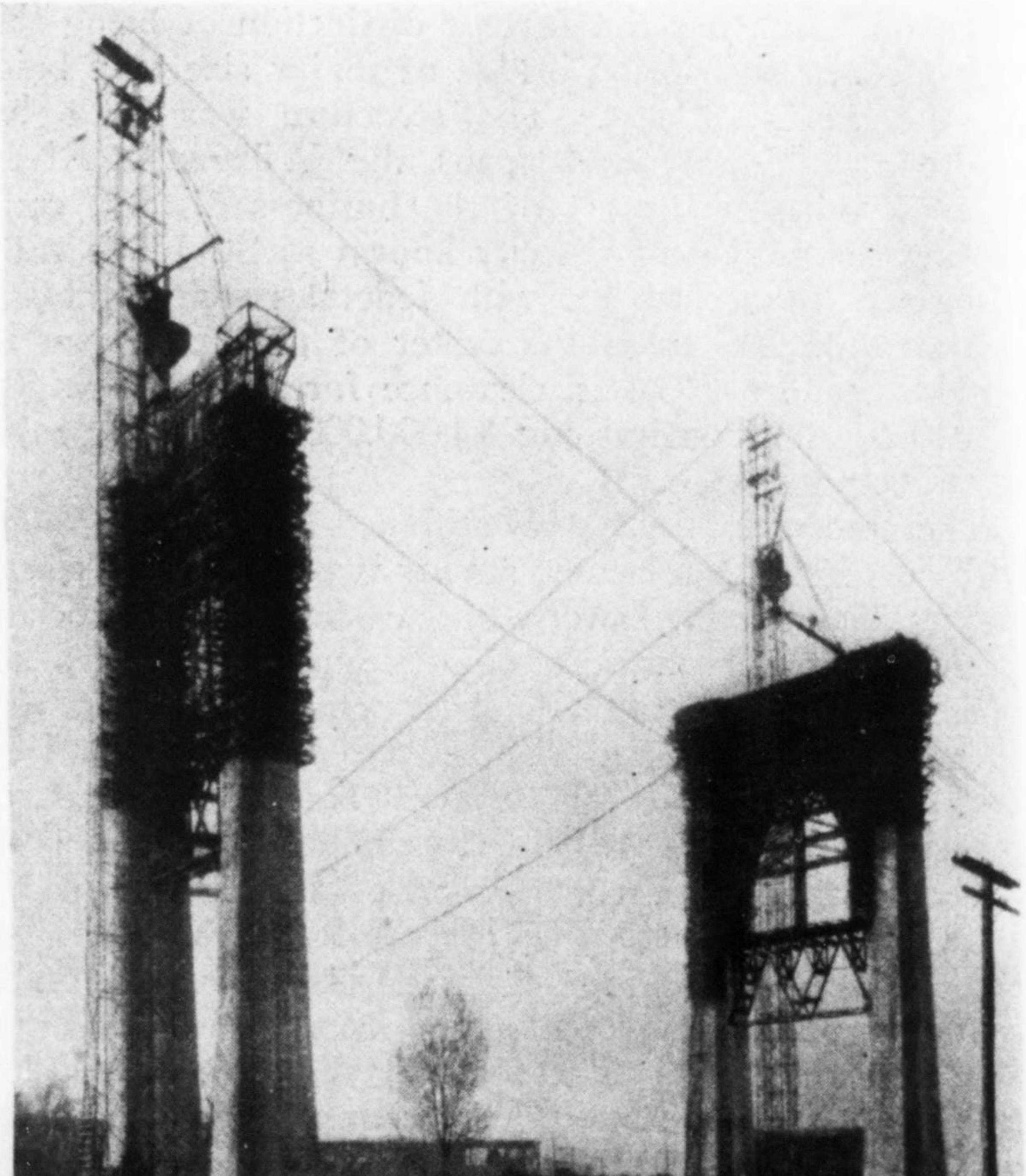

**Fig. 3—Concreting operations for viaduct piers**
Pier reinforcing is a structural-steel frame, which was essential in attaining correct shape and line by providing a rigid support for the forms.

tion procedure was described and illustrated in *Engineering News-Record*, Nov. 13, 1930, p. 770. Each tower is surmounted by two 50-ft. copper-sheathed steel spires, supporting aviation beacon lights equipped with flashers.

*Rope-Strand Cables*—The two main cables are 52 ft. apart and have a length of about 2,720 ft. between anchorage fastenings. Each cable consists of 91 $1\frac{1}{2}$-in. galvanized-wire bridge strands, which, in turn, are made up of 51 galvanized wires, varying in diameter from 0.100 to 0.196 in. and aggregating a gross metallic area of 1.337 sq.in. All wire for the main cables, suspender ropes and hand ropes was specified to be cold-drawn and manufactured by the acid open-hearth process. Each main cable strand was specified to have an ultimate strength of 135 tons, a yield point of 200,000 lb. measured at 0.7 per cent elongation, and a modulus of elasticity of not less than 24,000,000 lb. up to a load equal to 50 per cent of the specified ultimate strength.

It has always been recognized that a suspension bridge cable composed of wire ropes or strands cut to predetermined length would have advantages in economy over the parallel-wire type of cable. The main considerations that prevented the use of rope-strand cables for long-span suspension bridges carrying modern traffic were the lack of uniformity and dependability of the elastic properties for such ropes, as well as the much lower modulus value of the twisted strand compared to the individual wires composing it. A parallel-wire cable can be depended upon to have a modulus of elasticity equal to that of the wires composing it—namely, very close to 27,000,000 lb. per sq.in.; whereas, a rope under its first stressing has a modulus of about 12,000,000 lb., which may vary considerably depending upon size of rope and care in manufacture. Since the stiffness of the cable is of such vital importance in overcoming any excessive elasticity in a suspension bridge, it was essential to overcome these disadvantages of twisted strands in respect to elastic properties before consideration could be given to their use in a bridge cable.

To accomplish this purpose, each strand was subjected to a prestressing treatment prior to cutting. A load of 150,000 lb. per strand applied for one-half hour was sufficient to close the wires in the strand and thus remove the "inelastic" stretch to an extent sufficient to guarantee a modulus of elasticity of 24,000,000 lb. for the strand after such treatment.

Strands for the St. Johns bridge cables were treated accordingly. After application of the 150,000-lb. load for a half-hour period, the load was decreased to 70,000 lb. (closely approximating the dead-load stress in the completed structure), and the strand was then cut to predetermined length and socketed. Each strand was then coiled on a wooden reel and shipped to the site. Such a unit weighed about $6\frac{1}{2}$ tons.

**Fig. 4—Layout of cable-stringing operations**

A system of hauling ropes and the cages suspended from track ropes permitted elimination of footwalks for erection. Each precut and socketed $1\frac{1}{2}$-in. strand, 2,720 ft. long between anchorages, was delivered on separate reel and hauled across the river by continuous loop rope. The 182 strands in the two cables were strung in 23 working days.

### *Cables*

The economy of the twisted-strand cable over the parallel-wire type lies mainly in the field work, since the major operation of building the wires into strands (of necessity a field operation for the parallel-wire type of cable) becomes a shop operation and may be carried on while work preliminary to cable erection is in progress. It should be possible to erect rope strands in 60 per cent of the time required for parallel wires.

In an effort toward further erection economy the contractor did not use a footbridge for a working platform in stringing the cables. In place of this conventional erection practice, the installation consisted of a system of track ropes, cages and hauling lines to draw the strands across the river. The fixed ropes consisted of four 1-in. strands, known as track ropes; two of them spaced on 10-ft. centers served each cable. These track ropes supported and acted as guides for steel cages located at center and quarter points of the main span and center of the side spans. The cages served as working platforms for erection and adjustment of the main cable strands. The movable ropes consisted of an upper 1-in. hauling rope and a lower $\frac{3}{4}$-in. hauling rope, each forming a continuous loop serving both cables and passing through tightening frames located at the east anchorage to take up the stretch in the rope developed by use.

Each of the hauling ropes could be moved independently by means of electric machinery installed on the west anchorage. The steel cages could be fastened to the upper hauling rope when it was desired to shift their positions along the track ropes. The longer $\frac{3}{4}$-in. hauling rope was the actual means of pulling the strands across the river.

The reel containing the strand was set in a reel stand on the east anchorage. The strand socket was then attached to the hauling rope by a coupling, shown in Fig. 5, and the strand was pulled across the river over the tops of cable bents and main towers to the west

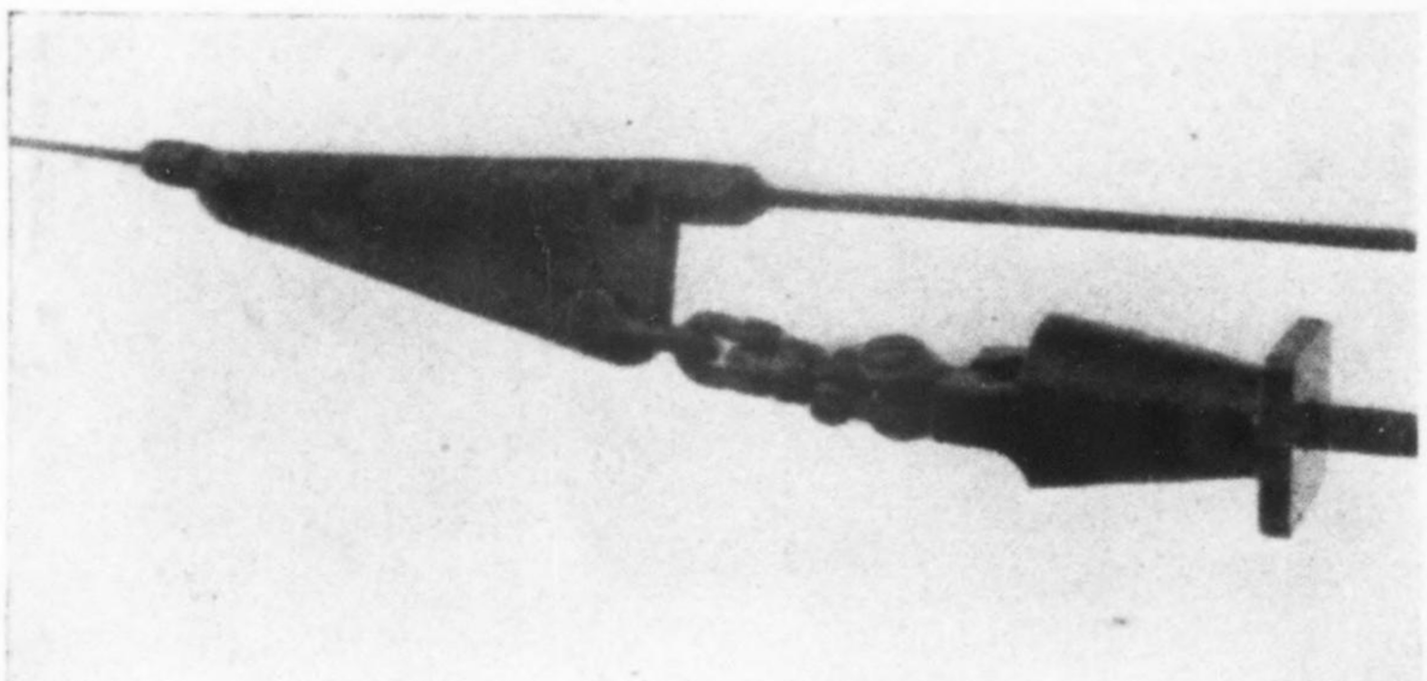

**Fig. 5—Swivel connection of hauling rope to strand socket**

This special fastening, permitting strand to twist independently of rope, was an important field development in cable-stringing, eliminating previous delays caused by twisting of the two units.

anchorage. To control the sag of the strand as it was being hauled across the river, steel U-shaped guides were hung from the upper tramway rope at 120-ft. intervals and the bridge strands pulled through these guides. The general arrangement of this cable-stringing operation is shown in Fig. 4.

The attaching of the strand sockets at the anchorages consisted in locking the sockets between a series of vertical plates pinned to the main anchorage bars. Each socket bears against a series of castings bolted between the vertical plates. Final adjustment of the strand to precalculated sag was controlled by means of shims inserted between the strand socket and the casting.

### *Strand Twisting Eliminated*

Actual erection and adjustment of the main cable strands was accomplished in 23 working days. The main difficulty met at the start was the twisting of the strand around the hauling rope, which necessitated untwisting—an operation both cumbersome and time wasting. This condition was finally corrected by the use of a special swivel that permitted the strand to twist independently of the hauling rope (Fig. 5). This freedom to twist naturally resulted in a slight change of strand length, which was corrected by the shim adjustment. The maximum number of strands erected per day was sixteen. Adjustments were made before sunrise or after sunset or in cloudy weather to eliminate the uncertain effect of the sun.

Following the completion of the strand stringing, the cable bands were placed at points measured along the cables in the field. Since the 91 cable strands as erected formed a regular hexagon, the inside faces of the cable-band casting formed a corresponding figure and, in addition, were grooved to fit the individual outside strands. With these bands in position the suspender ropes were hoisted. These are $1\frac{5}{8}$ in. in diameter, two of them being looped over each cable band to provide four parts of line at each point of connection to the stiffening trusses. Each suspender rope was also prestressed prior to cutting, a load of 100,000 lb. being applied for a period of half an hour. This load was then reduced to 35,000 lb. and the rope marked for cutting under that load. The specified ultimate strength for each rope was 212,000 lb., and the modulus of elasticity 14,000,000 lb. Suspender-rope erection was followed immediately by erection of the stiffening trusses.

### *Stiffening-Truss Erection*

Prior to hoisting the stiffening trusses the members were riveted into double-panel units, 36 ft. long and weighing 10 tons each. These units were hoisted from barges by means of falls suspended directly from the main cables, and wooden segmental collars were placed around the cables to protect the galvanized wires from abrasion. The schedule of erection for the suspended steel was planned to keep the weights balanced about each main pier to prevent excessive tower deflection. Erection was begun at the center of the main span and at the cable bents and progressed symmetrically from these points toward the main towers.

Cable wrapping, begun upon completion of steel erection, was preceded by two coats of paint applied to the cables. Then the flat surfaces of the cables were covered with segmental strips of cedar about 6 ft. long to transform the hexagonal cable cross-section into a circular one more suitable for wrapping. The cedar strips had been treated by immersion for a period of 20 min. in linseed oil at a temperature of 200 deg. F. The wrapping consisted of a continuous serving of No. 9 soft annealed double-galvanized steel wire applied by a motor-driven machine on each cable. The end of each cable band was counterbored to permit tucking in the wrapping wire, and the space was then caulked with lead wool. Following wrapping, three coats of paint were applied, the diameter of the completed cable being $16\frac{3}{4}$ in.

The pouring of the concrete deck slab and sidewalks was carried on simultaneously with the cable wrapping in accordance with a schedule laid out to prevent excessive main-tower deflections, as was done for the erection of the suspended steel.

The bridge was built by Multnomah County at a cost of very nearly $4,000,000—about $250,000 less than the estimate of the engineers. Robinson and Steinman, New York, designed and supervised the construction of the project. The work was let in seven main contracts as follows: Gilpin Construction Co., Portland, Ore.—foundations; Wallace Bridge & Structural Steel Co., Seattle, Wash.—suspension bridge superstructure; John A. Roebling's Sons Co., Trenton, N. J.—cables; La Pointe Construction Co., Portland, Ore.—west approach roads; Columbia Steel Co., Portland, Ore.—approach superstructure; Lindstrom & Feigenson, Portland, Ore.—roadway slab and sidewalks; National Electric Co., Portland, Ore.—electrical work.

The principal subcontractors were the following: Pacific Bridge Co., Portland, Ore.—river piers; J. H. Pomeroy Co., Portland, Ore.—erection of suspension-bridge structural steel; American Bridge Co.—viaduct steel.

The Robert W. Hunt Co. supervised the inspection of the fabrication and erection of all steel, and the Northwest Testing Laboratories inspected all concrete. The entire project was completed in a period of $21\frac{1}{2}$ months.