Bridging the Delaware River at Philadelphia—Part I

Project a Century Old Realized at Last by Public Enterprise on a Basis of Amortization of Cost by Tolls —Many New Problems—Rapid Construction Results in Opening Bridge Before Predicted Day

A two-page insert plate, facing p. 534, accompanies this article.

WITH the opening of the Delaware River bridge between Philadelphia and Camden on July 4, the world's largest suspension bridge went into service. The structure is the tardy realization of a project dating back fully a hundred years but not carried out until the states concerned, New Jersey and Pennsylvania, undertook the work jointly. As they are now operating it as a toll bridge to refund the cost, it probably ranks as the largest public toll enterprise ever carried out.

Both in technical and in esthetic respects the bridge is a distinguished achievement. Its unusual size and the prospective traffic demands called for the highest order of engineering skill and created many serious special problems, some of which led to experiments resulting in fundamental contributions to future suspension bridge engineering. The undertaking also presented an outstanding architectural opportunity. This received careful attention from the beginning and the joint effort of engineer and architect was directed toward attaining idealized line and balanced proportion for most pleasing effect.

One of the noteworthy technical achievements in the work was the completion of the bridge, ready for regular traffic, three days earlier than the day promised in the engineer's preliminary report, written a year before construction began. At that time, early in 1921, they stated that the recommended type of bridge at the selected location could be completed ready for traffic by July 4, 1926, the date of Philadelphia's sesquicentennial celebration of the Declaration of Independence. The bridge was opened to regular traffic on July 1.

Origin of Project—Proposals for building bridges across the Delaware at Philadelphia date back at least to 1812, but cost and technical difficulty kept all of them from success. Instead there was an active development of ferry service, which until the current year has supplied the only highway crossing south of Trenton.



FIG. 1—DELAWARE RIVER SUSPENSION BRIDGE, VIEW FROM CAMDEN TOWARD PHILADELPHIA
As seen after completion of main structure, with only finishing and approach work to be done before admitting traffic.

Within the last 15 years, however, demands for a bridge or tunnel crossing of the river became insistent. Reports were made by several special boards and commissions, establishing that a bridge was feasible and could be built at a cost that made the undertaking worth more detailed study. These preliminaries led to legislative action by the two states in 1919 providing for an interstate compact to join in the construction of a bridge, and such a compact was soon after formulated and approved. Under its terms the bridge has been built by the joint action of bridge commissions of the two states, with representation also of the city of Philadelphia, which undertook to meet half of Pennsylvania's cost.

Upon the organization of this joint body, it selected a board of three engineers to direct the design and construction: Ralph Modjeski, chairman and chief engineer; George S. Webster, and Laurence A. Ball.

Site and Type—The location of the bridge was selected from among a number of proposed sites, the five most prominent of which are indicated in Fig. 2. The choice was based on a desire to place the bridge near the east-west axis of Philadelphia, Market St., and toward



FIG. 2—LOCATION OF BRIDGE, AND, SHOWN DOTTED, SOME ALTERNATIVE ROUTES CONSIDERED

the side of the largest extent and growth of the city, that is, northward.

A clear span between pierhead lines, corresponding to a spacing of main piers of 1,750 ft., and a clearance height of 135 ft. were necessitated with regard to shipping. Property conditions, at least on the Philadelphia side, rather narrowly fixed the land-span length. The layout thus indicated was suited for either cantilever or suspension bridge construction. Subsoil conditions made either type of structure feasible, though they rather favored the cantilever span; bedrock formed a nearly flat floor 60 to 100 ft. below tidewater, the slope being downward toward Camden. However, comparative designs showed the suspension bridge to be 10 to 20 per cent cheaper, and as it permitted construction under a number of separate contracts, which was best suited to the plan of annual appropriation of funds, it was recommended by the engineers and adopted by the commission.

The cost was estimated at $28,900,000 (construction $20,800,000, land $6,800,000, contingencies and engineering $1,300,000). In the actual execution, however, these figures were overrun by 12 per cent in construction and 70 per cent in land. The actual cost was about $36,300,000.

Project Economically Justified—Traffic estimates based on ferry data (Fig. 3) showed a cross-river traffic of about 2,500,000 vehicles in 1921, and led to a conservative forecast of 6,000,000 in 1930, increasing to 15,000,000 to 20,000,000 vehicles per year 12 or 15 years later. These traffic volumes at any reasonable valuation of the crossing privilege—25 to 50c. per vehicle—assured an earning power ample not only to carry the annual charges but also to amortize the total cost within 20 years. Subsequent developments have even bettered these figures. The adopted toll rate of 25c. per vehicle now promises to retire the whole cost of the bridge in less than 15 years.

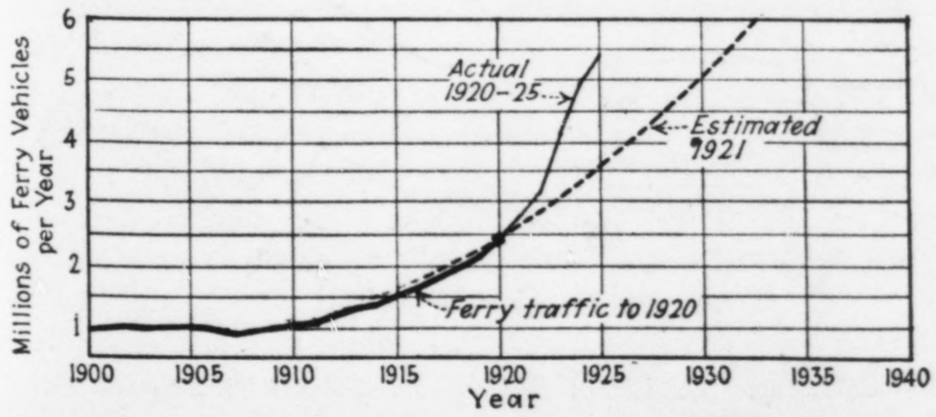


FIG. 3—FERRY TRAFFIC DATA FOR PHILADELPHIA-CAMDEN CROSSING

General Design—A great main highway artery was to be created by the bridge, connecting the road systems

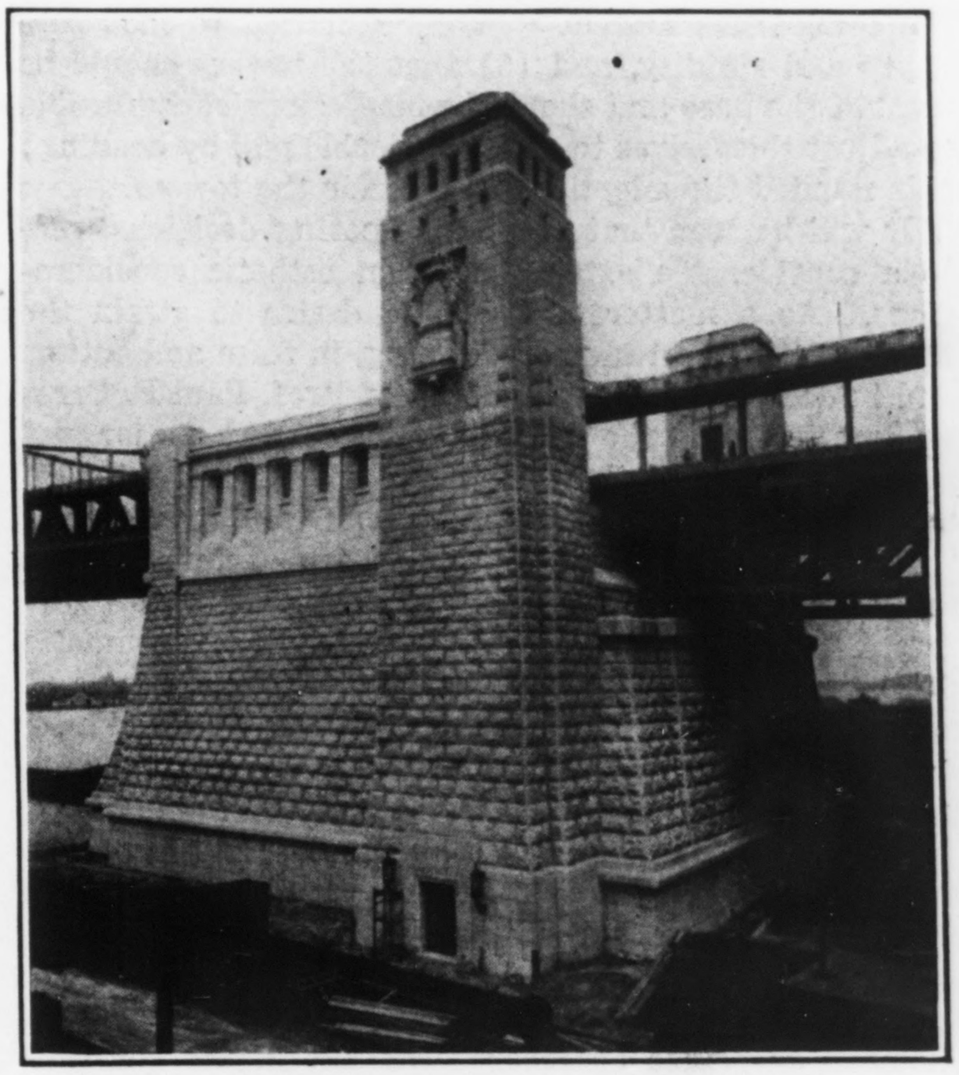


FIG. 4—CAMDEN ANCHORAGE
Illustrating the architectural development of the bridge.

of two populous states and also serving for intercommunication between Philadelphia's 2,000,000 and Camden's 150,000 of population. With this object in view, the engineers chose a six-lane roadway, 57 ft. wide, four tracks for street cars or rapid-transit tracks, and two 10-ft. footwalks, as representing adequate and practicable arterial capacity. Accordingly, the spacing between trusses or cables was set at 89 ft., which contemplated bracketing one track on either side outside the trusses. The footwalks were placed above roadway level. In the first design a plane of upper laterals was provided between the top chords of the trusses, and the footwalks were carried on these laterals. Later restudy of this design, mainly from the esthetic viewpoint, led to elimination of the overhead bracing. It was found

to be practicable to carry all lateral forces down to floor level by developing the bridge cross-section as a rigid frame, and the sidewalks could then be bracketed to the posts.

A bold decision was made by the engineers in adopting at the outset two-cable construction (though all recent large suspension bridges had been built with four cables) as giving greater structural simplicity and definiteness of stress distribution. The two cables (parallel-wire construction being presumed) would be 30 in. in diameter, half again as large as the largest ever built, those of the Manhattan Bridge. But the engineers felt certain that cables of this size could be built satisfactorily and would be as efficient as smaller cables. Both the construction experience and the results of special tests later confirmed their judgment most gratifyingly.

Other features of the suspension system early decided upon by the engineers were (1) that the land spans should be suspended from the cables rather than independently supported, (2) that the stiffening trusses should be kept wholly below the cables, (3) that the rather large sag of 200 ft. should be used, (4) that the stiffening-truss system should be limited to moderate depth and rigidity, and (5) that the towers should be fixed at the base and should be made sufficiently flexible to adjust themselves to change of cable pull by bending; this implied the adoption of steel for the towers.

It will be seen that these engineering decisions were to a considerable extent based on esthetic considerations. As a matter of fact, the desire to attain the best esthetic effect in proportioning, in form and outline and in detail led to the engaging of Prof. Paul P. Crret as architect, and the determination of both major and minor features of the structure was carried out with constant interchange between engineer and architect.

Foundations—Under the rather favorable rock and soil conditions at pier and anchorage sites, two different types of construction were adopted: pneumatic caissons for the main piers (in the river) and groups of open-dredged piers for the anchorage (on land). Fig. 4 shows the completed Camden anchorage and drawings in the second article will show details of foundations.

The pneumatic pier work was simple and not abnormal in size, but the caissons were distinguished by a steel working chamber with all-steel cutting-edge construction braced by brackets so as to leave the chamber free of bracing struts, as well as by a communication chamber or refuse chamber cored out in the concrete

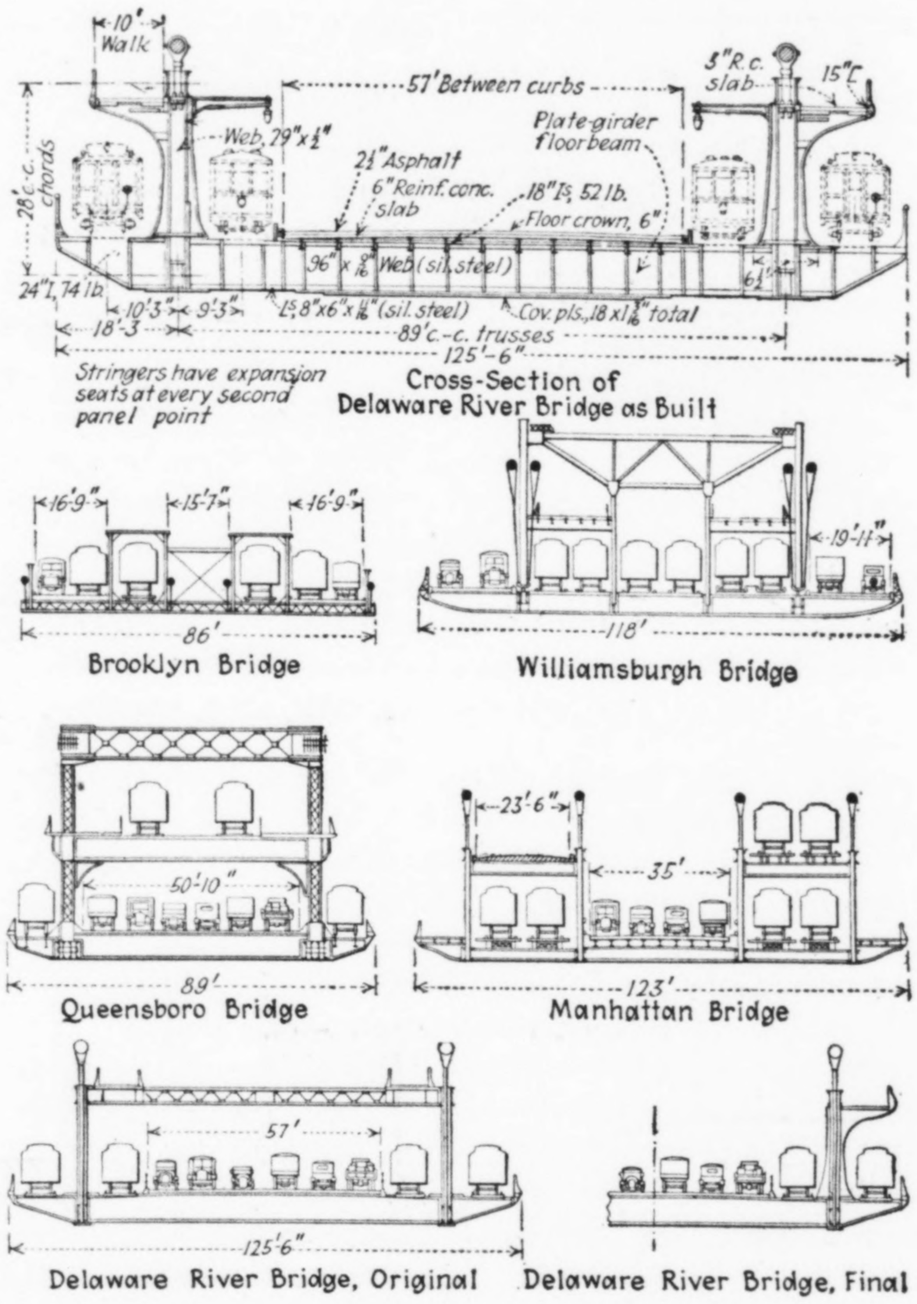


FIG. 5—MODERN TRAFFIC DEMANDS CALL FOR AMPLE DECK SPACE

Comparison of the several cross-sections of large bridges brings out not only the large roadway capacity of the Delaware River bridge, but also the open arrangement of the traffic space. Note that the first design showed an upper lateral system, later eliminated in order to make the roadway more nearly like an open highway.

above the roof. The latter, provided as a safety measure and as a refuge during blasting, proved so convenient that with its exit air lock it was used to the exclusion of the regular man lock. The caissons, 68 ft. 8 in. by 143 ft., were built to full height in a shipyard and were launched complete, so that no subsequent cofferdam framing was required. They passed through mud and sand to a depth of 58 ft. below high water at the Philadelphia pier and 82 ft. at the Camden pier, with maximum air pressures of 28 and 34 lb. respectively. They and the granite-faced piers surmounting

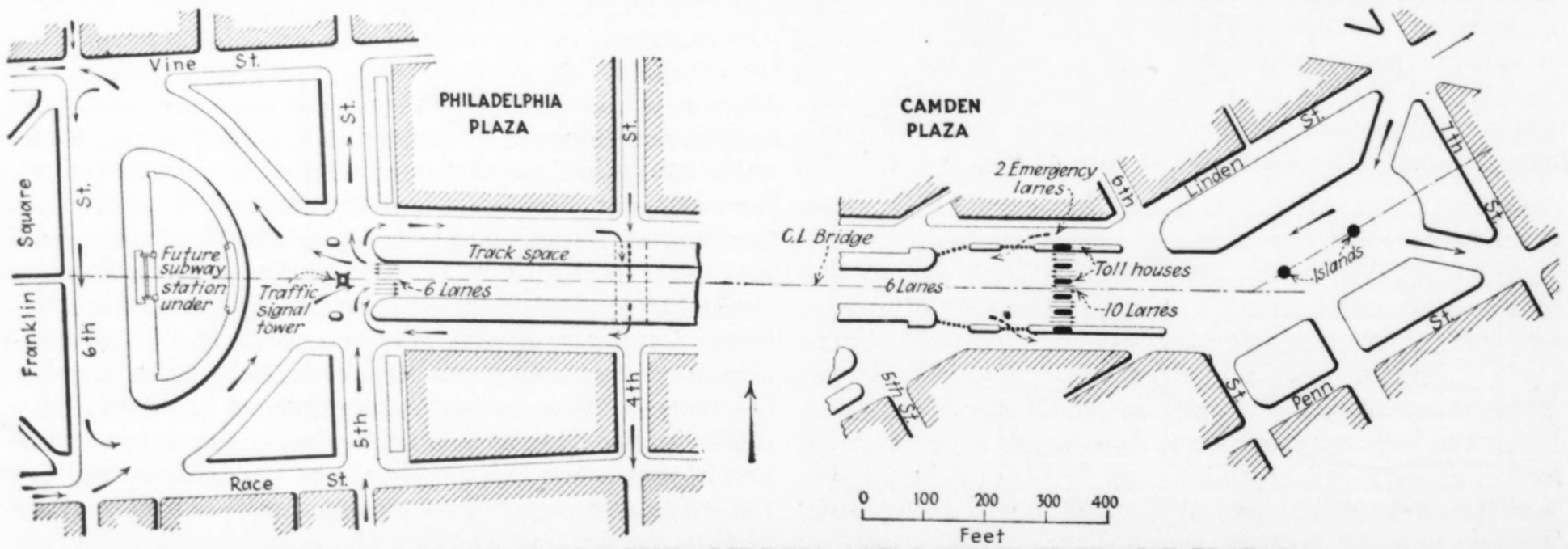


FIG. 6—LAYOUT OF THE PHILADELPHIA AND CAMDEN ENTRANCE PLAZAS

them were constructed during 1922 by the Keystone State Construction Co. in conjunction with Holbrook, Cabot & Rollins Corporation, at an average cost of $28 per cubic yard of pier and foundation (total quantities 26,900 and 31,600 cu.yd., final contract cost $744,839.15 and $897,039.88 for Philadelphia and Camden piers respectively).

A much less simple problem was presented by the anchorage foundations. While some other large suspension bridges, where rock is not exposed or found at shallow depth, have anchorages supported on piles or wooden grillage, it was thought proper here to go to rock, the depth below ground being about 75 ft. at Philadelphia and about 110 ft. at Camden. An anchorage structure of the required length for due stability at this height was so large as to make it advisable to divide the foundation into several units. Adapting this division to the method of functioning of the foundation, a long rectangular foundation pier was provided along either side of the anchorage, and a series of small circular piers under the rear portion; the former to take all of the horizontal pull and carry the main part of the anchorage weight in the completed condition, while the smaller rear caisson had the important function of carrying the excess weight at the rear of the anchorage before the cable pull was brought to bear on the structure. The large caissons were 40x125 ft. at Philadelphia and went down 61 ft. below high water; those at Camden were 40x140 ft. and went down 102 ft.

Both large and small piers are reinforced-concrete caissons sunk by open excavation through dredging wells, as described more fully in *Engineering News-Record* of June 4, 1925, p. 920. Their sinking was greatly facilitated by the possibility of pumping them

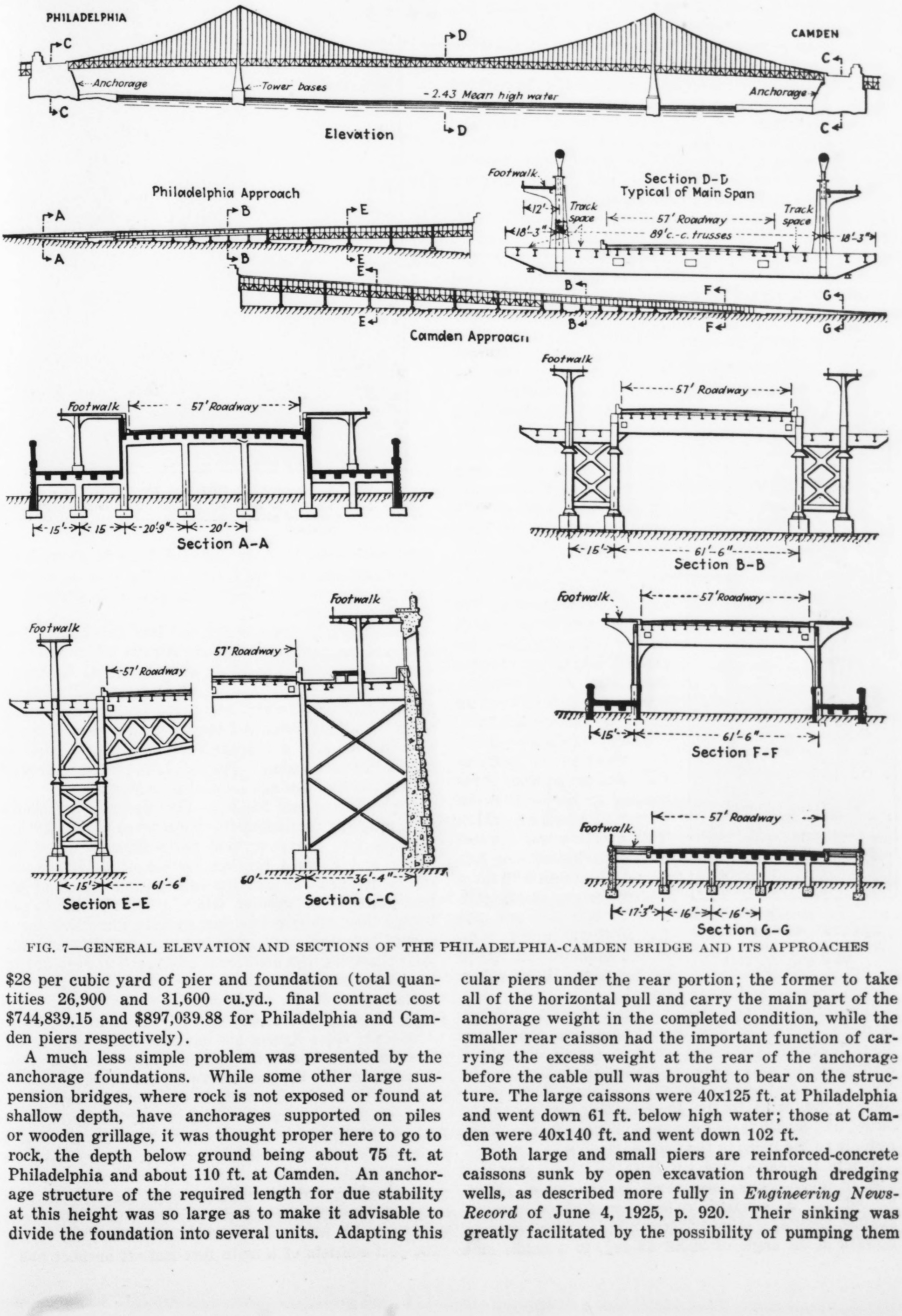


FIG. 7—GENERAL ELEVATION AND SECTIONS OF THE PHILADELPHIA-CAMDEN BRIDGE AND ITS APPROACHES

out when the cutting edge approached rock so that the last part of the excavation could be done in the dry, which assured a satisfactory and even foundation bed. The rock, a schistose material, was disintegrated to some depth in its upper 5 ft. or more, and a foundation bed of the desired hardness and solidity was found only after excavating some distance through the decomposed part. This was also true at main pier foundations.

The anchorage body was built in two sections. The first section, built by the same contractor as the main piers, involved a total volume above the foundations of 33,300 cu.yd. for the Philadelphia and 33,900 cu.yd. for the Camden anchorage, furnished at an average cost of $7 per cu.yd. These figures include the concrete forming the seats for the anchorage bents and the incasement of the anchorage girder grillage. The work was done under contracts which included the foundations (27,500 cu.yd., Philadelphia, at average cost $19.80 per cu.yd.; 51,400 cu.yd., Camden, at average cost $16.60 per cu.yd.) and the furnishing of the anchorage girders and eyebars (2,300 tons), the total contract cost being $916,841.05 for Philadelphia and $1,217,811.58 for Camden. The granite masonry inclosure and floors which form the upper part of the anchorage, built later under a contract awarded to Keystone State Construction Co., included a total of 34,500 cu.yd. for Philadelphia and 36,100 for Camden. The contract cost of this work on the two anchorages, including additional foundations, steel work, elevators, equipment and interior finish, was $2,991,835.15.

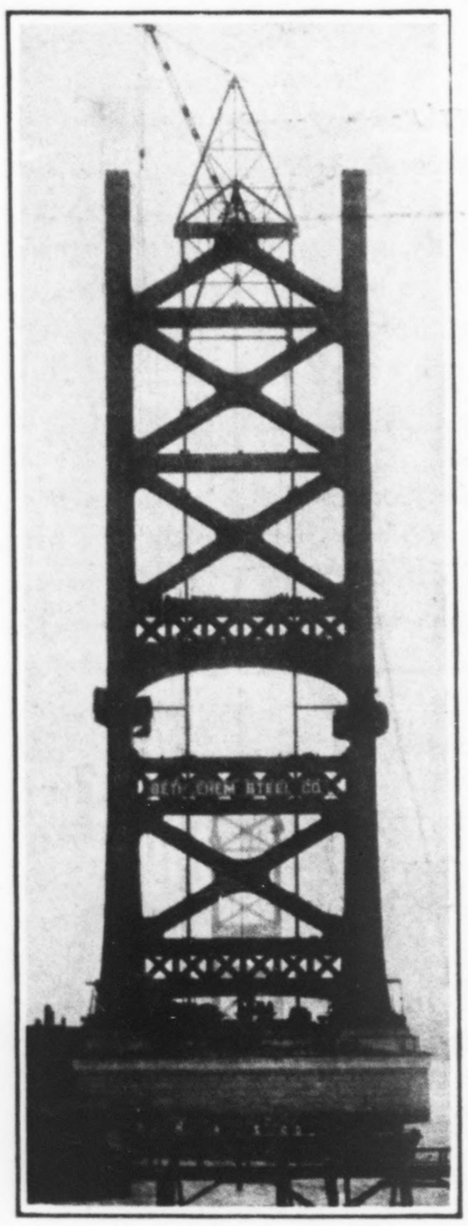


FIG. 8 — STEEL TOWERS ERECTED BY TRAVELING DERRICKS
View Feb. 21, 1924

Cable Anchors — After the adoption of general form and proportions it remained to decide between wire and eyebar cables. This question was studied by comparing a complete wire-cable design with three eyebar designs (with cable sags of 200, 220 and 240 ft.) at 40,000 lb. per sq.in. in the eyebars and 72,000 in the wire. Wire cables proved cheaper and were adopted.

Connection of the cables to the anchorage masonry is made by chains of eyebars connecting the strand shoes inserted in the end loops of the 61 strands of each cable to nine 42-in. box girders bedded in the concrete. The cables strands radiante from their fanning-out point to the strand shoes, set in square array, and thence their lines are continued by the chains radially to the piers in the anchor girders.

Where the cable fans out it is held by a cast-steel bell or toper collar (see detail on the insert plate), backed by a straight collar for further security against slipping. Above this point the cable continues upward at an angle of about 45 deg. to a saddle on a

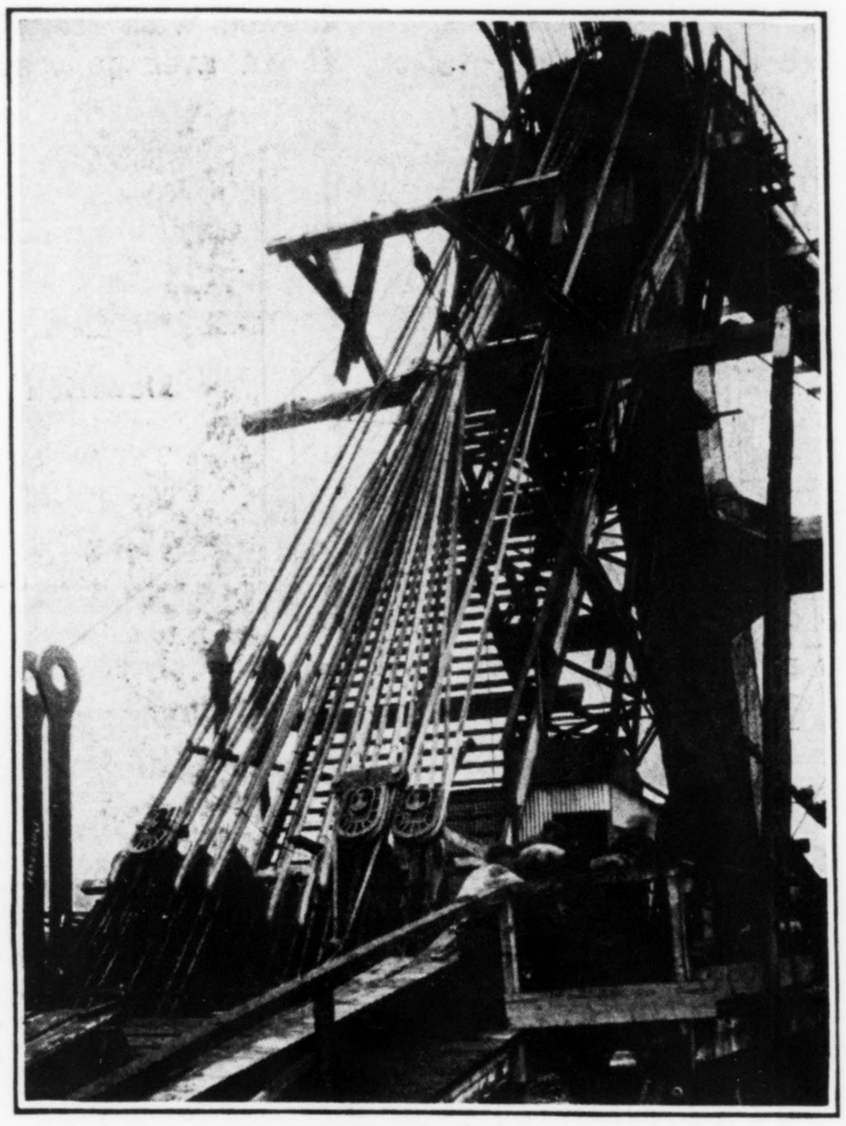


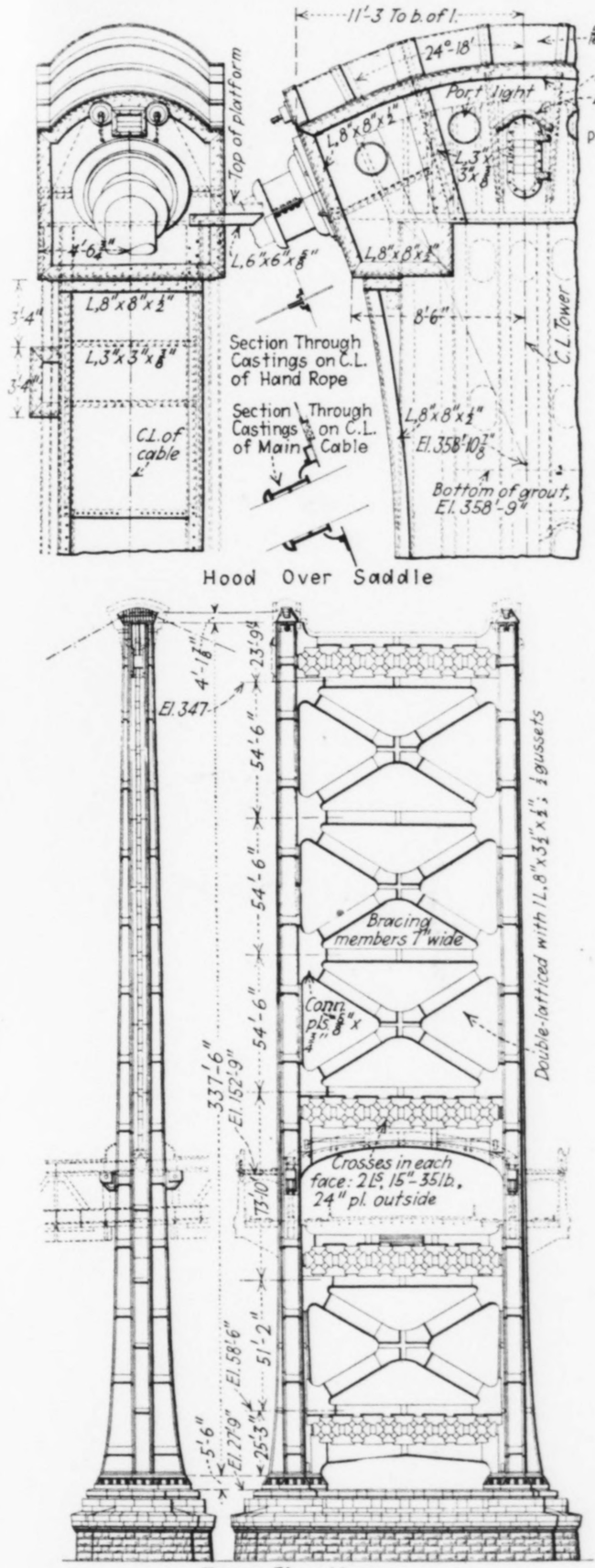
FIG. 9—HOW THE BRIDGE IS ANCHORED
Each cable strand is held by a pair of eyebars extending down to a group of girders bedded in the lower part of the anchorage concrete.

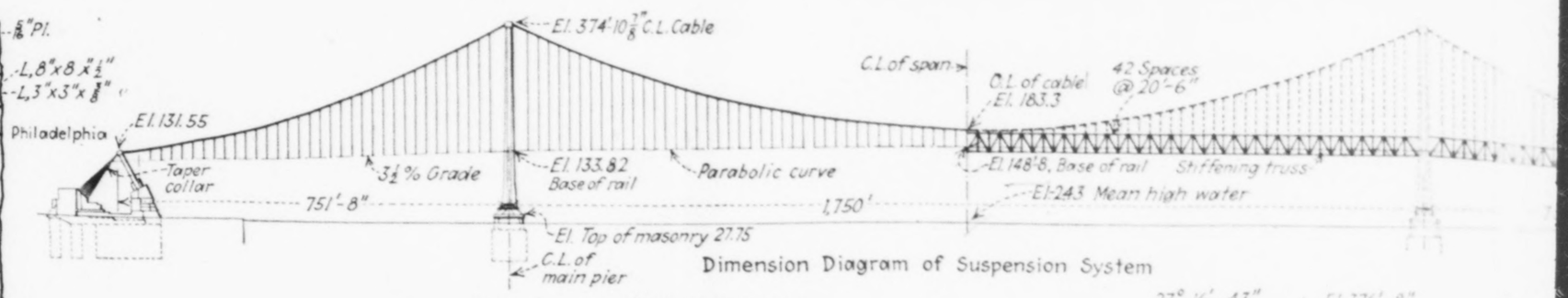
steel anchorage bent, is deflected here to about horizontal position, and then rises on the side span cable curve to the main towers, and so ultimately proceeds to the opposite anchorage.

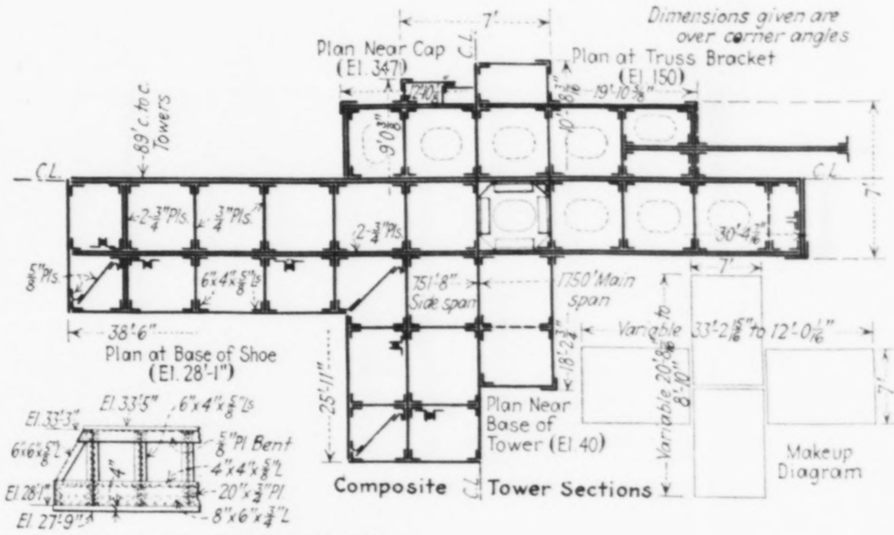
The anchorage bents, carrying a load about two-thirds as great as the main towers, consist of two silicon steel posts, about 7x8 ft. in cross-sectional dimension and 1,283 sq.in. metal section, 76 ft. long, built of three main longitudinal webs and three transverse staying webs. At their lower end they rest on cast-steel shoes, and on top carry a cast-steel saddle closely similar to the main tower saddle. The saddle is roofed with steel plates and its sides are incased in concrete.

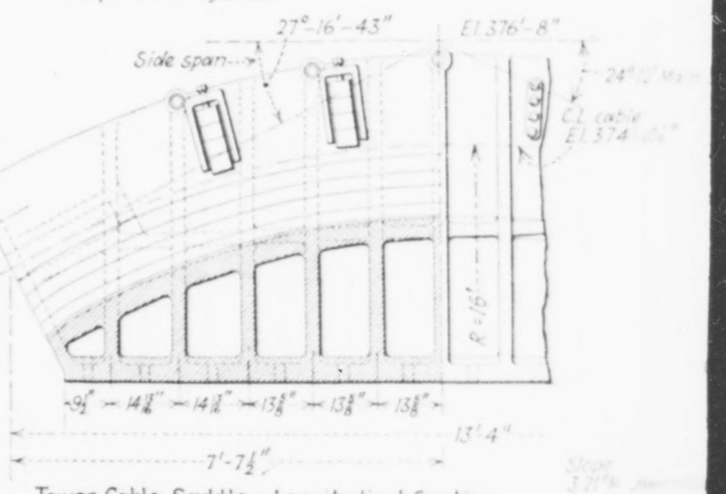
Main Towers and Saddles—The tower design, which in a measure dominates the appearance of the bridge, involves two heavy vertical posts directly under the cables and a rigid bracing system of diagonals and struts detailed for massive appearance. Each of the two posts is of greater width longitudinally to the bridge than transversely, but in both directions has a curved tapering profile, which favors stability and permits obtaining the additional cross-section required by the larger bending moments near the base of the tower.

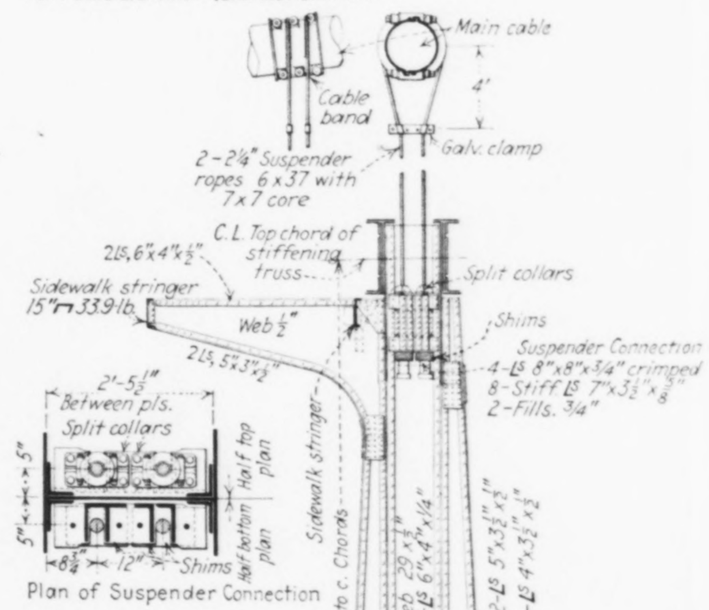
Each post of the tower carries a maximum vertical load of about 35,000,000 lb., of which 24,600,000 is dead-load; these figures are almost exactly equal to the corresponding horizontal pull components in one cable. In cross-section the post is cruciform. The large arm, parallel to the bridge axis, is composed of three $\frac{3}{4}$-in. to $1\frac{1}{2}$-in. webs $3\frac{1}{2}$ ft. apart and of width ranging from 12 ft. at the top of the tower (343 ft. above the masonry) to 38 ft. 8 in. at the base. These webs are connected at about $3\frac{1}{2}$-ft. spacing by vertical diaphragms of $\frac{3}{4}$-in. plate, and the middle three of these diaphragms extend outward to form the shorter arm of the cross, whose width ranges from 10 ft. to 26 ft. 1 in. Thus, the post consists of a main fore-and-aft member and

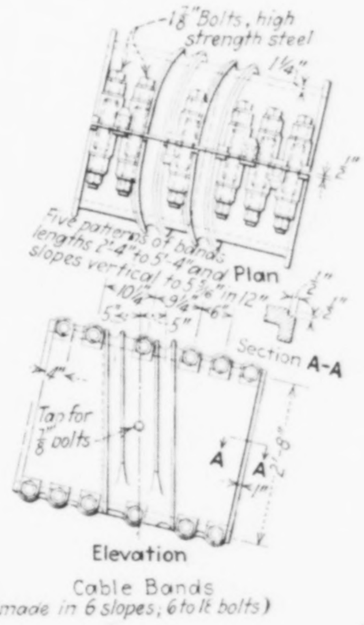


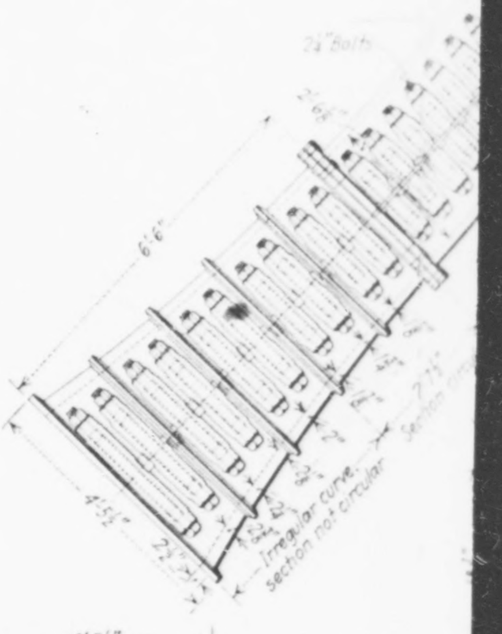


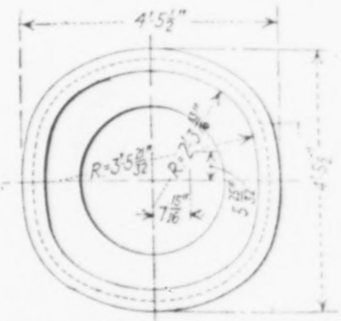




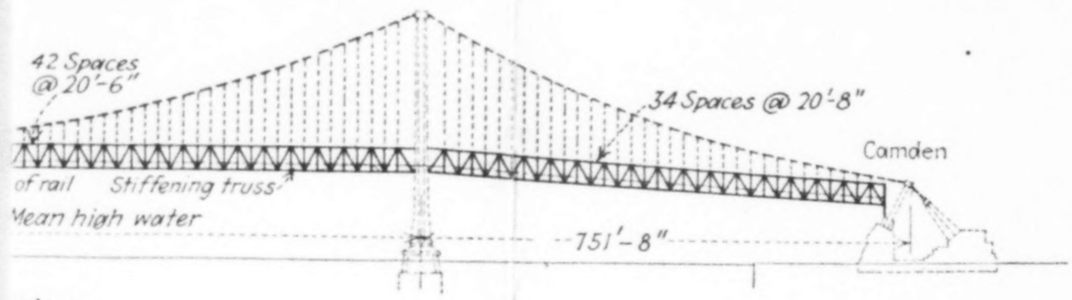




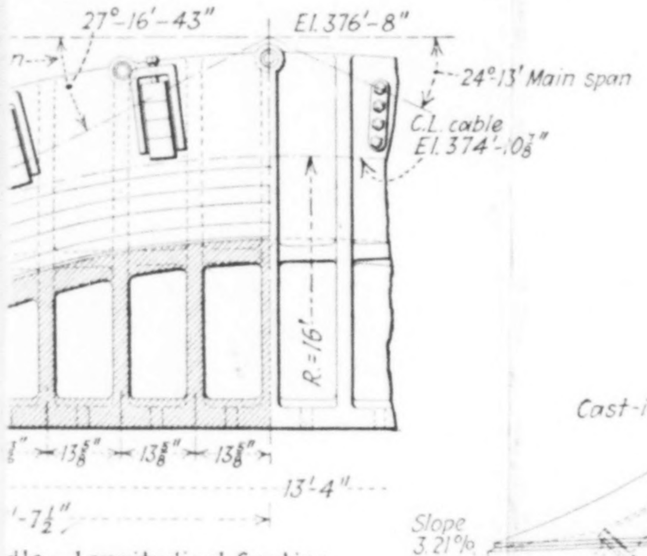




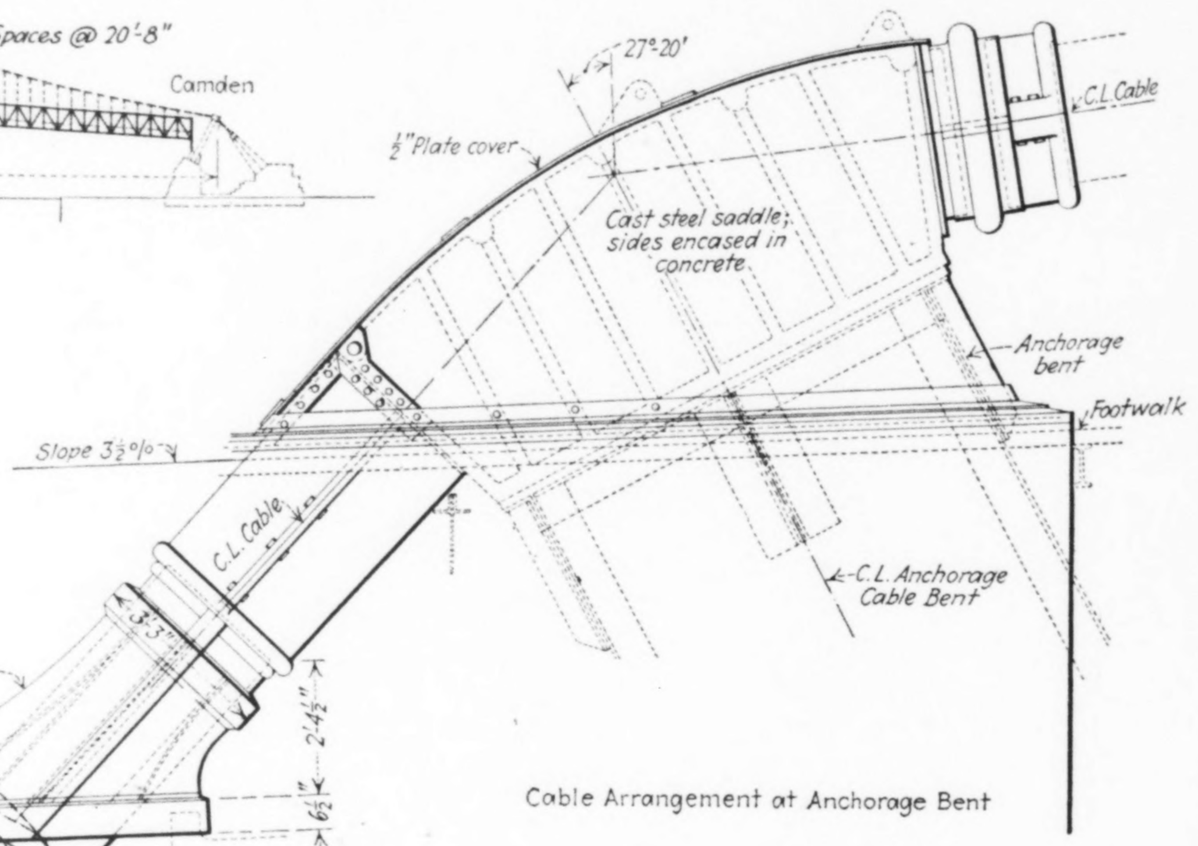
Delaware River Suspension Bridge Between Philadelphia, Pa., and Camden, N. J.—Dimensional Drawing and



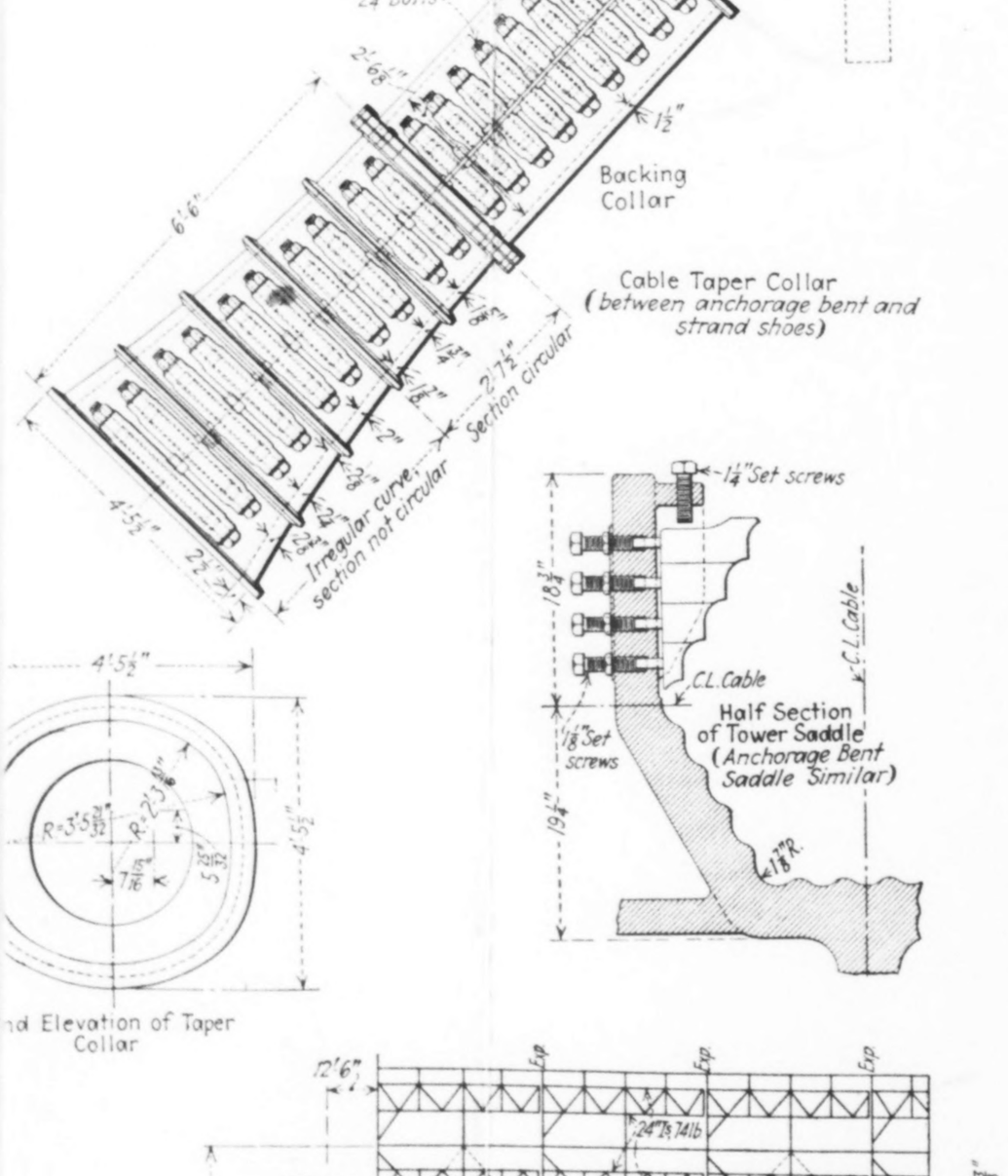
ystem



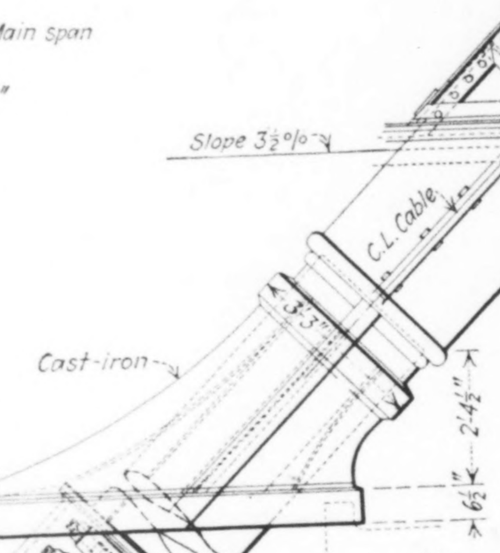
dle – Longitudinal Section



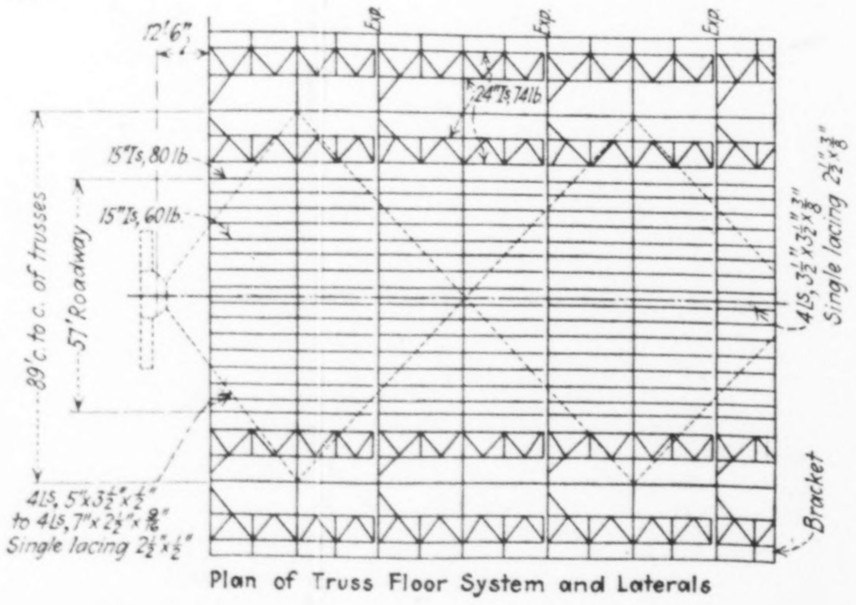
Cable Arrangement at Anchorage Bent



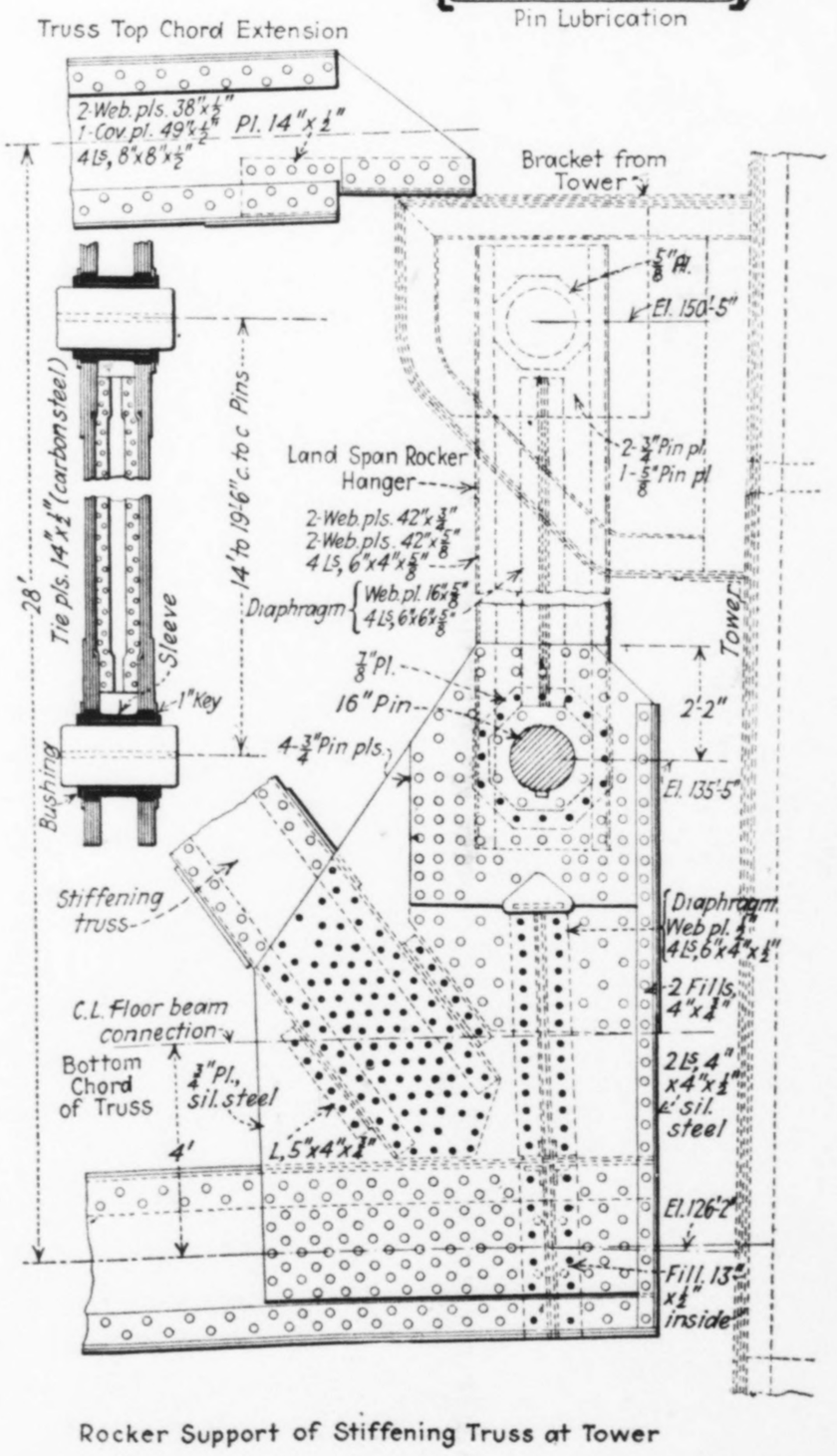
Cable Taper Collar
(between anchorage bent and strand shoes)



nal Elevation of Taper Collar



Plan of Truss Floor System and Laterals



Rocker Support of Stiffening Truss at Tower

Pin Lubrication

imensional Drawing and Details of Towers, Cables and Rigid-Frame Structure

minor transverse member integrated into a single cellular structure and further consolidated by horizontal diaphragms about 12 ft. apart. The latter have central manholes for access in inspection and painting, ladders being provided for the purpose. Three vertical splicing planes divide the tower cross-section into four construction sections, and transverse splices are 10 to 30 ft. long.

The towers are made of silicon steel (except for the transverse diaphragm and the sway-bracing members), to allow the use of a high unit stress corresponding to the desired flexibility of the tower under varying cable pull. They are seated on built steel shoes 5½ ft. high, resting directly on the masonry and held by twenty 3-in. holding-down bolts, which extend into the pier granite and concrete from 10 to 16 ft.

On the post is seated the cast-steel cable saddle, grooved to fit the hexagonal form in which the cable was laid up. The upper part of the cable periphery is held by profiled lugs screwed to the saddle sides. The weather protection of this tower top differs from that of the anchorage bent saddles, a complete plate-steel hood being built over the saddles and around the tower top (see detail on insert sheet).

The maximum cross-section of one tower post is 5,200 sq.in. Both towers together weigh 9,645 tons, to which is to be added 161 tons of saddle castings.

Cables—In the construction of the cables the same diameter and kind of wire were used as in most of the recent large suspension bridges: galvanized, hard-drawn steel wire, 0.196 in. in diameter after galvanizing, with ultimate tensile strength about 220,000 lb. per sq.in. Each cable consists of 18,666 wires (61 strands of 306 wires). The individual strands are each an endless wire (the separate reel-lengths of wire being spliced together by screwed sleeves) looped around the strand shoe at each end. The cable when completely laid was squeezed into circular shape by a power squeezer, giving it an outside diameter of 30 in. The cast-steel cable bands, constructed as detailed in one of the drawings, were clamped directly on the bare cable; between them a spiral wrapping of annealed galvanized steel wire was put on by a motor-driven wrapping machine under steady tension on the wrapping wire. The cables contain 6,780½ tons of wire including the wrapping.

At the anchorage bent saddles, where the cable section changes from circular to hexagonal and wrapping therefore would not give a fully tight protection, a shell cover is provided for protection. At the main saddles the wrapping finishes against the end grooves of special castings, bolted to the end faces of the tower hoods.

Two suspender ropes are looped around each main band, so that the suspender consists of four 2¼-in. ropes socketed to engage the upper ends of the stiffening-truss posts. Shims for adjustment and locking plates to hold the suspender sockets in place, are provided at the connection. These ropes have a strength of about 400,000 lb. each when tested in the straight length, and in excess of 380,000 lb. when tested over a 32-in. sheave (like a cable band groove), while the actual maximum load at any one panel point can never exceed 400,000 lb. on all four ropes.

Stiffening Truss and Roadway—The panel length of the stiffening trusses is 20 ft. 6 in. in the main span and 20 ft. 8 in. in each side span, which is the same as the suspender spacing. The trusses are shallow (28 ft. deep). Their chords and the end rockers are nickel steel, while web members and laterials are of silicon steel.

The truss system is continuous from tower to tower and from tower to anchorage, which construction was considered by the engineers to give the maximum economy, most effective stiffening, and least local stress and wear at hinges or the like. At the towers, the center span and side-span trusses are suspended by means of rocker hangers from brackets riveted to the tower posts; bushed and lubricated pins connect these hangers to brackets and trusses respectively. A similar rocker, in compression, supports the side-span truss on the base of the anchorage bent.

The maximum bending moment in one truss is about 200,000,000 ft.-lb. in the center span and 222,000,000 ft.-lb. in the side span, while the maximum shears are 1,263,000 and 1,682,000 lb. respectively. The total steel weight in trusses and floor is 18,700 tons, of which a little more than half is in the stiffening trusses.

Wind-stress transfer at the towers and anchorages is made by a jaw connection located on the center line of the bridge. Longitudinal movement is permitted by the jaw but lateral movement is prevented.

The floor consists of 8-ft. floorbeams between trusses, rigidly riveted to the truss posts (the shape of the three-member frame formed by the truss posts and floorbeams is shown in the cross-section of the bridge on the insert sheet) and prolonged outside the trusses by brackets carrying the outer tracks. Between these floorbeams are fastened 18-in. I-beam stringers 3 ft. 10 in. on centers, with expansion seats at every other panel point. The concrete floor slab of the roadway rests directly on these stringers. In view of the heavy load concentrations which a modern highway floor must be proportioned to carry a 30-ton truck with 15,000-lb. wheel loads and exerting a 50 per cent impact was contemplated—requiring a strong slab, while at the same time it was desirable to hold the dead weight to a minimum, heavy reinforcement and effective shear reinforcement were required. The main (transverse) reinforcement consists of "Rivet-Grip" trusses 4½ in. deep, set transverse of the bridge, 6 in. on centers on suspended spans and 7 in. on approaches. Longitudinally there are upper and lower temperature rods. A 1:1½:3 concrete, of about 2,700 lb. strength at 30 days, was placed around this reinforcement to form a 6-in. slab (1:2:4 concrete in 7-in. slab on the approaches). This slab was ultimately topped off with a 2½-in. sheet asphalt surfacing on the suspended structure, and with 3-in. granite block paving on the approaches. The sidewalks are floored with 3-in. concrete slab.

At alternate panel-points, where the stringer system has expansion seats, the floor slab and pavement are interrupted, and are retained by 8x3½-in. steel bulb angles set on the stringers transverse to the bridge, each angle being effectively anchored back to the concrete of its roadway section. Over the rocker connections at towers and anchorages, where large expansion movements occur, specially designed toothed-grid expansion joints allowing for about 12 in. of movement are provided. On the approaches a heavy sliding-plate joint is used, the plate seated on heavy iron castings.

[*The history of the planning of the Philadelphia-Camden bridge and the general design features by which the requirements were met as to the bridge itself and its approaches are given above.*

Part II of this article, to appear in a following issue, will describe the erection of the bridge.—EDITOR.]

Bridging the Delaware River at Philadelphia—Part II

Three Grades of Steel Used in the Superstructure—Choice of Loading—Features of Design Calculations—Tests Made to Verify Design—Construction of Towers, Cable Spinning, and Suspended Structures

Second Half of Article: Part I Appeared in Engineering News-Record, September 30, p. 530

THE PLANNING, design, and some of the construction features of the Philadelphia-Camden bridge over the Delaware River, which is the largest suspension bridge yet built, were described in Part I, Sept. 30. This second section continues the discussion of design and construction.

Materials—Three grades of structural steel were utilized in the superstructure: Silicon steel for the main compression material of the tower posts, the web members and lateral system of the stiffening trusses, and some of the heavy girders in the approaches; nickel steel for the chords of the stiffening trusses, and medium structural steel for all other structural parts. The reasons for using the two special steels were either economy in weight (as in the webs of the stiffening trusses) or desire to secure great flexibility within the elastic limit of the material (in the towers, and still more so in the truss chords). However, as the differential for silicon steel over carbon steel was usually less than 1c. a pound, and that for nickel steel over silicon steel about 1c., there was distinct economy in the use of these high-strength materials under the unit stresses which the engineers considered suited to them for the particular uses in hand, and the carbon steel was economical only where the special steels would have resulted in thicknesses under the minimum desirable.

Loading—As already mentioned, the deck accommodations adopted by the engineers comprise a six-lane roadway 57 ft. wide, four standard-gage railway tracks (two for rapid transit and two for surface-car service), and two 10-ft. footwalks. Maximum loadings for these several traffic elements totaled 12,000 lb. per lineal foot of bridge, composed of two rapid transit trains at 2,000, two surface-car trains at 1,500, 70 lb. per square foot of roadway, and 50 lb. per square foot of footway. The roadway loading 4,000 lb. per lineal foot of bridge was found to be one-fourth heavier than the average of present roadway loads, and was considered to be sufficient to cover future increase of loading and concentrations under extraordinary conditions. The sum of the units, 12,000 lb. per lineal foot of bridge, represents continuous maximum loading on all of the deck facilities simultaneously; as this is a virtually impossible condition, the amount was regarded as a congested load, to be provided for at extra high stresses, and the working design load was taken at one-half the congested load, or 6,000 lb. per lineal foot, to which ordinary unit stresses were applied.

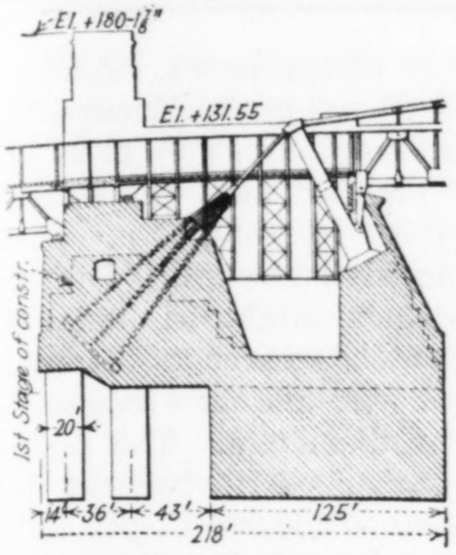
The weight of the suspended structure is about 26,000 lb. per lineal foot in the main span and 27,000 lb. in the side spans. Live-load was combined with these weights, with temperature variations of 55 deg. F. above or below closing, and in special cases with subnormal dead-load on part of the bridge, as when pavement repairs are going on. Wind pressure was taken at 50 lb. per square foot on the towers, and 30 lb. on the stiffening trusses, counting 1½ times the elevation in both cases. In designing the towers, to provide for all conceivable contingencies and to give extra strength and rigidity in the transverse bracing between posts, the assumption was made that a lateral force of 1,000,000 lb. in addition to wind reactions might be applied to the tower top, as from the impact of an airship. Of the wind load on the suspended roadway and trusses, 1,350 lb. per lineal foot, part is resisted by the cables due to their sharing in the transverse deflection (350 lb. in the main span and 50 lb. in the side span) while the remainder goes through the truss laterals. No impact allowance was figured anywhere except in the form of the 1,000,000-lb. special lateral load at tower top, and in a 50 per cent impact allowance in proportioning the floor slab.

Some Elements of the Calculations—For calculation of the relative action of towers and cables the modulus of elasticity of the cables was taken at 27,000,000 and that of the towers at 29,500,000 lb. per square inch. With these data, and with the various maximum distributions of load and a preliminary design of structure, the equilibrium of main and side spans was investigated, to determine tower deflection. It was found that the range of movement of the tower top for extreme positions of the working load would be 25 in., the range for congested load 39 in., and the extreme range, with subnormal dead-load, 44 in. The maximum horizontal pull in one cable was found to be 35,600,000 lb., and the total vertical load on one tower (both posts) 35,000 tons. These were the initial data in the design.

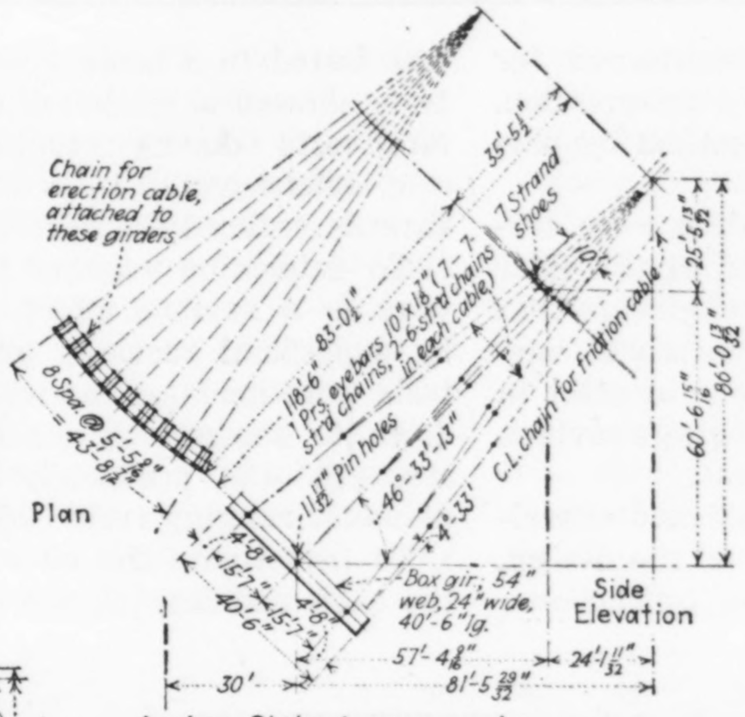
Under these various forces, the curve of the deflected tower was calculated, beginning with an assumed profile and section and working by trial and error, and from this the direct stresses and the bending stress due to longitudinal deflection of the tower were calculated. Secondary stresses in the tower due to bracing and portal action were well below 5,000 lb. per square inch. For dead-load, congested load, wind and temperature, in worst combinations, the sum of axial and bending stresses was held below 24,000 lb. per square inch, and the total, including secondary stress, below 27,000 lb. per square inch. This is for silicon steel.

For the cables, with a required minimum yield-point of 144,000 lb. per square inch of wire, the maximum allowable combined stress was set at 72,000 lb. Each cable contains 562 sq.in. of steel, and the maximum cable stress for extreme loading is 40,110,000 lb. (71,500 lb. per square inch; under working load 60,800 lb.). Secondary stress due to bending of the cables (mainly near the towers, where angular changes approaching 1 deg. may occur under extreme conditions) and inequalities of stress distribution through the cable are taken as being allowed for by the margin between wire strength and allowed working stress.

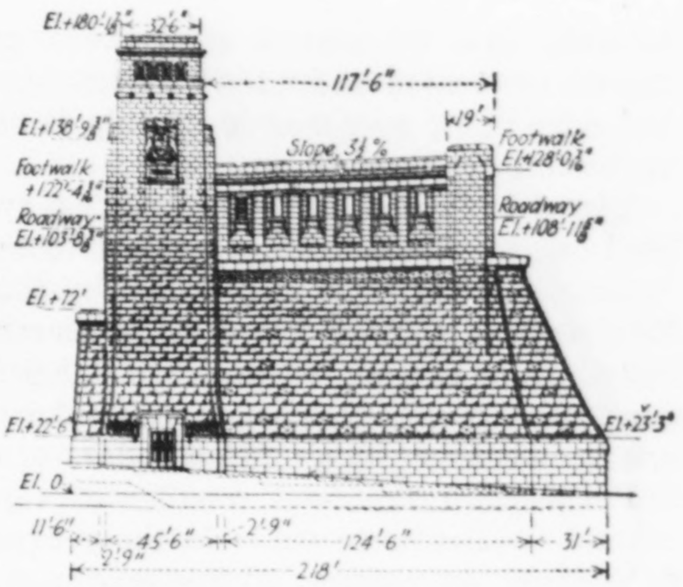
In connection with the cables, the design and con-



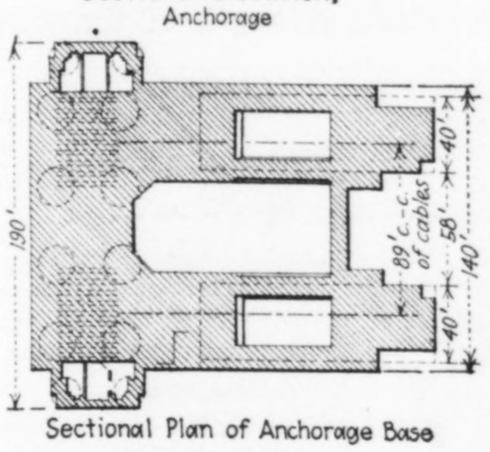
Sectional Elevation, Anchorage



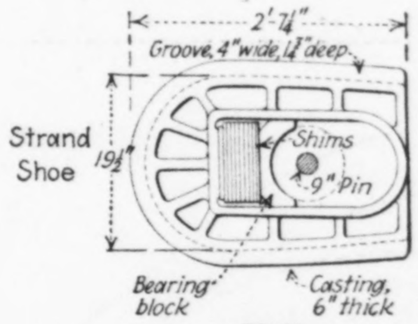
Anchor Girder Arrangement

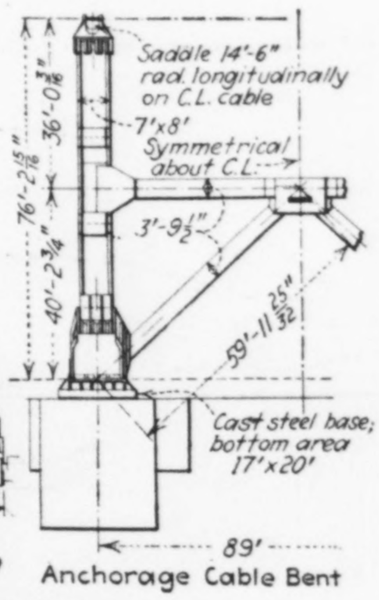


Complete Anchorage, Side Elevation

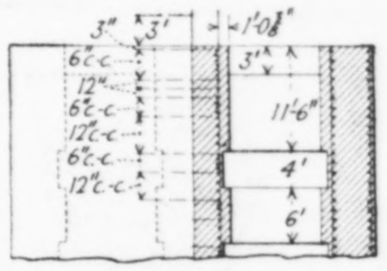


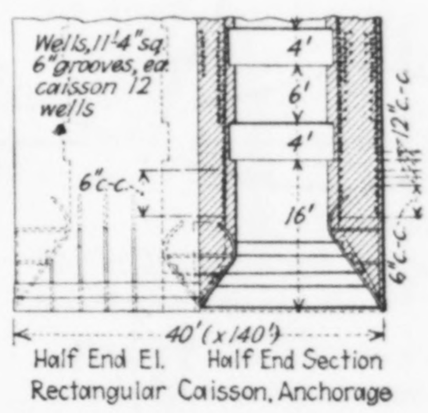
Sectional Plan of Anchorage Base



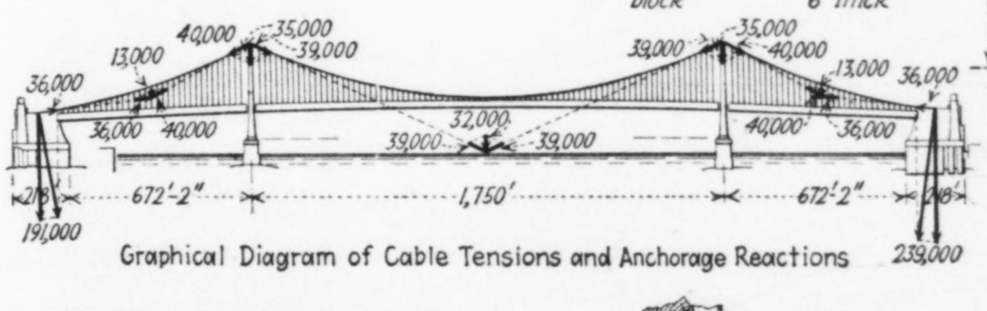


Anchorage Cable Bent

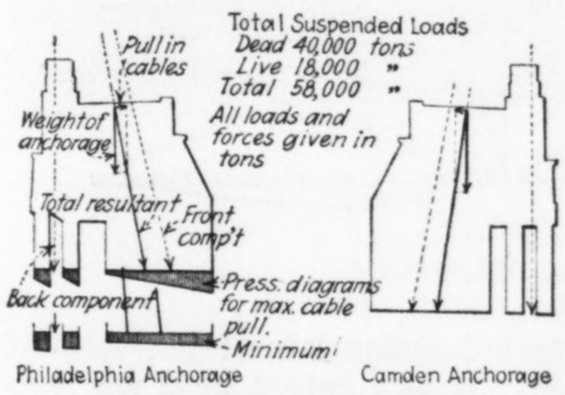




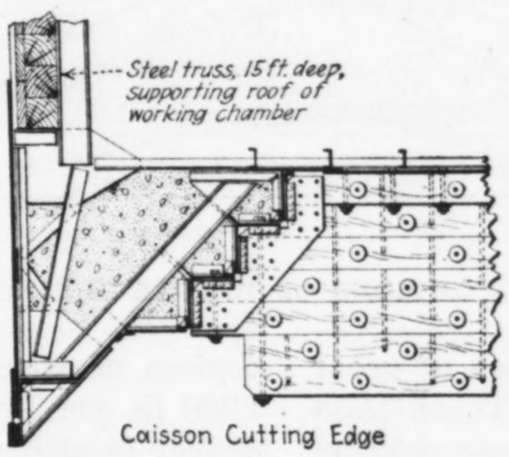
Rectangular Caisson, Anchorage



Graphical Diagram of Cable Tensions and Anchorage Reactions



Anchorage Foundation Pressure



Caisson Cutting Edge

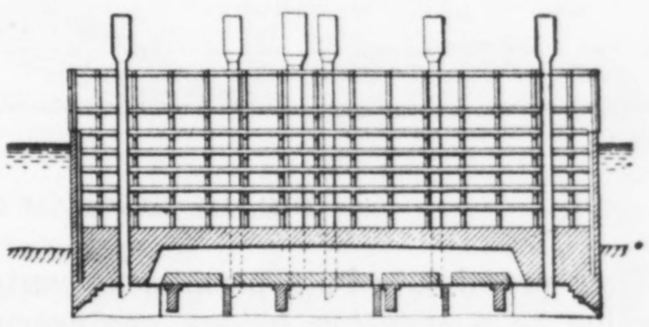

Longitudinal Section of Main Pier Caisson

FIG. 10—ANCHORAGE AND CAISSON AND CABLE WORK DETAILS, PHILADELPHIA-CAMDEN BRIDGE

struction details of the cable bands were important, as on some previous bridges it was known that cable bands had slipped downward along the cable and necessitated difficult repair operations. The solution was found in finishing the bore of the cable bands with a slightly rough cut ($\frac{1}{16}$ in. feed, using a roughing tool), fixing their diameter accurately to match the previously determined diameter of the compacted cable, providing cable-band bolts of adequate strength, and tightening these bolts to a definite tension or under a definitely known wrench pull.

The anchorage bents over which the cables passed downward to their sockets presented problems quite like those of the towers and were dealt with in similar manner. With only 76 ft. of length as against the 343 ft. of the tower, and with about two-thirds of the load per post, these bents nevertheless are subject to a $\frac{1}{2}$-in. bending under live-load, and therefore, like the towers, were built of silicon steel. The limited width available for each of these columns argued against the use of a masonry support (such as used in the Manhattan Bridge), since this would have been less safe than a steel post in case of chance transverse impact.

Because of the normal distributed character of highway traffic and the esthetic advantage of a shallow as compared with a deep stiffening truss, only a moderate degree of stiffening action was aimed at. The great weight of cables, trusses and floor, moreover, was available to resist local distortion. But since relatively large stiffening truss movements may occur under extreme conditions of loading it was desirable to use a material of high elastic limit. A steel of 55,000 minimum elastic limit was therefore specified for the truss chords, and under this specification a 3.2 per cent nickel steel was furnished. The working stress allowed in the chords is 40,000 lb. per square inch in tension and 35,000 lb. in compression. In the top chord the unit stress was reduced 3,000 lb. on account of the lateral deflection introduced by the bracketing of the sidewalk and the omission of overhead bracing. The web members were made of silicon steel, and in these 32,000 lb. was allowed in tension and 32,000 — 140 l/r in compression. The

laterals, also of silicon steel, were proportioned for 32,000 in tension and 30,000 — 100 l/r in compression, but something over 10,000 of this is absorbed by participation in chord stress.

The calculations of the stiffening system were carried out by the deflection method instead of the older "elastic method." It is reported by the engineers that this method showed lower truss stress by about one-fourth, and thereby saved some 3,000 tons of steel in the trusses alone, or, when tower and anchorage savings are included, nearly a million dollars of cost.

Verification Tests—A number of significant experimental studies were made in the course of the design, as a check on certain of the engineering conclusions.

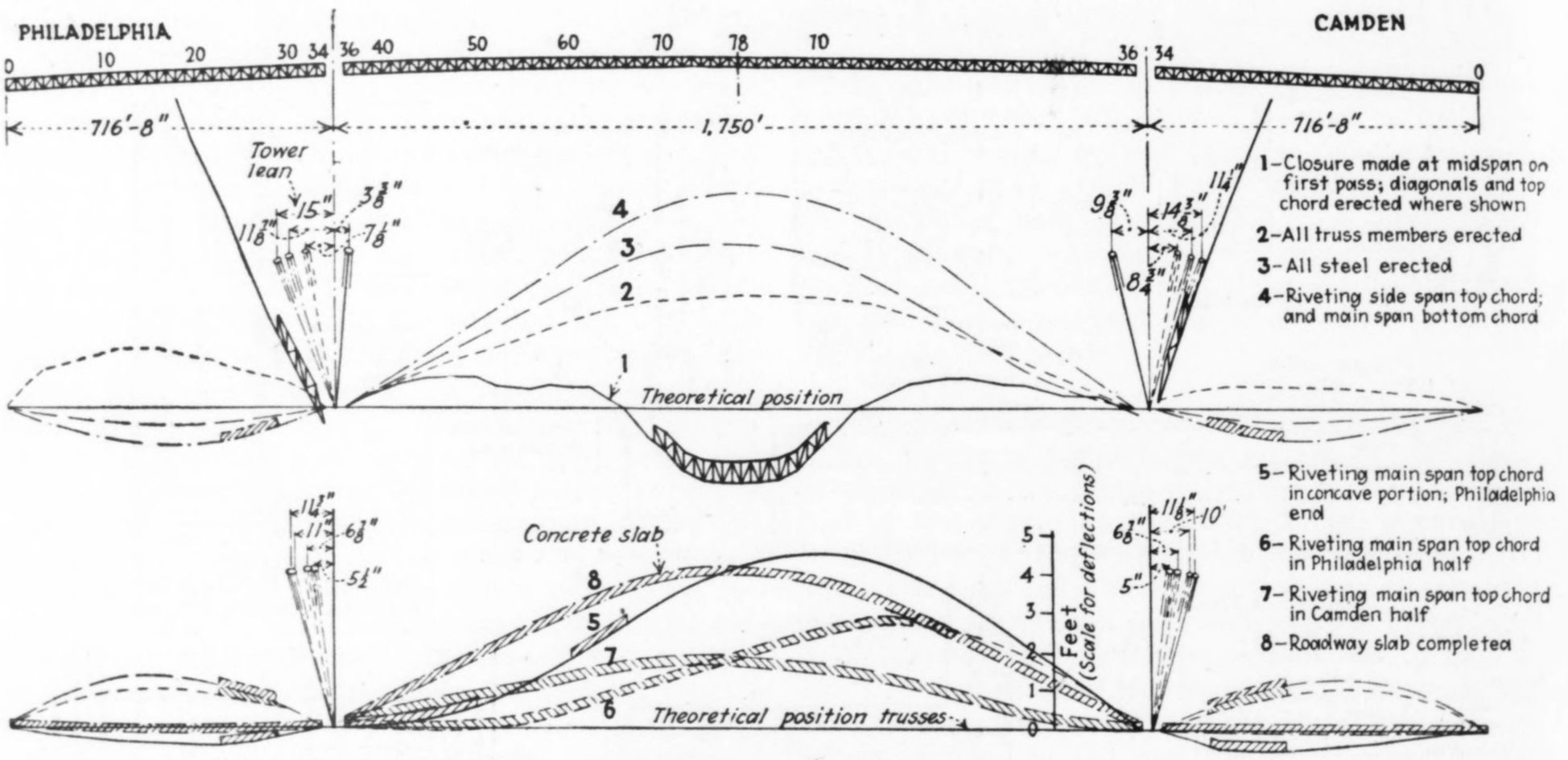


FIG. 11—DIAGRAM OF DISTORTIONS OF BRIDGE DURING STEEL ERECTION

They promise to be a permanent contribution of great value to suspension bridge engineering. These tests have been described somewhat fully in a paper read by Clement E. Chase before the Franklin Institute (*Journal* of the Institute, October, 1925).

To verify a vital element of the tower design, hence of the stability of the bridge, tests were undertaken to show whether the web plates are in danger of buckling or crinkling in their free width between edging angles. The design had been made according to an established rule which limits the width of a web plate in compression to 30 times the plate thickness, but the experimental basis for this rule was considered inadequate to justify its use in a structure of such extreme proportions as these towers without further verification. Fourteen web plates bordered by flange angles, 35 in. wide over angles (or 26 in. wide between inner rivets) and 10 ft. long were tested in endwise compression, the web plates varying from ⅜ in. to 1½ in. in thickness. They failed at loads of 43,000 to 46,000 lb. per square inch (one failed at 40,500), the effect of web thickness being somewhat obscured by the different physical properties of the steel in the thick and thin plates. The tests indicated that the rule is conservative and the towers amply safe.

The cable wire splice, consisting of a right-and-left-thread sleeve, ends of wires being mitered at 45 deg., was tested in a large number of specimens. Advance tests showed a minimum of 95 per cent efficiency, the later tests (during actual cable spinning) 97 to 98 per cent efficiency, in spite of the fact that the swaged threads reduced the wire section 18 per cent.

To determine whether the high strength of drawn wire is a surface effect (which might be destroyed by superficial rusting) or extends through the wire, tests were made on the whole wire and on wire ground down to successively smaller diameters. The tensile strength was practically unchanged throughout the diameter ranging from 220,500 to 236,000 lb. per sq.in.

To investigate the effect of lateral pressure (as in the cable saddles) on the tensile strength of the wire, a piece of wire was tested under lateral pressure of 4,300 lb. per lineal inch. It was only 1 or 2 per cent weaker in ultimate strength than a wire tested without lateral pressure.

Wires tested in tension while bent around an 18-in. sheave (simulating the strand shoe) broke in the straight portion and showed a strength identical with that of wire tested straight, between grips.

With regard to the doubt as to whether cable assembled in strands would develop nearly the full efficiency of the wire (George S. Morison 35 years ago found only 80 per cent strand efficiency), strands of 37, 80 and 306 wires were tested, showing 96, 95 and 94 per cent efficiency respectively. In one specimen of the second set all wires were spliced, in the other none, yet both gave exactly the same strength. The 6 per cent reduction shown by the 306-wire strand is believed to be greater than occurs in the cable, as the strand laid up for test (around strand shoes) could not be laid up with as uniform tension as was obtained in building the bridge.

To determine the flexibility of a section of cable and therefore permit estimating secondary stresses, a length of cable 9.4 in. in diameter tested as a beam proved to be almost as stiff as a solid beam under quite small loads, but under moderately larger loads was 15 to 45 times as flexible. An analysis made from the results

permitted an estimate to be made of the secondary stresses in the main cables of the bridge.

The friction of cable bands was tested on this 9.4-in. cable and later on a 30-in. cable. Friction tests of flat blocks of various degrees of roughness on pieces of wire had showed friction coefficients up to 25 per cent, but band-slipping tests on the two sizes of cable indicated a longitudinal resistance of about 60 per cent of the gripping force, presumably due to bulging. The bands were designed to utilize 15 per cent friction.

Since the suspender ropes pass around the cable bands, tests were made of such ropes bent around sheaves, no previous data on wire-rope-center cable tested in this way being available. Tests around a 32-in. sheave showed a loss of strength of 11 per cent as compared with rope in direct tension. Suspender rope was also tested for modulus of elasticity, which was found quite variable depending on the previous history of the rope. Cable was tested for degree of compaction and other minor tests were made, all of direct assistance in the design, but of less importance for record.

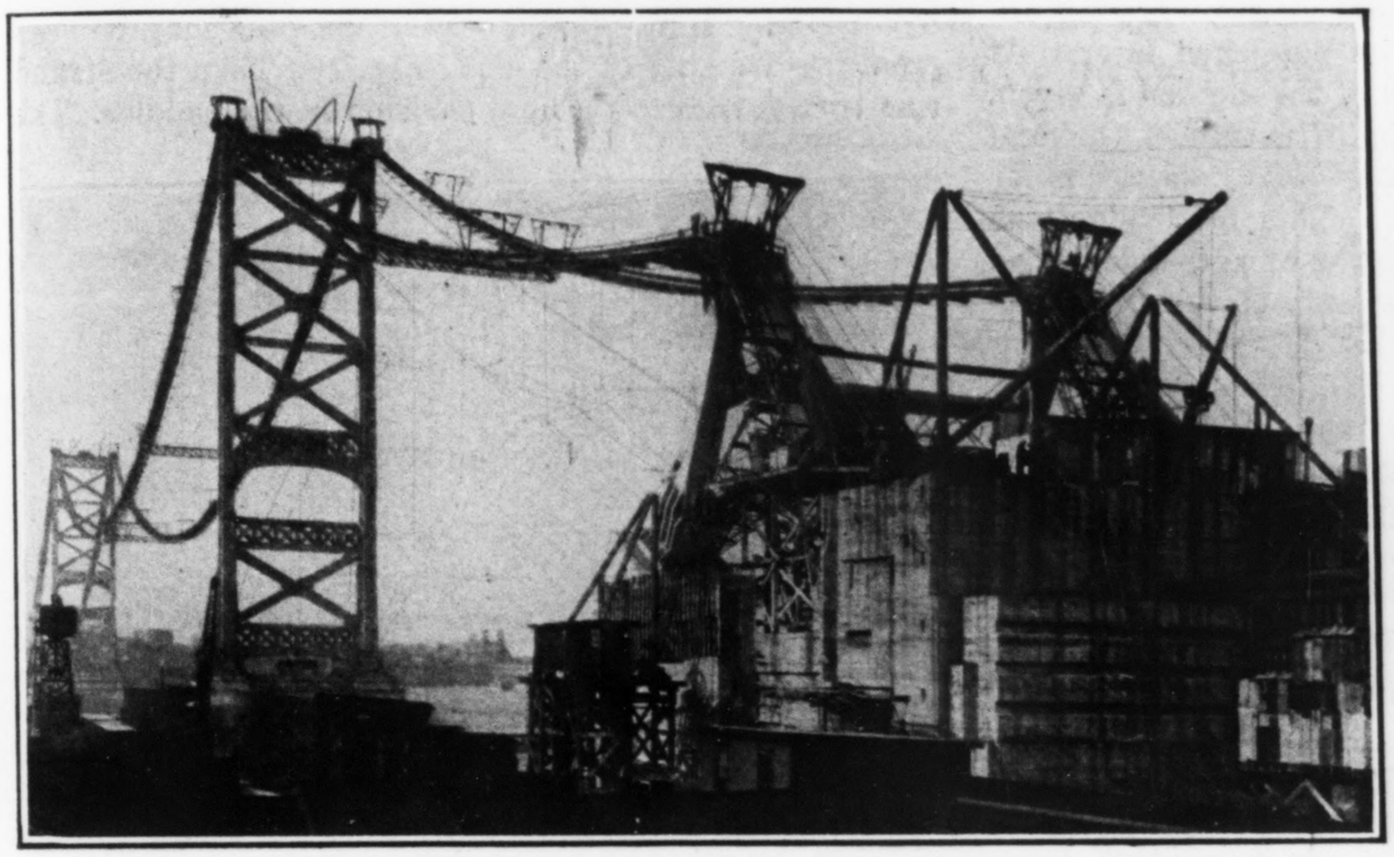


FIG. 12—CABLE-SPINNING UNDER WAY

A footbridge under each cable, carried on six 2¼-in. wire ropes, carries the guide frames, for an endless traveling rope over each cable position, by which four wires are laid at each trip from anchorage to anchorage.

Some Features of the Construction Work—Before setting the tower shoes on the completed granite top of the main piers, the pier tops were chipped level on the entire area to be covered by the 4-in. steel slab. This chipping was done by first cutting down a number of distributed small spots to the correct elevation, as determined by engineers' level, then cutting down channels to connect these spots and working them to the true level line, and finally connecting the areas between these lines. The finished surface is true to $\frac{1}{32}$ in.

Erection of the towers, carried out by the Bethlehem Steel Co., contractors for the fabrication and erection, went forward with little complication other than that introduced by the great weight of the sections and the necessity of entering the vertical splices between the four sections into which the post cross-section was divided for fabrication purposes. The lowest sections of the tower were erected by a floating derrick, but after the first panel of sway-bracing between the posts had been erected, vertical I-beams were attached to this bracing, to serve as a runway for a vertically traveling derrick platform capable of moving up with the advancing steelwork, and the derricks on this platform placed all the remainder of the tower steel.

Some difficulty was experienced in making the vertical splices in the base sections of a Camden tower post. Though the joints had been machined true and had been fitted together in the bridge-shop yard, it seemed that one of the erection pieces was slightly drawn out of shape in riveting the 2-in. base plate to it, and then could not be forced true by all the loading at hand. The base plate was finally cut loose, and the section could then be drawn readily into good contact and match.

Riveting the tower sections presented the problem of supplying hot rivets to the inside of the tower. This was handled most ingeniously by heating the rivets at the base of the tower and sending them up to the riveters by a pneumatic delivery tube.

In the trusses and other heavy members of the bridge the rivets are generally 1 in. in diameter, and many large grips occur, especially in the towers. All rivets are of soft steel, 52,000-60,000 lb. per sq.in. ultimate strength, even in the nickel-steel and silicon-steel members.

As erected, the tower was plumb. In the finished condition of the bridge, it was desired to have the posts bent back 5 in. from plumb position, under normal dead-load and abnormal temperature, to equalize stress conditions between the opposite extremes of loading. Thus in connection with the stretching of the cables from anchorage toward mid-span as their own weight and the weight of the suspended superstructure came upon them, meant a progressive movement of the tower top toward the river as the erection of the cables and suspended structure should proceed, and therefore required either that the saddles be slipped forward on the towers at some stage of the work, or that the towers be pulled back before cable spinning started. The latter course was adopted, and by means of special pull-back ropes fastened to tower tops and leading back to the anchorage the towers were pulled back 11 in. As each strand of cable was completed and was transferred from its temporary sling support over the tower-top saddle to its permanent place in the saddle, it was shifted riverward of its true position, thus raising the land-span curve higher than its final position, so as to take over part of the pull-back force on the tower and put the cable in the proper position for equalization of the subsequent stretch of the cable.

Spinning Cables—Spinning the cables was done by the method which has been long standard in suspension-bridge construction, in fact since John A. Roebling's early work, and was nearly identical with the procedure followed in the cable spinning of the Bear Mountain bridge. However, as done here it represents the highest point of a perfected art, and remarkable speed and efficiency of working were attained. A more detailed

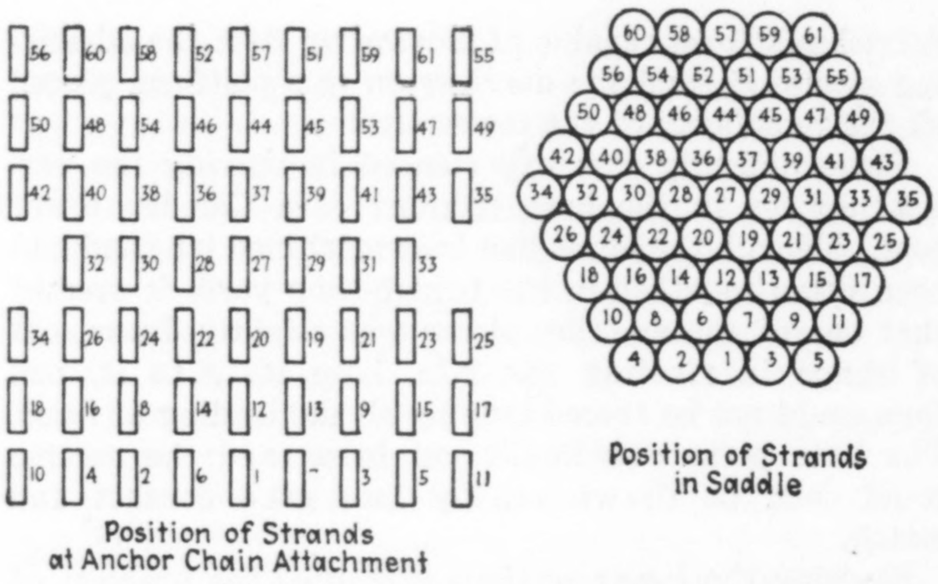
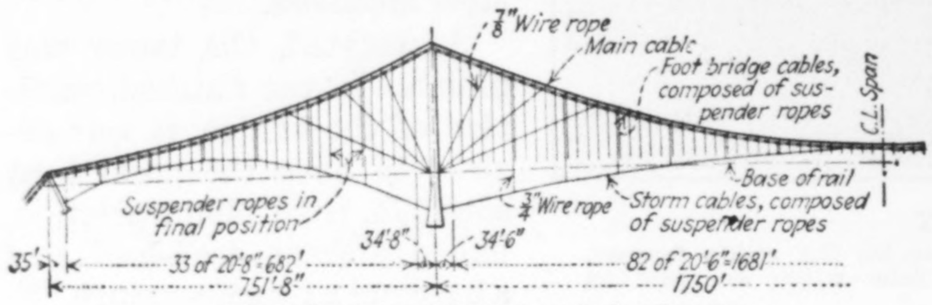


FIG. 13—ARRANGEMENT OF STRANDS IN CABLE

account of the procedure will be published later. At present only a sketchy indication of the method is given.

A temporary footbridge was built just under the final cable position, with two parallel 10-ft. platforms or walkways, one under each cable. This footbridge was carried by four sets of three lengths of suspender rope, two sets (or six 2¼-in. ropes) for each of the two walkways of the footbridge. This bridge was stayed by inverted catenaries or storm cables hitched to the tower bases. Above the footbridge two endless traveling ropes were arranged, one over each cable position. Each traveling rope extended from anchorage to anchorage, over supporting sheaves on the tower tops, and passed around horizontal sheaves at the anchorages. Two sheave carriages were fastened to each



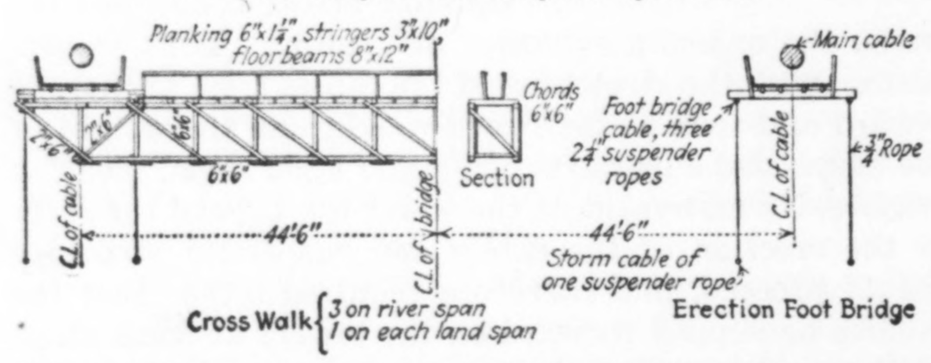


FIG. 14—DETAILS OF FOOTBRIDGES FROM WHICH CABLE SPINNING WAS DONE

traveling rope, and as this rope moved the carriages traveled across the river. A bight of cable wire from a large reel of wire was placed over the traveling sheave and was carried by it across the river, laying one dead and one live length of wire into a strand. Thus four strands, two in each cable, were in construction simultaneously. As soon as a bight of wire was across, and the bight was removed from its sheave and looped around the strand shoe at the far end, both wires were adjusted to the sag of the strand, the adjustment being first carried out from the initial strand shoe progressively along the length of the dead wire, and then backward across the river along the live wire. The wires when adjusted were clamped to the strand shoes, and the traveling sheave moved back over the river with another bight, taken however from a reel at the anchorage it was now leaving, in fact the reel whose wire it had carried on the previous trip in that direction. Thus each carriage was working on two strands.

Usually one one-way trip required about 8 min., so that 60 wires per hour could be laid under ideal conditions. The best day's work done was 578 wires, but generally it took four days for a set of four strands, or 1,224 wires.

When a strand was completed, it was seized every two feet with flat band clips of soft steel, devised by H. D. Robinson as an improvement on the wrapped wire seizings used in former cable works, and then was adjusted to correct sag as indicated by a previously strung guide wire of carefully adjusted length between anchorages. This adjustment of the strand was made by inserting shims in the strand shoe, as may be seen from the drawing of the shoe, Fig. 10. Then the strand was transferred to its final position in the saddles. The

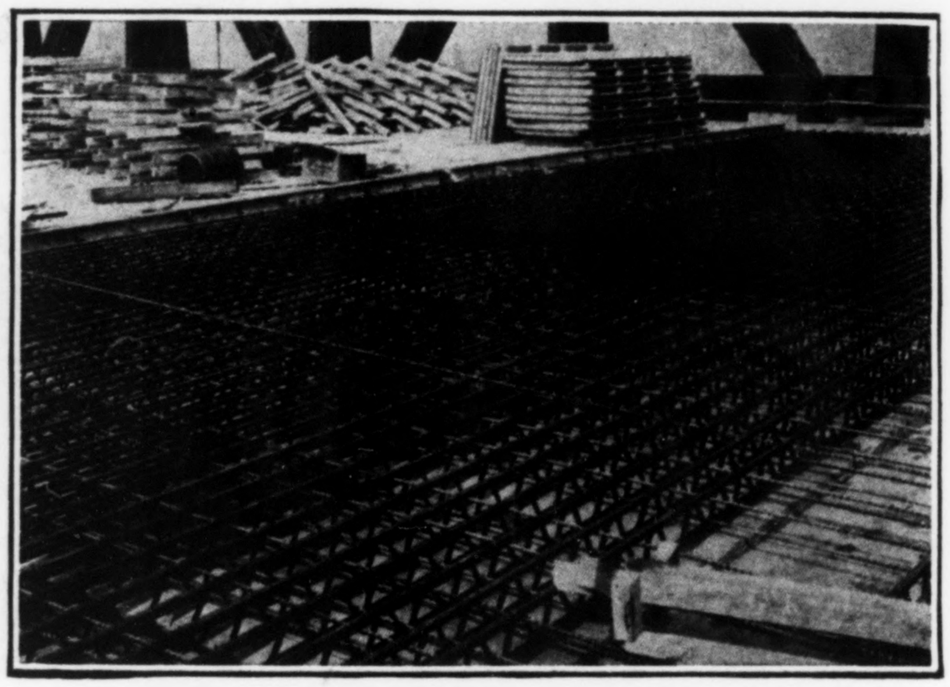


FIG. 15—FLOOR SLAB REINFORCEMENT
Trusses 4½ in. deep spaced 6 in., set transversely, and longitudinal temperature rods top and bottom.

relative position of the strand shoes and the corresponding strands in the hexagonal section of the cable in the saddles is indicated by a sketch.

After completion of the cable the assemblage of strands was compacted and formed to true circular section by a squeezer made of six 30-ton hydraulic presses seated radially on a ring and bearing on the cable through segmental shoes. Then the suspender bands were placed, the suspender lengths were carefully computed from the known position of the cable and the modulus of elasticity of the suspender rope, the lengths of suspender rope were cut to the required dimensions and socketed and placed around the band, and the structure was ready for the erection of the suspended steel work.

Cable construction covered the period from May to December, 1924, except for the compacting. The 37,332 wires of both cables were laid up in 24 weeks. The work was done by the Keystone State Construction Co., using wire manufactured by the American Cable Co. at the plant of the Page Steel & Wire Co. at Monessen, Pa.

Stiffening Truss Erection—This work, done by the American Bridge Co., involved a number of consecutive erection stages, to avoid heavy partial loading. Work was begun by 106-ft. booms rigged to the towers, which placed the posts, chords, floorbeams and stringers of three or four panels, on which traveling derricks could be erected.

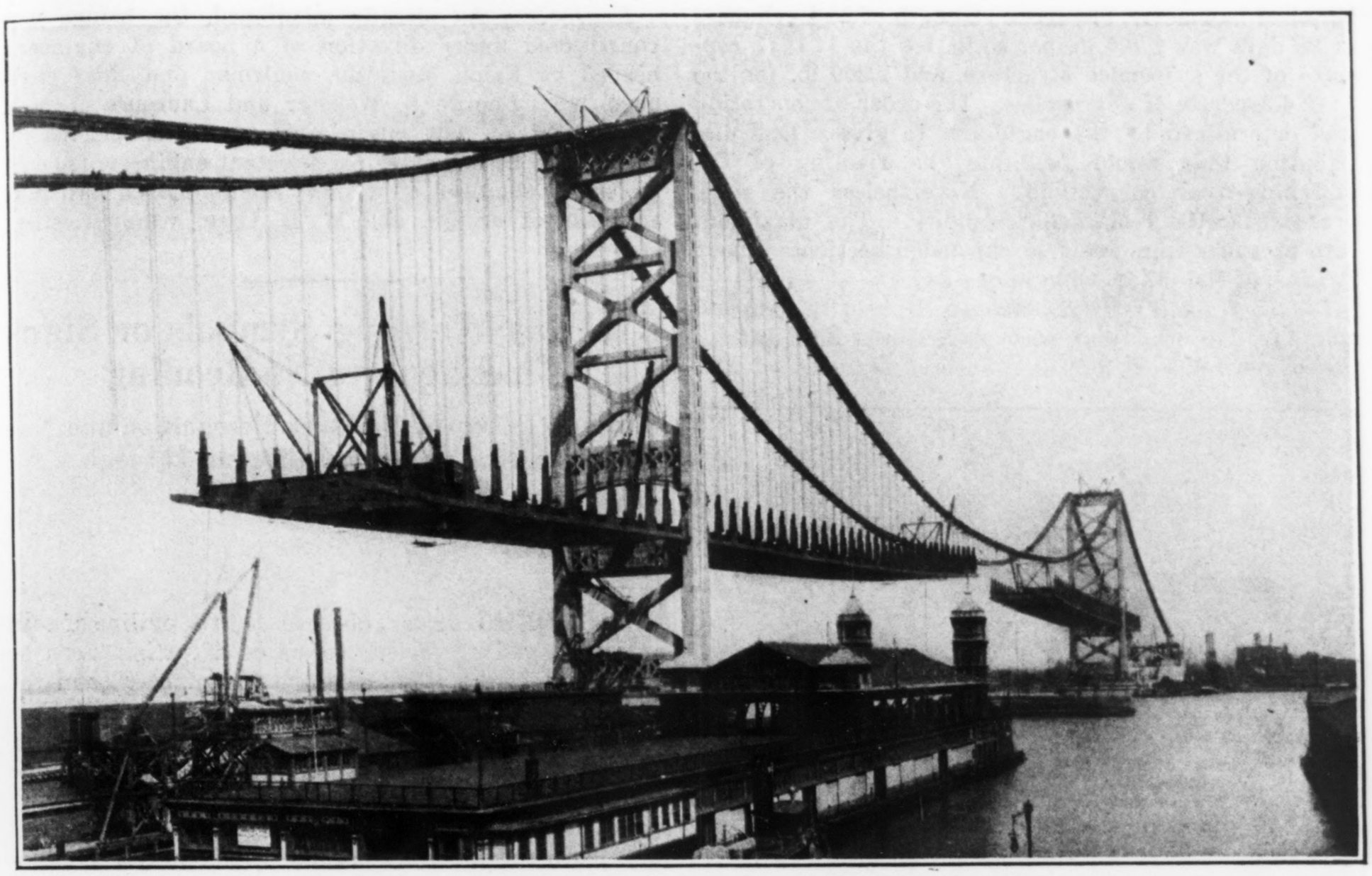


FIG. 16—FIRST OPERATION IN ERECTING THE STIFFENING TRUSSES
An abnormal distortion of shape occurs temporarily during the steel erection, due to the unbalance of loading. The construction procedure was so controlled as to minimize this distortion.

After this preliminary, in the first stage a traveler moved out from each tower toward midspan and another toward the anchorage, and placed the least weight of material that would carry the travelers; it erected the posts (hanging them to the suspenders), the bottom chord, the floorbeams and enough stringers to carry the travelers, except that near midspan the diagonals and top chord also were put in place. The largest distortions occurred in this stage, as it was necessary to erect 7,000 lb. of material per lineal foot of bridge, while the cables and accessories weighed only two-thirds as much. In the second stage the travelers returned, setting the truss diagonals. In the third, moving outward again they erected the top chord, while in the fourth pass they placed the remaining floor material, the sidewalks and the outer track.

Throughout these moves the structure was constantly changing shape (see diagram, Fig. 11), and it was a matter of most careful forethought to arrange conditions so that splices could be made. The diagonals were mainly connected during the fourth pass, but the chord splices were pinned and riveted while the floor-slab concrete was being poured, in order to utilize the arbitrary load distribution that could thus be obtained. The bottom chord of the main span and the top chord of the side span were connected first, while pouring part of the side-span slab (which raised up the main span). The other chords were riveted progressively during the concreting of the main-span slab, which proceeded outward from the west tower across the span, but to connect the splices near the center an extra loading of 300 tons had to be placed at midspan to weigh down the span curve. Also, the web of one splice was so refractory that recourse was had to lengthening the chord members by heating with oil torches to bring the splice faces to bearing.

A feature of the erection was the necessity of disconnecting and reconnecting half the suspenders in setting the top chord in place. The suspenders engage the posts, and pass through holes in the top chord. As the top chord was built in two-panel lengths the intermediate suspender had to be taken off the post and passed through the holes in the chord before it was finally in place. To do this a special pulling rig was built, by which the suspender was pulled down, away from its bracket seat on the post, and released by taking out the adjustment shims provided under the bracket; later it was reconnected by a similar procedure.

Altogether 18,500 tons of steel were erected in the trusses and floor, which was done in less than 9 months. As noted, steel erection and floor concreting overlapped, and both operations were comprised between March 3, 1925, when the first steel was erected, and Dec. 8, 1925, when the last section of slab was poured.

Construction of the floor slab was carried out by F. J. Boas. Wooden forms designed for repeated re-use were provided. They were handled from a scaffold one panel long, the full width of the bridge, supported by beams passed through manholes in the floorbeam webs. The forms could be stripped and moved ahead very rapidly. Concrete was hauled out on the bridge from a central mixing plant at the west tower by means of one-yard dump-body motor trucks running on plank roadways laid over the stringers. A somewhat wetter concrete than desired for maximum strength was used, to give smooth soffits, with excellent results in this respect. The forms were vibrated by pneumatic rub-

ber-shod hammers. The mean strength of test cylinders at 28 days was 2,700 lb. per sq.in. for the 1:1½:3 concrete of the suspended structure, and 2,200 lb. for the 1:2:4 concrete of approaches. The order of concreting was determined by the engineers, to give a load distribution that would facilitate the riveting of the stiffening-truss connections. Nevertheless the work proceeded with remarkable rapidity. The maximum rate of concreting was five expansion sections (about 200 ft.) of slab 57 ft. wide in one day.

Lighting and Traffic Provisions—Unusually complete and effective provision was made for traffic safety, traffic control and lighting. These features of the

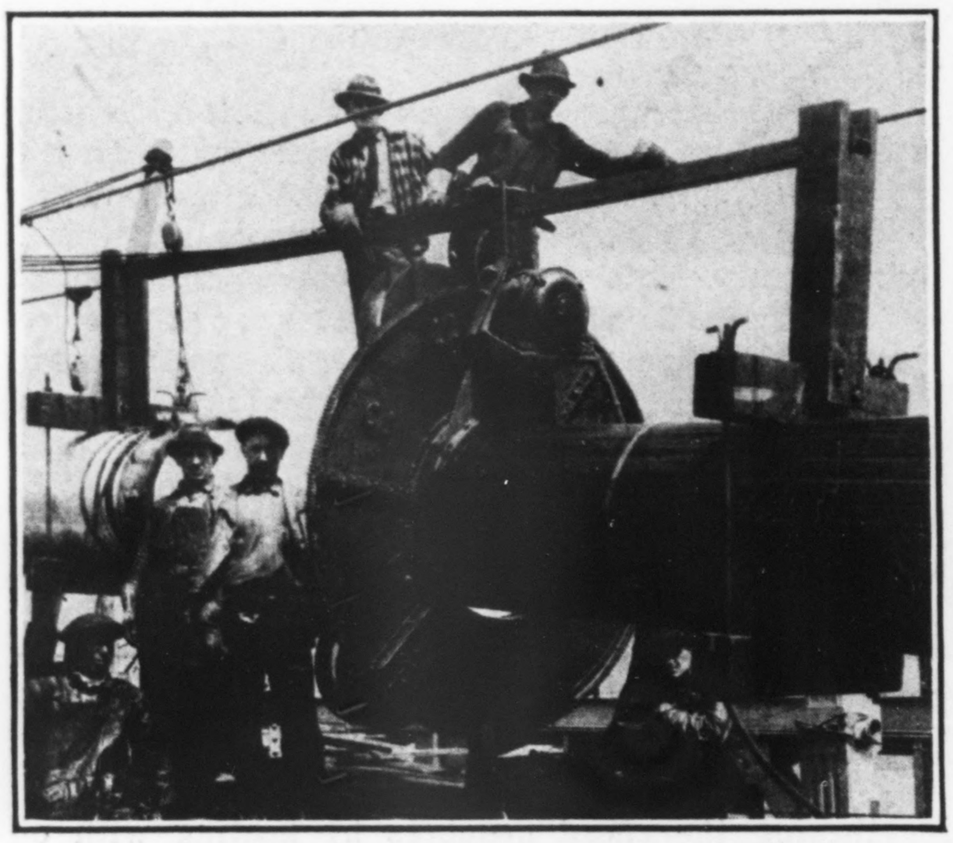


FIG. 17—WRAPPING THE CABLE BY POWER
One of the most important finishing operations. A soft steel galvanized wire is wrapped in a close layer around the 30-in. cylinder of cable wire.

bridge will be dealt with later, in view of the exceptional character of the bridge due to its size and its toll operation. Briefly, a very heavy two-level steel curb is provided at the edge of the roadway, and outside and above this is a heavy pipe railing. The track spaces are not in use or fitted for service at present, as no arrangements have been made for street-car operation. Therefore the inner track space is floored with galvanized grating and curbed with steel-mesh fencing, as a further guard against vehicle accidents. Lighting is provided by a lamp standard (or bracket from the trusses) every 120 ft. on either curb line, with 1,000 cp. per lamp.

To systematize traffic, white-paint lane borders have been painted on the roadway, and the middle line is protected by cast-steel buttons about every 100 ft. Toll collection is done wholly at the Camden end, where a roadway widening between the end of the approach incline and the terminal plaza gave room for installing seven tollhouses commanding five two-lane passages, two being fitted with scales for check weighing of trucks.

Traffic—Vehicle traffic turned to the bridge in unexpected volume as soon as it was opened. During the first two months about 1,500,000 private passenger cars crossed, and in addition there was a bus traffic reaching 1,700 per day and a truck traffic of nearly 1,000. This exceeds the most liberal estimates made in advance of opening, and makes certain the business success of the undertaking.

Engineers—As already mentioned, the bridge was constructed under direction of a board of engineers, headed by Ralph Modjeski, chairman and chief engineer, with George S. Webster and Laurence A. Ball as associates. The engineering staff was headed by Clement E. Chase, principal assistant engineer, Leon S. Moisseiff, engineer of design, Allston Dana, assistant engineer of design, and M. B. Case, senior resident engineer.

Highway Warning Symbols or Signs That Require No Reading

Outlines Designed for Quick Recognition and No Lettering to Become Illegible Through Disfigurement or Glare

By E. T. Scott
Los Angeles, California

AUTOMOBILE drivers observe that the outline of road signs and of various forms of advertising can be made out clearly at much greater distances than the words or symbols painted thereon can be read. This suggested to the writer that outline symbols would be a more effective warning to the motorist than the printed sign, because the former would convey meaning more quickly and thus would give the driver more time to prepare for what lay ahead. Eight outline symbols were accordingly made in the shapes shown in the accompanying drawing, Fig. 1. These were set up along the roadway and photographed under day and night

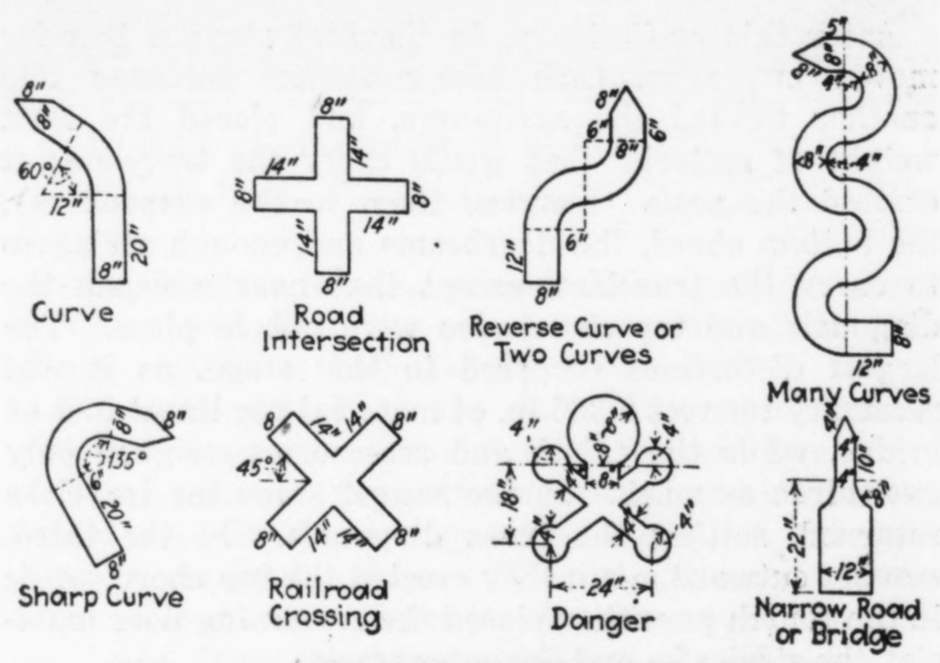


FIG. 1—DIMENSIONS OF HIGHWAY OUTLINE WARNING SYMBOLS

conditions, with the results shown in the accompanying pictures.

The symbols were cut out of sheet metal and to the metallic outlines three coats of ordinary white paint were applied. They were mounted on posts painted black in order that the outline would be kept distinct and were set up along the highway where they could be compared with the usual warning signs. The night pictures were made using only the illumination afforded by the headlights of an automobile stopped on the highway 75 to 200 ft. from the outline symbols.

While the pictures largely speak for themselves it may be noted that in Fig. 2 the symbol indicating a curve is instantly recognized although the curve on a standard type road sign alongside can hardly be made out and the lettering is altogether illegible.