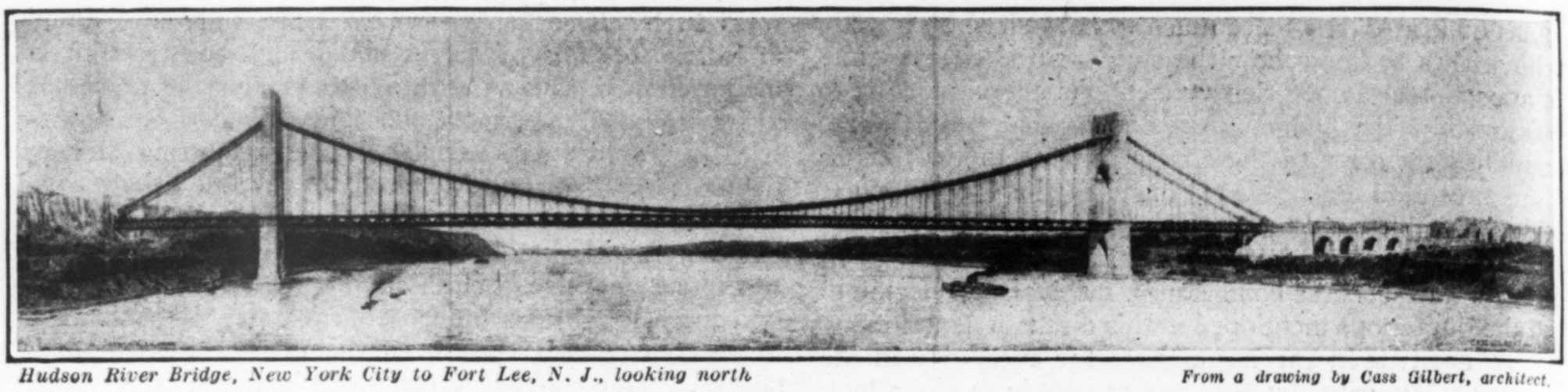


Hudson River Bridge, New York City to Fort Lee, N. J., looking north — *From a drawing by Cass Gilbert, architect.*

Design of 3,500-Ft. Suspension Bridge Across Hudson River

Huge Public Toll Bridge to Connect New Jersey with New York City—To Be of Suspension Type, Using Either Wire or Eyebar Cables—Stiffening Trusses and Lower Deck Will Be Omitted in First Stage of Construction—Long-Panel Floor Framing

(With insert plate)

THREE months ago construction began on what will be by far the largest bridge in the world. It is a highway crossing over the Hudson at New York City, connecting the city with the suburban region of northern New Jersey, and extends from Fort Washington Point, in the northern part of the city, to the borough of Fort Lee on the New Jersey side. It is to be a suspension structure of one span, 3,500 ft. between main towers, nearly double the span of the 1,800-ft. Quebec cantilever and exactly double the 1,750-ft. Camden-Philadelphia span. Although a public structure, it is to be operated on a toll basis.

The Port of New York Authority is constructing the bridge at the direction of the legislatures of the states of New York and New Jersey. The work is financed by an initial appropriation from the two states and an issue of Port Authority bonds. The cost of the structure will probably approach $75,000,000 in its final stage, but it is thought that only about $55,000,000 to $60,000,000 will need to be spent by the time the bridge is opened for traffic, with the initial section of the ultimate roadway width in place.

During three years the project has been under study by the engineering staff of the Port Authority, at the specific direction of the two states. Following a favorable preliminary report a year and a half ago, the legislatures in 1926 made a first appropriation of $1,000,000 each (with the declaration that four subsequent appropriations of similar amount would be made available), to form the nucleus of the financing, and the Port Authority was directed to proceed with the construction of the bridge on the basis of these funds and an issue of its own bonds secured by the toll revenues to be derived from the operation of the bridge. Just as in the case of the two bridges connecting Staten Island and New Jersey across Arthur Kill, which the Port Authority had previously been directed to build, and which are now nearing completion, the state specified that traffic over the bridge shall pay tolls to cover the interest and amortization of the bonds and return the state contributions. Following this legislation, the Port Authority last December obtained approval of the War Department for its proposed span and clearance height, and immediately thereafter authorized an issue of $60,000,000 of bonds and sold a first lot of $20,000,000 to a banking syndicate on a basis of approximately 4¼ per cent.

Since that time the development of final plans has been in progress, and early in the current year had progressed far enough to permit letting a contract for the west or New Jersey main pier. This pier is now under construction, as are also the anchorage and approach cut in the Palisades on the New Jersey side. Development of superstructure plans has now progressed so far that a call for bids upon the entire steel superstructure is expected to issue within the current month.

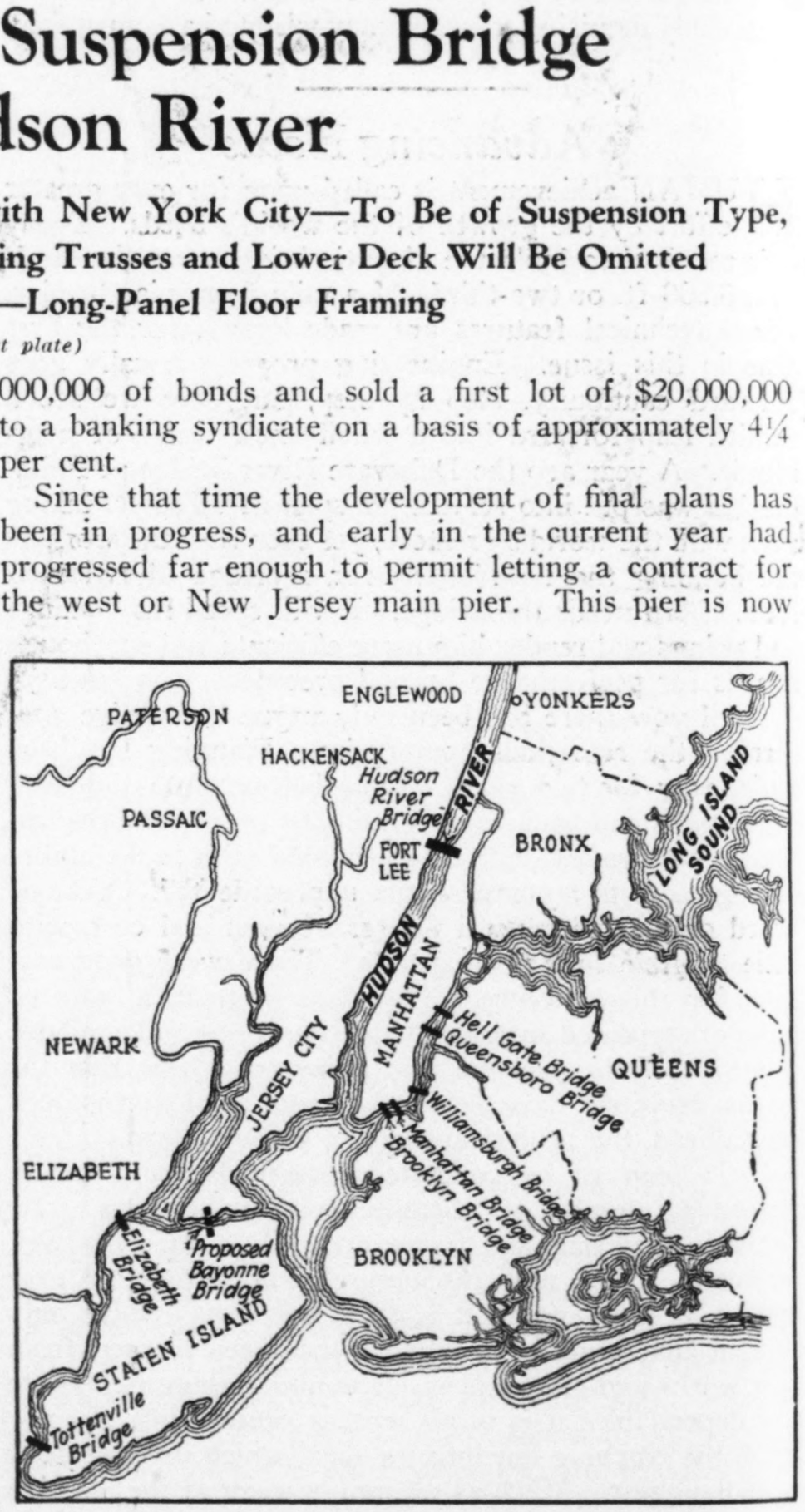
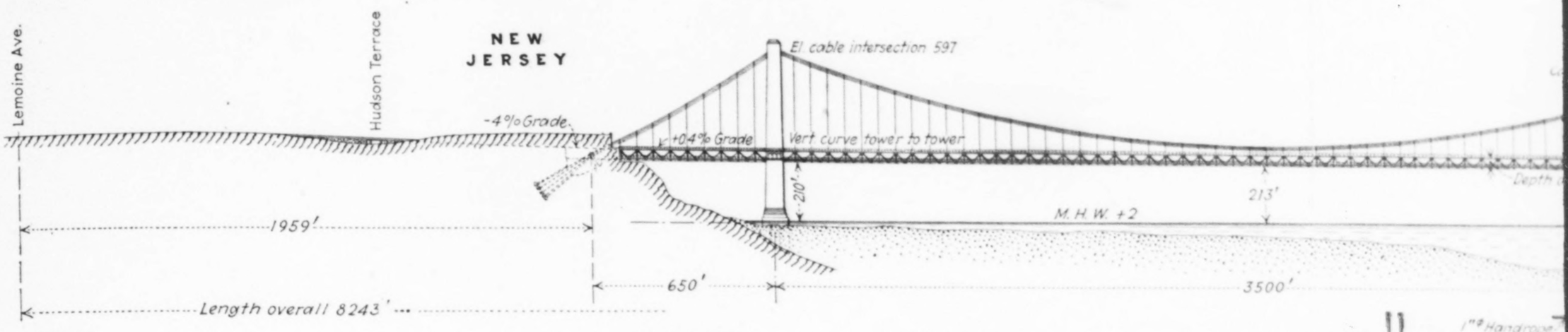
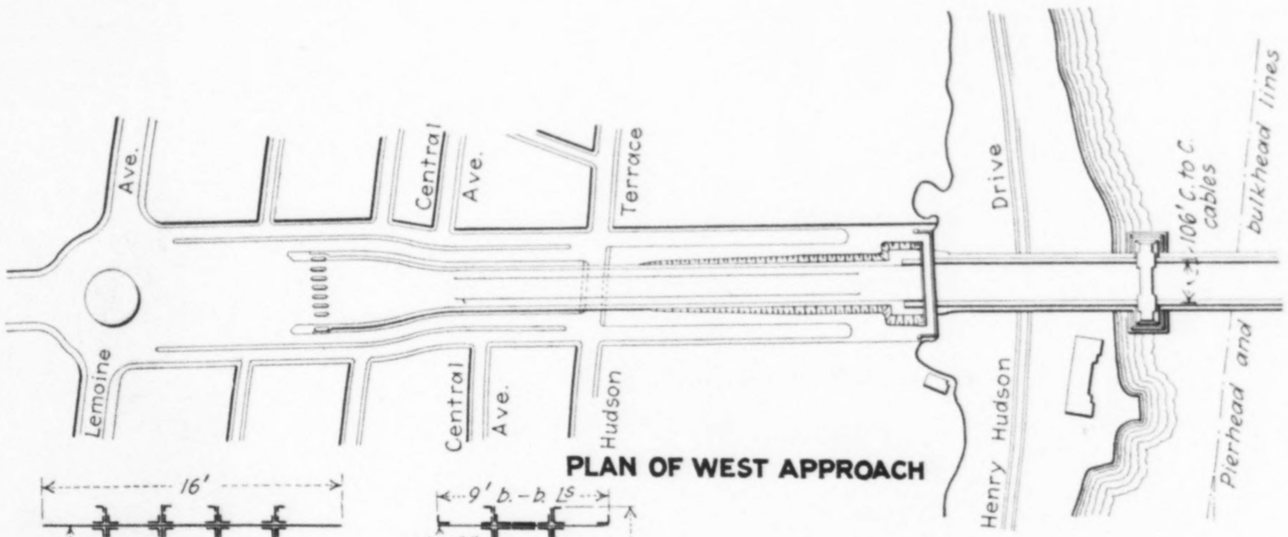


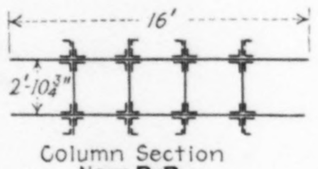
FIG. 1—HUDSON RIVER BRIDGE IN RELATION TO NEW YORK METROPOLITAN DISTRICT

Genesis of Project—The Fort Lee bridge project has a long historical background. Bridging the Hudson in its wide reaches past New York City by a great single-span structure has been advocated and studied for more





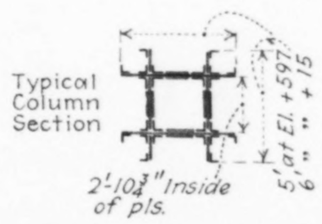
PLAN OF WEST APPROACH

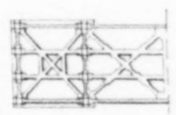


Column Section Near **B-B**



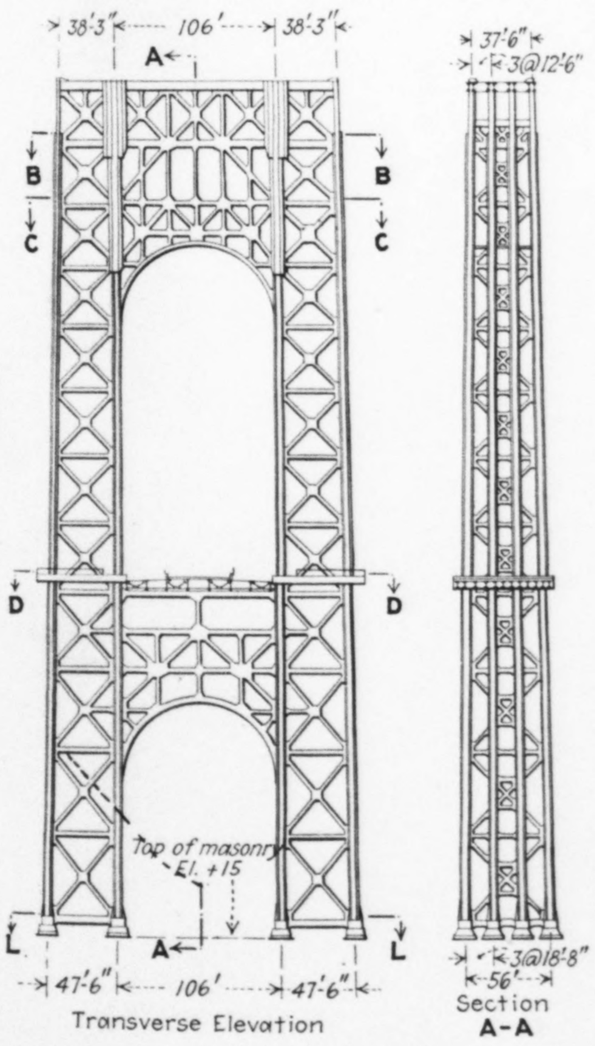
Column Section at **C-C**





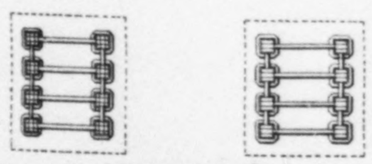
Half Section **D-D**

TOWER COLUMN SECTIONS



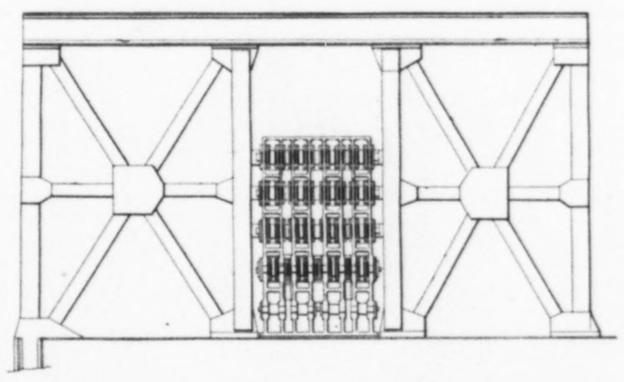
Transverse Elevation

Section **A-A**

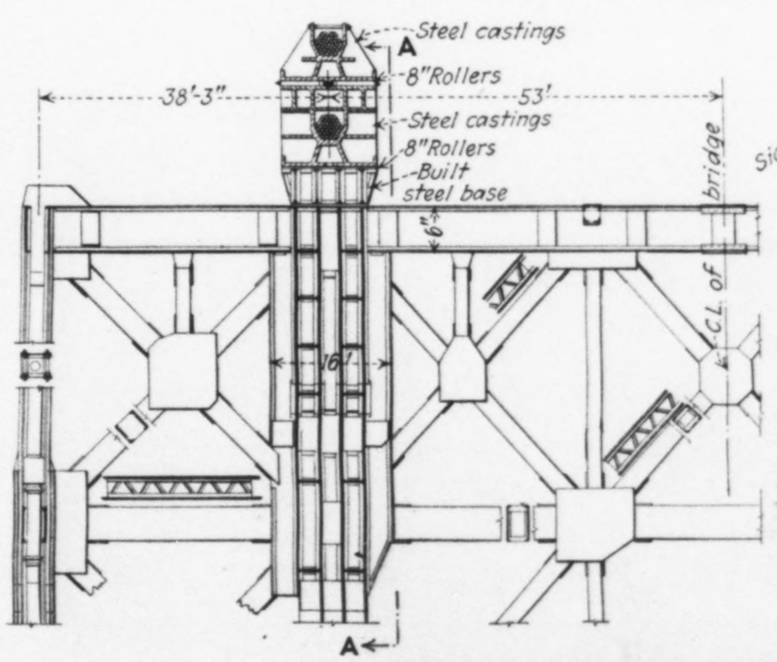


Section **L-L**

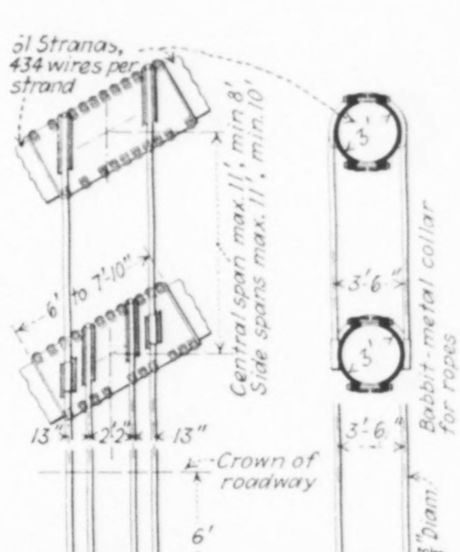
ARRANGEMENT OF SKELETON TOWERS
(First Stage)



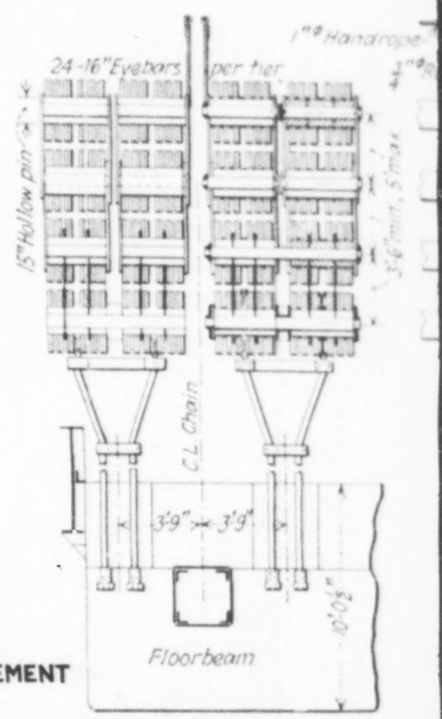
SUPPORT OF EYEBAR CABLE IN TOWER



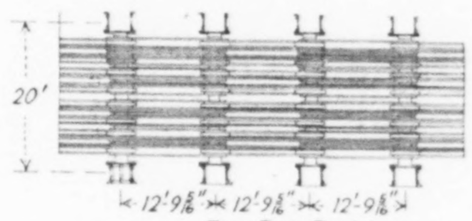
SUPPORT OF WIRE CABLE IN TOWER



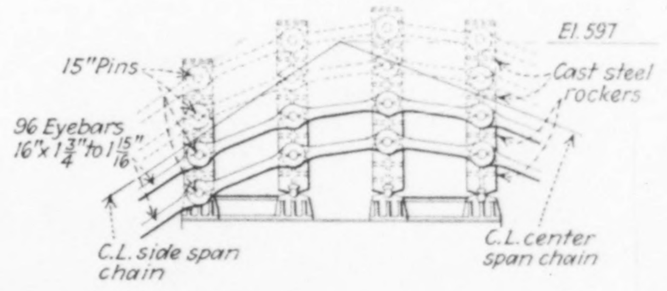
WIRE CABLES AND SUSPENDER ARRANGEMENT

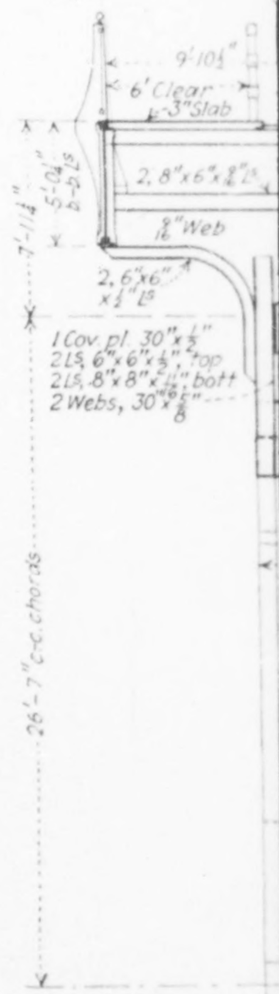


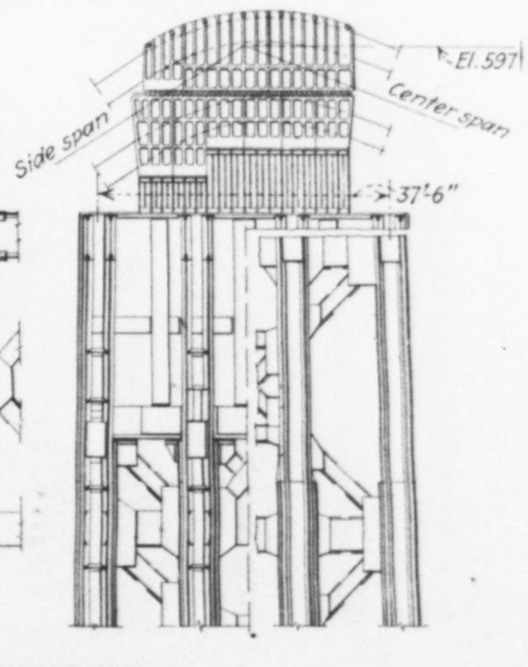
CROSS-SECTION OF EYEBAR CA



Eye-Bar Packing of Tier 2



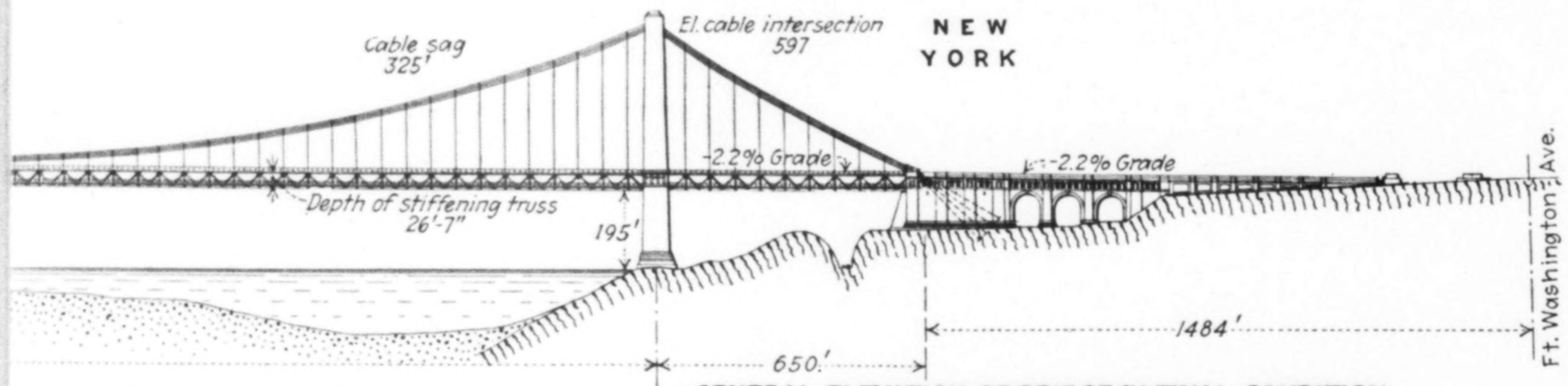




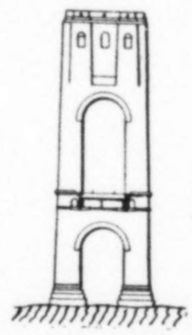
SUPPORT OF WIRE CABLE IN TOWER

Principal Construction Features of 3,500-Ft. Suspension Bridge Across the Hudson Rive

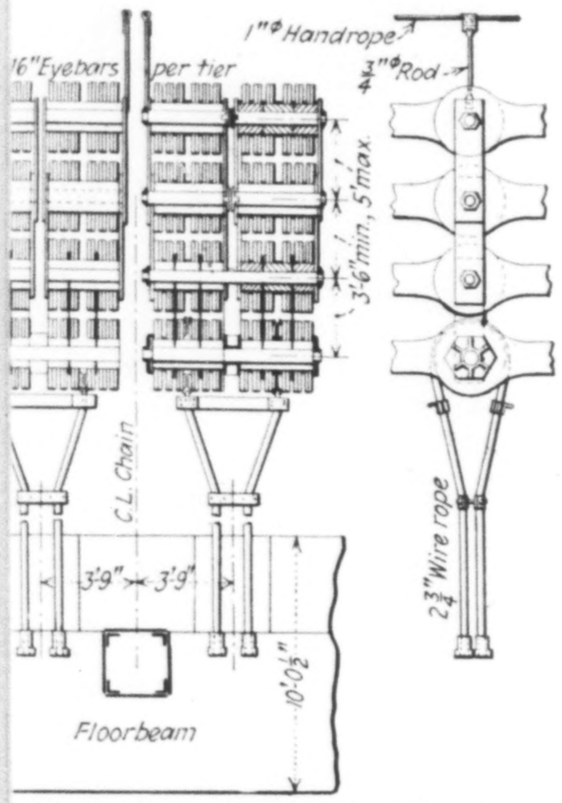
To accompany article, "Design of 3,500-Ft. Suspension Bridge Across Hudson River"



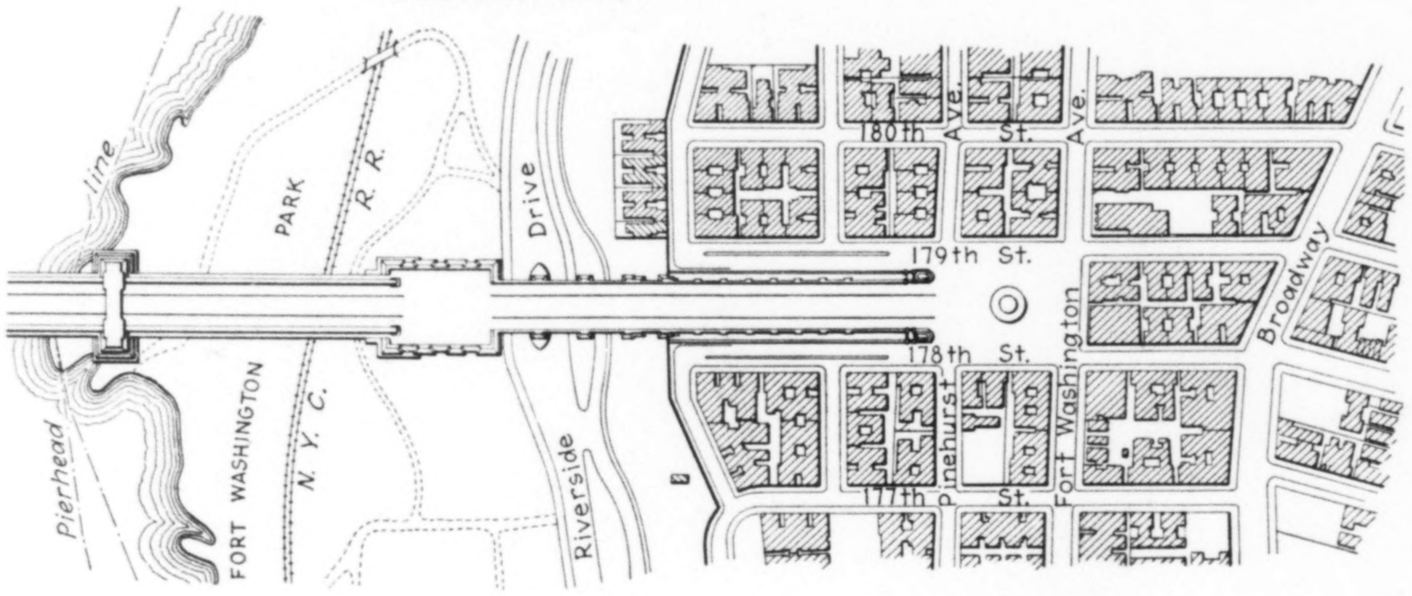
GENERAL ELEVATION OF BRIDGE IN FINAL CONDITION LOOKING NORTH



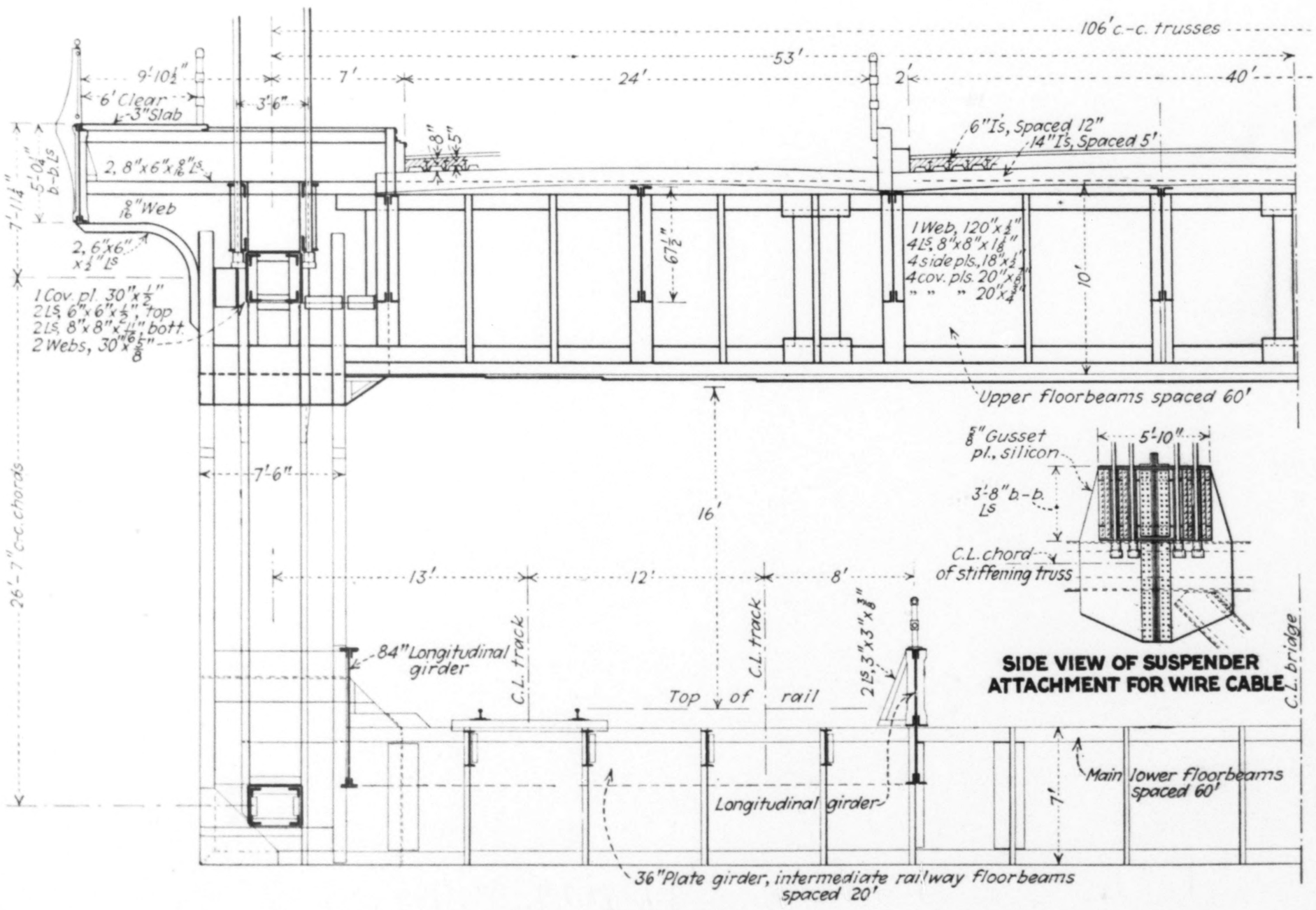
END VIEW OF MASONRY-ENCASED TOWER



CROSS-SECTION OF EYEBAR CABLE AT SUSPENDER



PLAN OF EAST APPROACH



HALF CROSS-SECTION OF DECK (Second Construction Stage Shown in Fine Lines)

s the Hudson River Between New York City and Fort Lee, N. J.

than thirty years. A project for a bridge at 57th St., developed by Gustav Lindenthal, as the final outcome of planning dating back to the '90s, is even now in an active stage. This and other projects had in view direct access to the heart of mid-town New York. But in 1910 the Interstate Bridge and Tunnel Commissions, a joint body of the two states, looked some six miles farther north, to the Fort Washington-Fort Lee narrows, and outlined a tentative plan for a cantilever bridge with river piers. Thereafter it proceeded to investigate foundation conditions by borings, and then found that no rock was within reach over a width of more than 3,000 ft. This disappointing discovery caused the commissions to conclude that any Hudson River bridge within the limits of New York City would have to cross the river by a single span—which meant 3,000 ft. or over. With this conclusion it abandoned further consideration of the Fort Lee site.

From that time on no attention was given to this site until 1923, when O. H. Ammann, now bridge engineer of the Port of New York Authority, made a study of

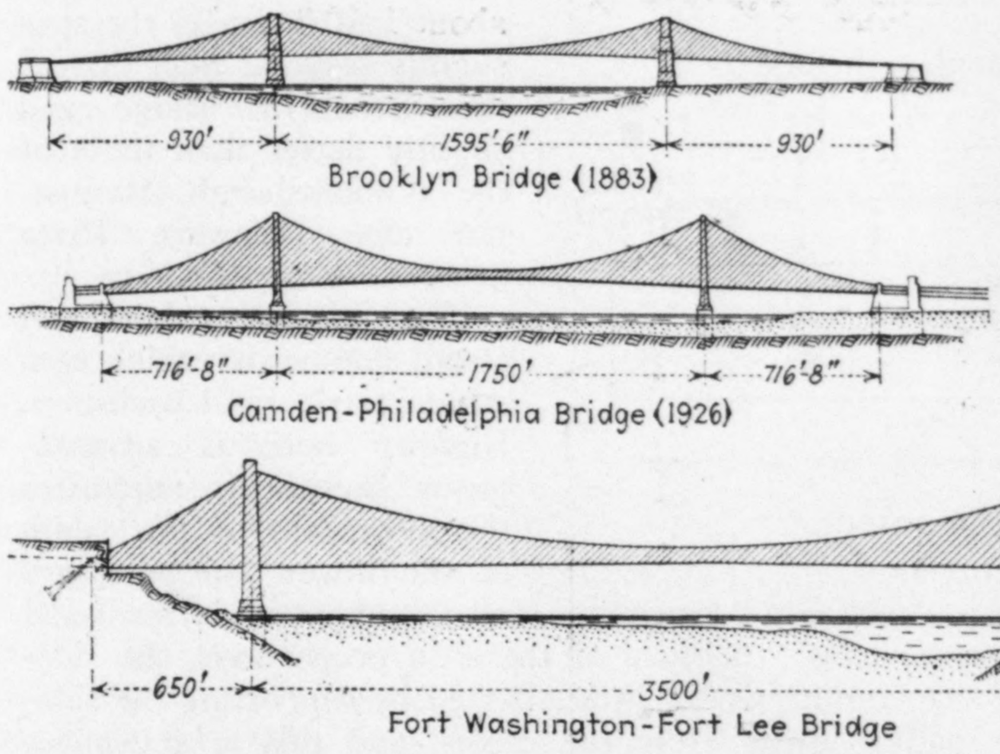


FIG. 2—THREE HISTORIC SUSPENSION BRIDGES COMPARED

the possibilities of the site, and recommended construction of a single span suspension bridge on the present location. The proposal obtained such popular support that a year later the legislatures of the two states authorized the Port Authority to study its possibilities. The resulting preliminary report, rendered in the early part of 1926, showed that the crossing was physically feasible and would pay for itself on a toll basis. The legislative decision to proceed with the work then followed.

Site—How the new bridge is related to the general territory of New York City and its neighbors immediately to the west is brought out by the regional map, Fig. 1. Its direct street connections are shown in connection with the general plan on the insert plate facing p. 212. At the east end the bridge ties into the general street network of the Washington Heights residential region of New York City and has ready connection with the Washington bridge across the Harlem River, leading to the Bronx and to the outward roads north and east. On the west side it taps several main highways, though more complete approach routes will probably be developed in the future.

Both banks of the river in the region of the bridge are rock ridges, 200 to 300 ft. high above tide, with crests from 500 to 1,500 ft. back from shore. Rock is exposed on the New York or east bank, forming a rocky cape, Fort Washington Point, whence it shelves down steeply to the navigation channel. On the New Jersey side rock slopes down more slowly, and reaches a depth of about 150 ft. below water at the pierhead line. Except at the shores, throughout the 3,200-ft. width between pierhead lines the rock surface lies below the depth of 200 ft. The overlying material ranges from soft mud to plastic clay.

These conditions of site and topography dictated the general layout—a single span of at least 3,200 ft., suspension type, and a clearance height of about 200 ft. Navigation interference was not a factor under these circumstances. Detail study of foundation possibilities placed the east pier above water on Fort Washington Point, and the west pier 100 ft. back of the pierhead line, where rock was at less than 100-ft. depth and the foundation could therefore be examined in the dry (the pneumatic process being available at this depth if found necessary). These positions fixed the main-span length at 3,500 ft. between centers of towers.

Capacity—With such a prodigious span, a great width of structure was essential for lateral stiffness. The main carrying members are spaced 106 ft. apart. This indicated a wide choice in the matter of traffic capacity. But in fixing upon the capacity to be provided for, conflicting considerations had to be faced: the fact that this first bridge crossing in the lower 50 miles of the Hudson would inevitably become a great traffic artery, and, on the other hand, cost limitations and the desirability of restricting any single arterial crossing to the reasonable absorbing power of the adjacent street network.

The future traffic function of this bridge, elaborate studies made by the Port Authority staff and the Regional Plan of New York and Vicinity indicate, is certain to be many times greater than its importance in the next decade or so, since the crossing will form an active ingress and egress route for New York, will be the main bypass route in the coastal highway system around the central part of New York City, and will connect the rapidly growing and highly important Westchester County suburban area with equally important New Jersey residential areas. Nevertheless, it was concluded that a traffic capacity of eight highway lanes, supplemented by a smaller number of rapid transit tracks (or possibly bus lanes or ordinary traffic lanes) represents a reasonable maximum to be concentrated in this single arterial connection. A plan has accordingly been worked out to provide eight highway lanes in one width, on an upper deck, and from four to six tracks or lanes on a lower deck. Comparison with the capacity of a few other great urban bridges is given by Fig. 3.

Cost considerations, however, at once entered into the matter here. Were the bridge to be built for full capacity from the beginning, many millions of dollars of investment would not be utilized for a number of years. Accordingly, the idea of building the towers, cables and roadway in two stages—first for half capacity and later for full capacity—was embodied in the preliminary plan, and was retained in the final plan. It has now been modified, however, by the decision that towers and cables shall be built to full capacity initially,

while the two-stage plan is retained for floor and stiffening trusses.

General Design—Various elements of the general design of the bridge are determined by the local topography, as already indicated. Not only is the position of piers closely fixed by the ground conditions, but the length of side spans is similarly determined within narrow limits. On the New Jersey side, the Palisades cliff

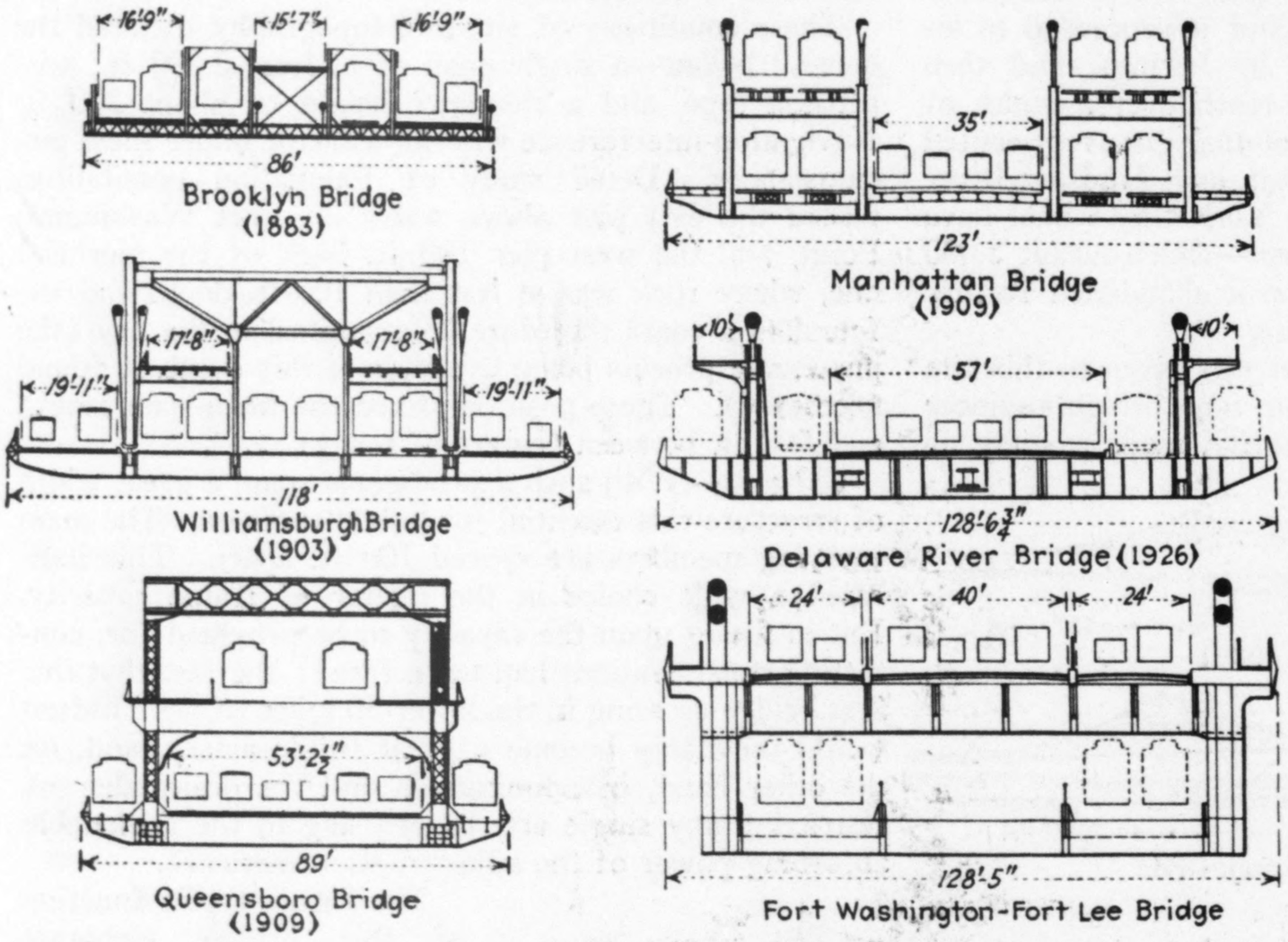


FIG. 3—TRAFFIC FACILITIES OF SIX GREAT CITY BRIDGES

naturally terminates the side span and at the same time provides ample anchorage facilities; anchor chains bedded in deep tunnels driven diagonally 250 ft. into the cliff are contemplated. On the New York side the ridge is farther back, but the ground permits building a masonry gravity anchorage at a point that makes the two side spans equal and the bridge therefore symmetrical in appearance. Continuing the roadway thence eastward to the high ground by means of a masonry viaduct proved to be cheaper than attempting to span this entire distance by a prolongation of the suspended structure, an expedient which would produce an unsymmetrical structure and would have the further disadvantage of increasing the flexibility of the structure and therefore making its strains considerably more serious, besides requiring provision for large movements of the cables at the top of the east tower.

The proportioning of the cable catenary was determined mainly by esthetic judgment, the economic balance being reasonably even over quite a range of sag ratios. The ratio selected, about 1:10.8, makes the span slightly less flat than that of the Brooklyn bridge and slightly flatter than those of the Williamsburgh, Manhattan and Delaware River bridges.

The relatively short and steep side span cables constitute fairly rigid backstays. Rigidity here is advantageous because it minimizes the movement of the cables at the tower top necessary to equalize the horizontal stress components in main and side spans. Because of the span proportions, the side-span rigidity is but little affected by supporting the side-span roadway from the cables, and this arrangement, with its economic and esthetic superiority over inde-

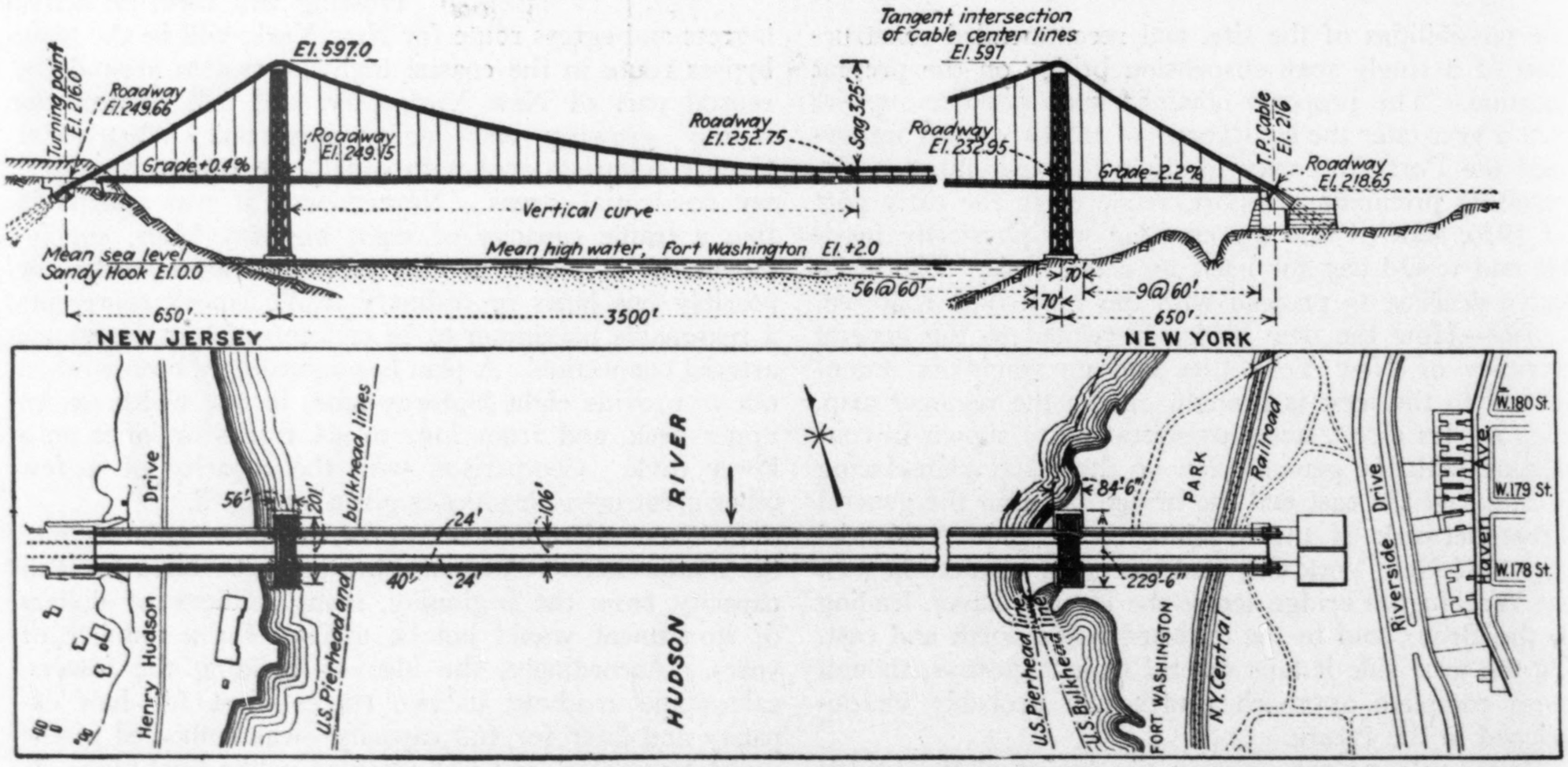


FIG. 4—INITIAL CONSTRUCTION STAGE OF FORT WASHINGTON-FORT LEE BRIDGE (APPROACHES NOT SHOWN)

In the first construction stage the full tower and cable capacity will be provided, but the concrete and stone encasement of the towers will be omitted. Only the upper deck will be built in this stage and the stiffening trusses will be omitted, except for the upper chord, which will serve as the chord of the horizontal wind truss in the plane of the roadway floor.

pendent support, was adopted. The esthetic effect is portrayed in two architect's sketches reproduced here.

Cables—The determinative features of the huge suspension bridge, naturally, are the cables, the towers, and the stiffening system and associated main floor construction. All of these features have been the subject of unusually extensive studies on the part of the engineers who developed the design, and the subject also of much criticism and discussion outside.

For the main supporting cables, the possibility of using either high-strength forged eyebars or parallel-wire cables of the conventional type has been kept in view from the outset. Exhaustive studies have shown both types of structure to be feasible and satisfactory, being capable of rapid construction by known processes, and being dependable and permanent in service. Alternative designs for the two types were therefore worked out at an early stage of the development, with a view to calling for competitive bids. This decision has been adhered to, after repeated restudy of the designs in the light of various criticisms, directed mainly against the eyebar cable.

The two kinds of cable necessarily will have different weight, the eyebar cables being about 2½ times as heavy as the wire cables. While this increased weight is an advantage in point of rigidity, the two designs have been worked out to the same outline and the same stiffening-truss proportions, without discrimination on the score of comparative stiffness.

For either type of cable, sizes much larger than anything previously attempted will be involved. In the case of wire cables, there will be four cables 36 in. in diameter, two cables on each side of the bridge, one directly above the other (forming a single two-cable carrying member). In the case of eyebar cables, each side of the bridge will be carried by eight groups of twelve eyebars averaging 16 x 2 in., arranged in four tiers comprising 96 eyebars, attaining a maximum of 3,456 sq.in. of carrying section. Both forms are so developed that two-stage construction is possible, the lower half of the cable being built first and the upper half later.

As previously noted, two-stage construction was a feature of the first plan for the bridge, with a view to postponing for five or ten years an expenditure ranging from $15,000,000 to $30,000,000 and thereby saving a very large sum by reduction of interest. As concerns the cables and towers, however, a thorough review of the facts of the situation early this summer led the engineering staff and its consulting engineers to conclude that they should be completed to final capacity by the time the bridge is opened to traffic, although they might as a matter of the constructor's convenience be built in two successive operations, the second half being erected while the floor construction is going on. Accordingly, alternative bids will be asked on construction of the cables in one stage or two (the second stage to be completed before the bridge is opened to traffic).

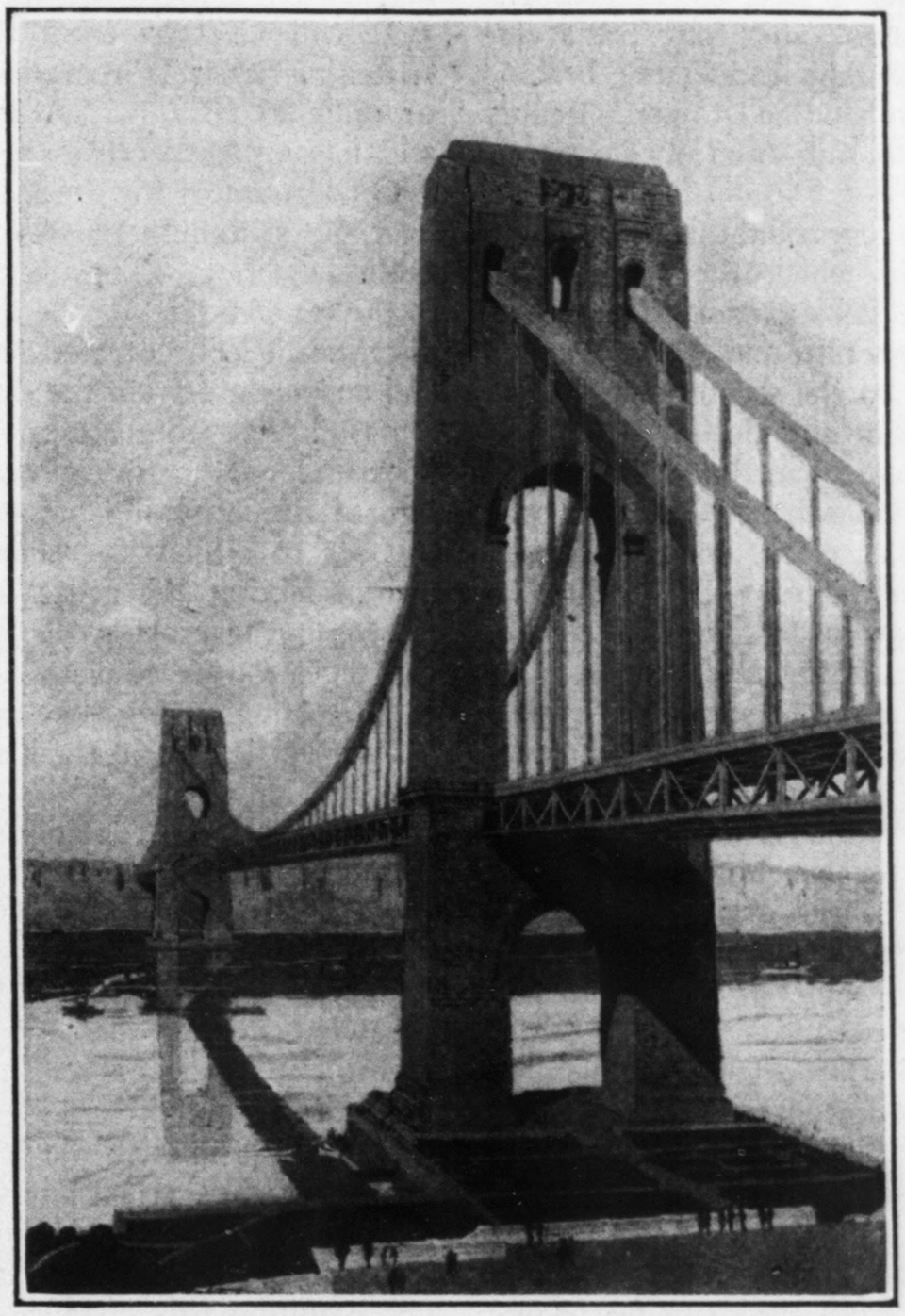


FIG. 5—PERSPECTIVE DRAWING OF FORT WASHINGTON-FORT LEE SUSPENSION BRIDGE NOW UNDER CONSTRUCTION

Steel towers support the saddles, but the steelwork is to be encased in stone-faced concrete. The roadway is a double-deck structure, with ultimate provision of eight lanes for highway traffic on the upper deck. The floor is supported by two cable groups, which may be of either parallel wire or eyebar construction. In the case of wire cables, each group will consist of two separate cables 36 in. in diameter. The towers will reach a height of 625 ft. above water.

The suspenders and their attachment to cables and floor have been studied almost as fully as the cables themselves. They present a problem somewhat different from that found in prior suspension bridges because of the use of two cables on either side of the bridge and because of the unusual floor panel length, namely 60 ft. (as compared with previous extremes of 7½ ft. in the Brooklyn bridge and 20 ft. in the Delaware River bridge). The adopted hanger constructions are shown by details on the insert plate accompanying this issue. Rope suspenders were chosen for both types of cable, the ropes resting on cable bands of the usual form of construction in the case of wire cable, and being looped over pin-spools in the case of the eyebar cables.

One factor that determined the adoption of rope suspenders in the latter case was the importance of a large degree of lateral flexibility in the suspender. Since the bridge floor is so narrow compared with the great span length that it will be quite flexible laterally, while the heavy cables are relatively resistant to wind deflection, a large part of the wind load will be transmitted to the cables by way of the suspenders. On this account it is necessary that the suspenders be capable of assuming a sidewise inclination sufficiently large that the horizontal component of suspender pull will transfer the wind shear from floor to cable without bending the suspenders. The lower ends of the suspenders in both instances are attached to the floorbeams by having their end sockets engage stiffened brackets on the sides of the floorbeams.

Towers—Esthetic considerations played a large part in the determination of all the general features of this bridge. They were most commanding, perhaps, in relation to the towers. Cass Gilbert, the noted architect, was called into consultation from the earliest stages of

the Port Authority planning, and has been active in developing the architectural features and ornamentation of the bridge throughout. He agreed with the engineers in favoring a masonry-encased steel-frame tower. The form developed is, in side elevation, that of a slender, gently tapering shaft, plain in outline and marked by only enough vertical relief to emphasize its function and size. Seen in the direction of the bridge axis the tower is broad, and is cut by a lofty vaulted passage through which the roadway passes. Structurally, the great pier is a cellular assemblage of reinforced-concrete walls surrounding and encasing a steel tower comprising two groups of eight columns and four transverse planes of truss work, assembled by connecting bracing and horizontal diaphragms into a unified tower. Beauty and permanence are served by an exterior facing of stone.

It was planned at first to proportion this tower as a concrete-and-steel structure, in which the steel would support the loads of the first construction stage (half roadway capacity) and the concrete and stone, applied later, would carry a large part of the second-stage loads. This ingenious construction plan and design have been under discussion quite as animated as that centered on the question of eyebar or wire cable. Criticism was mainly based on the claims that plain, uncovered steel towers (Manhattan bridge type, later used in the Philadelphia-Camden bridge) are superior to composite towers both economically and structurally, that the use of masonry architectural form for a structure whose strength depends mainly on the steel framework violates esthetic principle, and that the load distribution through the composite tower would be indeterminate. Repeated and thorough restudy of the towers, however, has led to the retention and approval of the original design in all substantial features with only one major change, namely, strengthening the steel skeleton so that it will be able to carry the complete final loads, both dead and live, leaving the concrete and masonry encasement to serve as permanent weather protection and architectural dress, though it will also give the tower a greater marginal strength and thus provide amply for possible future increase in carrying capacity.

This change, determined upon only within the last three months, resulted in part from the growing recognition in the course of the engineering studies that the initial traffic demand may prove unexpectedly heavy, taxing the initial roadway capacity almost at the outset and making it necessary to construct the additional roadways immediately after opening and before there would be time to build the inclosing shell of the towers. In part, also, the decision constituted a recognition of the wholly exceptional psychological factors surrounding a structure of a size so far beyond precedent. It was felt that such ample guarantees of strength should be provided in these huge towers that immediately upon its opening the bridge would be sure to engage complete public confidence. These views also influenced the decision in favor of constructing the full ultimate cable capacity at once.

Stiffening Truss—In the Fort Lee bridge, for the first time in the modern construction of large suspension bridges, the stiffening truss assumes a very minor position. The great span length and the resulting weight of cables and floor result in a large inherent stability of the structure against deformation, while on the other hand the distributed nature of highway traffic means a small deforming moment. No stiffening truss is considered necessary, therefore, in the initial construction stage for vehicular traffic, but the action of a stiffening truss is believed to be desirable when and if trains are run over the structure.

Combining these conclusions with the plan to construct two decks, the upper or highway deck built in the initial stage and the lower or rapid-transit deck in the second stage, the design provides for placing in the first stage only the upper chord of the stiffening truss (serving as chord of the wind truss), and adding the web members and bottom chord when the lower deck is added.

It will be seen from the general drawing that the stiffening truss is remarkably shallow, having a depth of only 26 ft. 7 in. It is discontinuous at the towers and has uniform cross-section throughout a span.

On account of the great length of span and large dead-load the stiffening trusses have little effect in resisting cable deflection. They become necessary as an adjunct to the construction of the lower deck and are effective in distributing concentrated loads, thus relieving the cables and suspenders from the direct action of such concentrations and preventing local distortions of the cables and floor under the action of train loads especially. The design of the stiffening trusses consists in finding the unit stresses to which the deflections of the cables subject them. Fixing the areas of the chord sections of the trusses is therefore a matter of selecting nominal sizes that will produce efficient compression members, the size of the section having very little effect on the unit stresses.

Both vertical deflections due to live-load and temperature and lateral deflections due to wind forces are taken into account. The combined unit stress as found for the worst case of loading in each span determines the grade of steel used. For the eyebar design the chords are of carbon steel in the side spans and silicon steel in the center span, while for the wire cable design, with its somewhat greater deflection, the chords are of silicon steel in the side spans and nickel steel in the center span, with correspondingly higher unit stresses.

In connection with the bridge cross-section, shown on our insert plate, attention is called to the fact that in the upper deck there is a main stringer system, transverse subfloorbeams at 5-ft. spacing, and a reinforced-concrete slab spanning this distance with the aid of longitudinal reinforcing members embedded in the concrete. The main floorbeams, as already stated, are spaced 60 ft. A lateral bracing system is included in the floor, in the plane of the top flanges of the stringers, forming with the stiffening-truss chord a complete wind truss. The lower deck has intermediate floorbeams at 20-ft. spacing carried by longitudinal plate-girder main stringers spanning the 60-ft. panel between main floorbeams.

Three roadways occupy the upper deck, designed to accommodate two, four and two highway traffic lanes respectively. It is intended to build the central four-lane roadway first, adding the side roadways only when traffic increase demands. The lower deck as originally designed comprised brackets extending out either side of the stiffening-truss posts for a sufficient distance to accommodate a single-track width of deck. The finally developed design, however, provides a full floorbeam for the lower deck in place of the bracket construction, mainly to increase the convenience and adaptability of the lower deck for four or more tracks or other lanes of traffic. This construction is slightly more costly and involves greater dead-weight than the bracket construction,

but is believed to offer sufficient advantage to make this extra expense worth while.

Materials—Silicon steel is used generally for the main columns of the towers and for the main material in the floor system. Carbon steel is used generally for the tower bracing and tower and floor details. The cable eyebars (if used) will be heat-treated eyebars with a minimum ultimate strength of 105,000 lb. per sq.in. and minimum yield point of 75,000; they are proportioned for a working stress, including secondary stress, of 50,000 lb. per sq.in. The wire cables (if used) will consist of No. 6 galvanized wire having a minimum ultimate strength of

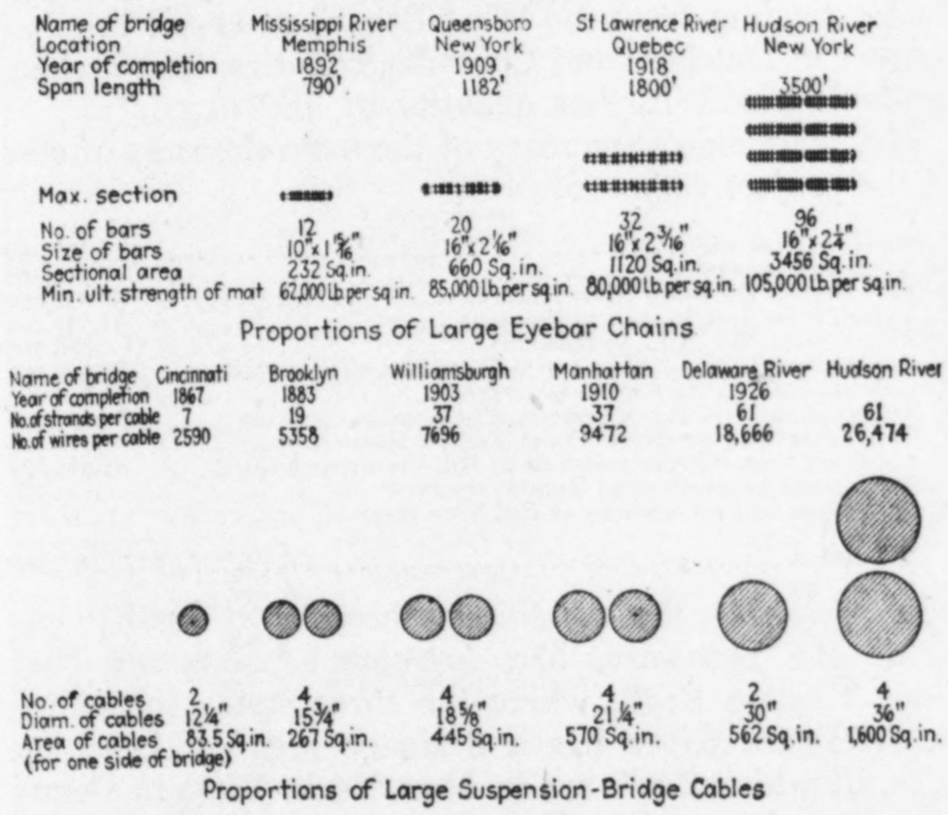


FIG. 6—SOME NOTABLE BRIDGE CABLES AND EYEBAR CHAINS

220,000 lb. per sq.in. and minimum yield point of 150,000; their working stress, exclusive of secondary stresses, will be 82,000 lb. per sq.in. The grades of steel used in the stiffening trusses have already been mentioned.

Loads—The final dead-loads for the eyebar-cable design are 50,000 lb. per lin.ft. in the center span and 56,000 lb. in the side spans. For the wire cable design they are 39,000 lb. per lin.ft. in the center span and 40,700 lb. in the side span.

Live-loads on both roadway and track spaces were taken at figures corresponding to the heaviest modern traffic. Because of the effect of longitudinal distribution, their equivalents in load per lineal foot of bridge, as used in proportioning the floor system, suspenders, stiffening trusses and main carrying members, vary with the loaded length and number of lanes involved. Thus in the design of the stringers, where load on only one 60-ft. panel is concerned, the system of loading is such as to represent a live-load of 31,000 lb. per lineal foot of bridge (both decks, ultimate condition). In the floorbeam design (two panels of both decks loaded) this figure becomes 26,000 lb., while in the suspender design (five panels loaded) it reduces further to 18,000 lb. For the case of load on the full 4,800-ft. length of bridge, as in the proportioning of cables, towers and anchorages, the equivalent average load becomes 8,000 lb. per lineal foot of structure.

Construction Program and Organization—Contracts have already been let for two items of the bridge, and work on them has been in progress for some months. The foundations for the west (New Jersey) tower are being built by Silas B. Mason, on a bid of $1,160,000. The west anchorage and approach cut excavation, in rock, is being done by Foley Bros., on a bid of $694,000.

These two contracts are to be completed next spring and next fall respectively. In the meantime the contract for the superstructure steelwork will have been let; bids are to be asked within the current month. The east anchorage contract is expected to be let early next year. Erection of the towers should begin next summer. The program calls for completing the bridge and opening it to traffic early in 1932.

The design and construction of the bridge are being carried out by the engineering staff of the Port of New York Authority. J. E. Ramsey is chief executive officer. O. H. Ammann, bridge engineer, with E. W. Stearns as assistant, is in general charge of the design and construction. L. S. Moisseiff is advisory engineer of design and Allston Dana is engineer of design. Traffic matters are in charge of Robert A. Lesher, traffic engineer. Field operations are under M. B. Case, engineer of construction. The work is being done with the assistance of Prof. William H. Burr and Gen. George W. Goethals, consulting engineers, Daniel E. Moran, consultant on foundations, and Cass Gilbert, architect.

Southern Land Reclamation—A Report on Possibilities and Methods

WHAT can be done in land reclamation and settlement in the southern states has been studied by the Bureau of Reclamation under direction of Congress. The principal study was made for the Bureau by a board of special advisers, composed of Howard Elliott, Daniel C. Roper, and George Soule. Each of the southern states was asked to designate a tract of land available and suitable for group settlement after reclamation, and the tracts so selected were then studied by the board in conjunction with specialists from the Bureau of Reclamation and the state officials and agricultural colleges. Tracts of 6,000 to 25,000 acres were submitted by North and South Carolina, Georgia, Florida, Alabama, Mississippi, and Tennessee. The required reclamation was found to consist of clearing, in some instances drainage, and usually breaking the land and putting it into an initial crop; the Florida tract, west of Fort Lauderdale, would require also pumping plants for permanent drainage. Individual farms ranging in size from 20 acres (Florida) to 120 acres (Alabama) are recommended, according to the type of product and cultivation suitable to the soil and climate. The crops contemplated are such as have a ready market and as will maintain and increase soil fertility, in due rotation. Each of the tracts in question will provide for 100 to 150 home-owning farmers.

It is recommended by the board that further study leading to the development of definite settlement plans be based on specific soil surveys; and that the settlement projects be worked out on the basis of expert planning and supervision, provision of money to enable settlers to complete the farm improvement and equipment on long-term loans at low rates of interest, centralized control of settlement, and co-operation in the work between state and federal bureaus.

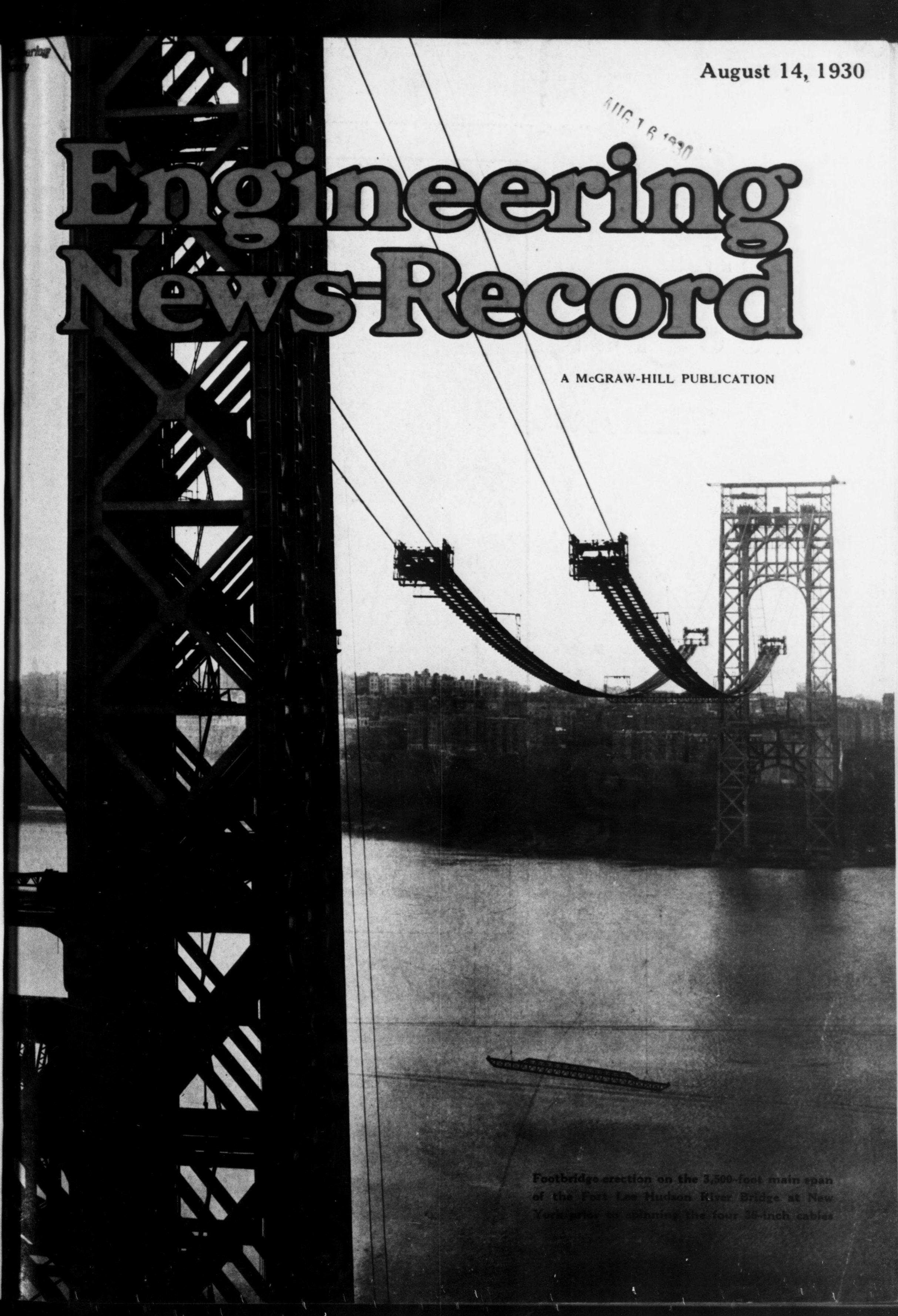
Most of the land studied is idle or under unprofitable farming at present, the latter mainly because of tenant farming. The South Carolina, Georgia, and Alabama tracts are now being given further study by the Bureau of Reclamation and state authorities.

August 14, 1930

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Engineering News-Record

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Footbridge erection on the 3,500-foot main span of the Fort Lee Hudson River Bridge at New York, prior to spinning the four 36-inch cables



Spinning Four 36-In. Cables for the Fort Lee Bridge

Elaborate Electrically Operated Plant of Cable Contractor Spins 28,400 Tons of Wire in 9¾ Months on Hudson River Structure—Spinning Wheels Travel Mile Between Anchorages 60 Per Cent Faster Than on Any Previous Project by Virtue of Driven Reels and Improved Tramway Supports—Stepped Footbridges on Side Spans

By E. Warren Bowden

Assistant to Chief Engineer, The Port of New York Authority

THE CABLES for the new bridge over the Hudson River between Fort Washington Park in New York City and Fort Lee in New Jersey are now rapidly nearing completion. On August 7 the last wire of the total of 105,896 wires was spun, signalizing the placing in a single project of more wire than the sum of that contained in the next seven largest suspension bridges now in existence. In addition, the spinning was completed in 9½ months, the first wire having been placed Oct. 18, 1929. The wire in the four 36-in. diameter cables weighs 28,400 tons.

The entire bridge is of unusual proportions, its center span of 3,500 ft. being practically double that of any bridge previously built. The massive steel towers supporting the four cables rise more than 600 ft. above the water level and thus exceed in height all but the very tallest buildings in our large cities. Other articles treating various phases of the project have been published in *Engineering News-Record* as follows: "Design Considerations of Complete Project," Aug. 11, 1927, p. 212; "Design of Steel Towers," May 24, 1928, p. 819; "Deep Open Cofferdams for New Jersey Pier," Aug. 16, 1928, p. 232; "Construction of New Jersey Tunnel Anchorage," Oct. 25, 1928, p. 611; "Pouring 106,000-Cu.Yd. Block of Concrete for New York Anchorage," April 11, 1929, p. 591; "Manufacturing High-Modulus Footbridge Rope," May 1, 1930, p. 714.

The work of spinning the cables involved many interesting problems and many improvements incidental to production on such an unprecedented scale. The contractor has used new types of footbridges and unusual methods of erection, including an improved design of tramway rope support, power drive and control of reels, an unusual and efficient installation of electric signal and operating equipment. The latter not only permits a close control of the entire spinning operation from a central dispatcher's office but also permits wire adjustments to be made at the towers one-third of a mile away by the observer at midspan. Also by a system of light signals, the progressive stages of wire adjustment are co-ordinated most effectively.

Elaborate Preliminary Studies

Although the method of wire-cable spinning introduced by John A. Roebling and first used in spinning the wires for the railroad bridge across the Niagara gorge in 1854 has remained fundamentally unchanged, each wire cable erected since that time has contributed something to the refinement and progress of the art. Increased spans using larger quantities of materials have necessitated constant refinements and improvements in the machinery used, but no suspension bridge heretofore has offered as manifold problems nor has justified as intensive experimentation and study in the solution of them as has the Fort Lee bridge with its 101,000 miles of wire to be spun under erection loads more than twice as large as any previously encountered.

For a period of many months the Port Authority engineers engaged in a detailed study of cable problems, weighing the relative merits of various cable arrangements, saddle designs, anchorage loads and erection problems in general. Bids were secured on both eyebar-chain and parallel-wire design. In the latter a complete set of plans was prepared for cables arranged in pairs, either one above the other or side by side. Also, bidders on wire cable were given the alternative of choosing the method of simultaneous or successive erection. In the case of successive erection, one cable of each pair would be spun first, the suspended structure hung, leaving as a final operation the spinning of the remaining two cables.

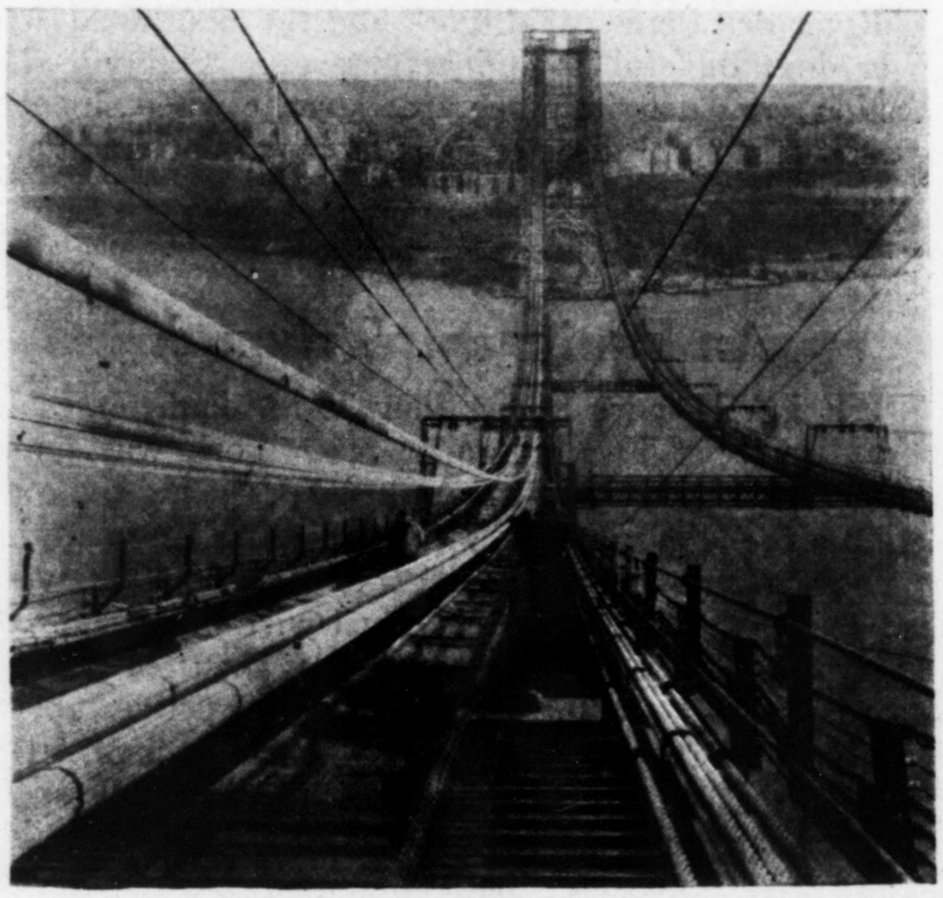


FIG. 1—MAIN-SPAN SPINNING LAYOUT—FORT LEE BRIDGE

Mesh used under cables to reduce weight of footbridges. Note steel headframes carrying four tramway ropes over each footbridge.

The John A. Roebling's Sons Co., the successful bidder, proposed at first to use this successive erection plan with cables side by side. However, as plans for accomplishing the transfer of load to the second set of cables began to mature, it became increasingly evident that the disadvantages of such a plan more than outweighed its

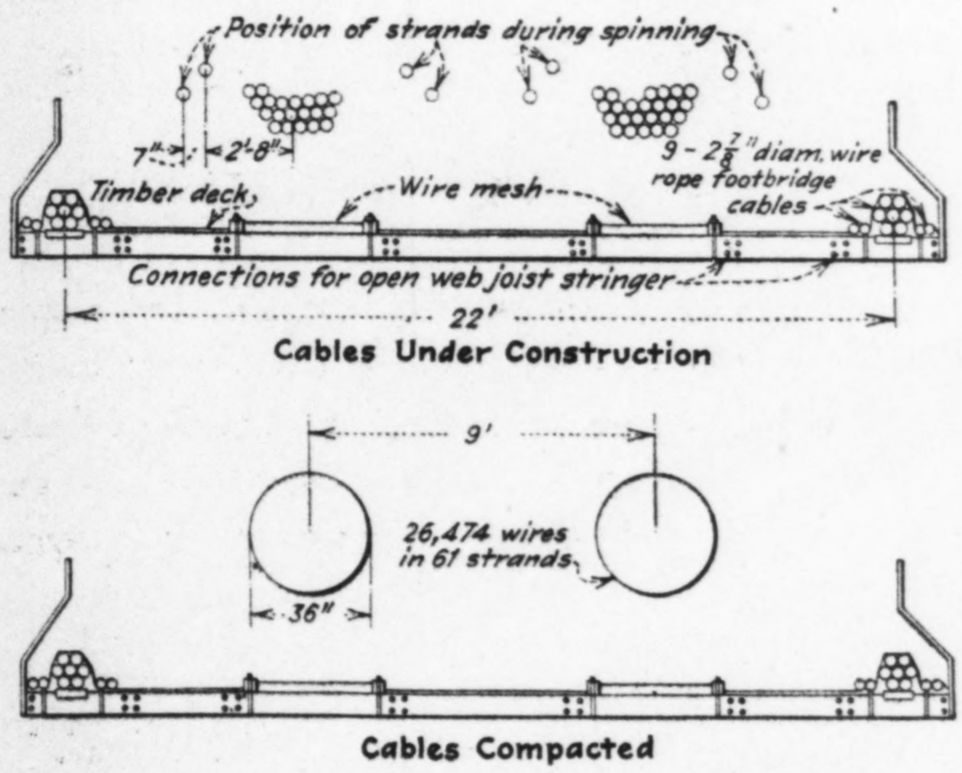


FIG. 2—RELATIVE ARRANGEMENT OF CABLES AND FOOTBRIDGE

Note grouping of footbridge ropes—seven ropes in a hexagon and one rope at either side.

advantages, which consisted mainly in the speeding up of erection of the suspended structure. At the suggestion of the Port Authority, the contractor, therefore, changed his plans to the system of simultaneous erection and undertook to complete the four cables in the time originally contemplated for two.

The Cables

The four cables are grouped in two pairs, 106 ft. apart, each pair centering over one side of the bridge. The cables of each pair are spaced 9 ft. centers. They are of the parallel-wire type, and each contains 26,474 cold-drawn galvanized wires, 0.196 in. in diameter. When completed the cables will be compacted to a diameter of 36 in. Each individual wire contained in the cables has a strength of more than 7,000 lb., representing an ultimate strength for the material of more than 240,000 lb. per square inch.

For purposes of construction, the cable was divided into 61 4½-in. diameter strands, each of which contains 434 wires. Each strand was independently spun and independently attached at the anchorage to a pair of eyebars forming the upper links of a chain which extends down into the concrete of the anchorage to anchor girders.

On the New York side the cable pull is resisted by a 260,000-ton mass of concrete and granite masonry. On the New Jersey side the cables are attached to eyebar chains which are anchored in concrete in tunnels extending approximately 150 ft. into the solid rock of the Palisades.

The cables are supported in cast-steel saddles at the towers and at the pedestals in front of the anchorages. Each tower saddle covers an area 8½ ft. in width by 28 ft. in length, is roughly 12 ft. high and weighs nearly 200 tons. The necessity for such unusual proportions is better understood when we consider that the load to be carried in each saddle may be as great as 30,000 tons. Each saddle is supported on a nest of 40 8-in. diameter rollers, which during erection permit the saddle to roll to a position corresponding to balanced horizontal pull from main and side spans, but which, after the completion of the bridge, will be locked in fixed position.

The anchorage saddles are much lighter and serve both as cable support and as splay casting. They are supported on rocker nests to provide for the necessary motions due to stretch under load and temperature changes.

Footbridges

The footbridges, of ample width to provide working room for two cables, are supported by two groups of nine steel-wire ropes, 2⅞ in. in diameter. When later removed and cut to length, these ropes will serve as suspenders in the permanent structure.

It has been the usual practice in previous cable-spinning operations to provide footbridge ropes in continuous length from anchorage to anchorage, determining the final length for cutting and socketing in the field after the ropes have stretched under their own load. For the Fort Lee bridge the contractor elected to provide separate rope lengths for main and side spans, prestressed at the manufacturing plant to loads corresponding to those to be encountered under working conditions, thus permitting cutting and socketing before shipping to the bridge site.

In erecting the footbridge ropes, the main-span ropes were laid out between the tower bases on the bed of the river and attached by pennant lines to the side-span ropes, which had been stretched out from either anchor-

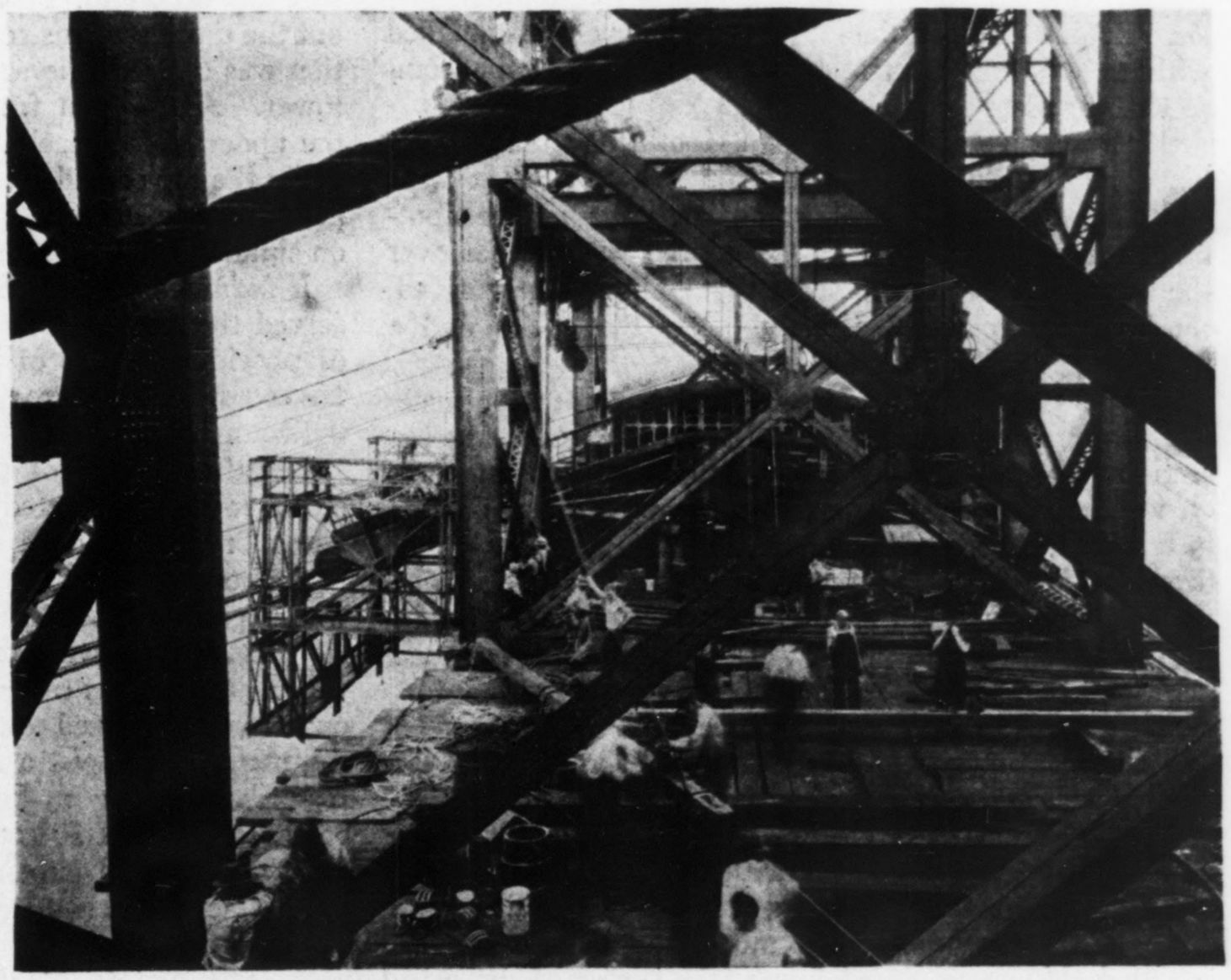


FIG. 3—ATOP THE NEW YORK TOWER, FORT LEE BRIDGE

Arched walkway in background follows outline of saddle which is 12 ft. high and 8½x28 ft. in plan. Steel superstructure shown is temporary, used to support bridge crane which handles completed strands and to carry other erection equipment. On footbridge ropes at left is carriage used to erect main-span footbridge sections. It is in position to be loaded.

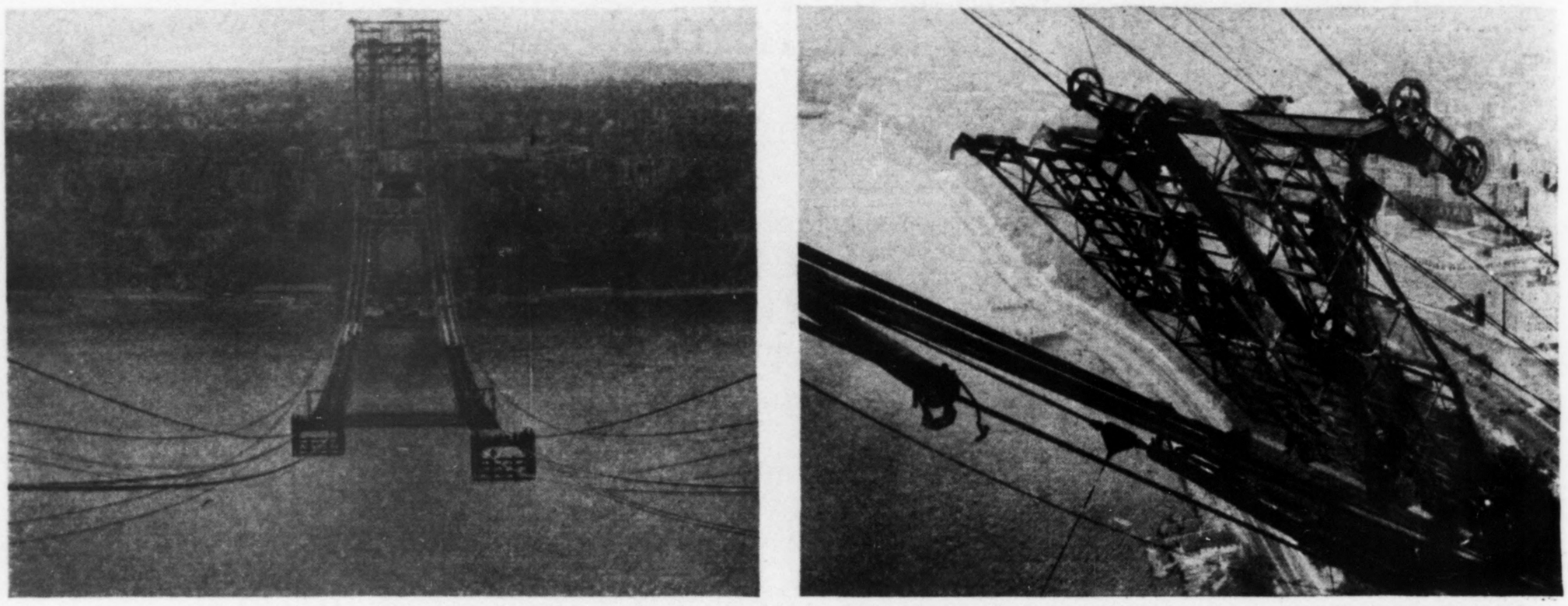


FIG. 4—ERECTING FOOTBRIDGE SECTIONS ON FORT LEE BRIDGE

Left: Main-span erection carriages; loaded with ten sections at platform on the face of the tower; run on footbridge ropes; equipped with hoist to handle sections into position. Right: Side-span erection carriage bringing stepped footbridge panel into position. Note how hooks at top will fit over pipe crossbeam fastened to footwalk ropes.

age. After being hoisted to the tower top and lowered into a supporting frame, the pennant line was replaced by a permanent connection consisting of threaded rods passing through clamps fastened to the ropes at the sockets. The connection thus provided permitted about 5 ft. adjustment in rope length at each tower. No adjustment was provided at the anchorages, at which point the nine ropes forming a group are held in a single structural frame, pin-connected to a chain of anchoring eyebars.

The first footbridge rope was hoisted to the tower tops on July 9, 1929, and eight days later all were in place.

The entire area of the tower top, approximately 200 x 40 ft., is surmounted by a structural framework about 50 ft. high, composed of 30-in. column sections securely braced, carrying transverse beams and longitudinal girders. Running upon the latter is a girder crane equipped with two 65-ton electric hoists for handling the completed strands.

Main Span—The main-span footbridge decking is made up of light welded structural sections, 25 ft. wide and 12 ft. long, clamped immediately below the footbridge ropes. The sections are floored with timber over the working areas, but in order that weight may be saved, the timber is omitted directly below the cables and a heavy wire mesh substituted.

To erect the footbridge sections the contractor made use of four large carriages (Fig. 4), each carrying ten sections, which moved out on the footbridge rope groups on wooden wheels; 1-in. endless tramway ropes, erected in temporary position and driven by 100-hp. electric motors located on the superstructure, hauled each carriage from its loading position at the tower to the center of the main span. The sections were raised from the ground in skips lifted by the overhead cranes on the tower top, which were permitted to run beyond the tower faces by means of cantilever runway brackets. From the skip the sections were transferred to push cars running on a platform located on the river face of the tower at an elevation which permitted the car with its load of sections to run under the erection carriage. From this position the group of sections was hoisted into the erection carriages.

The decking was placed first at the center of the main span, working thence toward the towers. All main-span sections were erected in eight working days.

Side Spans—In the side spans the footbridge-rope groups do not follow the curve of the cable but approach the anchorage at a higher elevation. It was therefore necessary to suspend the side-span decking from these groups in order that the desired working position below the cables might be secured. The side-span sections are made up of light trusses in the form of flights of steps and landings, as shown in Fig. 4. Three sections placed side by side and connected by their timber platforms form a panel. In the space directly beneath the cable the timber is omitted, heavy wire mesh being used as in the main span.

The 11-in. diameter pipe floorbeams which occur at each side-span panel point were first spaced along 2-in. ropes, and each pipe was suspended at its ends from the main rope groups above. In erecting the sections, carriages running on $1\frac{3}{4}$-in. ropes in temporary position carried out the three sections composing a panel at a time. Erection was carried forward from the anchorage toward the tower. As is plain from Fig. 4, hook-shaped angles at the upper end of each section engaged a pipe floorbeam, while the lower end was bolted to the section previously placed. The erection of side-span decking was carried on simultaneously with the work in the main span.

Windbracing—The five crosswalks in the main span served the double purpose of stabilizing the system and of providing means of passage between the footbridges. No crosswalks are provided in the side spans, which have the comparatively short length of 700 ft.

For the purpose of resisting wind uplift a system of storm ropes was erected below the main-span footbridges. An interesting feature of this storm system as installed is the counterweighting applied to the main storm ropes which pass completely across the river. The counterweights at the towers maintain equal tensions under the varying conditions of temperature and deflection. From experiments carried on with models at the plant, the contractor developed a rather elaborate system of wire-rope crossbracing to booms suspended below the footbridges and about 500 ft. from the towers. These booms were difficult and expensive to erect and their effectiveness is open to some question, but the action of the storm system as a whole has been highly satisfactory with very little motion of the footbridges during winds of high velocity.

The elaborate character of the spinning plant is evident from Fig. 5, which shows the layout at the New Jersey

anchorage and on top of one of the main towers. So far as equipment is concerned, that in the New York anchorage is almost identical with that at the New Jersey end. One endless tramway rope, operating in either direction and carrying two spinning wheels attached at directly opposite points, is provided for each cable. Each rope is driven at the New Jersey anchorage by a 100-hp. electric motor; at the New York anchorage the ropes pass around idler sheaves. Tension is maintained by adjustable sheaves at the New Jersey end. The steel tramway headframes, placed at 200-ft. intervals along the footwalks, are innovations, those on previous bridges being made of timber. The reeling machines shown in Fig. 5 are driven by hydraulic motors providing exact control and fast operation of the spinning wheels. The floating counterweight shown in the tower is necessary because of the fast operation being used by the operator to assure constant and proper tension in the wire being spun. A telephone and electric signal system for dispatching and wire-adjustment operations makes the spinning operation as efficient and as fast as progress of other parts of the project warrant.

The wire to be spun was manufactured in coils about 4,000 ft. in length. At the plant, the coils were spliced and wound on special metal reels, having a barrel diameter of 6 ft., a loaded weight of more than 13,000 lb. and a capacity of 25 miles of wire. The reels were moved by rail from the plant at Roebling, N. J., to New York harbor on special cars equipped with steel cradles. The cars were delivered by car float to the tower bases, where the reels were transported to the anchorages by inclined cableways.

The Guide Wires—Preparation of the guide wires, the standards of length against which all strands are compared while being spun, constitutes the first operation of a cable-spinning job. This is done by measuring its length from observation of sags determined with surveying instruments. On the Fort Lee bridge the difficult observations usually necessary to obtain side-span lengths were entirely avoided by mounting the wire on small rollers at the points of support in the tower saddles. The wire was thus able to move to a condition of equilibrium with equal tensions on the wire on both sides of the saddle. Observations of main-span sag alone gave sufficient data for computing the entire length.

For each cable four strands were spun at a time, and during the process were supported in temporary position on brackets fastened to the outside of the saddle castings. When marked for length, one guide wire was placed in each of these spinning supports and its ends were attached to strand shoes at the anchorages. The strand shoes are of usual design, on this bridge capable of retaining 217 loops (434 wires), and provide for a total adjustment of 10½ in. by means of shims.

The Spinning Process—During the spinning process, the strand shoes were mounted on a "strand leg," a bulky steel casting, weighing approximately 5 tons, which holds them in a vertical position in advance of their final location at the eyebars. The leg must safely withstand a pull of 130 tons as well as bending arising from the necessity of leaving one side open for placing loops of wire around the strand shoe, as shown in Fig. 8.

In spinning the strands the reels of wire were mounted on unreeling machines at both anchorages. The free end

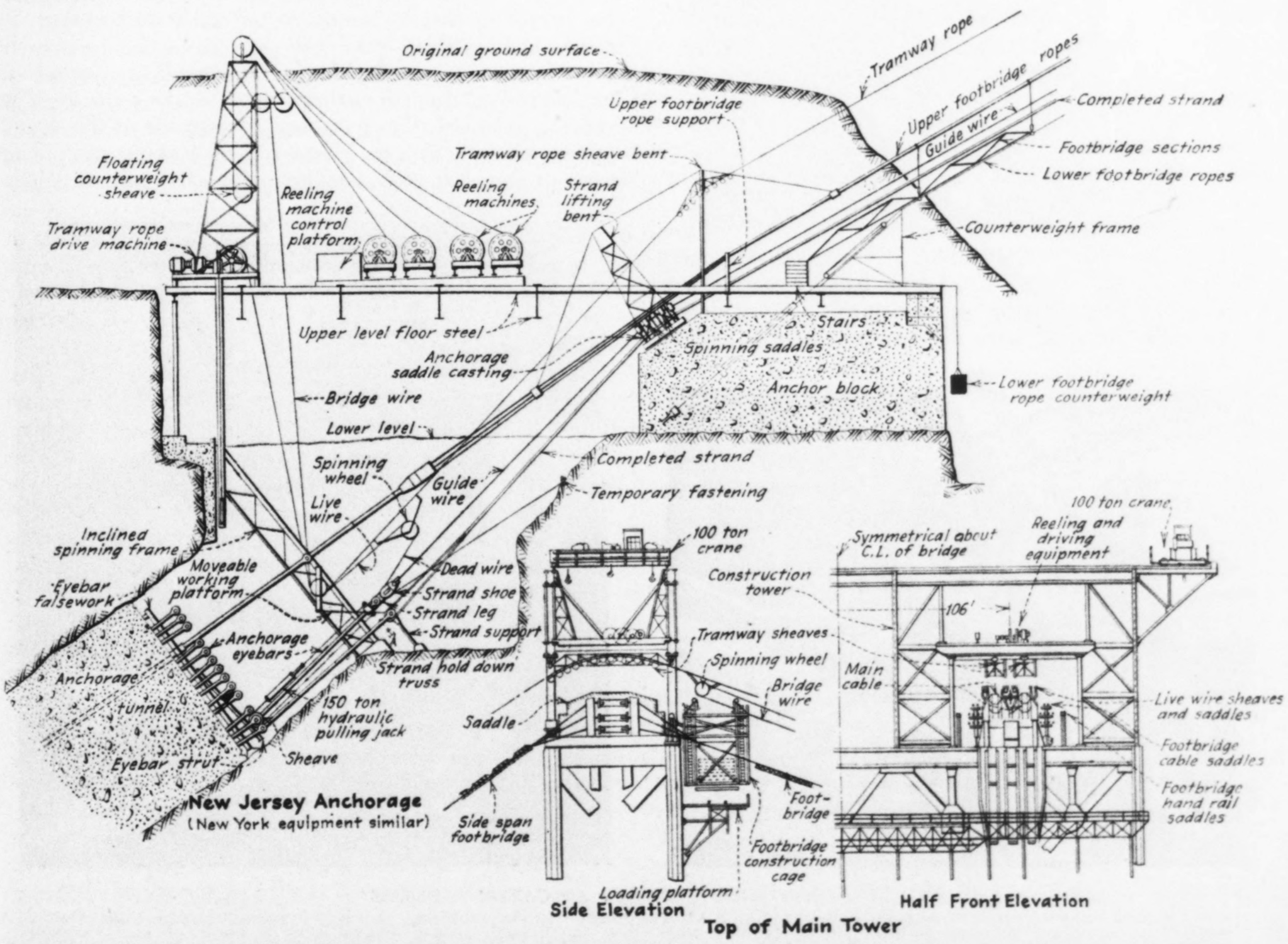


FIG. 5—LAYOUT OF WIRE-SPINNING EQUIPMENT, FORT LEE BRIDGE

of the wire, from a reel located, say, on the New Jersey side, was temporarily fastened at a fixed point in front of the strand shoe and the wire looped around the strand shoe. The bight of wire thus formed was placed over the 5-ft. diameter spinning wheel, which is deeply flanged to receive it, and the spinning wheel was propelled across the river by the hauling rope, unwinding the wire from the reel by progressively lengthening the bight. Reference to the left end of the "first trip" diagram in Fig. 7 will assist in making this operation plain.

Arriving at the New York anchorage, the wire loop was removed from the wheel and placed around a strand shoe. The companion spinning wheel, attached to the same hauling rope, similarly and simultaneously pulled out two wires from the New York anchorage which were looped over a strand shoe at the New Jersey end. Thus the two spinning wheels for each cable spun four strands at one time, as shown in Fig. 7. All wire for two of the strands originated at one side of the river and all wire for the other two strands originated on the opposite side.

Of each of these two pairs of wire, the "dead" wire—that is, the branch of the loop whose end was fixed at the starting point—was successively adjusted behind the traveling wheel, so as to hang at predetermined sags in the first side span and in the main span. The sags were measured by comparison with the guide wires. After the loop was placed around the far strand shoe, the dead wire was adjusted in the last side span, and the "live" wire—that is, the branch of the loop which unwound from the reel—was successively adjusted in the last side span, the main span and the first side span, and placed around the strand shoe at the point of beginning, thus forming another loop to be pulled across by the spinning wheel.

This operation was repeated until the 217 loops or pairs of wire of a strand had been placed. The ends of the wire last pulled across was then spliced to the end of the wire fastened in front of the shoe at the start. Thus the strand was formed by a continuous length of wire. With all wires in place, the strands were lightly compacted by hand clamps and seized at intervals of 5 ft., ready to be picked from the spinning supports and lowered into position in the saddles. Views of this operation are shown in Fig. 6. Large balance beams were provided at the towers to accomplish this. Grommets or slings made of $\frac{3}{8}$-in. wire rope were placed under the

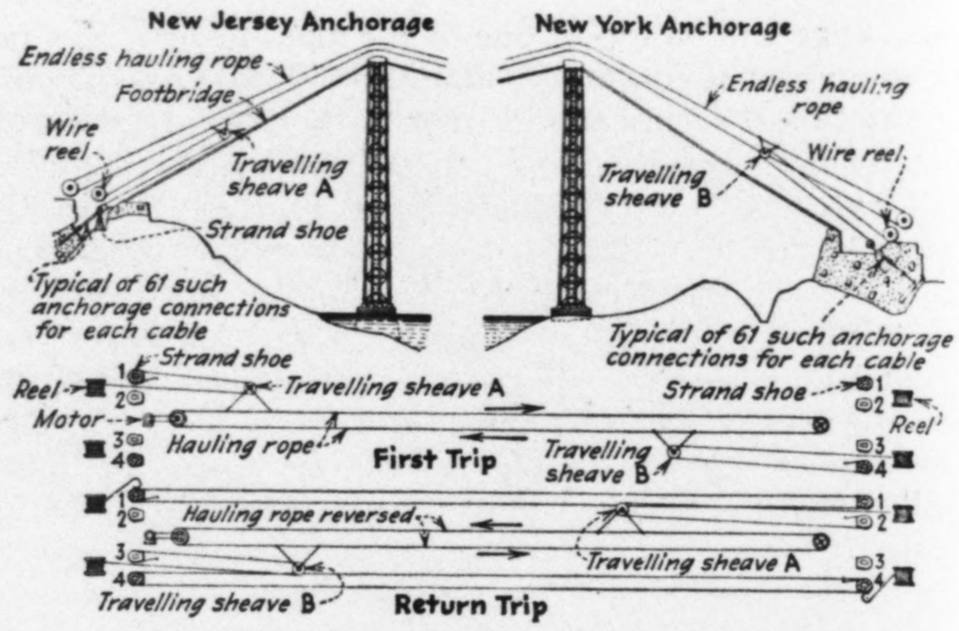


FIG. 7—DIAGRAMMATIC LAYOUT OF CABLE-SPINNING OPERATION

Reels and strands shown for one cable only. Note that on first trip wires are placed in strands 1 and 4 and on the return trip in strands 2 and 3.

strand and by means of grommet bolts a uniform tension was obtained on all the slings. The balance beam was then picked by the two hooks from the girder crane operating on the erection steelwork overhead. Approximately the same operation was performed at the anchorage saddles, where lighter loads permit the use of smaller balance beams, which are picked by 60-ton jacks.

Strand Adjustment—As the finished strand was lowered into the saddle the strand leg was jacked back, the attachment of strand shoe to the strand leg remaining until the shoe was finally fastened on its pin at the eyebars. This jacking was accomplished by means of a 150-ton hydraulic jack placed in a two-part wire-rope system which passed around a segmental sheave mounted on the pin at the lower end of the anchorage eyebar, as shown in Fig. 5.

A feature of the jacking arrangement was a set of bypass rods arranged so that they could be pinned off in varying positions, thus guarding against loss of the strand in case of jack failure. Two sets of jacking equipment were provided at each anchorage for each pair of cables.

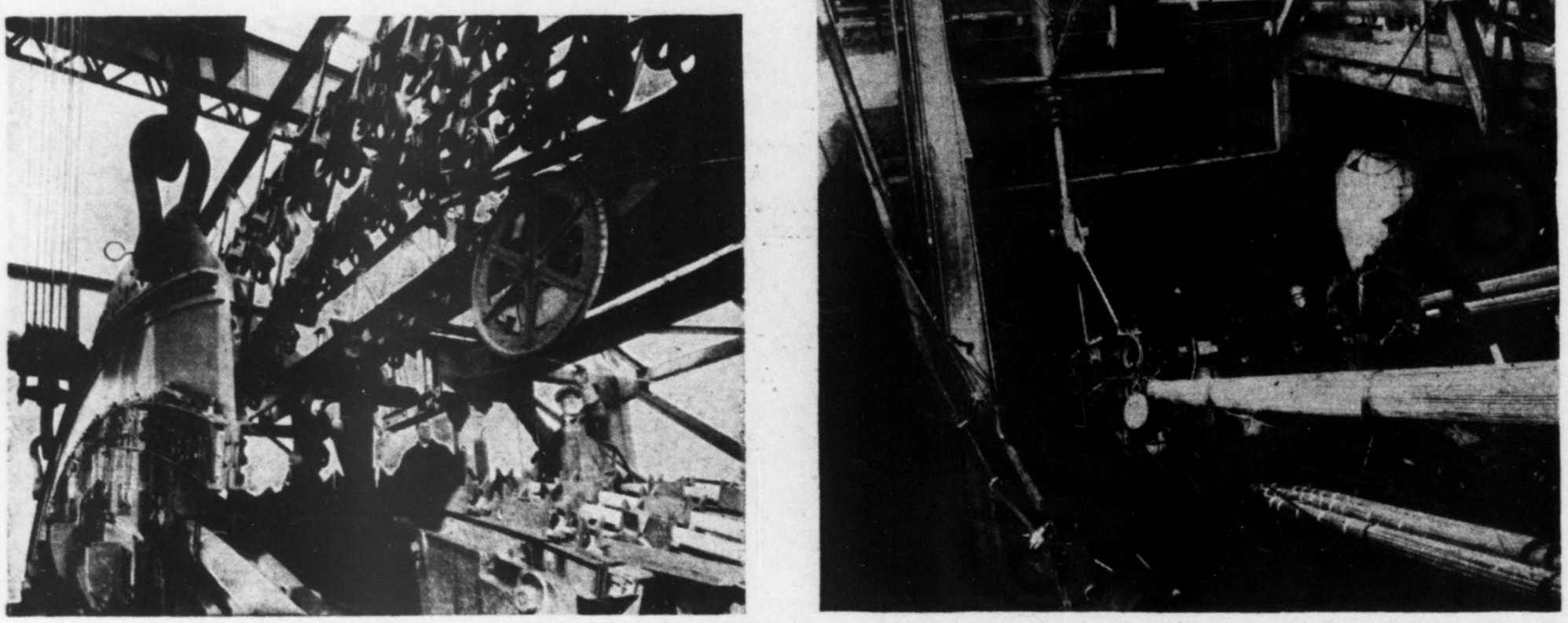


FIG. 6—HANDLING STRANDS AT CABLE SADDLES

Left: Strand being transferred to main tower saddle by balance beam attached to 65-ton hooks of overhead crane. Note other strands in temporary spinning supports; also spinning wheel passing through trolley blocks with a bight of cable wire. Right: Hydraulic hoist lifting strand into New Jersey cable-bent saddle which already contains three completed strands.

The first strand in each cable was set to the desired sags in the main and side spans by means of observations with survey instruments. All subsequent strands were adjusted to conform to the first and all adjustments in the main and side spans were obtained by jacking at the anchorage.

During the early stages of spinning, the saddles were set in position approximately 2 ft. shoreward of the center line of towers and were temporarily blocked against the possibility of rolling. Later this blocking was removed and the saddles permitted to roll, to balance the horizontal pulls exerted by the strands.

Plant Improvements

Certain items of equipment as installed for this work by the contractor are of unusual interest and have operated very successfully in the spinning and adjusting operations. An outstanding improvement relates to the speed of the spinning wheel, which was increased to 650 ft. per minute, compared with a maximum of 400 ft. per minute previously attained in work of this nature. Moreover, the speed was maintained continuously throughout the entire trip between anchorages, no slackening of speed being required as the wheels passed over the towers. Speed in adjusting also kept pace with the increased speed of the spinning wheel. These important advances in the art are due principally to three distinctive features: power drive for the reels, improved tramway-rope supports, and an electric signal and control system.

On each anchorage two unreeling machines (Fig. 9) were provided for each cable. Heretofore the wire has been pulled from the reels by the action of the spinning wheel, a brake being used to control its motion. This problem of control became of great importance in consideration of the heavier reels used on the Fort Lee bridge, the greater speed of travel of the spinning wheel and the greater accumulation of slack over the 5,000 ft. of length between the anchorages.

The unreeling machines were designed to engage the drum of the loaded reel, a 100-hp. electric motor furnishing the power to drive four unreeling machines. They were controlled by a hydraulic system, which was readily manipulated and permitted rapid changes in speed. The operator synchronized the speed of the unreeling machine with the speed of the spinning wheel by the use of a counterweight frame or tower which was erected in front of the reeling machine location on the top of the anchorage, as shown in Fig. 5. The wire coming from the reel passed over a sheave or wheel at the top of the counterweight tower, thence looped down around a weighted sheave and passed again over a wheel at the top of the frame. By observing the position of the counterweighted sheave as it rode on the suspended loop in the frame, the operator was able to drive the unreeling machine at the rate required by the spinning wheel that was pulling out the loop.

The method of attachment of the spinning wheel to the tramway rope is especially important, and on this work the offsetting "gooseneck" used on former projects was eliminated. Instead, the frame was hung directly below the rope and at the rope consisted of plate connections $\frac{1}{4}$ in. thick. Between the two points of attachment of the frame the diameter of the tramway rope was enlarged to that of the connection by a continuous series of interlocking sectional rings. Beyond the frame the diameter then tapered to the original rope section.

The tramway rope retaining wheels on the overhead frames were arranged in sets such that two sheaves or wheels located just above the rope took a slight reaction from the running rope under ordinary conditions, while at the time of passage of the spinning wheel its weight forced the rope down on a pair of wheels which had outside flanges only, were set at an inclination to each other and were so spaced as to permit the free passage between them of the $\frac{1}{4}$-in. plate connections of the spinning wheel frame.

On this work the necessity of visual signals between the adjuster at midspan and the men operating the "come-along" was eliminated. The "come-alongs," which grip the wire and permit of its being pulled up or slacked off as desired, were operated by a motor controlled by the observer at midspan. This enabled him to adjust the wire to correct sag and having accomplished this, he signaled the fact ahead by turning a switch which lighted a lamp on an indicator panel at the adjustment point, where the wire was then clamped off and the signal passed along.

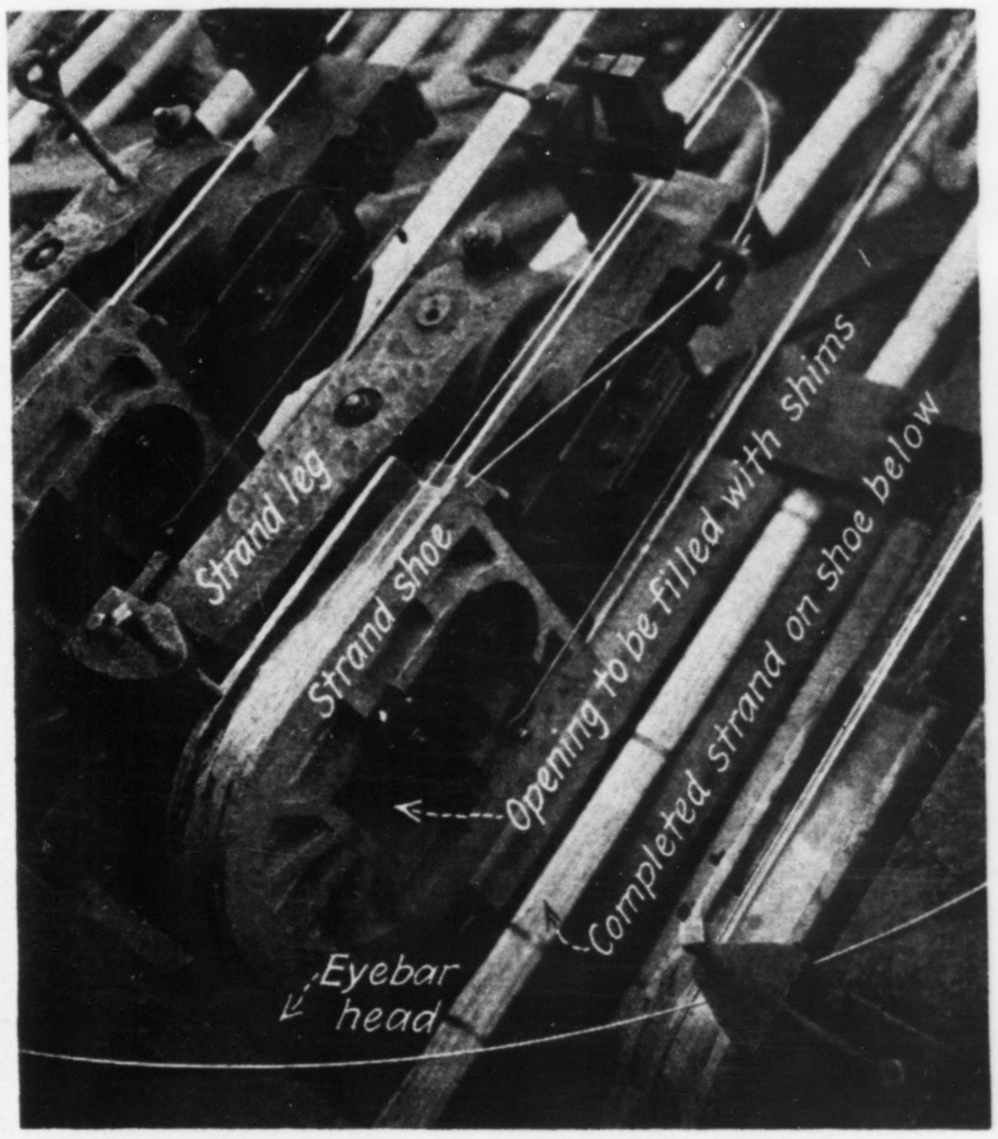


FIG. 8—SPINNING ARRANGEMENT AT STRAND SHOE
Strand leg is a large U-shaped casting which holds shoe during spinning in vertical position ahead of its final position which is pinned to the eyebar head at left. Shoe is pulled back to final position by 150-ton jacks shown in Fig. 5.

A number of control points for making emergency spinning wheel stops were located along the footwalks. A complete system of telephones was also maintained.

The despatcher house, located in the cut just behind the New Jersey anchorage, was an innovation on a spinning operation. At this point record was kept of the progress of the wheel. If stopped at any point on the walk the annunciator system immediately gave the position and the operator was able to obtain the reason by phoning to the indicated location. Unnecessary delays and misunderstandings were thus eliminated and lost time due to wheel stoppages reduced to a minimum.

The adjustable working platforms, shown in Fig. 5, riding on inclined trusses at the anchorages, and the girder crane and runway for handling reels to the New Jersey anchorage may be mentioned as other important features of the plant.

The hydraulic splicing equipment warrants a brief

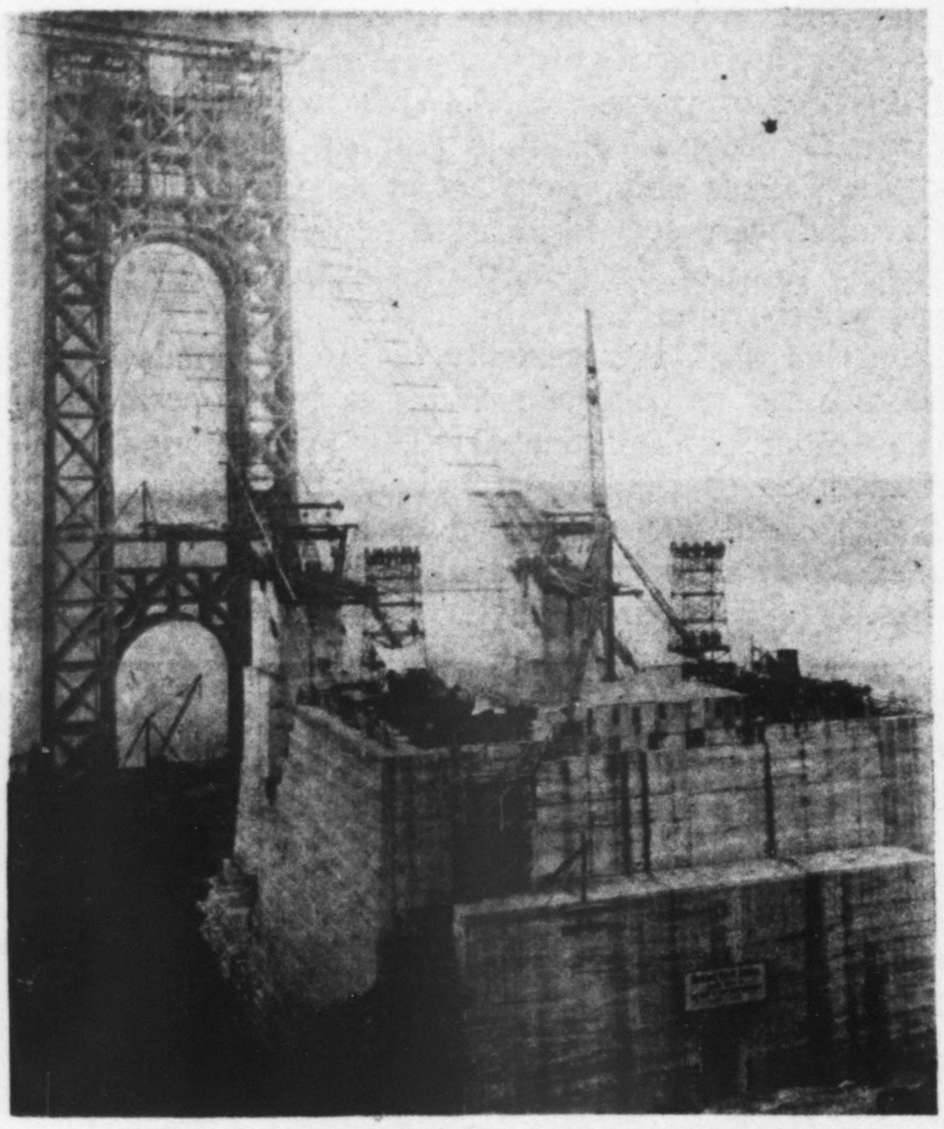


FIG. 9—CABLE-SPINNING EQUIPMENT AT THE NEW YORK ANCHORAGE

Footbridges under construction. Note two counterweight towers on top of anchorage which assure proper tension in the spinning wire. Note also hubs for wire reels.

description. Machines identical with those used in making splices in the shop were installed at the anchorages. The end of a wire to be spliced was so pressed that an elliptical section was produced at each groove, no attempt being made to secure a thread. The ferrule or coupling was roughened on the inside. In making a splice the ferrule was laid between the jaws of a hydraulic press, the wires to be spliced entered at either end, and a pressure of about 200,000 lb. maintained for one minute. Such splices proved almost 100 per cent efficient.

Spinning Organization

As a matter of efficient organization, the work was carried on in such a way that spinning was in progress on three cables while the fourth was in the process of strand adjustment. Such an arrangement allowed for the best utilization of equipment. Also, by this plan of operation only three spinning crews were required and these could be kept continuously occupied. It also proved advantageous to have a group of men at each anchorage who did nothing but adjust strands. An average of a dozen men were employed at each anchorage for this purpose. Crews of eight men were also required for adjusting at the tower tops. During spinning approximately 65 men were scattered throughout the length of each cable. A total of about 375 men was engaged in the cable-spinning operation.

The minimum time required for spinning a set of strands was nine calendar days. Adjusting and preparing the next set-up of eyebars and strand legs, etc., required another two to three days, making a total period of about twelve days.

Steps Following Completion of Spinning

Following completion of the spinning the cable must be compacted between points of support. The squeezing will be accomplished in two stages. The first stage with a light squeezer will reduce the cable to a diameter of about 38 in. This operation will then be repeated using a heavier squeezer to reduce the cable to 36 in. The squeezer completely encircles the cable and pressure is applied radially on twelve segments.

Cast cable bands will be placed at each panel point for supporting two loops of suspender rope. The suspender ropes will be measured, cut and socketed at the site after removal from their present position as footbridge supports.

The wrapping, which is the last stage of the cable-manufacturing process, cannot be done until the suspended structure and roadway slabs are in place, in order that the cables may have attained as much of their stretch as possible. A soft annealed double-galvanized steel wrapping wire of 0.148 in. diameter will be used, wrapped under tension by machine. At the cable bands the wrapping is discontinued just under the lip of the band and lead wool is calked into the connection, the result being, when complete and painted, a very effective and relatively impervious protection for the cable wire.

Administration

The Fort Lee bridge is being built by the Port of New York Authority at an estimated cost of $60,000,000. Nine contracts covering various phases of the construction work have been let to date. The contract to furnish the wire and erect the cables, to furnish and erect the cable bands and suspenders and anchorage steelwork was awarded to the John A. Roebling's Sons Co., of Trenton, N. J., on Nov. 4, 1927, at an estimated contract price in excess of $12,000,000.

The design and erection of the bridge is under the direction of O. H. Ammann, chief engineer, the Port of New York Authority. Allston Dana is engineer of design and Leon S. Moisseiff advisory engineer of design. The field work and all phases of the erection are under the direction of Montgomery B. Case, engineer of construction. Homer R. Seely is resident engineer in charge of cable-spinning operations. C. C. Sunderland is chief engineer of bridges for the John A. Roebling's Sons Co., which company maintains a corps of engineers on the work with W. E. Joyce as resident engineer.



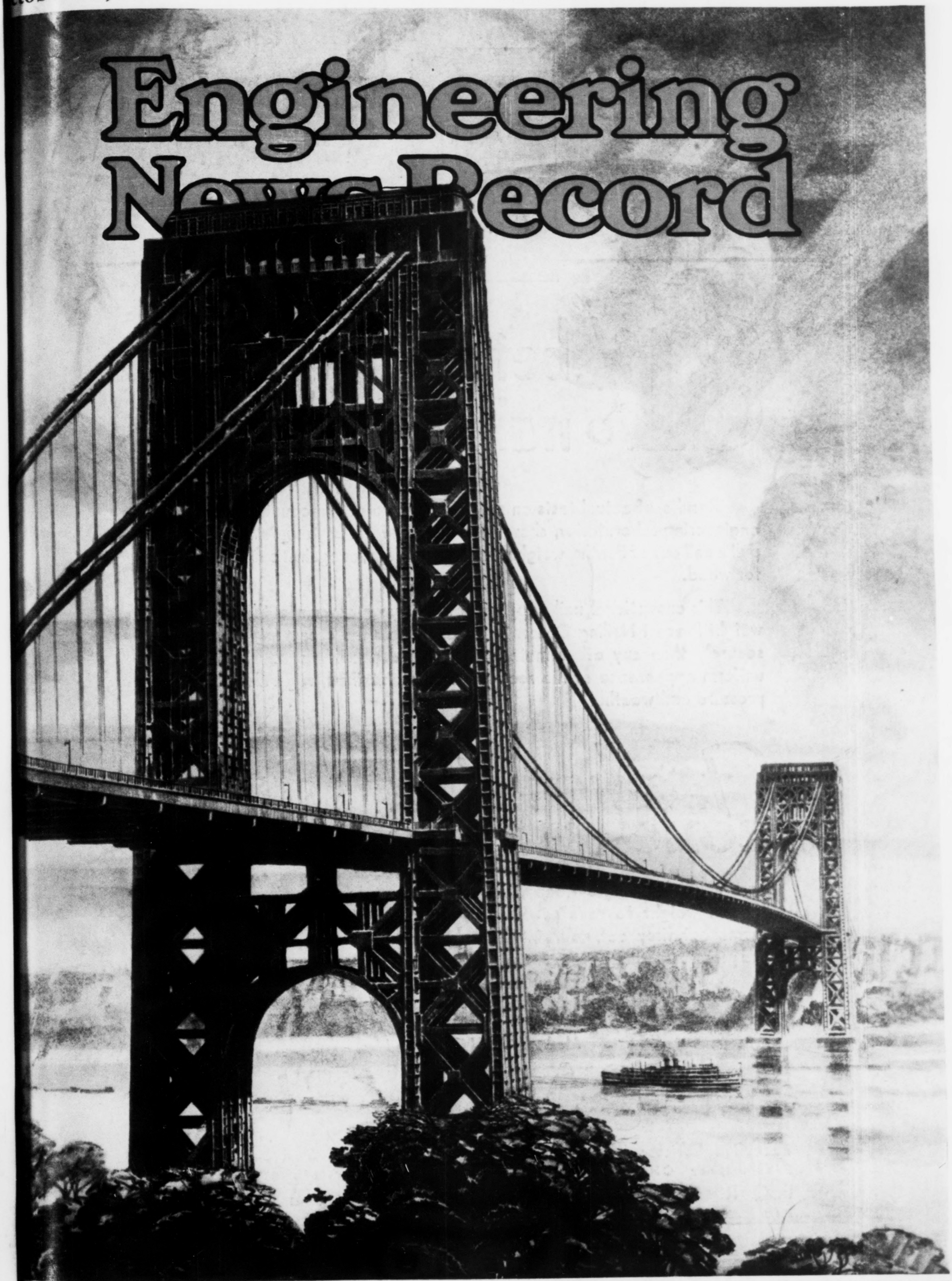
THE 3,500-FT. FORT LEE SPAN AT NEW YORK

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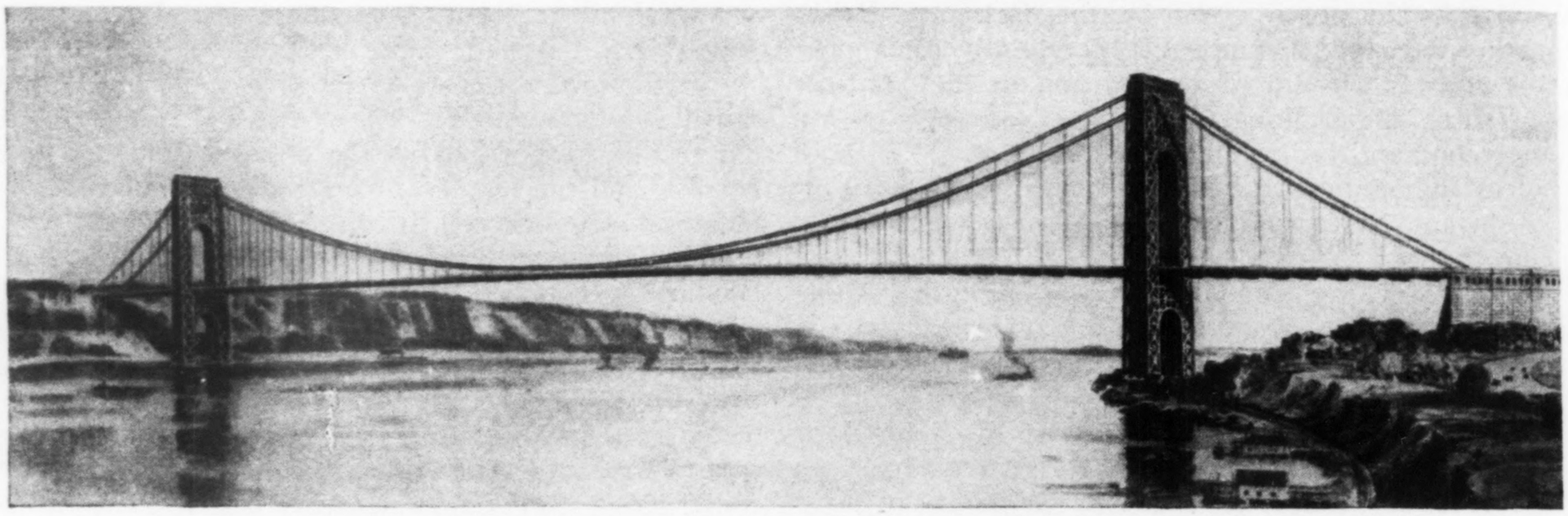
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Engineering News-Record



Hudson River Bridge at New York

—From a lithograph by J. Floyd Yewell



George Washington bridge over Hudson River connects 178th St., New York, with the Borough of Fort Lee in New Jersey by means of a main suspended span of 3,500 ft.

Bridging the Hudson at Fort Lee

The Engineering Design

Its controlling conditions, and the main elements of the 3,500-ft. suspension crossing just opened to traffic

MORE than 50 years of unsuccessful planning preceded the final achievement of the just completed suspension bridge across the Hudson between New York, N. Y., and Fort Lee, N. J., though the first beginnings of the executed project go back only eight years. The great span, nearly twice as long as the longest previous bridge spans—Quebec and Detroit—was designed as a suspension bridge because of the foundation conditions and because of its superior economy. In arrangement and detail it is characterized by great structural simplicity. The present structure is only the first stage of the final bridge, provision being made for adding a lower deck to carry rapid-transit trains. The main carrying members are four parallel-wire cables 36-in. in diameter; eyebar chains had also been designed, but the wire cable bids were lower. The floor is wide and heavy; because of the great cable and floor weight, the stiffening and wind systems are relatively unimportant. Unique steel towers support the cables; they may later be inclosed by concrete walls. One anchorage is of tunnel type, the other of gravity type.

Four years ago, on Nov. 4, 1927, contracts were let for building the superstructure of the Fort Lee Bridge over the Hudson River at New York, since named the George Washington Bridge. In preparation for the bidding, two alternative designs had been prepared, one with wire cables and the other with eyebar chains as main carrying members. These two designs were described in detail in *Engineering News-Record* of Aug. 11, 1927, p. 212. The wire-cable design brought the lowest bids, and was adopted for execution. On Oct. 25, 1931, the finished bridge will be opened to traffic, though parts of the approaches are still in temporary condition.

During the four years of construction the natural evolution of the design brought about numerous detail changes. Most of these are minor, but it seems proper at this time to record them and to acquaint the engineering world fully with the design as it was actually carried out. This is done for the main elements of the structure in the present article, while the tower and floor construction and the approaches are dealt with in immediately following articles. Other articles on the bridge have appeared in this journal as follows: "Design of Steel Towers," May 24, 1928, p. 819; "Deep Open Cofferdams for New Jersey Pier," Aug. 16, 1928, p. 232; "Construction of New Jersey Tunnel Anchorage," Oct. 25, 1928, p. 611; "Pouring 106,000-Cu.-Yd. Block of Concrete for New York Anchorage," April 11, 1929, p. 591; "Manufacturing High-Modulus Footbridge Rope," May 1, 1930, p. 714; and "Spinning Four 36-In. Cables for Fort Lee Bridge," Aug. 14, 1930, p. 242.

Distinguishing Characteristics

The new bridge occupies a position of peculiar note not only because of its great size but also because of its long history and its distinctive structural character. More than 50 years of planning went before its successful accomplishment. Its 3,500-ft. span outreaches in one prodigious bound the greatest prior achievenments in bridging—the 1,800-ft. spans of the Quebec cantilever over the St. Lawrence and of the Ambassador suspension bridge over the Detroit River. It is marked by many features of individuality, among others (1) double-deck construction; (2) provision for building the structure in two stages, of which only the first has so far been completed; (3) massive steel towers made up of four four-column arch bents; (4) four 36-in. parallel-wire cables set in pairs over the interior columns of the bents; (5) very shallow stiffening trusses in the final construction, and no stiffening trusses whatever in the first stage; (6) a rather flexible windbracing system in the floor, the main cables chiefly supplying the transverse stiffness; and (7) a highway deck of concrete-embedded steel joists.

To record its main dimensions: the main span is 3,500 ft. long, the suspended side spans 650 ft. each. The primary supports are two steel towers, 588 ft. high from mean water to center line of cables. They support four cables 36 in. in diameter, two side by side on either side of the towers. The cables hang in vertical

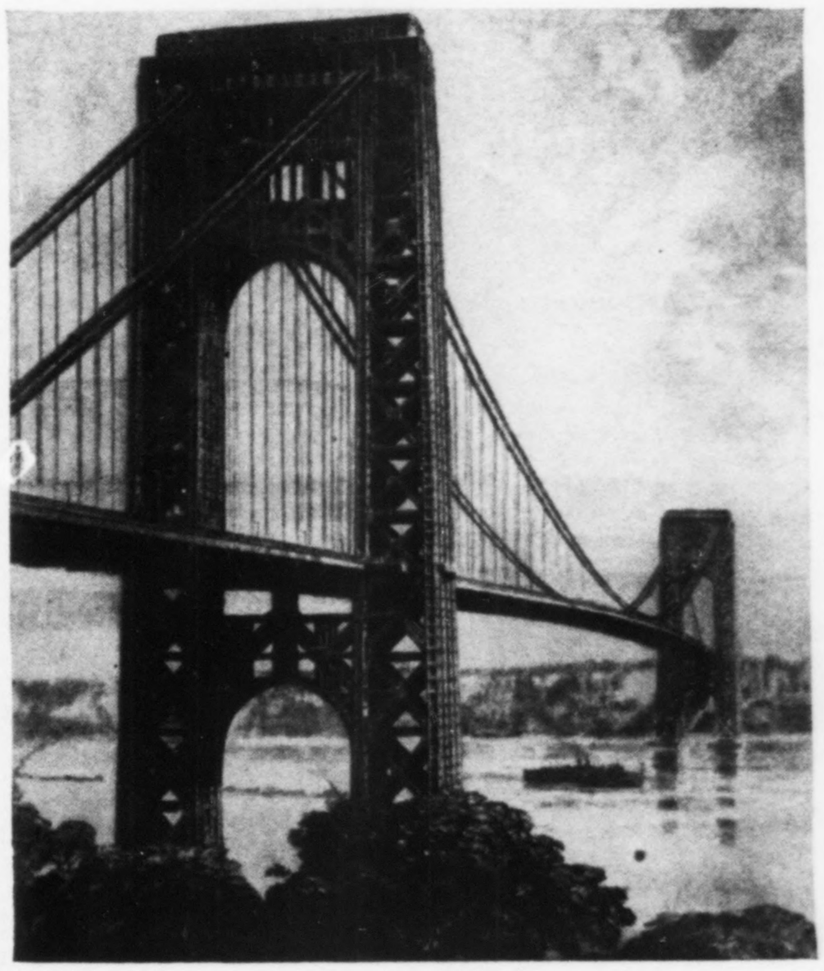
planes. The present (upper) deck is 106 ft. wide and is planned to carry ultimately eight vehicle lanes. Each end of each floor beam is hung from four $2\frac{7}{8}$-in. suspender ropes looped over the cable bands. The future lower deck is to carry four rapid-transit railway trucks. When it is built, a light stiffening truss 29 ft. deep will be constructed on either side, in the space between the two decks; there is no stiffening truss at all in the present condition, but the upper chords of the future trusses are in place, serving as chords of the wind truss. The roadway is a $7\frac{1}{2}$-in. concrete slab incasing longitudinal bulb beams set 15 in. apart; there are two roadway strips 28 ft. 9 in. wide, separated by a 30-ft. middle strip left unpaved for the present. The future lower deck will afford a navigation clearance height of 213 ft. at midspan · the present clearance height is some 40 ft. greater.

Fifty Years of Planning

Bridging the Hudson at New York has been the subject of dreams and projects for more than 50 years. The width and depth of the waterway, its soft clay bottom, with rock far out of reach, the densely populated shores and the busy traffic both along and across the river were among the factors that either stimulated or discouraged such planning. Many designs were made, numerous locations were studied and divergent views on important questions of bridge design were developed, among them the views set forth in that engineering classic, "Maximum Spans for Suspension Bridges," prepared by a board of army engineers nearly 40 years ago.

Since the first bridge was proposed trans-Hudson traffic has increased greatly in volume; its center of gravity has moved north away from the congestion and high land values of downtown New York, and its governing conditions have changed so that while formerly a railway bridge was sought for, today the chief need is for highway crossing. These changes in volume and character of traffic largely determined the design of the present bridge, if indeed they did not make the bridge possible today where previously every project met failure through one cause or another. But also the establishment of an interstate public cooperation with independent financing powers and the right to levy tolls to repay construction cost was a vital element; all the earlier projects were for construction by private companies as an enterprise for profit.

Earlier sites of bridge projects were at Tenth, 23d, 57th, 59th and 68th Sts. One or two of these contemplated a pier at midstream, but after borings showed great depth of soft ground they were changed to single-span projects. Most of them were for a suspension bridge, but cantilevers and arches were also proposed; spans ranging from 2,500 to nearly 4,000 ft. The most noted of early projects is still active. It is that of the North River Bridge Co., with Gustav Lindenthal as chief engineer since its inception 43 years ago, in which the Pennsylvania R.R. at one time took interest. The present site of this project is at 57th St. Not only is it the only project still active but it has a close relation to the bridge now completed because the engineer of the latter, O. H. Ammann, was long connected with the 57th St. project and took part in working out its suspension bridge designs.



Massive steel towers of unprecedented design suspend the roadway 250 ft. above the waters of the Hudson

Investigation of the 178th St. site dates back to 1910, when the Interstate Bridge and Tunnel Commission, a joint body of the states of New York and New Jersey, considered the possibility of building there a 1,400-ft. cantilever highway bridge with river piers; a year later, when borings showed that rock could not be reached in the main part of the river width, the scheme was abandoned. It marks quite definitely the time when highway traffic needs had attained such importance as to claim first attention, in contrast to the earlier planning for railway bridges.

But though the Bridge Commission had abandoned its study, yielding to the superior desirability of a highway tunnel 10 miles south—later built, the present Holland Tunnel—study of a bridge at 178th St. was taken up anew in 1923 by O. H. Ammann, then an engineer in private practice, later bridge engineer and now chief engineer of the Port of New York Authority. Using this investigation as a basis, the states of New York and New Jersey in 1925 passed legislation empowering the Port of New York Authority to build a bridge in the general vicinity of Fort Washington Point on the New York side and Fort Lee on the New Jersey side. In 1926 the states made the initial contributions to an eventual $10,000,000 advance of funds, and directed the Port Authority to proceed with the construction of the bridge. Later the Port Authority issued $50,000,000 of $4\frac{1}{2}$ per cent bonds, secured by toll revenues to be derived from operation of the bridge.

Basic Site Selection Factors

In determining upon the location of a bridge at 178th St., Manhattan, the geography and topography of the site as well as potential traffic were decisive factors. In so far as geography is concerned, the site has two principle advantages: At the Manhattan end the approaches occupy much less expensive property than would be the case for a bridge farther south, while at the New Jersey end the bridge taps relatively undeveloped but highly desirable suburban residential territory and also affords access to the industrial cities of Paterson and Passaic. A bridge in this location also permits traffic from the north and east to bypass lower Manhattan and, in conjunction with the proposed Tri-Borough bridge over the East River, provides a continuous route from New Jersey to Long Island outside of the congested sections of New York City.

An important consideration in fixing the location of the bridge at 178th St. is the fact that careful studies of pro-

spective vehicular traffic showed that such a structure would pay since ferries had proved inadequate. In 1926 it was estimated that the number of vehicles crossing the Hudson River in 1930 would be 19,800,000. Actually the recorded number of vehicles crossing the river during 1930 was 25,500,000—nearly 30 per cent greater than the estimate. This same 1926 forecast placed the number of vehicles that would use the Hudson River bridge in 1932 at 8,600,000, increasing in 1946 to 15,000,000. On the basis of the 30 per cent recorded increase in trans-Hudson traffic between 1926 and 1930 a new forecast was made in 1930, which placed the number of vehicles crossing the Hudson River bridge in 1932 at 8,700,000, in 1934 at 10,100,000, and in 1946 at 25,100,000. Based upon the 1926 forecast of traffic and an average toll rate of 50c. per vehicle and 5c. per passenger in vehicles, exclusive of driver, the annual net revenue was estimated to increase from a minimum of $5,250,000 in 1932 and to be sufficient, with a substantial surplus, to meet interest and sinking fund payments during a period of amortization of about 25 years, as well as to refund with interest the amounts advanced by the two states.

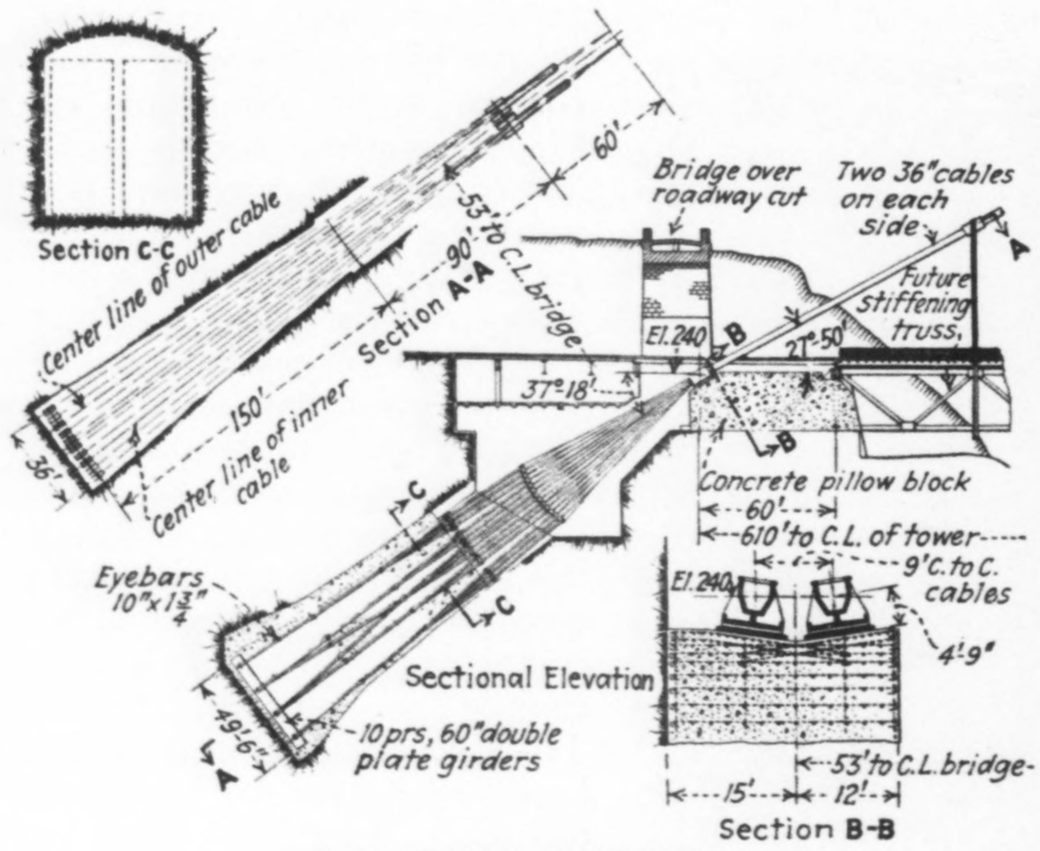
Topography—The topography of the site proved ideal for a long single-span bridge, in fact demanded it. The ground on both sides of the river is high. In New Jersey 500 ft. from the shore line it reaches a height of 280 ft., while in New York about 1,000 ft. from the shore line a height of 200 ft. is attained. Assuming that a 200-ft. clear height at the center was necessary for navigation clearances, the upper roadway of a bridge would be about 240 ft. above the water, which corresponded closely to the topographical conditions.

On the New York side rock is exposed, forming a rocky cape, Fort Washington Point, from which it shelves down steeply toward the navigation channel. On the New Jersey side of the river rock was found at depths of 115 to 170 ft. near the pierhead line, but it also falls off sharply toward the center of the river. It is overlain by material ranging from soft mud to plastic clay. By placing the New Jersey pier about 200 ft. inside the pierhead line, rock could be reached at a maximum depth of 75 ft. and an average depth of 50 ft. This rock was a hard, compact shale and sandstone in strata inclined toward the shore and thus ideal for foundations that needed to be designed with assurance that no tendency to slide toward the river gorge would occur. The Palisades cliffs are a hard diabase, an igneous rock of volcanic origin commonly known as traprock, which would prove ideal for the New Jersey anchorage of a suspension bridge. Tests from cores taken from the site of the New Jersey pier showed variation in strength from 3,000 to 24,000 lb. per sq.in. with a probable average of between 12,000 and 15,000 lb. per sq.in, or more than 30 times the maximum edge pressure of the tower base.

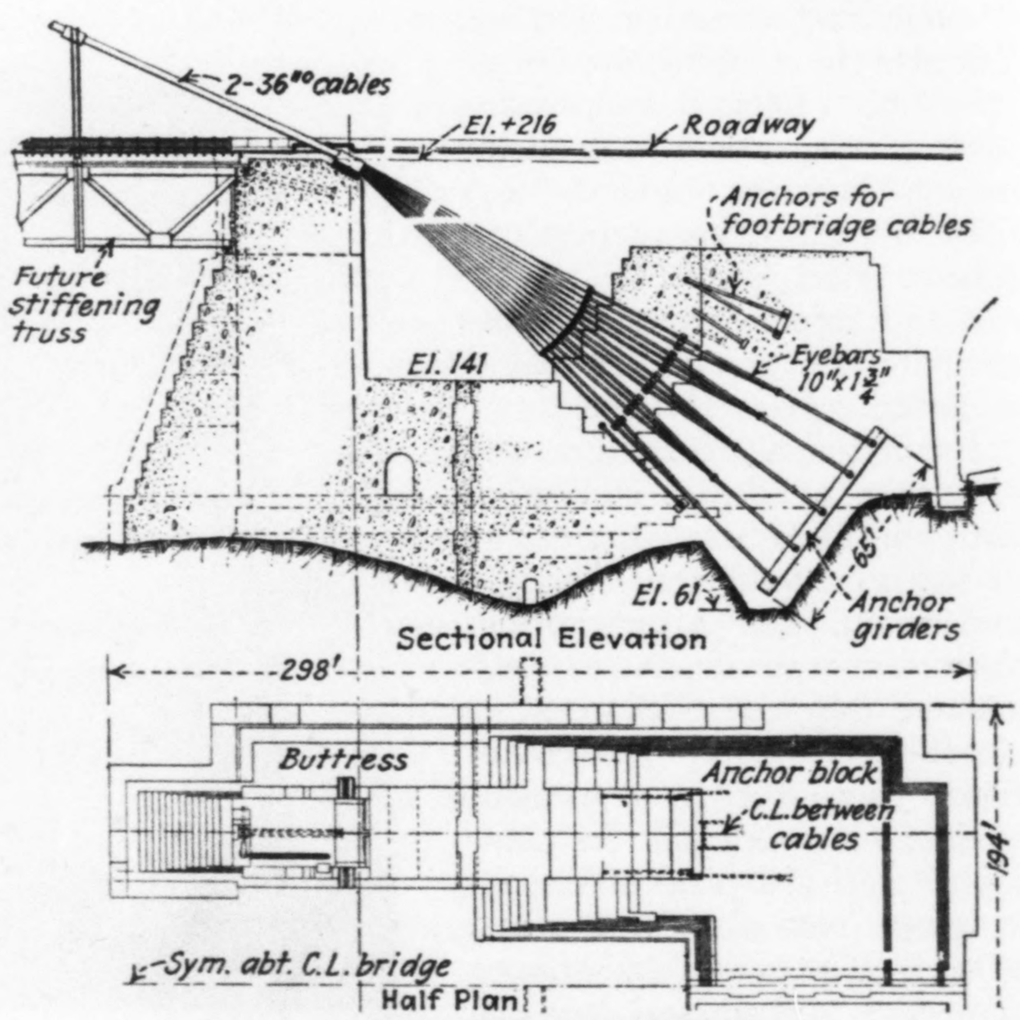
Suspension Design Chosen

Thus topographical and geological conditions indicated clearly the points of support for a suspension bridge and the location of the anchorages. The New York tower was placed on the extreme rocky point of Fort Washington Park, the New Jersey tower 200 ft. back of the pierhead line in order to secure rock at a reasonable depth, thus fixing a main span 3,500 ft. long between supports. The New Jersey anchorage logically was placed in a tunnel in the Palisades some 600 ft. west of the New Jersey tower. High ground in Fort Washington Park offered a symmetrical location for the New York anchorage, but one requiring a gravity structure on top of the ground. Careful study of costs showed that such an anchorage would be cheaper than carrying the side span back to higher ground east of Riverside Drive. In addition the short side span, of practically the same length as the New Jersey side span, preserved the symmetry of the bridge. Although all conditions of site and topography pointed to the use of a suspension bridge, other bridge types were considered, but they were rejected as too costly.

Capacity—Statistics on trans-Hudson traffic indicated

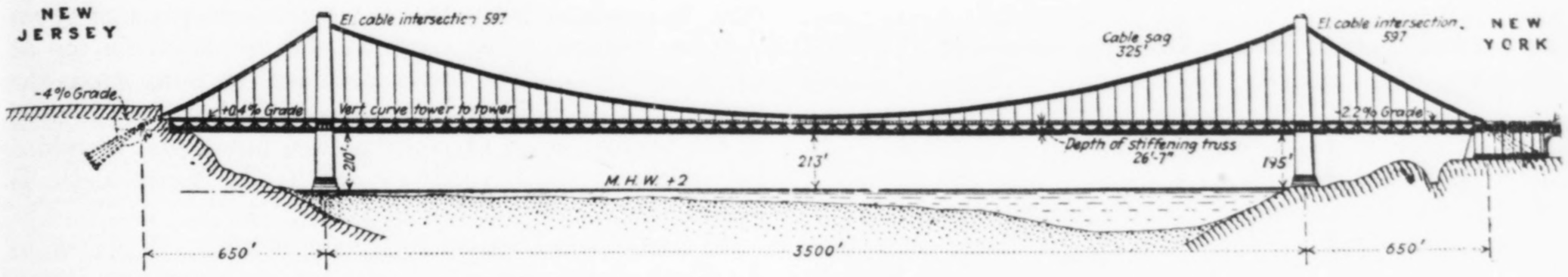


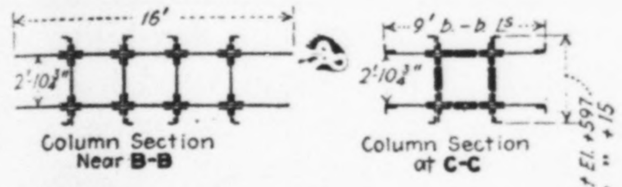
NEW JERSEY ANCHORAGE

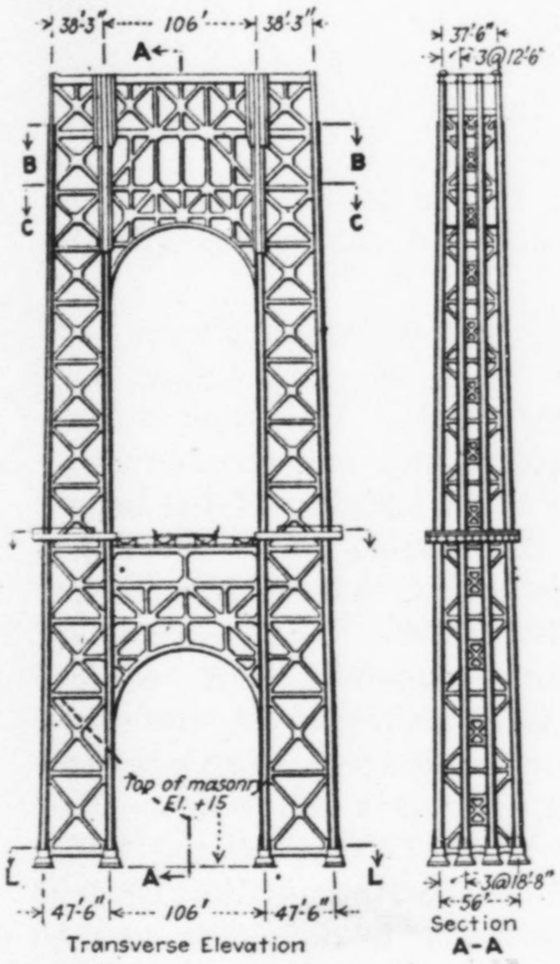


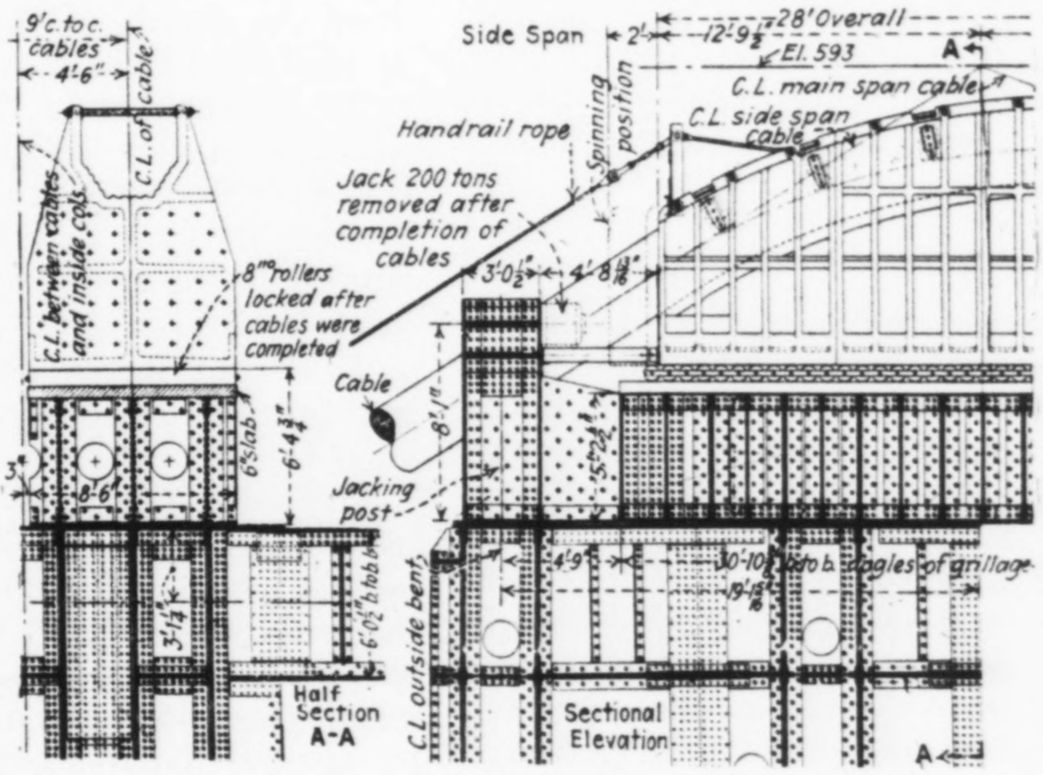
NEW YORK ANCHORAGE

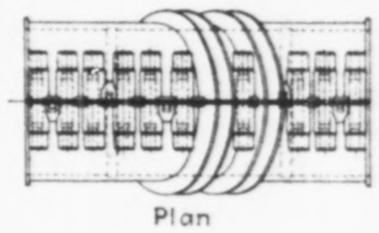
New Jersey end of bridge is anchored in a deep tunnel in the traprock Palisades, while a 106,000-cu.yd. concrete block forms a gravity anchorage on top of the ground at the New York end

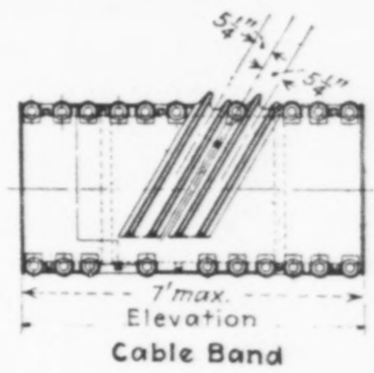




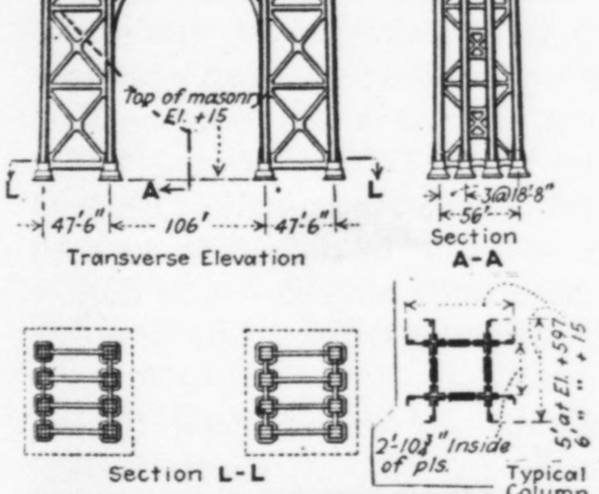




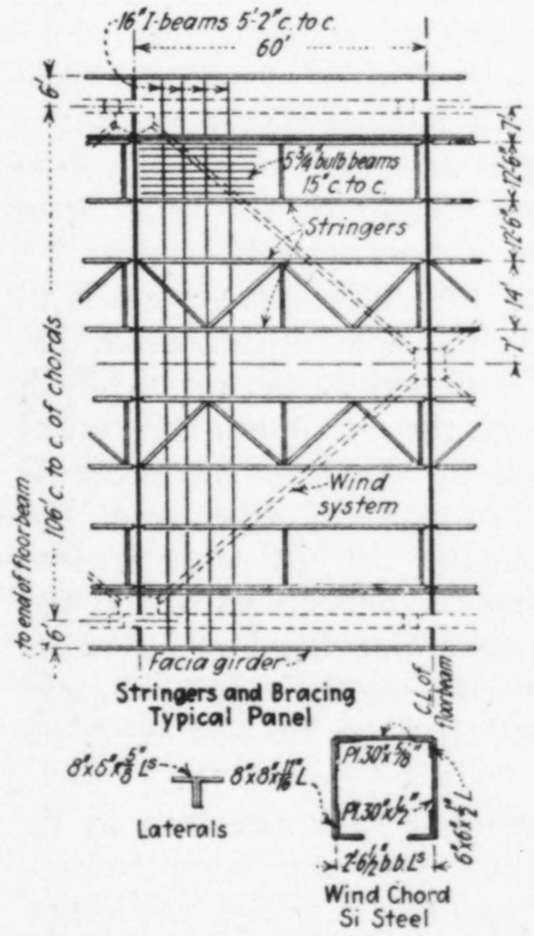


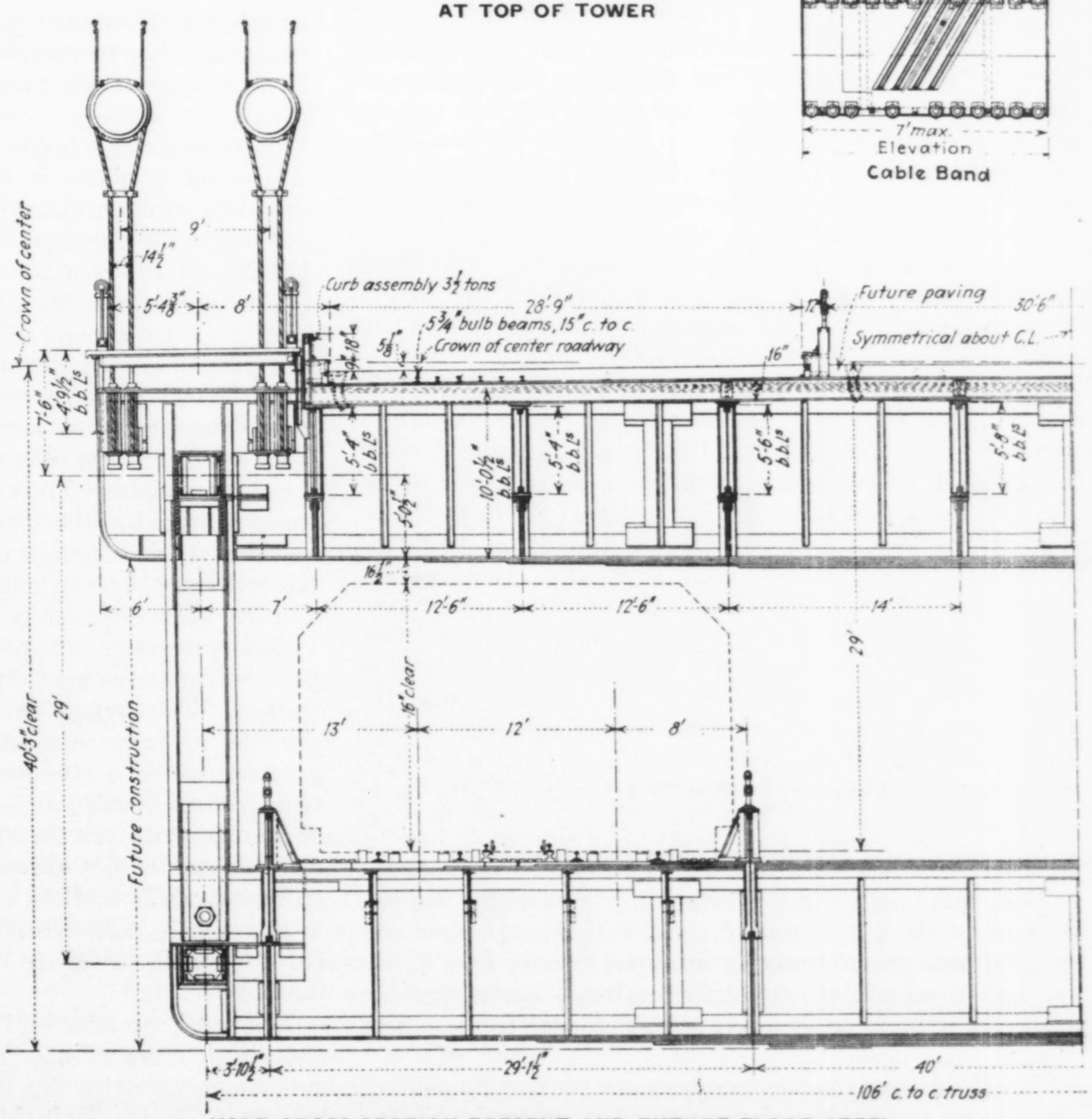


CABLE SADDLE ARRANGEMENT AT TOP OF TOWER



ARRANGEMENT OF SKELETON TOWERS (First Stage)





HALF CROSS-SECTION PRESENT AND FUTURE FLOOR STEEL

Principal design elements of 3,500-ft. span George Washington Bridge over Hudson River at New York

that traffic crossing the bridge during the first five years of operation could be handled by a four-lane roadway. Doubling this capacity would provide for 25,000,000 vehicles annually, the estimated traffic volume in 1946. This volume of traffic is provided for by a deck 90 ft. wide between curbs. Two 6-ft. sidewalks are provided. The possibility that rapid transit facilities might be required in the future made provision for them a necessity. Consequently the design includes a lower deck beneath the present highway deck to carry four lines of rapid-transit tracks, this lower deck to be added when needed.

Dimensions and makeup of the various parts of the bridge are given in the accompanying drawings. There follows a discussion of the principal elements of the design as they were finally determined.

Cables and Suspension System

The controlling criteria in designing the suspension system were structural simplicity, economy with necessary rigidity, provision of the widest possible competition in materials and erection methods and pleasing appearance. In line with the desire for simplicity, the designers chose a plain, unstiffened cable, supplemented by shallow parallel-chord stiffening trusses. To assure the widest possible competition, complete designs were made utilizing both eyebars and wire cables as the main carrying members. From an esthetic viewpoint the engineers determined upon plain massive towers and a flat catenary, producing a suspended structure light and graceful in appearance but giving the impression of being adequately supported at either end.

Bids were asked on both the eyebar and the wire cable designs, on simultaneous and successive construction of the two pairs (for first and final stages), and on side-by-side and vertically superposed position of the two cables on either side. The lowest bid was for wire cables, placed side by side, and constructed simultaneously. A two cable design had been considered, but was rejected because of the great size of cable required. The cables actually used, being 42 per cent larger than any heretofore built, called forth unusual erection procedure and equipment. (*ENR*, Aug. 14, 1930).

The two cables on either side of the bridge are 9 ft. apart. Each cable contains 26,474 cold-drawn galvanized wires 0.196 in. in diameter, having an ultimate strength of 240,000 lb. per sq.in. For purposes of construction the cable was divided into 61 strands $4\frac{1}{2}$ in. in diameter. The cables are supported on the towers in cast-steel saddles $8\frac{1}{2}$x28 ft. in plan, 10 ft. 9 in. high, weighing 200 tons. Carried on rollers during the spinning operations, these saddles are fixed in position in the completed structure. Standard cast-steel cable bands, carrying two loops (four lines) of suspender ropes are placed around the cables at each panel point. Final wrapping of the cables, accomplished after the floor was in place, was done with soft-annealed double-galvanized steel wire 0.148 in. in diameter.

Floor and Bracing

The floor and wind systems are unusually simple. Because of the extreme dead weight of the structure and its short side spans, insuring inherent rigidity and stability, both windbracing and stiffening truss action become relatively light in suspension bridge design for the first time.

The main upper floor members are deep silicon-steel plate-girder floor beams hung from the suspender ropes. Plate-girder stringers 60 ft. long are framed between the floor beams and carry 16-in. I-section crossbeams about 5 ft. on centers. The roadway slab is placed on top of the crossbeams. The lower deck will be hung from the upper deck by rigid frames. No stiffening truss is used in the present stage of construction, but it will be added below the present roadway when the lower deck is added.

A single wind truss, relatively flexible, is placed in the plane of the upper deck, the top chords of the future stiffening trusses being installed at the present time to form the chords of these wind trusses. The flexibility of this wind truss causes much of the lateral wind load to be transmitted to the towers directly by the cables, which, with such heavy cables as are used on this bridge, is an economical expedient. The design studies indicated, in fact, that the wind truss might have been entirely eliminated except for its effect in preventing local distortions in the floor.

The wind forces on the future lower deck will be transmitted to the wind truss and cables through the rigid frames.

Stiffening Trusses—The future stiffening trusses will by only 29 ft. deep, will be of silicon steel, and will be located wholly below the upper deck. A shallow stiffening truss was selected for appearance and economy and was made possible by the inherent stability of the heavy main span and the rigidity contributed to the structure by the relatively short side spans, whose steeply inclined cables act as rigid backstays. The remarkably shallow depth of the stiffening trusses, 1/120th of the length of the main span, is made apparent by comparison with other suspension bridges: on the Delaware River bridge at Philadelphia, this ratio is 1/63d, on the Manhattan Bridge in New York, 1/60th, on the Bear Mountain Bridge 1/63d and on the Ambassador Bridge at Detroit 1/84th. It is interesting to note that on the proposed Golden Gate bridge, with only a single highway deck, the depth of the stiffening trusses will be only 1/168th of the span.

The stiffening truss designed for the Hudson River bridge is practically flexible in that it exerts little restraint upon the distortions of the unstiffened cables, but is itself forced to deform with the cables. In the initial stage of construction, with no stiffening truss, the floor system is entirely unrestricted vertically except by the cable weight. The use of a flexible system for such loads as rapid-transit cars is a new development, but is justified in the opinion of the designers of the Hudson River bridge by the extreme weight of the main span and the short side spans with almost straight cables.

The governing function of the stiffening system is to prevent excessive gradients in the floor during cable distortion. The trusses were designed to permit safely a gradient of 5 per cent, although the maximum theoretical gradient under a combination of live load and extreme temperature is calculated not to exceed 2.4 per cent. In preliminary studies the stresses in the stiffening trusses were determined by calculating the deflections of the unstiffened cables and the corresponding bending moments in the trusses, subsequently correcting the results slightly for the small stiffening effect that the trusses exert on the cables.

The economy effected by using flexible trusses may be seen by a comparison of weights of stiffening trusses and windbracing in the Hudson River bridge and other bridges. On the Bear Mountain Bridge, having a span of 1,630 ft. and designed for highway traffic only, the

Principal Contracts, George Washington Bridge

Character of Work	Date of Signing	Contractor	Amount
Foundation—New Jersey tower	May 19, 1927	Silas B. Mason, Inc.	$1,060,000
Excavation—New Jersey anchorage and approach east of Hudson Terrace	June 18, 1927	Foley Bros., Inc.	930,000
Steel towers and floor system	Nov. 4 1927	McClintic-Marshall Co.	10,800,000
Cables, suspenders and anchorage steelwork	Nov. 4, 1927	John A. Roeblings Sons Co.	12,200,000
New York anchorage and tower foundation	May 4 1928	Arthur McMullen Co.	1,090,000
New Jersey approach, miscellaneous construction, Hudson Terrace	July 3, 1930	George M. Brewster & Son	323,000
New York approach, main approach ramp	Aug. 7, 1930	Cornell Contracting Corp.	800,000
New York approach, vehicular tunnel, West 178th St.	Aug. 7, 1930	Cornell Contracting Corp.	2,090,000
New York approach, Riverside Drive connection	Oct. 14, 1930	William P. McGarry Co.	1,050,000
New Jersey approach, miscellaneous construction	Feb. 2, 1931	George N. Brewster & Son	480,000
Main span and New York approach, paving, railings and miscellaneous construction	Mar. 20, 1931	Corbetta Concrete Corp.	460,000

average weight of stiffening trusses and wind bracing per foot of bridge is 2,650 lb. The comparable figure for the Hudson River Bridge is 1,110 lb. On the Manhattan Bridge (1,470 ft. span) and the Delaware River bridge (1,750 ft. span) both designed for highway and rapid transit, the average weights of stiffening trusses and wind bracing are 7,200 lb. and 5,730 lb. respectively. The comparable figure for the Hudson River Bridge is only 2,350 lb.

It is estimated that the use of a light flexible stiffening system on the Hudson River Bridge saves about $10,000,000, taking into account resulting savings in cables, towers and anchorages.

Towers

The towers have called forth much favorable and unfavorable comment, largely resulting from opinions on their appearance, which opinions naturally are variable and intensely individualistic. On most previous suspension bridges the towers have been designed as steel frames or bents, fixed at the base and flexible enough to take bending from unequal cable pulls. In the case of the Hudson River Bridge towers it was decided that flexibility was not needed and that their first requirement after load conditions were met was a massiveness of appearance for esthetic reasons. The cables are fixed to the tower tops, but the tower is not subjected to undue bending, since the short side spans reduce to a minimum the tendency of the cable bearings toward longitudinal movement.

Design studies showed that a steel tower and a steel-and-concrete tower compared favorably in cost. The first design was made on the basis of a composite tower in which the steel frame would carry at least the initial dead load of the structure and that the concrete incasement added later would bring the tower up to its ultimate required strength. Controversy arose as to the merit of such a composite tower, so that finally the steel skeleton was made strong enough to carry all loads, and provision was made for concrete incasement later if this proved to be desirable from an esthetic standpoint. This strengthening of the steel towers added about $800,000 to their cost.

The design of the indeterminate steel tower structures was described three years ago by L. S. Moisseiff (*ENR*, May 24, 1928, p. 819). The enormous load concentrations called for a frame of simple parts with clear and certain but not necessarily statically determinate stress action. The vertical reaction on top of each tower is 112,000 tons and the horizontal pull is 110,000 tons for both pairs of cables. Instead of the four-column tower bent with bracing in four planes that has often been used, the Hudson River Bridge tower was made up of four rigidly connected frames each including two pairs of braced posts connected by arched portal bracing below the floor level and at the top. Cables are disposed over the inside columns of the bents, one pair over each column, but the vertical load is distributed to all four columns of each bent by means of a heavy top girder and transverse bracing throughout the height of the tower in such a manner that exact proportioning of all column sections was possible. The only excess metal in the tower is in the upper portions of the exterior columns. Placing the cables over the interior columns instead of midway between the interior and exterior columns of the tower bents is one of the unique features of the design. If the latter alternative had been followed, the central planes of the cables would have been 144 ft. apart, which would have required floor beams of the same length. This would have occasioned a very considerable additional cost, about $2,800,000, as compared with the adopted design of floor beams 106 ft. long, made possible by locating the cables directly on the inner columns. The adopted design saved about 12,000 tons of floor-beam material, while the saving in the tower itself by making its symmetrical by centering the cables would have been less than $100,000.

The towers as built are 566 ft. high from foundation to top of steelwork. Each leg at the base is 47½ ft. wide and 56 ft. long center to center of columns. Typical column sections are shown on the accompanying drawing. The maximum section occurs on the interior line of columns in the top panel under the cables and contains 1,296 sq.in. of metal. The steel area of each column in the bottom panel of the tower is about 717 sq.in.

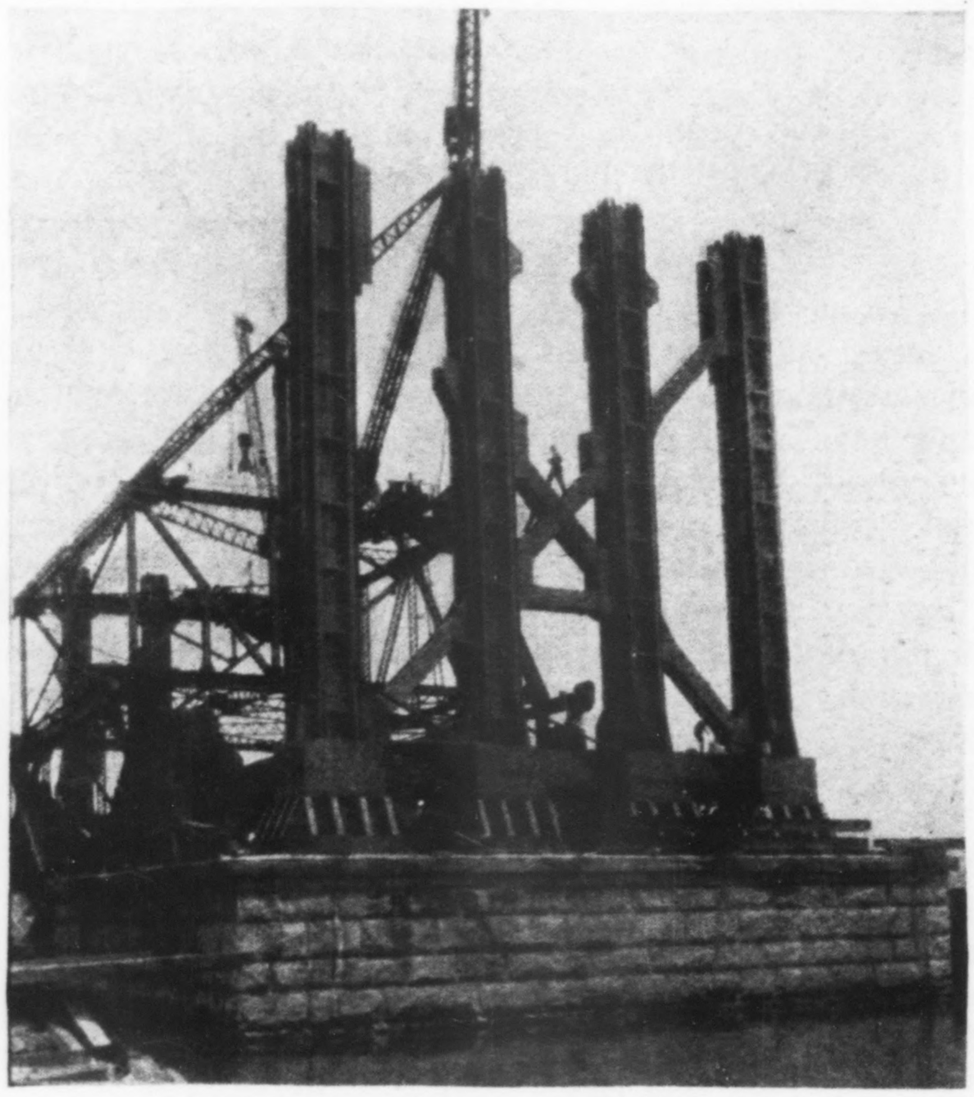
The cost of the bridge, its approaches and the highway connections as now built, with the construction work yet to be done in the near future (which includes the completion of the New York anchorage and the tunnel approach in 178th Street) and inclusive of right-of-way, interest during construction and cost of financing, will be within the $60,000,000 estimate made at the time the bridge was financed.

The cost of operation, inclusive of toll collection, policing and maintenance, is estimated at $350,000 per annum.

The George Washington Bridge was designed by the engineering department of the Port of New York Authority, of which O. H. Ammann is chief engineer, Edward W. Stearns assistant chief engineer, Allston Dana engineer of design, M. B. Case engineer of construction, Aksel Andersen assistant engineer of design, W. E. Cuenot chief draftsman, H. J. Baker engineer of steel inspection, and A. W. Munsell engineer of masonry inspection. R. Hoppen, Jr., and H. R. Seely were resident engineers.

The chief engineer also had the advice of the following consulting engineers: William H. Burr, the late Gen. George W. Goethals, Leon S. Moisseiff (on design) Daniel E. Moran (on foundations), Ole Singstad (on vehicular tunnel approach), Lewis B. Stillwell (on electric installations), and Joseph B. Strauss. Gustav Lindenthal, consulting engineer, rendered special advice on design questions, and Prof. Charles P. Berkey, acted as consulting geologist. Cass Gilbert was consulting architect.

Erecting Towers and Floor Steel on the Hudson River Bridge



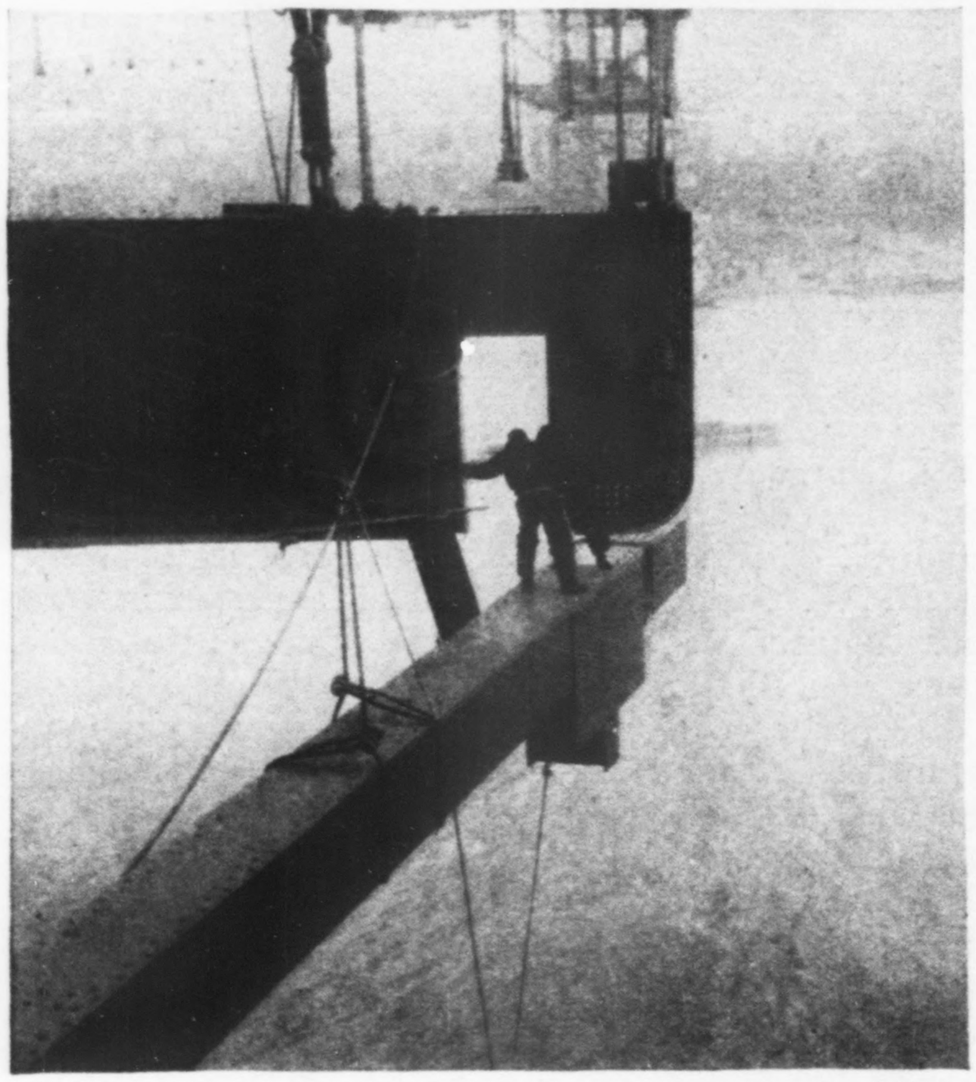


Fig. 1—Of the 60,000 tons of steel in the Hudson River bridge 40,000 tons was placed in the two towers in picks weighing from 37 to 80 tons each (top). Principal floor members were 62-ton floor beams and 12-ton wind chords (bottom)

Structural steel placed in record time under carefully organized erection procedure — Towers erected by platform travelers moving upward between tower legs and carrying two derricks, each operated by a 300-hp., seven-drum hoisting engine —A-frame travelers operating on the deck each way from each tower erected floor steel

By AUSTIN S. HALTEMAN
Erection Department, McClintic-Marshall Corp.

ERECTION of fabricated structural steel on the Hudson River bridge was an operation that both preceded and followed the spinning of the cables. Thus the two towers had to be completed before cable spinning could start, and the floor steel could not be placed until the cable work was finished. The structural-steel erector, therefore, had two separate jobs to perform, each not only different from the other but separated from it by a considerable time interval and capable of being carried out by two different erection gangs if need be. In describing the erection work it is logically considered as two distinct operations—tower erection and floor-steel erection—and it is so considered here.

Some elements of the preliminary planning, such as provision of steel-receiving facilities, establishment of field-office buildings, blacksmith shops, supply depots, etc., took into account the needs of both steel erection jobs; but for the most part this planning was directed toward setting up the tower-erection system, for this was the key operation in the time schedule, its completion governing the start of the cable spinning.

The most important of the operations preliminary to steel erection was the provision of facilities for receiving steel at the site. All steel for the job was shipped from the fabricating shops to ground storage at the New Jersey terminals of the railroads several miles down the river. As required by the erection forces, this steel was loaded on railroad cars, in turn loaded on a car float and towed to the bridge.

The New Jersey tower is about 100 ft. off shore in water deep enough to permit the car floats to be moored. Steel for the west half of the bridge, therefore, could be picked directly from the cars upon which it was received and placed in the tower. The New York tower, however, is on land, about 100 ft. in from the shore line. Here it was necessary to provide means for transferring the steel from the floats to the base of the tower. Docking facilities were most important. At the site, however, a sloping rock bottom was encountered, with rock at elevation + 15 about 50 ft. in from the shore line and at elevation — 140 at 150 ft. out from shore. After

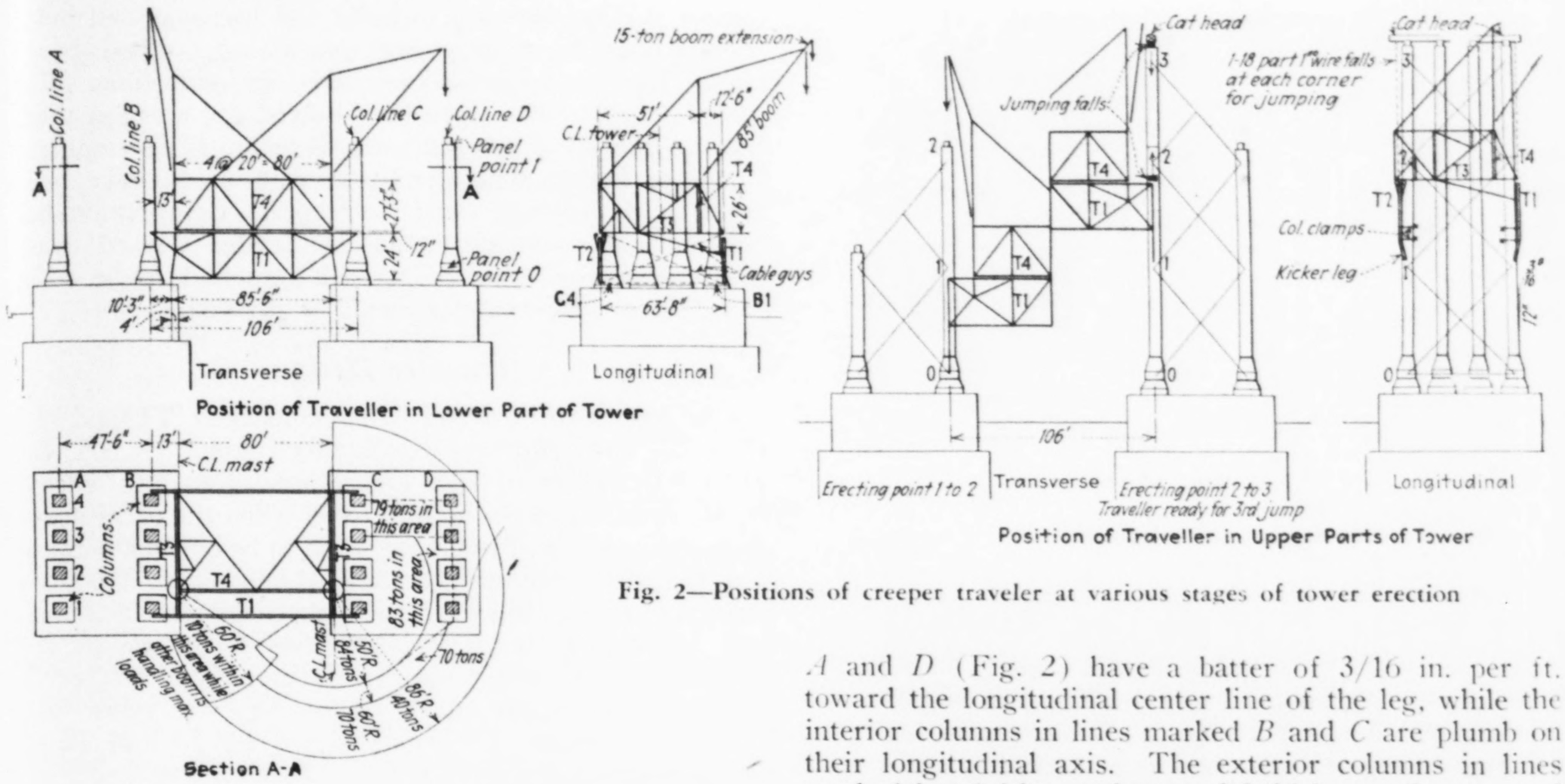


Fig. 2—Positions of creeper traveler at various stages of tower erection

careful studies of wharf designs a rock-filled crib type was selected. Five timber cribs were built, floored over with heavy planks and sheathed on the river side. These cribs were then tied back to the tower foundations and to anchors placed in the rock bank, in order to keep them from sliding down the steeply shelving rock. Rockfilling was placed in the cribs to a height about 3 ft. above high tide. This wharf, placed in May, 1928, was removed in good condition in July, 1931.

A stiff-leg derrick on the wharf unloaded the steel and passed it back to where the tower creeper derricks could reach it.

Office facilities were set up at the New York end of the bridge for general supervision of the work. However, actual erection of the two towers and to a large degree of the two halves of the floor steel was carried out as two separate jobs, each with its supervisory officers and supply depots on its own side of the river.

Tower Erection

The form and general design of the towers of the Hudson River bridge were radically different from any towers of previous suspension bridges, and consequently they called for an entirely new approach to the erection problem. Each tower has two legs, in each of which are eight columns arranged in rectangular plan and braced by struts at twelve levels in their height (less cable saddles) of 566 ft. and by heavy diagonal cross-bracing in each panel between the struts. The two legs thus formed are tied together below the roadway level and at the top by arch portal bracing. The panels vary in length from 56 ft. 6½ in. at the bottom to 38 ft. 9½ in. at the top. Each tower leg is 47½ x 56 ft. in plan, center to center of columns, while the two legs are 106 ft. apart, center to center of inside columns.

In addition to the fact that each tower contains two legs a considerable distance apart, two other characteristics of the design exercised an important bearing on the erection procedure and equipment. These characteristics were the batter of some of the columns and the extreme weight of some of the erection pieces near the top of the tower. The exterior columns in lines marked *A* and *D* (Fig. 2) have a batter of 3/16 in. per ft. toward the longitudinal center line of the leg, while the interior columns in lines marked *B* and *C* are plumb on their longitudinal axis. The exterior columns in lines marked 1 and 4 have a batter of 3/16 in. per foot toward the transverse center line, while the interior columns in lines 2 and 3 have a batter of 1/16 in. per foot toward the transverse center line. Erection equipment had to be designed to accommodate these battered columns, and it had to be of large capacity.

The inside columns in lines *B* and *C* are made wider at the top so as to support the cable saddle grillages adequately. Due to this design the *B* and *C* column sections, although growing shorter toward the top do not become much lighter, the erection piece between panel points 9 and 10 weighing 78 tons, while the heaviest erection pieces in the tower (those in the bottom panel of columns in lines *B* and *C*) weighed 80 tons. Columns

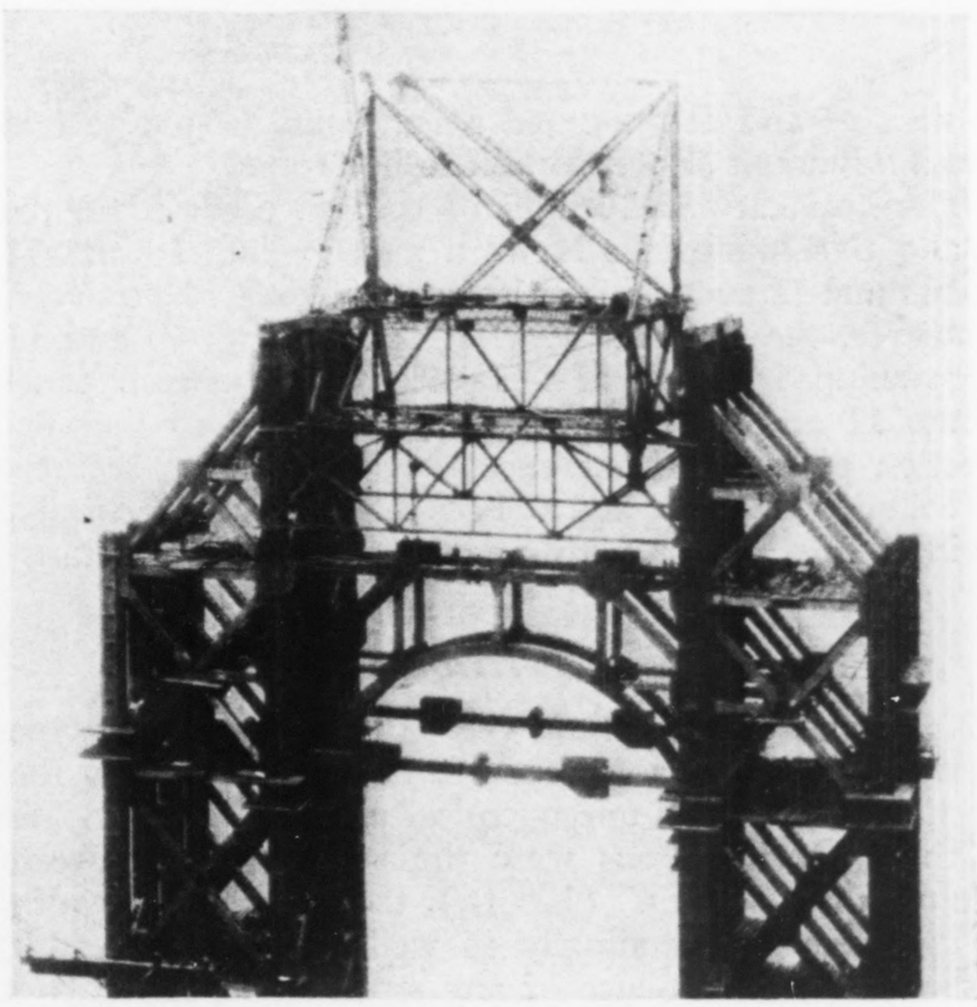
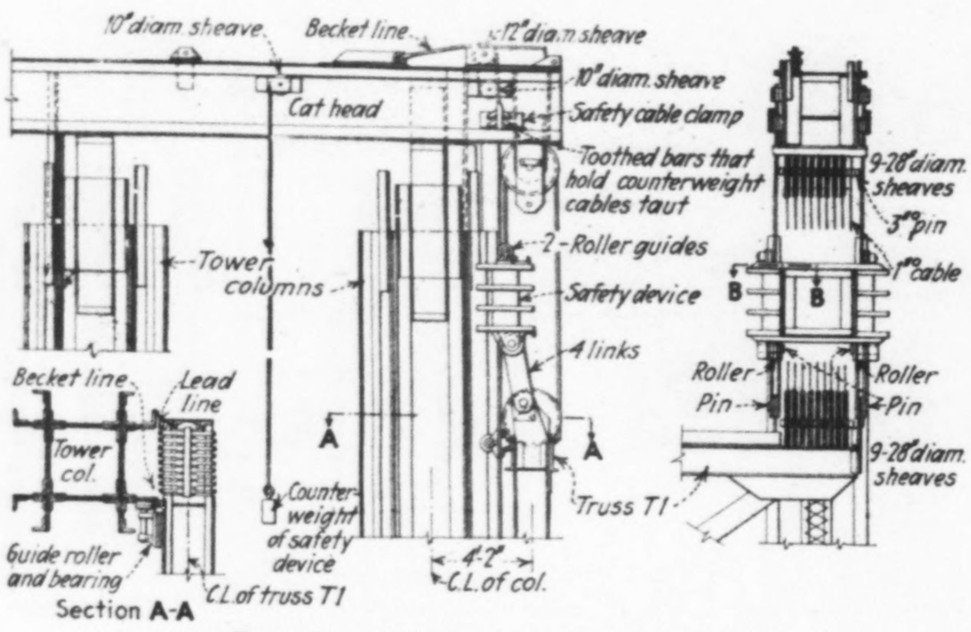
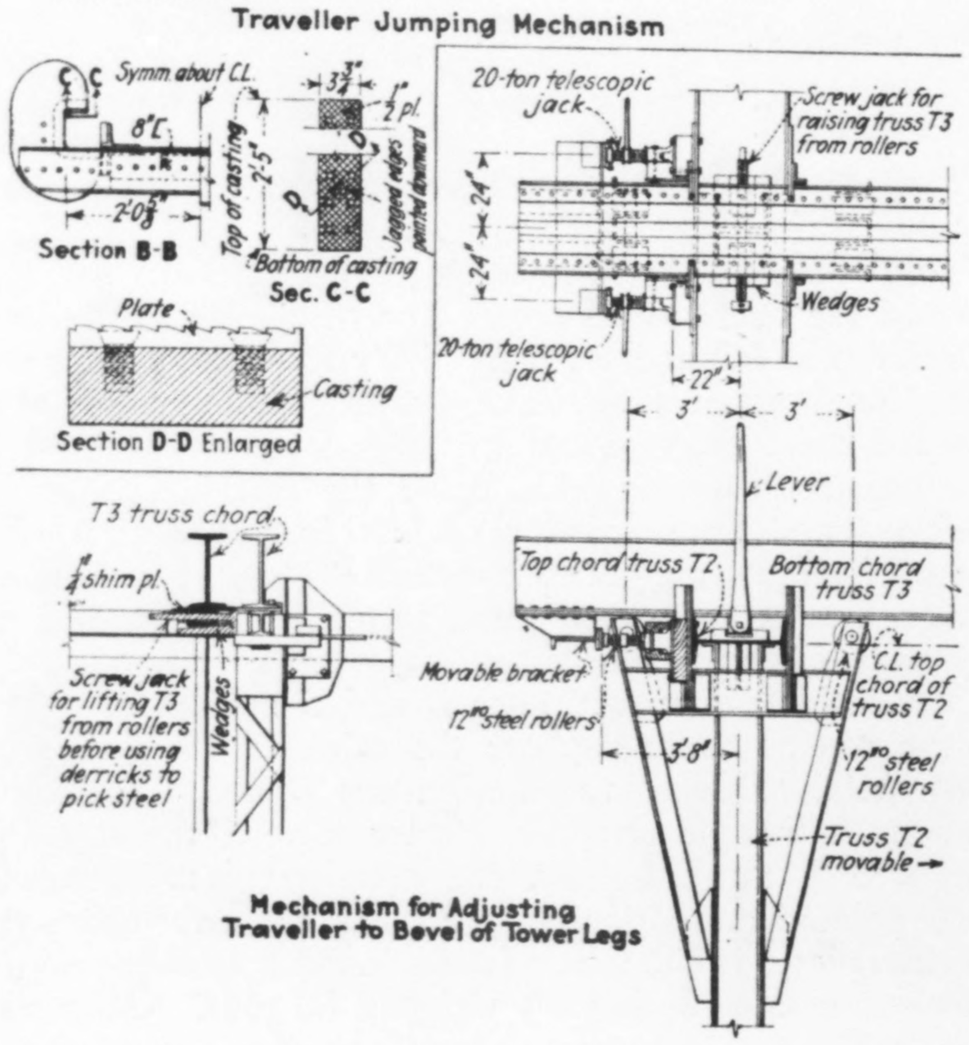


Fig. 3—Creeper traveler spanning between tower legs erected towers

Creeper has just been jumped. After erecting portal steel, as shown, derricks will complete outside half of tower panel below them and inside half of panel beside them before being jumped again.



Traveller Jumping Mechanism



Mechanism for Adjusting Traveller to Bevel of Tower Legs

Fig. 4—Mechanical assemblies used in operating erection creeper on Hudson River bridge towers

in lines *A* and *D* presented a maximum 67-ton section and a minimum 37-ton section to be erected.

The heaviest panel length of column occurred in the *B* and *C* column lines between panel points 10 and 11 and 11 and 12, and weighed 102 and 150 tons, respectively. However, the columns between panel points 10 and 11 were shipped in two 51-ton pieces, and between panel points 11 and 12 in three 50-ton sections, so that the erection picks were not excessive. These top sections of columns in the *B* and *C* lines were spliced vertically, giving a final section with six longitudinal and six transverse webs.

Determining Erection Procedure

Numerous considerations caused the conventional type of single-boom tower creeper to be discarded from the erection planning as uneconomical and too slow. Some of these considerations were the wide spread between column lines *B* and *C*, (106 ft.), the considerable width of the tower longitudinally as well as the total width transversely, the absence of any struts between the two legs except at the portals, and the large number of lifts to be made.

The traveler adopted was made up of two stiff-leg derricks mounted on a bridge or platform that spanned the 106 ft. between the *B* and *C* column lines. Each derrick had a tower leg to build and was operated independently of the other, except that maximum loads could not be handled by both derricks at the same time. The use of two derricks more than halved the erection time, because the 85-ft. booms used were much shorter, and therefore faster, than the boom that would have been required on a single-derrick creeper. Competition between the two derrick gangs also tended toward speed in erection, for a panel on both legs had to be completed before the traveler could be jumped.

Traveler Design

The two main traveler trusses marked *T*1 and *T*2 (Fig. 2), were supported on tower columns *B*1, *C*1, *B*4 and *C*4 by means of four kicker legs or toggles, except during erection of the bottom panel when the toggles and the lower corners of the trusses had to be omitted because they fouled the column pedestals. As soon as this bottom panel was passed, however, the kicker legs were attached, each leg resting on a bracket made from one of the regular column diaphragms, reinforced and stiffened. The horizontal thrust of the kicker legs was taken care of by clamping the *Uo* points of trusses *T*1 and *T*2 to the columns.

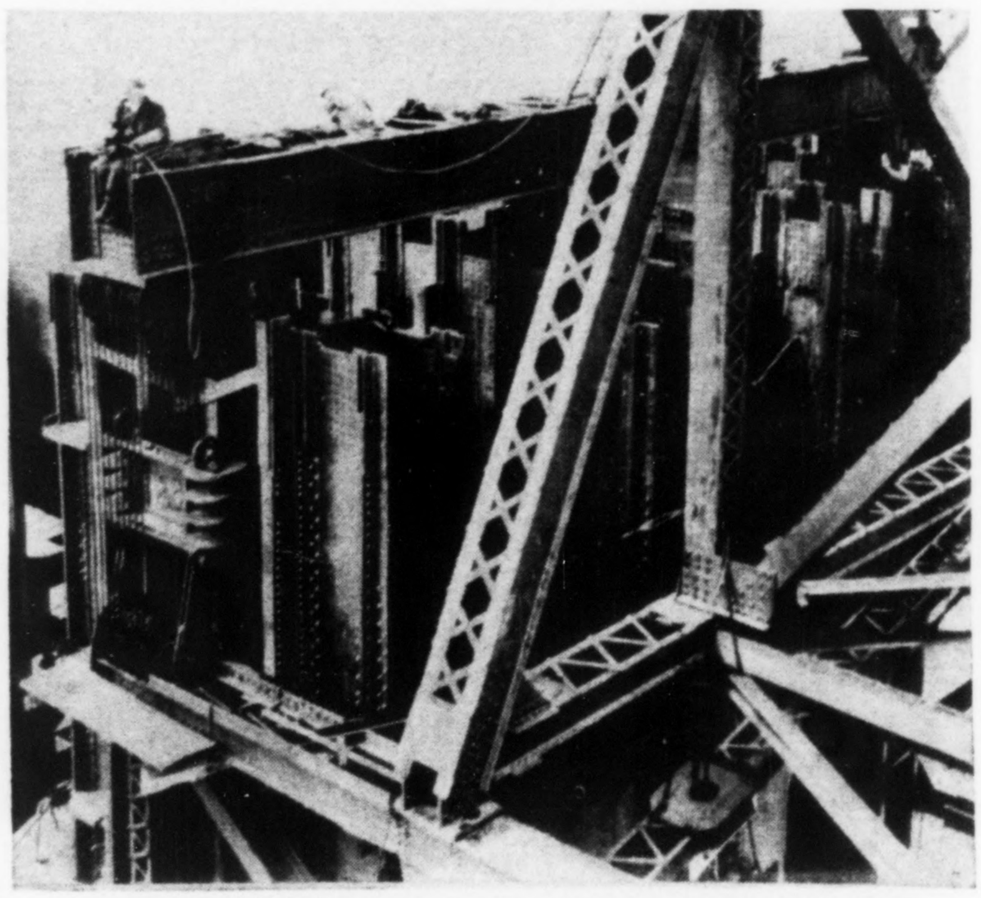


Fig. 5—Tower creeper was jumped by means of 18-part falls attached at each corner of the creeper and to a "cat head" beam placed over the tops of the erected columns

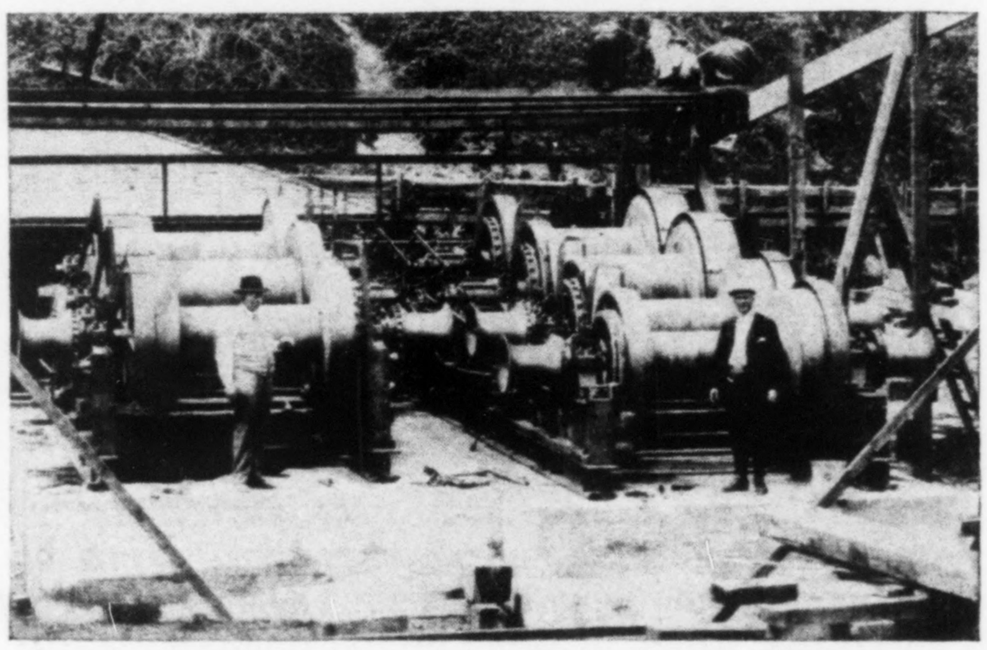


Fig. 6—Seven-drum electric hoisting engines operated each derrick on the tower creepers

The masts of the derricks were located 12½ ft. behind the river truss *T*1 and 13 ft. from the center line of the *B* and *C* columns, both to give the minimum reach for picking and erecting the columns and to provide clearance between the 10 ft. bullwheel and the columns. The top chords of truss *T*4 and the two trusses *T*3 served as sills for the derricks.

It will be noticed from Figs. 2 and 4 that the river end of truss *T*3 rested on and was rigidly attached to truss *T*1, while the land end of this truss rested on *T*2 through rollers. The latter allowed the trusses *T*1 and *T*2 to approach each other as they traveled up the tower, thus adjusting for the batter of the columns in lines 1 and 4. Two 20-ton telescoping jacks were installed to assure this adjustment, although they did not prove to be necessary as the trusses adjusted themselves to the batter automatically. After each traveler jump, truss *T*3 was jacked off the rollers before the derrick picked any steel; the screw jack and wedges by means of which this latter operation was accomplished are shown in Fig. 4.

Each traveler weighed 320 tons and was jumped by means of 18-part falls of 1-in.-diameter steel cable, one set at each corner of the traveler. The lower block was built into the ends of the top chords of trusses *T*1 and *T*2, while the upper block was placed in a "cat head," a temporary beam supported on the top of the columns in lines *B* and *C*. After the traveler had been jumped, these cat heads were removed and hung off on the outside of the tower until the next traveler jump, one panel above.

Four safety devices were installed in the jumping mechanism, one at each corner, consisting of an assembly of wedges that caused heavy casting members to be clamped onto the columns if the travelers started dropping due to breakage of the jumping falls. This clamping action was made more certain by attaching roughened steel plates to the faces of the castings that surrounded the tower columns. The jagged edges of these plates were pointed downward so they would bite into the column steel. The safety device was held from falling with the traveler by a counterweight whose cable passed over sheaves attached to the cat head. Details of these safety devices are shown in Fig. 4. Fortunately, no difficulty was experienced in jumping the travelers so that there was no occasion for testing the efficiency of the safety mechanism.

Method of Traveler Operation

Each traveler was operated by two seven-drum engine units, one for each derrick. Of these seven drums two were used for the main load falls, one for the auxiliary load falls, two for the boom falls and two for the jumping falls.

Both ends of the load falls and of the boom falls were brought to the engine, and both ends were pulled simultaneously. This decreased friction losses and therefore reduced the lead-line pull, halved the time required for lifts and did not necessitate as large individual drum capacity as would the more usual method of reeving.

Each seven-drum unit was powered by two 150-hp. 440-volt three-phase, alternating-current motors. These motors could be used independently or, by throwing one lever, could be coupled together to deliver the full 300 hp. on one pair of drums. The two drums pulling on the two leads for the load falls were coupled together so that both went ahead or slacked off simultaneously, thus giving at all times two six-part falls instead of one twelve-part fall. A similar coupling was provided for the two drums carrying the boom-falls leads.

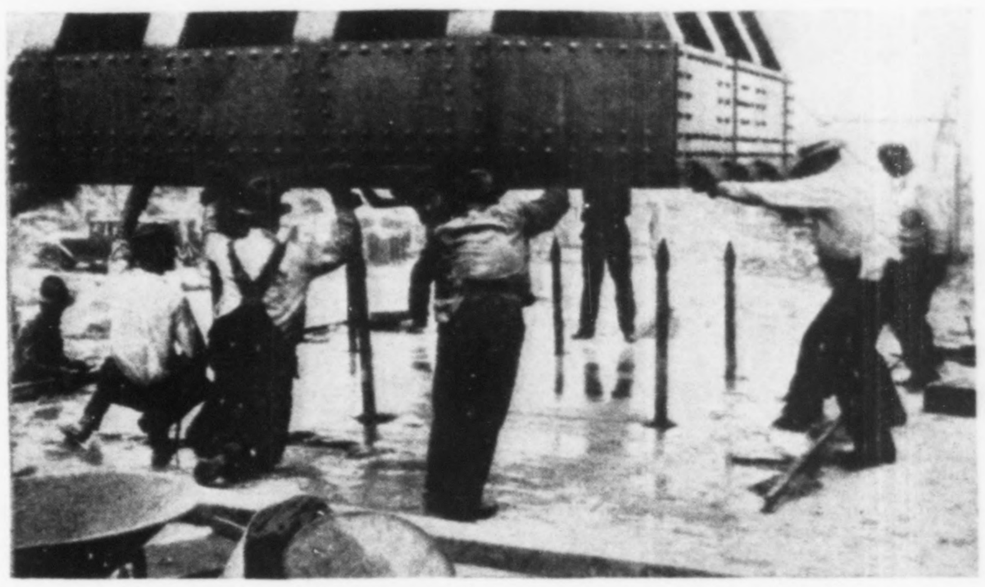


Fig. 7—In erecting column pedestals steel pads (beside screed rail in right foreground) shimmed to exact elevation were used to assure accurate setting of pedestal on mortar bed

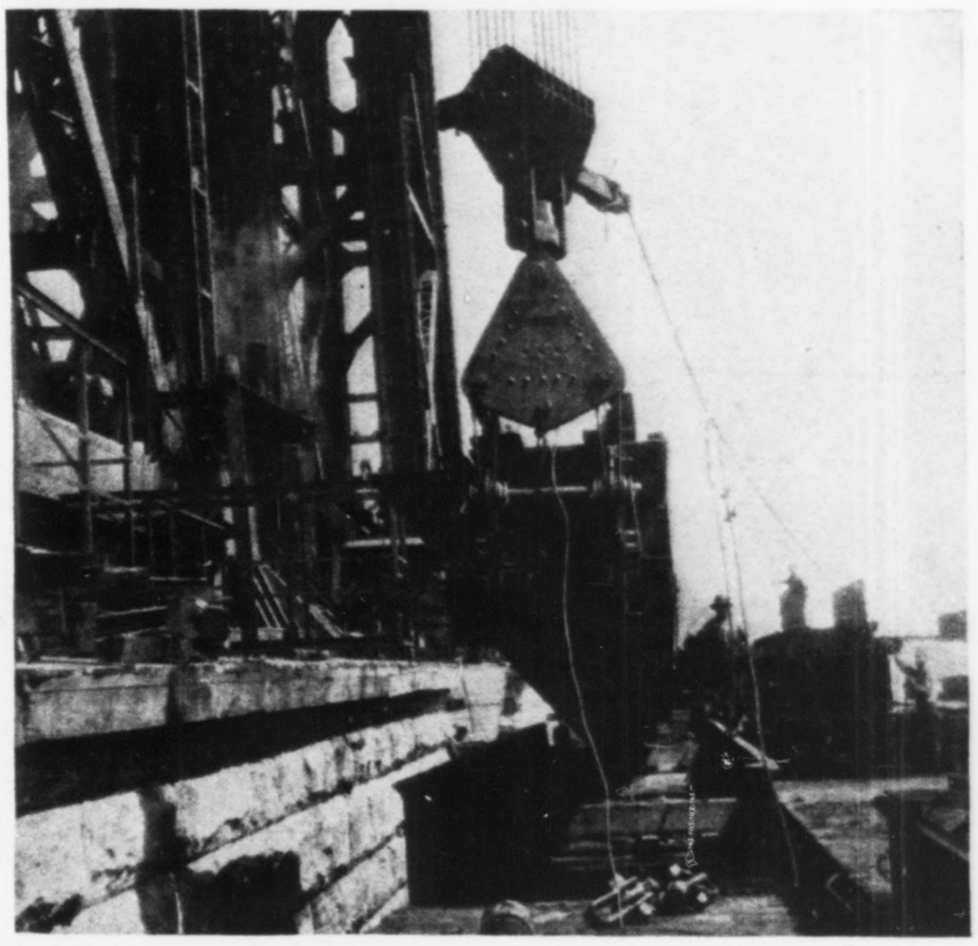


Fig. 8—Special hitches were used for unloading and hoisting the tower column sections

The four hoisting engines were located on the ground 150 ft. landward from the center line of the towers. They were 300-hp. seven-drum engines built for the job and gave entire satisfaction. They were probably the largest portable hoisting engines ever used on a construction job, each engine weighing 75 tons.

Signaling System

Over each drum or pair of drums there were two lights, one red and one white, controlled by the signal man on the traveler through a push-button control. There was also one bell to an engine unit, controlled from the push-button box. A white light signaled the engineer to go ahead and a red light to lower. If the signal system failed, the lights went out and the engineer stopped his engine, so that no damage resulted. This system proved entirely satisfactory, and results were even better than where the engineer can see the load or get his signals visually from the signal man. The derricks were swung by means of small electrically-operated swinging engines located on the traveler itself.

Erection Procedure

The first piece erected were the column pedestals. These were large structural units, 13 ft. 9 in. square and weighing 55 tons, and had to be set with extreme accu-

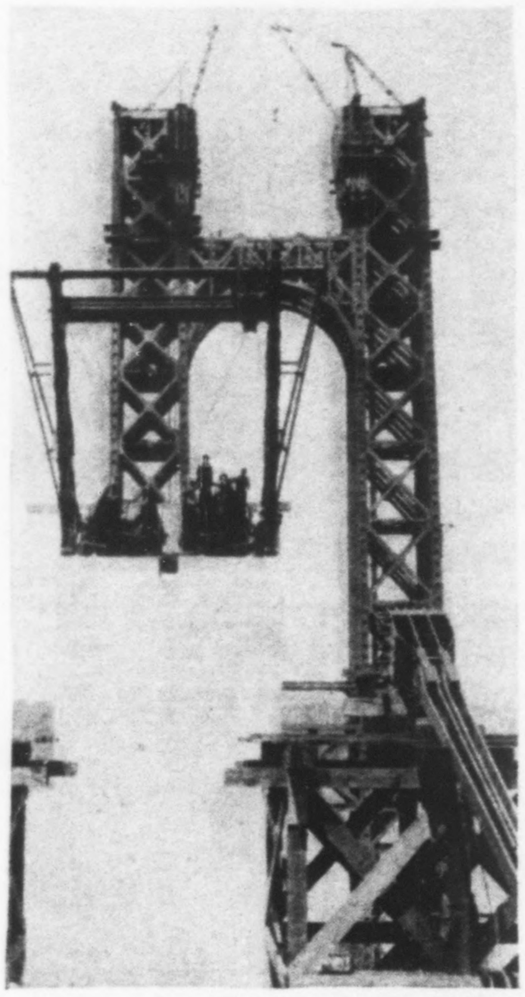


Fig. 9—At top of tower, creeper derricks were moved onto tower legs, creeper trusses removed and the top portal bracing set to complete the tower erection

In foreground are New York anchor block and eyebar anchors for footwalk cables.

racy. An ingenious method of setting them to exact position was devised. The pedestal was first set in place on steel pads on the foundation pier, as shown in Fig. 7, and bronze shims were inserted until it was level and at the correct elevation. The pedestal was then removed and a bed of stiff 1:1 mortar laid down and struck off $\frac{1}{8}$ in. high by means of a special screed bar slid along rails accurately set on either side of the pedestal location. The pedestal was then lowered over the anchor bolts, as shown in Fig. 7. If it did not rest on the steel pads under its own weight, another pedestal was placed on top, which forced it down. This method proved to be much quicker and cheaper than bush-hammering or grinding the piers, as has been done on some bridge jobs. It is also believed that better bearing was secured than is obtainable by any other method. One pedestal was wedged up and pulled off after the preliminary set of mortar, and the bearing was found to be perfect. After the mortar had set up, the steel pads and bronze shims were removed and grout was rammed into the space they had occupied.

Tower Steel Erection—The column sections to be lifted varied in weight from 37 to 80 tons. They had riveted to them all of the connection gussets for the struts and diagonals, so that these latter members could be merely lifted into place. In general, column splice material was riveted to the top of the column section, although splice material for the wing angles was received bolted. Erection, therefore, consisted in the placing of large and heavy, but completely assembled, units.

The erection procedure may be briefly summarized as follows: In their first position, with the traveler spanning between the two foundation piers, the derricks placed the column pedestals, the entire first panel of each tower leg and the inside columns (*B* and *C* lines) of the second panel. The traveler was then raised until the derricks were at the elevation of the second panel point, from which position the outside columns of the second panel, the inside columns of the third panel, the remaining half of the bracing in the second panel and half of the bracing in the third panel was erected. On the next raise this procedure was repeated.

Special hitches were used for picking and setting the columns and heavy portal members, the type of hitch depending upon the length and weight of the section. Hitches for upending the columns were also utilized. The columns were unloaded flat, after which the hitch was attached to the upper end. A triangle balance beam, hung from the 12-part main-load block of the traveler derrick, was connected to the hitch by wire cable grommets, the column upended and hoisted. When in approximate position in the splice and within 1 in. of bearing, all dirt and paint that had been scraped off the sides of the member were blown out by compressed air. The column was then dropped into final position, and the splice bolted and pinned. It required 17 min. to raise the last column section from the ground to the top of the tower.

There were 475,000 rivets, weighing 325 tons, in each tower. Special scaffolds, used for riveting the column splices on the outside, utilized a 2-in. plank roof to protect the riveters; the scaffolds were also inclosed in a hand-railing. These scaffolds were suspended from the column above, their position being adjustable for about 60 ft. vertically by means of winches, which were hoisted every panel by the derrick. Oil and electric forges were tried, but the most satisfactory was the coal forge with an extra large pan. The tower legs were floored over at each panel point with 4-in. plank immediately after erection, and this floor was not removed until all rivets in the vicinity were driven. This not only provided a working platform but gave protection to all men working below the raising gangs.

Tower erection work was suspended when steel had been erected to panel point 10, in order to take comparative elevations on the columns. Small differences in elevations (about 3/16 in.) were adjusted by the shop in fabricating the last two panels. Erection was not resumed until the next spring because of the bad weather

With the tower legs completely erected and the traveler at the top of the tower, the derricks were moved outward onto the tower legs. From this position they took down the traveler and erected the top portal. In order to maintain the correct distance between the column lines *B* and *C*, gage struts were used at panel points 2, 4, 6, and 8 on column lines 1 and 4 until enough of the portal had been built to permit their removal.

As a final operation, one of the derricks was moved so that its mast was on the river column line 4 and on the longitudinal center line of the bridge. The boom was lengthened to 105 ft., and this derrick then took down the second derrick, erected grillage girders for the cable saddles, placed the saddles themselves and erected the construction tower and crane runway (see cable-spinning article, *ENR*, Aug. 14, 1930) used for handling the main cable strands. This derrick also raised and placed in position the thirty-six $2\frac{3}{4}$-in.-diameter ropes used to support the footwalk required in spinning the cables.

The shop work on the towers was excellent, no serious difficulty in making a connection or getting a joint in bearing being experienced. Also, there were no drawing room errors. During the entire course of the work there was no accident that could be charged to failure of equipment or supervision. The amount of money spent on special equipment, derricks and engines for the erection of steel on this job was very heavy, but the lack of accidents and the speed of erection proved that the money was well spent.

The first steel was set in the New Jersey tower on June 25, 1928, and in the New York tower on August 8, 1928. The towers were turned over to the cable contractor on June 20, 1929.

Floor System Steelwork

Roadway steel to be erected on the Hudson River bridge comprised 19,188 tons, made up of 8,645 tons of carbon steel, 8,131 tons of silicon steel, 2,341 tons of beams and tierods in the roadway slab, 68 tons of cast

steel and cast iron and 3 tons of bronze. This steelwork differs from that used on any previous suspension bridge, in that there is, under the first stage of construction, no stiffening truss. The present floor steel includes the upper chord of the stiffening truss that will be necessary when a proposed lower deck is added sometime in the future, but it is now utilized as a wind chord for the existing floor system.

The component parts of the floor system are indicated in the plate on p. 643. The principal members is a built-up plate-girder floor beam about 10 ft. deep, attached at each end to two groups of four $2\frac{7}{8}$-in. wire rope hangers. The floor beams are spaced on 60-ft. centers by means of the wind chords, set in 6 ft. from the ends of the floor beams and entered from the bottom. Eight lines of plate-girder stringers about $5\frac{1}{2}$ ft. deep frame into the floor beams below the top flange and carry 16-in. I-beam cross-members on 5 ft. 2-in. centers. On top of these cross-beams $5\frac{3}{4}$-in. bulb beams are placed longitudinally, spaced 15 in. apart. These bulb beams are concreted into the $7\frac{1}{2}$-in. concrete floor slab. Two lines of built-up steel curbs divide the 90-ft. roadway into three lanes, of which only the outer two, each 28 ft. 9 in. wide, have been paved at the present time.

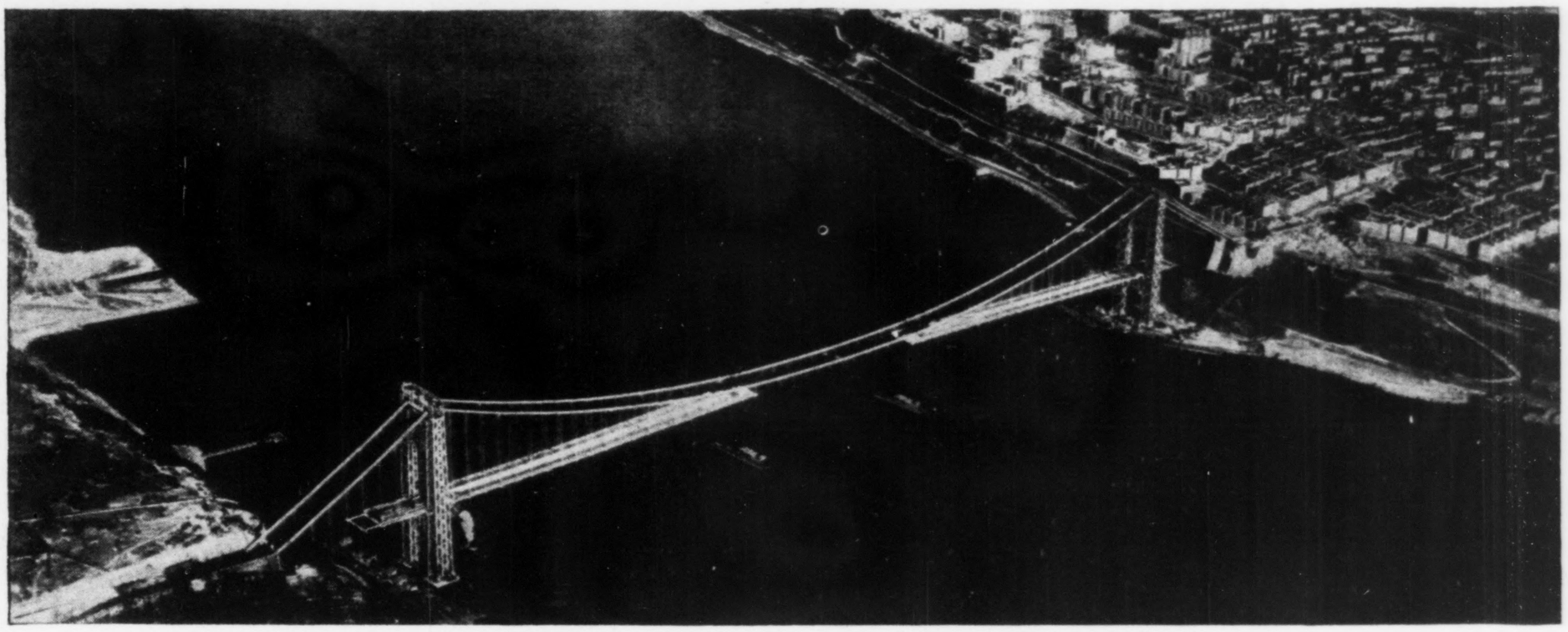


Fig. 10—Floor steel was erected each way from each tower by deck travelers

Floor Steel Travelers

Rapid erection was required by the contract, and the speed with which the main span floor could be erected was the determining factor. Erection of the floor was carried on simultaneously at four different points, travelers working both ways from each tower. Due to the large size of the cables, the entire steel deck could be erected by the travelers as they moved out, without excessive cable distortion. On most suspension bridges where the travelers have operated on the deck, it has been necessary to make two, and sometimes three, passes to complete the floor steel; otherwise the excessive local deflections would make assembly of stringer floor-beam connections and chord splices very difficult.

Main Span—Each of the main span travelers was, in reality, two A-frame derricks with cable backstays, coupled together. By using two derricks to pick the only heavy lift, the $62\frac{1}{2}$-ton main floor beams, lighter, shorter and faster booms could be used and a smaller number of parts of line was required in the main load falls than if a single extra large traveler had been used. As a consequence, the erection time was reduced more than half. After a floor beam had been set, the two A-frame travelers operated independently, each filling in half of the panel. By having two raising gangs, competition was created, and greater speed of erection obtained. If one gang finished its half before the other, it helped fill in the other half, so that the two travelers could be moved as a unit. A-frame cable-backstay travelers are easily adjusted to compensate for a change in grade, it being necessary merely to move the sheave at the end of the backstay to a different pin hole in the link plates (Fig. 13).

All of the A-frame travelers had been previously used on other jobs and required some rebuilding for the Hudson River bridge work. The travelers for the New York half of the main span had been used to erect the Port Authority's Outerbridge Crossing between Staten Island and New Jersey and on the Mt. Hope Bay bridge approaches. For use on this job the tops of the two A-frames were tied together with steel cable, and side guys or "quarter falls" were used on the north side of the north traveler and on the south side of the south traveler. The shoes at the bottom of the quarter falls were bolted to the top flange of a floor beam, these being the only tie-downs required at the front of the traveler. Although the engine provided sufficient counterweight to take care of the uplift, a rear tie-down was provided as an additional safety factor, and to take out vibration. The two front sills were fastened together with a removable ling that transferred shear only.

The four-drum, 150-hp., 440-volt, three-phase alternating current electric engines used were part of the seven-drum units that erected the towers. Of these four drums, one was utilized for the main load falls, one for the auxiliary load falls, one for the boom falls, and one for the traveler moving falls. The booms were swung by point lines, worked by small electrically operated swinging engines located on the traveler.

The travelers for the New Jersey half of the main span had been used to erect the Cooper River bridge at Charlestown, S. C. The A-frames of these travelers, having only an 18-ft. spread at the bottom, were not very stable, so an additional side brace leg was added. Like the travelers on the New York side of the river, the front sills were linked together for shear only.

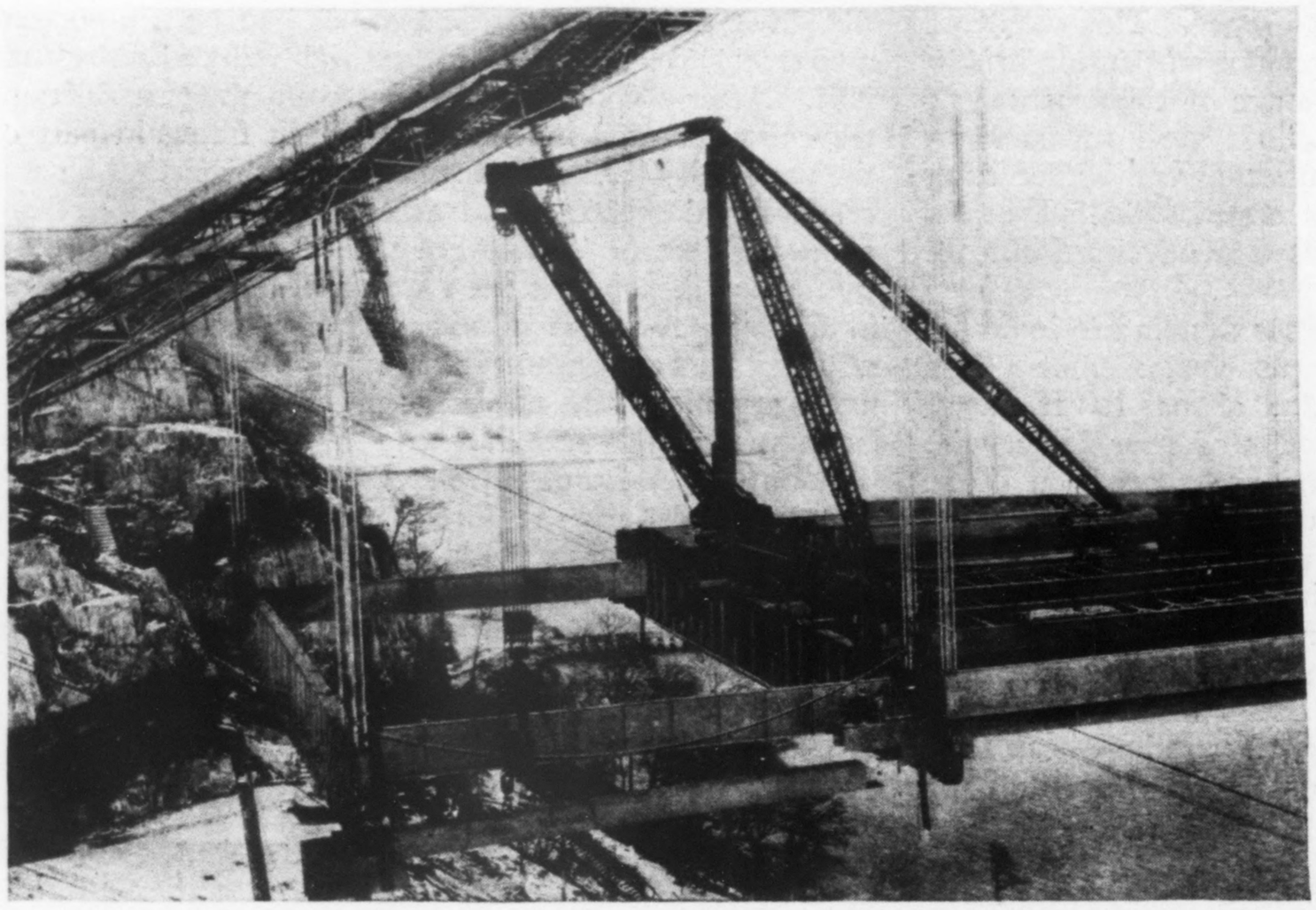


Fig. 11—Side-span floor steel was erected by stiff-leg derrick travelers operated from the ground by one of the hoisting engines previously used on the tower creeper

Two 150-hp., three-speed, three-drum gasoline engines were used, with one drum operating the main load falls, one drum the auxiliary load falls, and the third drum the boom falls. Gasoline engines were preferred to electric engines because of the great cost of the heavy conductors required on the latter and the expense of laying and removing these conductors. Electric engines were used on one side of the river because the erector did not own additional gasoline engines powerful enough to work at a fair speed, and the purchase of such engines was not deemed economical. The New York side was chosen for the electric engines because of the cheaper power rate. The New Jersey main-span traveler was moved by manila rope falls operated from the niggerheads. The engine had a swinger attachment that swung the boom by means of point lines.

The side-span travelers were alike, each being made out of one of the stiff-leg derricks from the tower travelers. A single-boom rig was used on the side span because, for the comparatively short spans, it would not be economical to make a traveler from other derricks that the erector owned, and pay the freight to and from the Hudson River bridge; besides, the tower stiff-leg derricks had been stored on the job and were thus available. With the main span partly erected before side-span erection was begun, there was no appreciable change in grade to be adjusted for, and so the stiff-leg derricks were perfectly satisfactory for side-span erection.

Side-Span Travelers

The engine used for each side-span traveler was one of the tower-traveler engines. These engines were left on the ground at the rear of the towers, since the leads did not need to be excessively long (only 550 ft.) to serve the side spans adequately. Special sheaves were provided to support the cable above the floor of the span, and no trouble was experienced in overhauling boom or load falls. A swinging engine mounted on the traveler swung the boom by means of a bull wheel.

Like the tower steel, the floor steel was shipped from the shop where it was fabricated to ground storage at the New Jersey terminals of the railroads. From there it was brought to the job as required by the erector, being loaded on railroad cars, in turn placed on a car float. Railroad tugs delivered the car floats to the tower foundation on the New Jersey side and to the erector's wharf on the New York side of the river.

When required for main-span erection, the car float was towed out and moored to a barge that was anchored directly under the panel being erected. The erection travelers on the bridge deck transferred the steel direct from the car float to its position in the bridge.

Although these barges carried lights and were left in the river at all times, it was not permissible to have the car floats moored to them overnight; accordingly, the car floats had to be towed out every morning and returned to shore in the evening. Four anchors were used on each barge, and the position of the barge was adjustable by slacking off and taking up on the anchor lines. When the limit of motion had been reached by working the anchor lines, the anchors were shifted by tugs.

Car floats carrying steel for the side spans were moored to the wharf on the New York side and to the tower foundation on the New Jersey side, from which points steel was raised to the bridge deck by a stiff-leg derrick located directly above. The steel was then loaded on push cars and trucked back to the side-span travelers, using the bulb beams as rails.

The floor beams were erected first, each hung from eight suspender ropes. Then followed in order the wind chords, stringers, bracing, cross-beams, bulb beams and

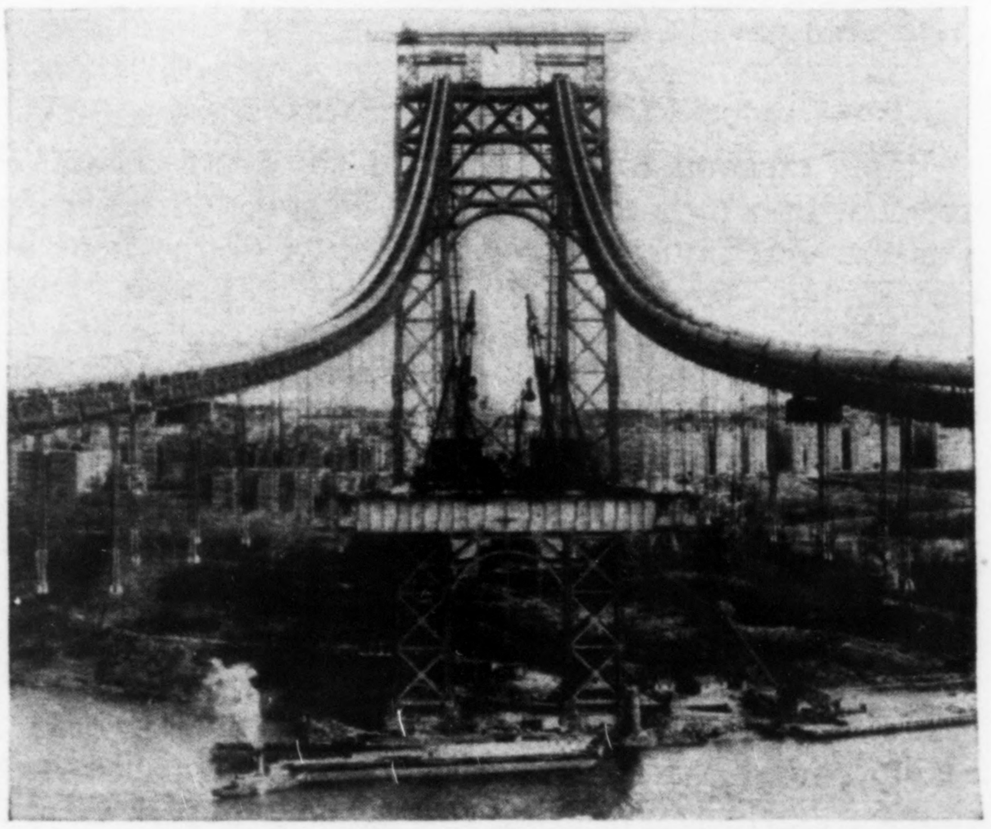


Fig. 12—Two A-frame derricks were used to lift the 62-ton floor beams, and then each derrick working independently filled in half of the panel of floor steel

tierods. The main load falls were used only on the 6$2\frac{1}{2}$-ton floor beams and the 12$\frac{1}{2}$-ton wind chords, all other steel being set with a two-part auxiliary falls. The wind chords could have been raised with the auxiliary falls, but the main falls were used to get the delicacy of motion required to enter the wind chord into the floor beam (Fig. 1) and make the connection.

On the main span a 12 x 12-in. timber 62 ft. long was used as a spacing strut and tie to keep the floor beam exactly 60 ft. out until the wind chord connection could be made. This timber was also used as a footwalk by the workers. For the side spans this strut was not necessary, since the floor beam was not exposed to heavy

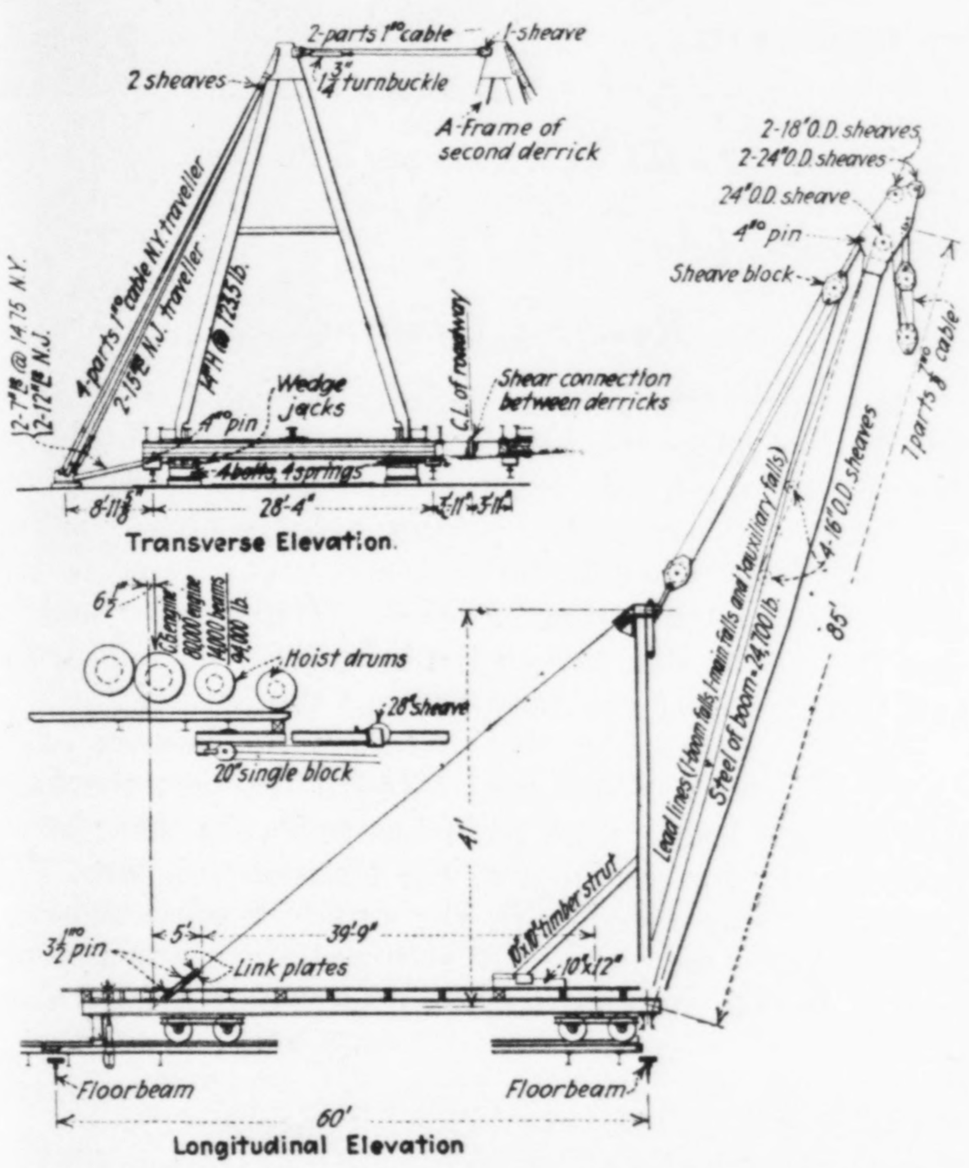


Fig. 13—General design of A-frame cable-backstay traveler used on main-span floor erection

Two of these travelers, used side by side as shown, speeded up erection and furnished the required capacity for picking the heavy floor beams.

winds. On side-span erection the workers rode the floor beam out from the deck.

No difficulty was encountered in closing the main span. In making this closure the New Jersey side was pulled toward the towers with turnbuckles, while jacks were used on the New York side. This was possible since the span at each tower was on a sliding surface, wind shear and half the vertical load of the adjacent panel being taken into the towers on sliding shoes. The closing chords were dropped in with $\frac{1}{8}$-in. clearance, the connection made, and the jacks and turnbuckles removed at the towers.

During the early stages of erection, before the short suspenders near the center of the span had a restraining influence, the lateral motion at the end of this span due to wind was sufficient to cause the end of the windward stringers in the panel adjacent to the tower to run off their seat. To prevent this the wind chords were fixed to the columns, thus transferring not only the wind shear, but also wind moment, the span forming a lateral cantilever truss. After erection reached a point where the suspenders were short enough to restrain the end, this tie was removed.

A total of 312,500 field rivets was driven in erecting the floor steel. Air was furnished by compressors located on the ground near the towers. Wind chord splices were not riveted until the two adjacent chords were in a straight line, since this was the position in which they had been assembled in the shop for reaming. About ten panels of the main span were erected before the first splice was riveted. At a few splices at the middle of the river this condition (of three chord sections in a straight line) could not be obtained, and the chords were riveted with a slight curve in them.

Personnel—The Hudson River bridge is being built by the Port of New York Authority, O. H. Ammann, chief engineer, Allston Dana, engineer of design, and Montgomery B. Case, engineer of construction. All structural steel was furnished and erected by the McClintic-Marshall Co. For McClintic-Marshall, David S. Gendell, Jr., was manager of erection, W. B. Fortune, foreman on the New Jersey side, and H. G. Reynolds, foreman on the New York tower. Sterling Johnston designed the tower travelers. The writer was resident engineer on the work.

Wheel Tracks for Low-Cost Roads

Two types of wheel-track construction for pioneer roads are in experimental use abroad. Economy and rapidity of construction dictated the design of an experimental stretch of gravel roads with concrete wheel tracks, by Leslie Scott, of the public works department of Southern Rhodesia. This road, 16 ft. wide, has a 6-in. surface of compacted gravel in which are two 6-in. concrete tracks 2 ft. wide and 5 ft. 3 in. c. to c. As described in *The Surveyor* (London) by Mr. Scott, shallow trenches 6x24 in. were excavated and filled with concrete, the surface being screeded to the road camber or crown of 1 in 36. For curing and the prevention of too rapid drying by tropical heat, the trenches were well watered before the concrete was placed and the concrete was then kept covered with wet sacking and sand for fourteen days. After eight months' service the road was in good condition and the gravel had withstood torrential rains without excessive erosion. As built by native labor the cost was about $2,250 per mile. Advantages claimed include benefits for motor traffic, rapidity of construction, long life and low cost of both construction and maintenance.

In India an early type of railway track has been adapted to highway construction, with steel angles held at proper gage by transverse bars. The upstanding legs of the angles are inward, and the space between them is filled with earth or broken stone, while the road surface outside is level with the horizontal flange or track. The angles are laid upon beds of rolled or tamped gravel 12 in. wide and with a depth varying according to the soil, the minimum being about 18 in. in earth. It is claimed that this construction gives easy passage for vehicles of all kinds, the angles serving as tracks and guides, but it permits the vehicles to turn out easily for passing. It has been developed by H. W. Perry, and according to the *Railway Gazette* (London), experiments have led to its adoption by the Madras Provincial Railway for a 24-mile line.

Ingenious Form System Speeds Up Paving of Hudson River Bridge Floor

Easily stripped sheet-metal pans placed between bulb beams of floor system—Concrete handled by industrial railway using bulb beams for track—Pouring schedule of 240 ft. of 29-ft. roadway and 13-ft. sidewalk per day maintained



Wide World Photo

BY TAKING advantage of the special design of the floor system of the Hudson River bridge the contractor on the roadway and sidewalk paving developed an ingenious and novel form system and an unusual concrete-distribution layout to maintain a fast pouring schedule. In placing 9,540 lin.ft. of roadway 28 ft. 9 in. wide and a sidewalk 12 ft. 10½ in. wide, a daily progress of 240 ft. was made. The form system for the roadway in which 5¾-in. longitudinal bulb beams, 15 in. c. to c., were incased consisted of sheet-metal pans, placed from the roadway deck and easily stripped from a traveler beneath the floor system. Wood-veneer panel forms were used for the sidewalk. Concrete materials were dry-batched at the base of the New York anchorage, hoisted to the deck level and then transported by railroad, using the bulb beams as tracks, to a paving mixer at the point of placement.

Several other finishing operations were included with the roadway contract. These were: placing of the roadway slab over the New York anchorage, 35,200 sq.ft., or 1,530 cu.yd.; filling the eyebar pits and keyways of in New York anchorage, 6,100 cu.yd.; placing 29,500 lin.ft. of railing and more than 50,000 lin.ft. of electrical conduits.

Design of Floor System

A typical cross-section of the floor system is shown in the plate on p. 643. For the present, traffic will be carried on two side roadways 28 ft. 9 in. wide, separated by steel curbs and railings from a future central lane 30½ ft. wide, which will not be paved until traffic demands it. All of the structural steel of the central roadway is in place. Two sidewalks, 12 ft. 10½ in. wide, are carried outside the roadway lanes directly under the cables, each protected by two substantial steel railings.

In the floor system the main cross-girders are suspended from the four 36-in. cables very 60 ft. Framing into these main cross-girders are ten longitudinal plate girders, eight under the roadway and two at the outside of the sidewalks. The roadway girders carry 16-in. transverse beams, 5 ft. 2 in. c. to c. Resting on top of the 16-in. beams are longitudinal bulb beams, 5¾ in. high and spaced 15 in. c. to c., which are incased in the roadway slab. These bulbs are continuous for the main 60-ft. panels. They are tied together with ⅜-in. threaded bolts, 12 in. c. to c. A reinforcing mat of ½-in. round bars was placed on top of the bulbs. The roadway slabs are nominally 5½ in. thick, with haunches incasing the bulb beams, adding 3 in. to this thickness. The sidewalk slabs are 3 in. deep. Expansion in the roadway and sidewalk slabs is provided at the panel points (every 60 ft.).

Roadway Construction

Because of the unusual rigidity provided by the four suspension cables the usual balancing of floor loads during construction of suspension bridges was not required. Consequently, paving work started at one end, the New York anchorage, and was carried straight across the bridge without a break. The regular schedule was to pour on one side of the bridge for two days, then cross to the other roadway for two day's work.

Form System—For the roadways the bottom of the concrete slabs was designed to span between the longitudinal bulb beams with haunches to incase these beams, covering the top surfaces of the transverse beams. The contractor was faced with the problem of designing a simple, easily set and easily stripped form for approximately 46,000 individual openings. The placing of any



Fig. 1—A traveling concrete plant high above the Hudson

1-yd. paving mixer moved across bridge in unpaved central roadway lane, which was also used by railroad hauling batched aggregates, the longitudinal bulb beams of the floor system serving as tracks. Concrete was placed by buggies from traveling platform bridge, shown at extreme left. Mechanical screed can be seen beyond this platform.

type of form from the top of the deck would be complicated by the myriad of cross-ties connecting the bulb beams. A 14-gage sheet-metal inverted pan, built to the contour of the arched bottom to fit between the bottom flanges of the bulb beams, was the answer to the form problem. They were rigid and yet light enough to be easily handled and were set in place from on top and stripped from a traveling scaffold below.

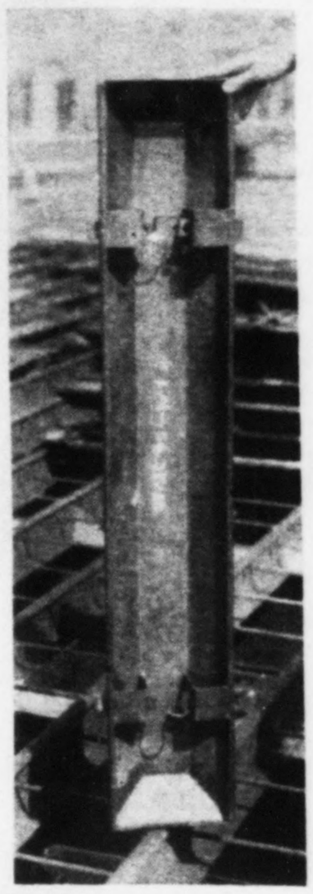


Fig. 2 — Bottom view of typical form pan

The pans, shown in Figs. 2 and 5, did not bear on the flanges of the bulb beams or cross-beams, but they were held in place by four sliding lugs which engaged the bottom flanges of the bulbs. The lugs worked through slots in the sides of the pan and in four welded straps, one for each lug. An arched wire spring held the lugs out into place when the forms were set. When stripping, releasing the springs permitted the lugs to be pulled back into the arch of the pan, then the form could be easily pulled from below.

In designing the pans it was contemplated that at least one of the cross-ties in each small panel would have to be removed to permit the forms to be set from above. But in actual practice it was discovered possible to slip the pans endwise into place without removing any of the rods. After each use the pans were thoroughly oiled. Most of the supplementary cross-beams of the floor system were regularly spaced 5 ft. 2 in. apart, and the majority of form pans were built in one piece to accommodate this length. But the distance between the main cross-girders and the adjacent cross-beams at each panel point varied somewhat. This condition required a form pan of adjustable length, made up of two parts, one sliding inside the other.

Plywood panel forms were used for the sidewalks. These panel forms were placed between sidewalk cross-beams 3 ft. 9 in. c. to c., two to the 12 ft. 10½ in. of sidewalk width. These forms were carried on 2x4-in. stringers wedged up on the lower flanges of the sidewalk beams. All sidewalk forms were set from deck level without the use of scaffolds below. These were also stripped without the use of scaffolds, the men working from the windbracing of the floor system which extended the full length of the bridge.

Traveling Platforms—Stripping of the forms was done from two traveling platforms suspended below the structural steel of the deck, one under each roadway (Fig. 6). The roadway platforms were 40x66 ft. and rolled on three rows of hangers fastened to the main longitudinal bridge girders. At the bottom of each hanger were four rollers that engaged the upper flanges of longitudinal I-beams rigidly fastened to the top of the platform. The platform hung 2 ft. below the bottom of the main cross-girders. As the traveler moved past the rear hangers they were released and reset at the front

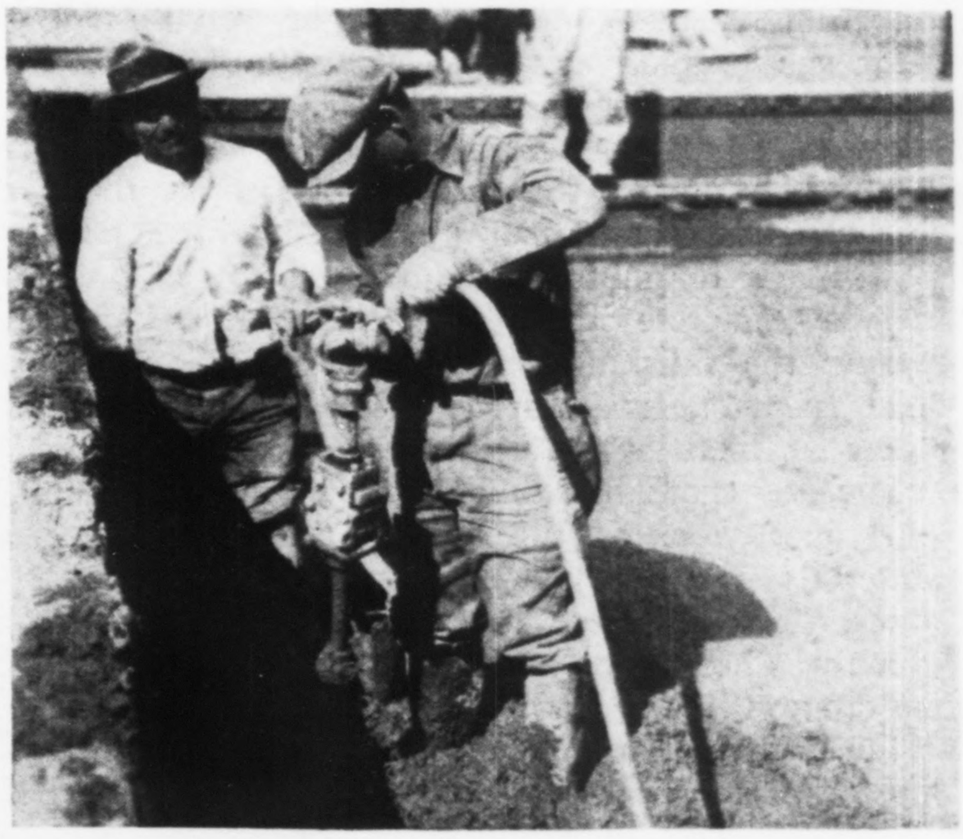


Fig. 4—A special pneumatic vibrator tamped the stiff concrete around bulbs and into corners

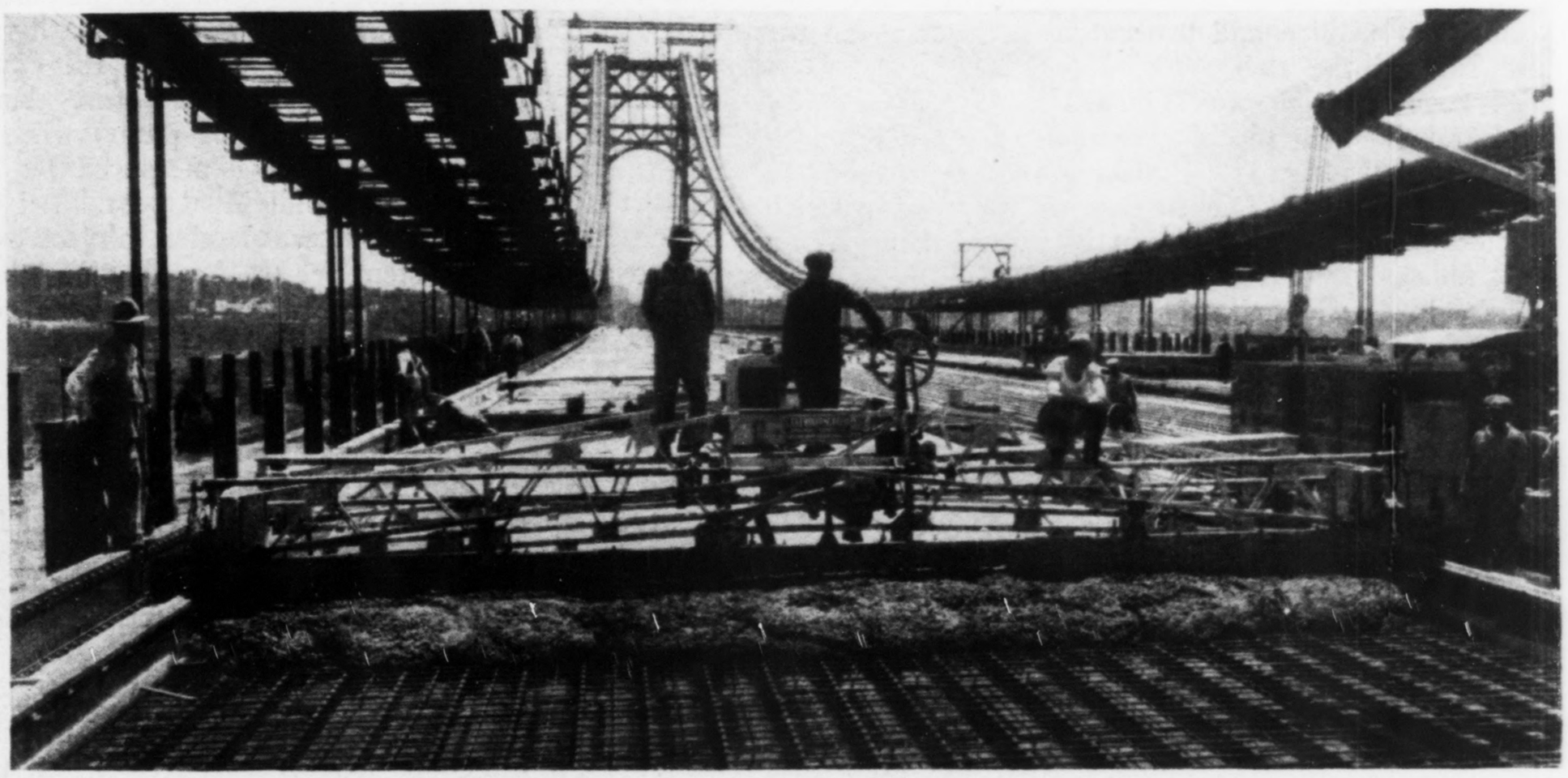


Fig. 3—This giant mechanical screed of 29-ft. span smoothed out the stiff concrete in the roadway slabs
An aggregate train can be seen at extreme right, with mixer crane ready to pick up a batch box.

end to provide a continuous trackway. Propelling power was provided by a hand winch and cable.

Because of the depth of the main cross-girders (10 ft.) the men could not work directly from the platform but stood on planks placed between the bottom flanges of the longitudinal bridge beams, protected by the traveler platform below.

Roadway forms were left in place two full days before stripping; those for the sidewalk were not stripped until four days after pouring. The forms were dropped on to the platforms and then handed up through openings in the floor systems to deck level to be transported ahead.

Concrete Distribution System—Full advantage was taken of the open central roadway lane in the concreting operations. The long rows of bulb beams with their closely spaced tierods furnished a substantial and safe deck from which to work. Furthermore, by using equipment of 28¾-in. gage the regularly spaced longitudinal beams provided a dozen tracks for industrial railroad operation the full length of the bridge. Accordingly, the contractor bought two special gage, 5-ton gas locomotives and several sets of cars to handle dry-batched aggregates. A 1-yd. paver mounted on crawlers mixed the concrete at the point of deposit. The paver moved forward on planks on the central roadway. It was specially equipped with a small derrick at rear for dumping aggregate boxes, a hoist tower carrying concrete chutes and a large derrick for transferring the finishing machine and distribution equipment from one roadway to the other. A view of the mixer and distribution equipment is shown in Fig. 1.

The aggregate batching plant was located at ground level just west of the New York anchorage block. Materials were dry-batched and loaded into 1-yd. boxes. Two of these boxes were placed on a flat car, which was hoisted up a 164-ft. tower to deck level. The gas dinkeys handled two cars at a time from the tower to the mixer. Jump-over switches at the tower and the mixer permitted easy maneuvering of trains.

Concrete was mixed fairly dry and stiff. To insure proper placing of the concrete around the steel work, a special pneumatic vibrator, shown in Fig. 4, was used. The vibrator proved to be very effective in keeping the bottom corners filled and preventing honeycomb on the under side of the slab.

The roadway slab was finished with a mechanical screed and finishing machine spanning the whole width of the roadway (Fig. 3). Finishing of the sidewalk slab was done by hand. The surface of the roadway slab was checked to ⅛-in. variation in 10 ft. and then given a broom finish.

After finishing, the slabs were covered with four layers of burlap, which was kept wet for ten days. Water for mixing and curing operations was supplied through 2½-in. pipe line that had been used as an air line by the contractor on the floor erection.

Concrete Mix—The concrete mix of the slabs was approximately 1:1.8:2.4, with a 5.75 water ratio. A cubic yard of concrete consisted of .62 cu.yd. of sand, .71 cu.yd. of ¾-in. trap rock and 1.75 bbl. cement. Each of the roadway panels, 28 ft. 9 in. x 60 ft., contains 38.5 cu.yd. of concrete and 1.6 tons of reinforcing steel. The sidewalk panels, 12 ft. 10½ in. x 60 ft. contain 72 cu.yd. of concrete and 42 ton of steel.

Railings—Setting the railings was an interesting operation. It was desirable that all the railing posts be plumb after the bridge floor came to its final position after loading with the slabs. Inasmuch as the posts were incased in the concrete slabs they were necessarily set far in advance of the slab laying. Because the curvature of the floor system was much greater before placing of the slabs, each post was set out of plumb when being placed, the amount of tip being measured by special templet designed for each individual post. All of the railing and conduit material was hauled to place over the industrial railway system set up for handling the concreting of the floors. This material was hoisted to deck level by a stiff-leg derrick on the bridge floor.

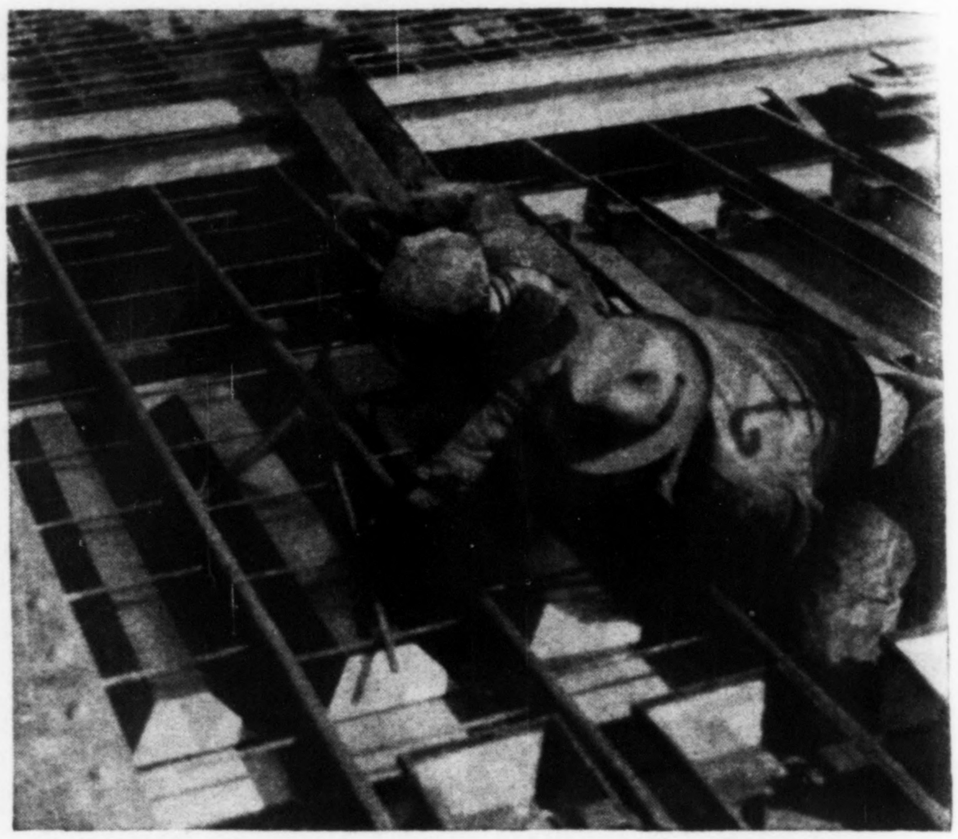


Fig. 5—The form pans were easily set from on top by two workmen

Pans are held by lugs resting on flanges of bulb beams. Illustration plainly shows tiebolts connecting bulb beams, which were a problem to any scheme of setting forms from on top. Reinforcing mat is yet to be placed on top of beams.

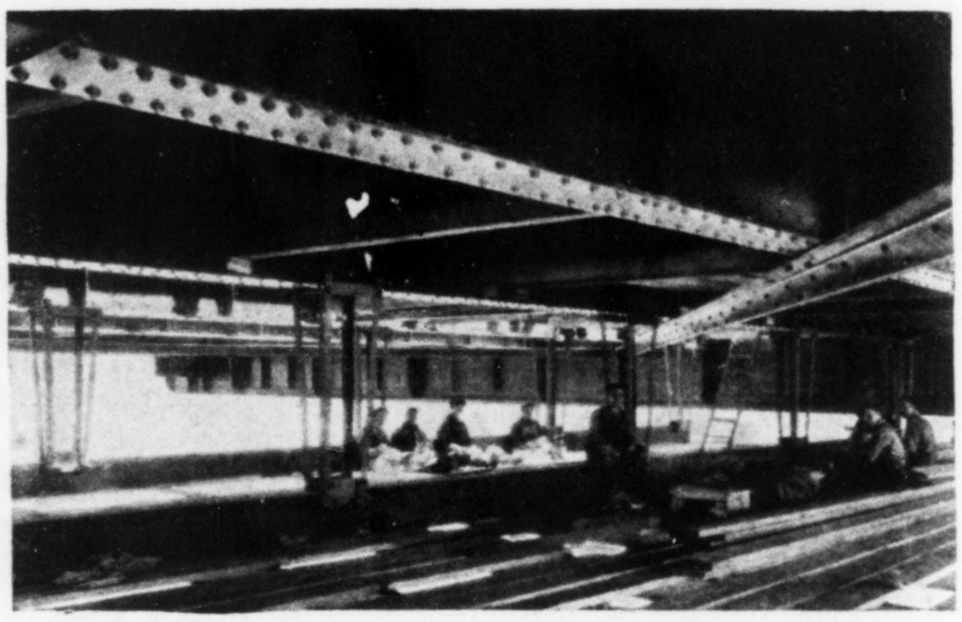


Fig. 6—Traveling platform from which roadway forms were stripped

Contractors and Personnel

The work was done under the direction of the Port of New York Authority, O. O. Ammann, chief engineer, Montgomery B. Case, construction engineer, and Homer R. Seely, resident engineer. Contract for the concrete work on the floor system was held by the Corbetta Concrete Corp., New York City. Roger H. Corbetta and Louis P. Corbetta were in personal charge, with Bruce Davis as superintendent and Joseph Lipset resident engineer for the contractor. The installing of the railings was sublet to the Globe Iron Works, New York, and the electrical conduits to the Wimpie Electrical Co., New York.

Complex Approaches Developed to Promote Efficient Use of Hudson River Bridge

Elaborate layout in New York providing connections to main north and south traffic arteries with no grade crossings, and separated on-bound and off-bound traffic, includes two ventilated tunnels several blocks long—New Jersey highway department furnishes similar facilities for traffic distribution at west end of bridge

THE APPROACHES to the Hudson River bridge —the greatest of the world's suspension bridges— represent major engineering and construction projects in themselves. Fundamentally, the problems at the New York and New Jersey ends were the same, that is to facilitate the movement of bridge traffic by providing separate on-bound and off-bound roadways, by avoiding all intersections at grade and by limiting roadway gradients to 6 per cent. Varying local conditions at the two ends of the bridge, however, made the actual planning and construction of the two approaches quite different.

On the New York side, the existing streets are flanked by expensive structures, largely apartment houses. Traffic on these streets was comparatively dense and could not be interfered with unduly. Approach connections were required to four main north and south streets—Riverside Drive, Broadway, Amsterdam Ave. and the Harlem River Speedway—the latter over a mile distant from the bridge. The problem was further complicated by the fact that Riverside Drive, which is located just east of the New York anchorage, is about 100 ft. below the bridge roadway, making an extensive ramp connection necessary.

At the New Jersey end the bridge terminates about 50 ft. below the top of the rocky cliffs of the Palisades, requiring a deep cut of considerable length to reach the surface. A fairly steep grade also was a necessity in this cut to permit the approach to pass over a street that parallels the river.

Unlike Manhattan, most of the territory adjacent to the bridge in New Jersey is undeveloped, and the existing streets of the village of Fort Lee are not flanked by such expensive structures as found on Manhattan. The problem here was to distribute the traffic to state and county highways and to provide connections to village streets rather than to city thoroughfares The difference in the solution is made clear by the fact that about 18 acres of land are utilized in the Fort Lee approach, while only 8 acres were acquired for the facilities at the New York end.

The New Jersey approach was worked out in collaboration with the New Jersey state highway department and local interests and is being built in large part by the state highway department. In New York the Port Authority collaborated with the city authorities but is doing all of the construction work itself.



Fairchild Aerial Survey

Fig. 1—Approaches to Hudson River bridge extend long distance from bridge to facilitate traffic collection and distribution

Amsterdam Ave., in the foreground of the view, on eastern edge of Manhattan Island, is reached by tunnels in 178th and 179th Sts., the first two cross-streets at right. New Jersey approach carries traffic nearly a mile west before making connections with state highways.

New York Approach Plan

The plan of the New York approach embraces the following principal elements:

1. Ramps, plaza and street connections west of Pinehurst Ave. between 178th and 179th Sts. (Initially only a part of this complete layout is being constructed.)
2. Connections with Riverside Drive.
3. Ramps, plaza and street connections in the vicinity of Fort Washington Ave. from Pinehurst Ave. to Broadway, between 178th and 179th Sts. (A part of this development will be deferred until bridge traffic requires it.)
4. Tunnels under 178th and 179th Sts., between Fort Washington and Amsterdam Aves. (The 178th St. tunnel is practically completed, and work will start immediately on the one in 179th St. so that it can be put in operation two years after the bridge opens.)

The roadway of the Hudson River bridge is 90 ft. wide but the present development leaves a 30-ft. strip down the center unpaved. The initial capacity, therefore, will be only six lanes, and the first stage of the approach development is planned for this capacity.

Main Approach West of Pinehurst Avenue

In this section the entire block between 178th and 179th Sts. has been appropriated for approach purposes. In Fig. 2 is shown the layout of the approach in this area as it finally will be built. Fig. 4 shows the initial stage of construction, which forms in reality the

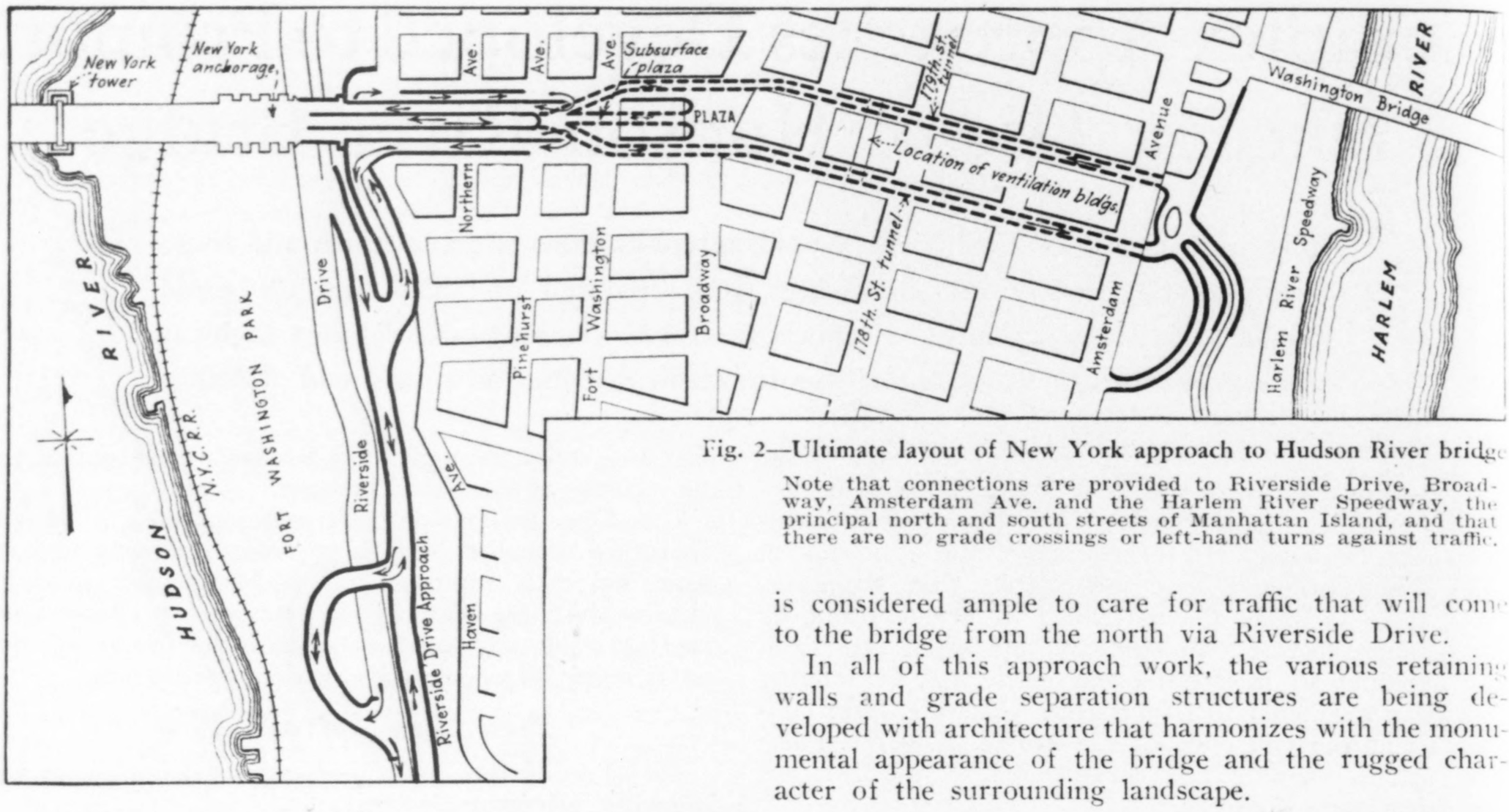


Fig. 2—Ultimate layout of New York approach to Hudson River bridge

Note that connections are provided to Riverside Drive, Broadway, Amsterdam Ave. and the Harlem River Speedway, the principal north and south streets of Manhattan Island, and that there are no grade crossings or left-hand turns against traffic.

core section of the finished approach. As finally built, the main ramp from the New York anchorage pier east to Pinehurst Ave., which also includes a concrete arch spanning 196 ft. over Riverside Drive and a parallel street, will be 145 ft. wide, made up of a central 60-ft. roadway and two 30-ft. side roadways, the rest of the width being occupied by sidewalks. The 60-ft. central roadway will drop away on a relatively steep gradient (— 4 per cent to Northern Ave. and — 1.82 per cent from Northern Ave. to Pinehurst Ave.) entering an underground plaza east of Pinehurst Ave. The side roads will slope downward more gradually to a surface plaza between Pinehurst and Fort Washington Aves. built over the underground plaza, and will afford connections with 30-ft. ramps to and from Riverside Drive. At present, only the central 60-ft. roadway is being built, the areas to be occupied by the ultimate side roadways being left vacant, as shown on Fig. 4, where they are marked "graded area." Widening of this main ramp to its ultimate width of 145 ft. is to be deferred until traffic volume on the bridge shall have reached 10,000,000 vehicles annually. At present, connection to the Riverside Drive ramps will be made at Northern Ave., where the grades of the central roadway and the ramps intersect, as shown in Fig. 3. The two side ramps, which connect with the Riverside Drive approach at Haven Ave., follow generally the existing grades of 178th and 179th Sts.

Riverside Drive Connections

Access to the bridge from Riverside Drive has been planned so as to provide complete separation of on-bound and off-bound traffic and to preclude any grade crossings at the intersections of the approach roadways with Riverside Drive.

Both the on-bound and off-bound roadways of this part of the approach are planned for a width of 30 ft., to accommodate three lanes of traffic, with the following exceptions: the on-bound roadway south of 172nd St. has a width of 35 ft. to provide for parking facilities, and the on-bound roadway west of Riverside Drive in Fort Washington Park has a width of 22 ft., which is considered ample to care for traffic that will come to the bridge from the north via Riverside Drive.

In all of this approach work, the various retaining walls and grade separation structures are being developed with architecture that harmonizes with the monumental appearance of the bridge and the rugged character of the surrounding landscape.

Layout Between Pinehurst Ave. and Broadway

The principal structure in this section is an underground plaza extending from a portal east of Pinehurst Ave. to the east side of Fort Washington Ave. and occupying the entire block between 178th and 179th Sts. Access to this plaza from the west is from the 60-ft. central roadway on the main approach ramp. At Fort Washington Ave. this underground plaza opens into tunnels under 178th and 179th Sts. that provide connections with Amsterdam Ave.

This underground plaza will be covered over with a steel superstructure, which in the final development of the plan will carry a surface plaza over which vehicles bound for Riverside Drive will pass, instead of turning off at Northern Ave., as in the present arrangement. Future development of the underground plaza also con-



Fig. 3—Main ramp construction, New York

In foreground is portal to subsurface plaza. View shows how only center roadway of ramp is being built at present.

templates two 20-ft. ramps rising eastward to a surface plaza west of Broadway. Construction of these ramps is deferred until such time as traffic over the bridge reaches a volume of 10,000,000 vehicles annually. Under the present scheme, access to Broadway will be via the existing roadways in 178th and 179th Sts., which connect with the bridge roadway at Northern Ave.

Connections to Amsterdam Avenue

In order to provide access to the bridge from Amsterdam Ave., and incidentally from the Harlem River Speedway and Washington Bridge over the Harlem River, two-lane, mechanically ventilated tunnels (Fig. 10) about 2,500 ft. long are planned in 178th and 179th Sts., extending from the subsurface plaza west of Fort Washington Ave. to a surface plaza east of Amsterdam Ave. The tunnel in 178th St. is being built at the present time. Work on the 179th St. tunnel will be commenced six months after the bridge is opened, to be completed 1½ years later. Work at the present time also includes the widening of 178th and 179th Sts. from 36 ft. to 44 ft. by decreasing the sidewalk widths. Final widening of these streets to 80 ft. is planned for the future.

Bridges and Ramps

Riverside Drive Arch—The arch bridge that carries the main approach ramp over Riverside Drive is designed for one of the heaviest unit loadings on record, 235,000 lb. per lineal foot. This compares with 47,000 lb. per lineal foot on the suspended span of the main bridge. Such a high loading results largely from the necessity for providing for a future lower deck capable of carrying four tracks of electric railway passenger lines.

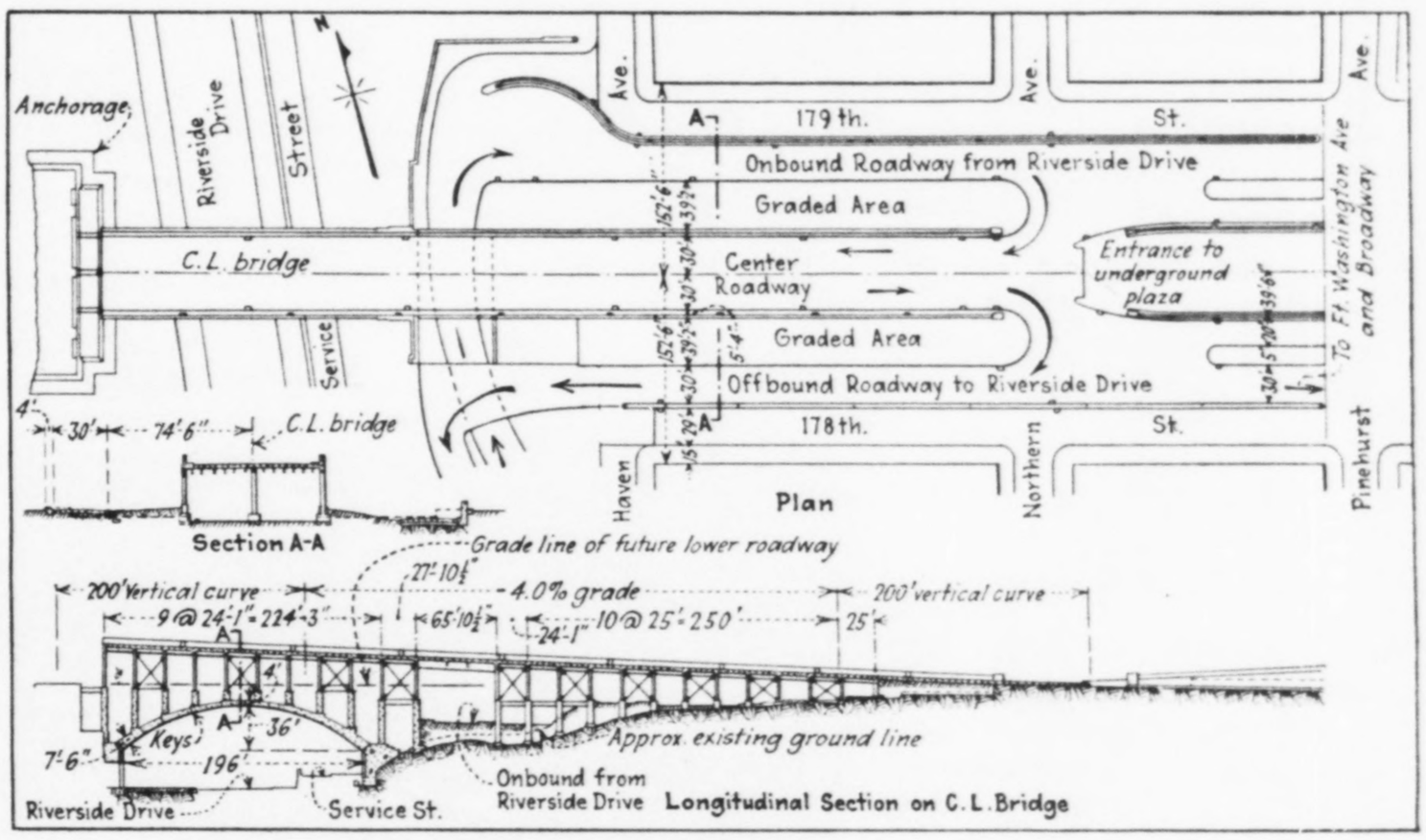


Fig. 4—New York ramp layout as it is being built

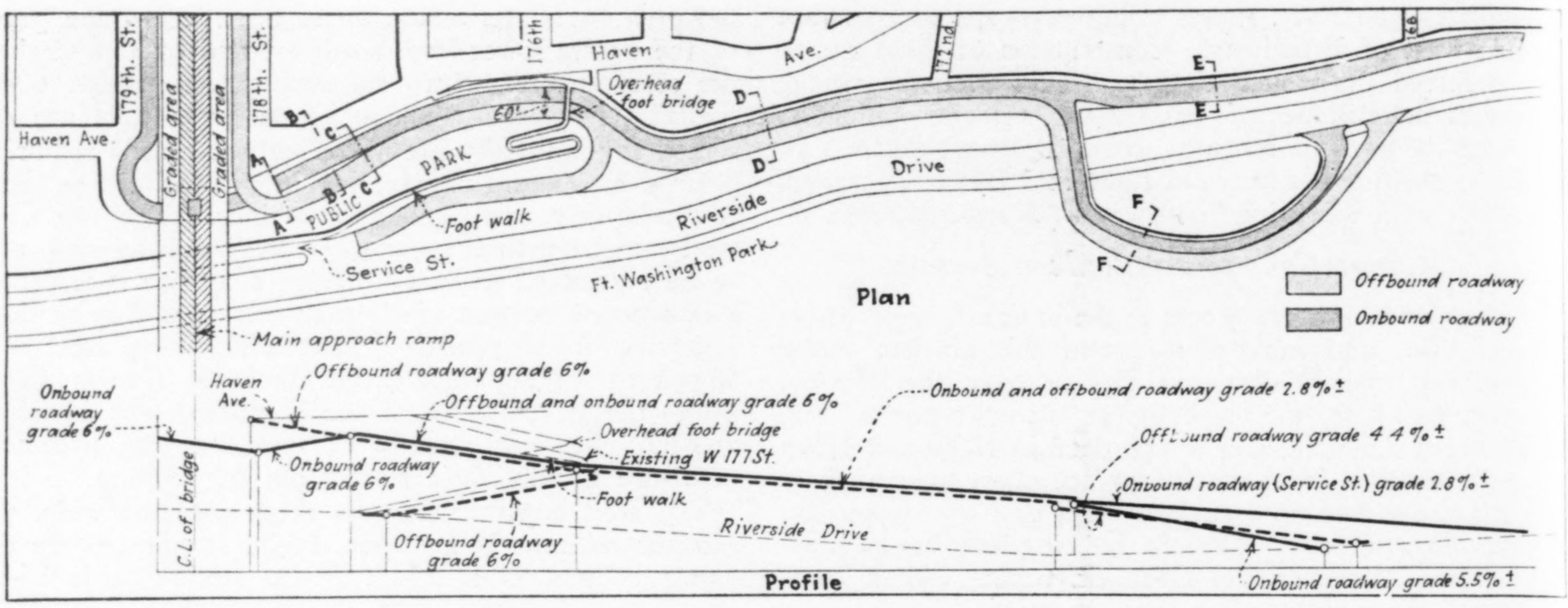
The bridge is of reinforced concrete, of barrel type, spanning 196 ft. across the Drive. The arch thrust at the west abutment is resisted by the main bridge anchorage, while at the east end it was possible to carry the thrust into the rocky hillside through a relatively small concrete block. The rise chosen—36 ft.—resulted from a balancing of practical and esthetic considerations, the practical factor being the elevation of the future lower bridge deck. A barrel design was deemed more efficient than one with ribs, in view of the flexibility it permitted in locating the railway tracks. Also the barrel design proved stronger.

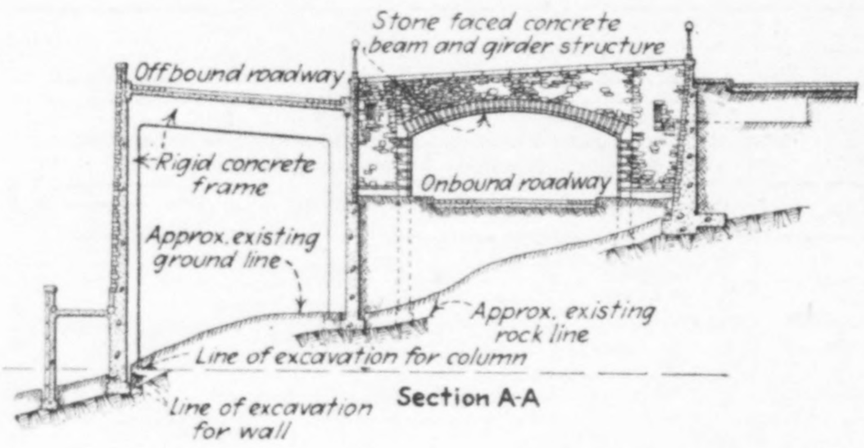
As built, the barrel is 65 ft. wide and varies in thickness from 7½ ft. at the springing line to 4 ft. at the crown. This bridge is really the core of the final structure, which will be 145 ft. wide. Future widening will consist in adding two barrels 40 ft. wide to each side of the present structure. No provision is made for connecting these additions to the present bridge except through the roadway superstructure. This superstructure is somewhat unusual in that it is a combination of concrete and steel. As shown in Fig. 4, concrete cross-walls 25 ft. on centers and poured integral with the arch barrel terminate about 3 ft. below the grade of the future lower-level roadway, and on top of them the steel superstructure supporting the present 60-ft. roadway is placed. The concrete cross-walls are designed for unusually heavy concentrated loads requiring a large amount of reinforcing in the top of the walls and relatively thick concrete sections. The use of a steel superstructure to support the roadway will also be an advantage when the bridge structure is widened. The concrete portion of the arch was poured in sections following usual practice. The reinforcing steel was lapped in all keyways except those at the haunches, where the steep incline of the barrel made it advisable to carry the rods straight through into the abutment to insure against their sliding out of position.

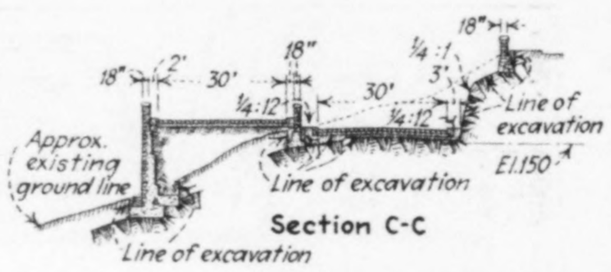
The steel superstructure is three columns wide, the exterior columns being 14-in. 96-lb. sections, while the center column is a 24-in. 76-lb. section with 15-in. 33.9-lb. channels as covers. The roadway is carried on 60-in. plate-girder crossbeams and consists of a 9-in. concrete slab reinforced with bar trusses 6 in. deep spaced on 6-in. centers. An expansion joint is provided in the floor at every fourth panel point, designed with particular attention to its waterproof quality. The arch is equipped with a complete drainage system of wrought-iron piping through which all the rain water on the roadway surface is carried to a point of discharge near one of the abutments.

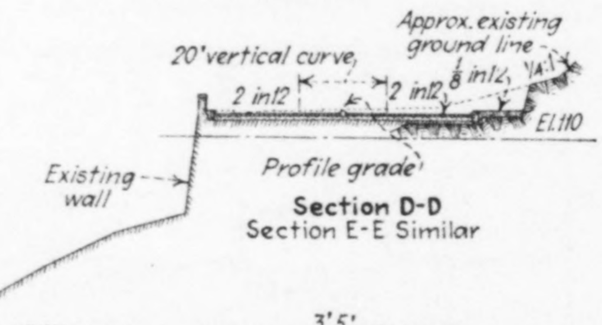
The initial construction provides spandrel walls of gunite placed on wire mesh, masking the structural superstructure. When the bridge is widened to 145 ft. it will be faced with granite to match the abutments and the rest of the approach work.

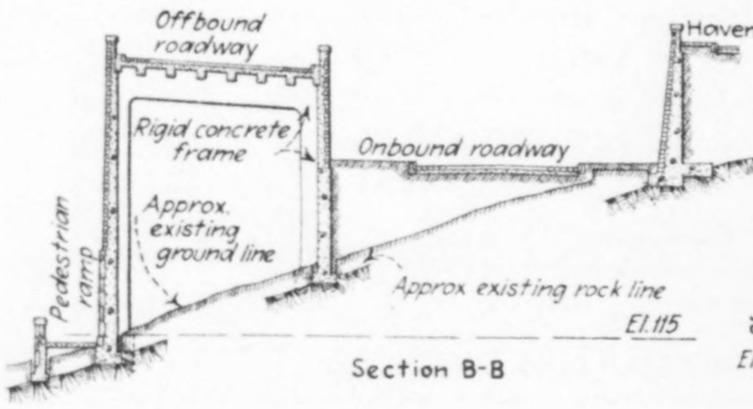
Main Ramp—The main ramp structure east of the arch is of conventional braced steel-tower construction resting on a concrete column substructure, the top of the latter being about 3 ft. below the future lower-level roadway. Immediately east of the Riverside Drive arch this ramp is pierced by the on-bound roadway of the Riverside Drive connection, and here the ramp roadway is carried by 92-in. plate girders on a 66-ft. span. In the rest of the ramp structure the longitudinal floor members are 24-in. I-beams. Cross-girders throughout the ramp are built-up plate girders 60 in.











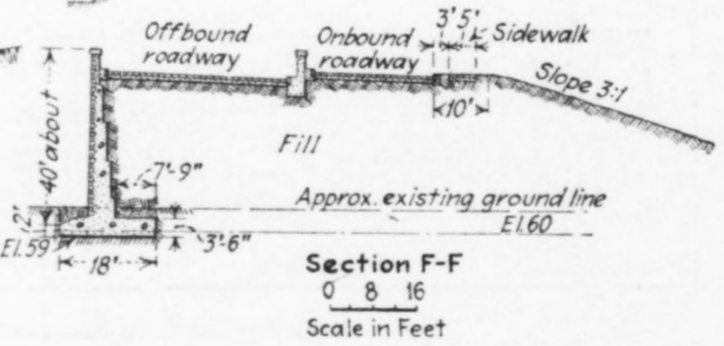


Fig. 5—Complete separation of on-and-off-bound traffic with no grade crossings and 6 per cent maximum grades are characteristic of Riverside Drive connections

deep. The east 100-ft. of the ramp is a retaining wall structure.

In the roadway design for this ramp an interesting detail was worked out to insure bond between the stringers and the concrete slab. On each 25 ft. length of stringer, four 5x3-in. bulb angle clips are fastened to the top flange and concreted into the slab. The bond thus secured makes the stiffness of the combined I-beam and slab about three times the stiffness of the I-beam stringer alone.

Near the point where the on-bound roadway from Riverside Drive pierces the main ramp, it also passes beneath the off-bound roadway leading to Riverside Drive. The grade separation at this latter point is made by means of a heavy concrete beam structure of about 40 ft. span. Because of the sharp curve in the off-bound roadway, this concrete beam framing is extensive and somewhat complex. The portal opening is faced with stone to simulate a rigid frame or a flat arch.

Riverside Drive Approach—The major portion of the structure in the Riverside Drive connection layout is of retaining-wall type, built into the hillside. This is true of all of the approach on the east side of Riverside Drive. An exception exists near the north end where a section of the off-bound roadway is designed as a rigid frame structure, to resist the earth pressure existing on the uphill side. A gravity retaining wall while satisfactory would have projected into the roadway area. The rigid frame by utilizing the resistance of both legs retained the earth equally well and provided a maximum roadway width.

The approach structure on the west side of Riverside Drive is interesting in that it consists of a tall retaining wall placed on a fill. Comparative estimates showed that this type of structure was cheaper than a viaduct. Also, there was considerable excess excavation from other parts of the work which could be economically utilized in filling between the retaining walls. The base of the wall was poured continuous since uniform settle-

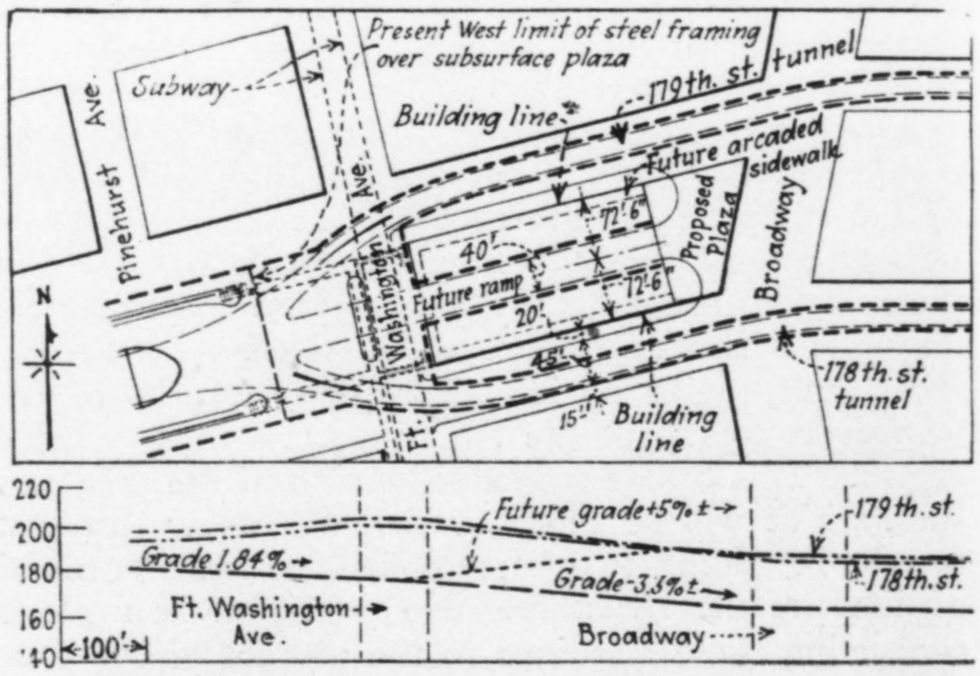


Fig. 6—Subsurface plaza between Pinehurst and Fort Washington Aves. affords access to tunnels and future Broadway ramp

Plaza on top of subsurface layout in final plan will furnish turning point for Riverside Drive traffic and provide connection with Fort Washington Ave.

ment was to be expected. The walls, however, are broken every 50 ft. by an expansion joint.

The portions of the approach on the east and west sides of Riverside Drive are connected by means of a reinforced concrete arch spanning 120 ft. over the Drive. It is irregular in cross-section, because the two roadways that it carries are at different elevations. The arch is comparatively flat and for this reason a broken joint is utilized at the center of the span cutting through the parapet walls, thus relieving the arch from any floor or wall action. As on the main arch, a complete drainage system of wrought iron piping is installed to carry away rain water.

The arch barrel is 75 ft. wide, the roadway being carried on concrete cross-walls placed on 11-ft. centers. The arch barrel is reinforced with $\frac{3}{4}$-in. rods on 18-in. centers transversely, 1-in. rods on 6-in. centers longitudinally and $\frac{5}{8}$-in.-diameter stirrups. An interesting feature of this structure is the relative size of the two abutments. Rock at the east end outcrops at the ground surface, requiring only a small abutment. At the west end however, it was necessary to excavate a hole 26 ft. wide, 44 ft. deep and 80 ft. long, in order to uncover sufficient rock of a satisfactory character for foundation requirements. The concrete required in this west abutment exceeds the total volume of concrete required in the arch superstructure and the east abutment combined. The arch is faced with stone.

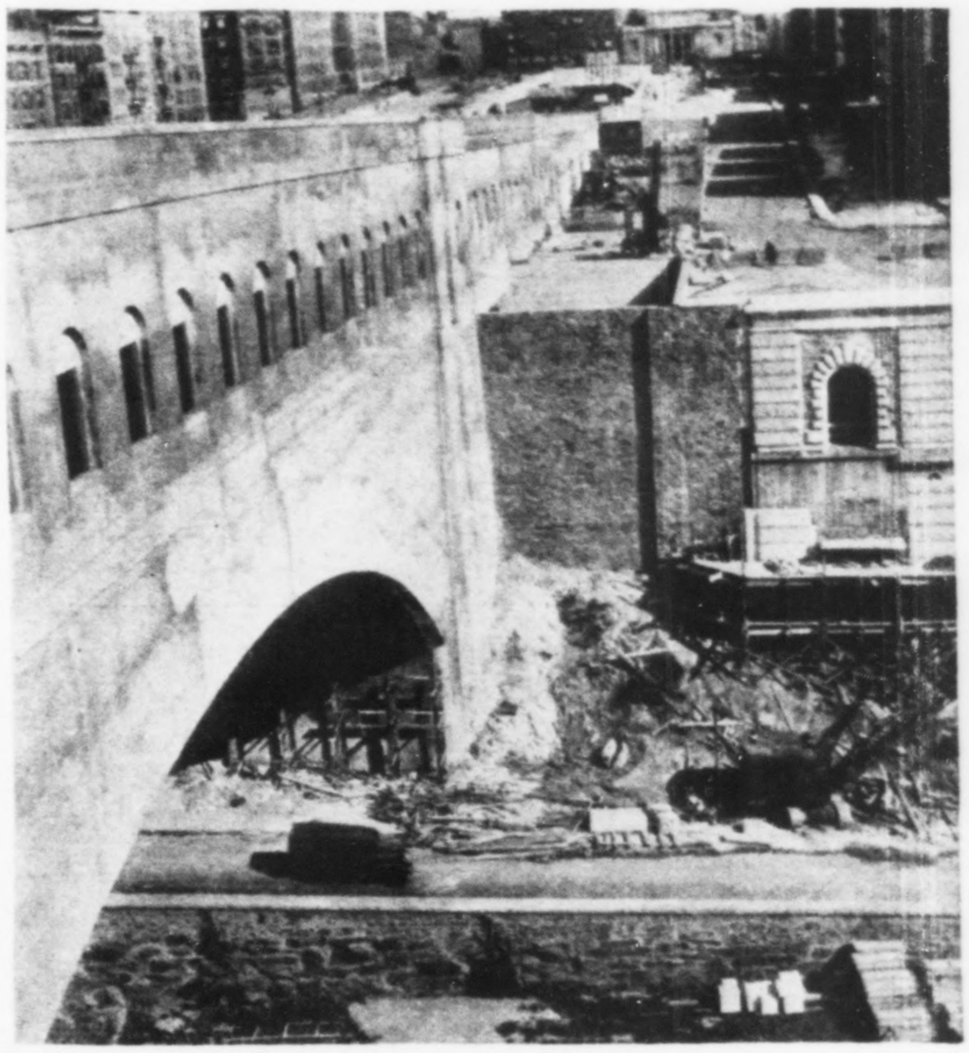


Fig. 9—Riverside Drive arch in present stage of development is masked by gunite curtain walls

View looking east from bridge anchorage, with off-bound roadway of Riverside Drive connection straight ahead.



Fig. 7—Riverside Drive connection roadways were largely cut out of a rocky hillside

The bridge in the foreground connects pedestrian ramps on either side of the approach roadway.

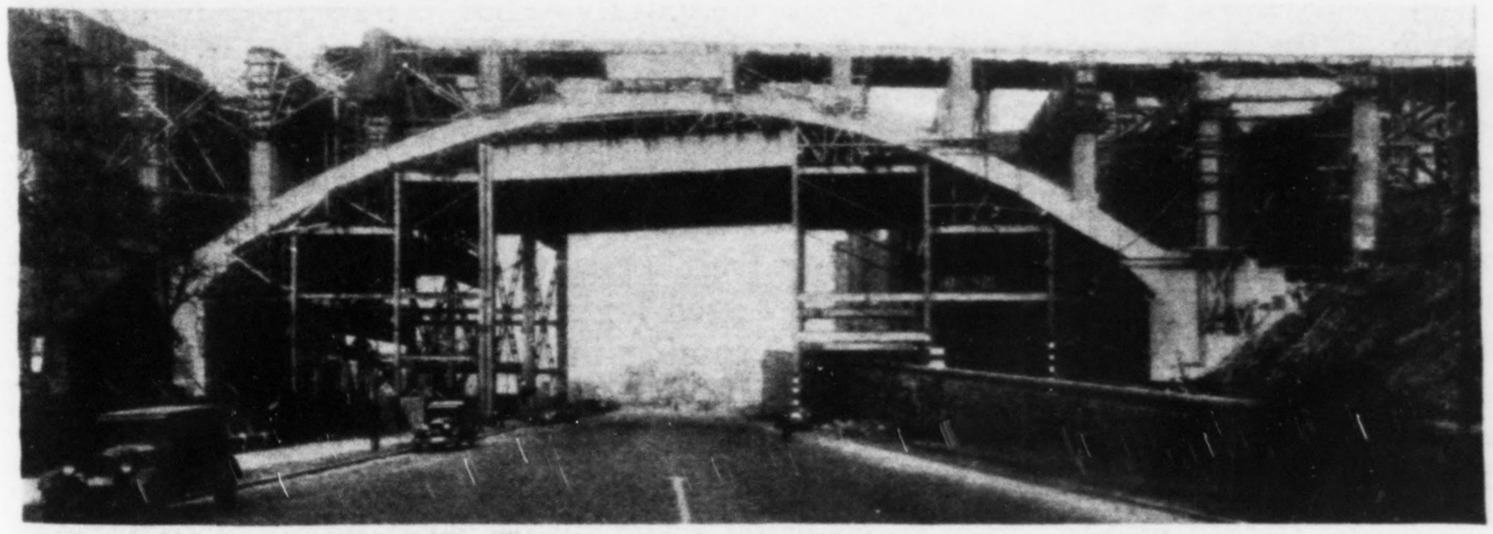


Fig. 8—Concrete arch carrying main ramp over Riverside Drive is designed to carry unusual load of 235,000 lb. per ft.

The future railway deck will rest on top of construction shown in view. Present highway deck is carried overhead on a steel superstructure, as shown in Fig. 4.

Approach Pavements—Several different types of pavements are utilized to meet the varying conditions in the different parts of the approach. The roadway paralleling and east of Riverside Drive is paved with sheet asphalt. Traffic here will not be heavy, and the grade is relatively light. On the curved portion of the approach west of Riverside Drive, asphalt block pavement is specified because of its non-skid properties. For the north half of the Riverside Drive connection near the main bridge, where grades are steep and several short and sharp curves occur, granite block is being laid. On the main approach ramp, the concrete slab will be utilized as the wearing surface. The tunnels are also to be provided with concrete roadways.

Tunnels—The tunnels under 178th and 179th Streets, from Fort Washington Avenue to Amsterdam Avenue, are of identical section and plan. In general, their design follows usual subway practice with the exception that ventilation ducts are provided. Each tunnel is a concrete box about 40 ft. wide and 20 ft. high in outside dimensions, made up of steel bents on 5-ft. centers

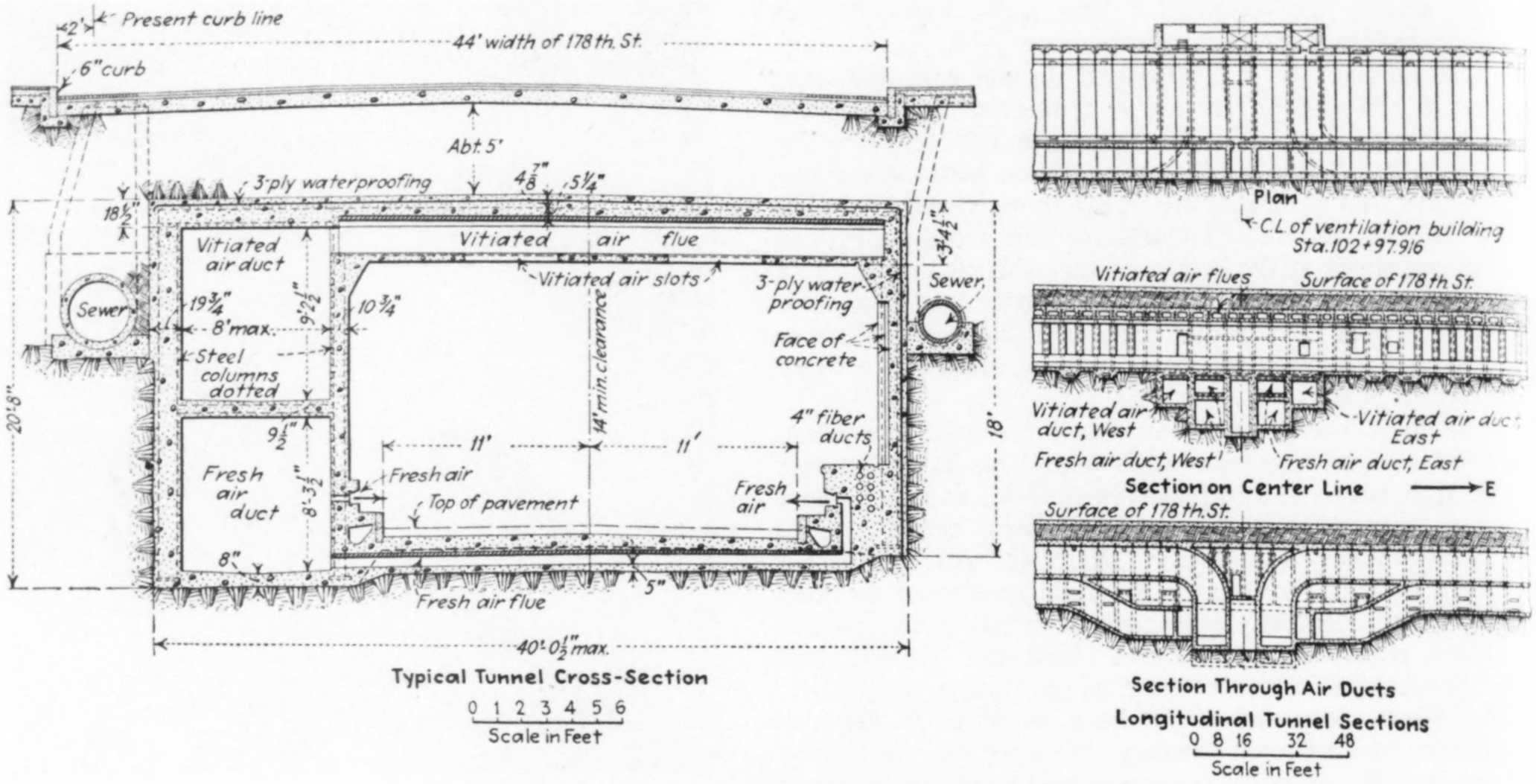


Fig. 10—Typical sections of ventilated tunnels in 178th and 179th Sts. that will carry traffic to and from the east side of Manhattan Island

incased in plain concrete at the air ducts and connected by concrete jack arches on the side opposite the ducts. Inside of this box the roadway occupies an opening 22x14 ft., providing for two lanes of traffic between curbs 22 ft. apart. The fresh and vitiated air ducts are located at one side of the box, separated from the roadway and from one another by concrete walls. The vitiated air duct is over the fresh air duct. Fresh air is admitted to the tunnel immediately above the curb on both sides of the roadway, fresh air flues on 15-ft. centers beneath the roadway providing connection with the discharge opening at the far curb. Vitiated air flues on 15-ft. centers occupy the spaces between transverse roof beams; air exhausted into these flues through slots in the ceiling above the roadway is drawn into the vitiated air duct beside the roadway, through which it finds its way to the ventilation building.

This ventilation building is placed about midway of the tunnel length in the block between 178th and 179th Sts. It serves four separate and distinct ventilating systems—that is, each tunnel is divided into two systems running from the ventilation building either way to the portals. Inasmuch as the ventilation building is on the opposite side of the tunnel from the air ducts, connection with them is through a transverse tunnel excavated in rock beneath the main tunnel roadway (Fig. 10).

The 178th St. tunnel, the first to be constructed, has been built by the usual cut-and-cover method. Its roof is only about 5 ft. below the street level, so that work was not difficult. At Fort Washington Ave. the tunnel passes over the line of the new Eighth Ave. subway, requiring careful but not otherwise difficult construction. A large-scale rock excavation job was also required in building the subsurface plaza west of Fort Washington Ave. A plan of this plaza as well as the overhead plaza, which, in the final plan, will serve as a turning area for Riverside Drive-bound vehicles, is shown in Fig. 6.

Plan of New Jersey Approach

The New Jersey approach to the Hudson River bridge is primarily a collecting and distributing agency for traffic, in contrast to the layout at the New York end, which is made up essentially of connections to streets that intersect the center line of the bridge. It was deemed desirable in planning the New Jersey approach to accomplish the actual traffic distribution at as great a distance from the bridge as economical and physical considerations would permit, and at the same time to avoid all semblance of a plaza where undue concentration of traffic could occur. Accordingly distribution facilities are spread over about a mile of the approach that terminates on the edge of a plateau overlooking the Hackensack River valley. Three state highways from the north, northeast and south, respectively, are made to converge near the western limit of this plateau, and a network of highways and grade-separation structures results that gives the appearance of complication but is designed to eliminate all grade crossings and safely speed up traffic. The remainder of the approach between this point and the bridge contains numerous simple under- and over-crossings and ramp connections to existing streets in the village of Fort Lee.

Extending west from the bridge the main approach road is a depressed highway varying from 60 to 100 ft. in width and passing in order under Lemoine, Center, Linwood and Fletcher Aves. The first street west from the bridge, Hudson Terrace, is crossed on a viaduct.

In describing the approach layout it may be considered as being made up of three principal parts: (1) the toll plaza between Hudson Terrace and Hoyt St.; (2) the connections to the several streets listed above; and (3) the distributing branches leading to the three

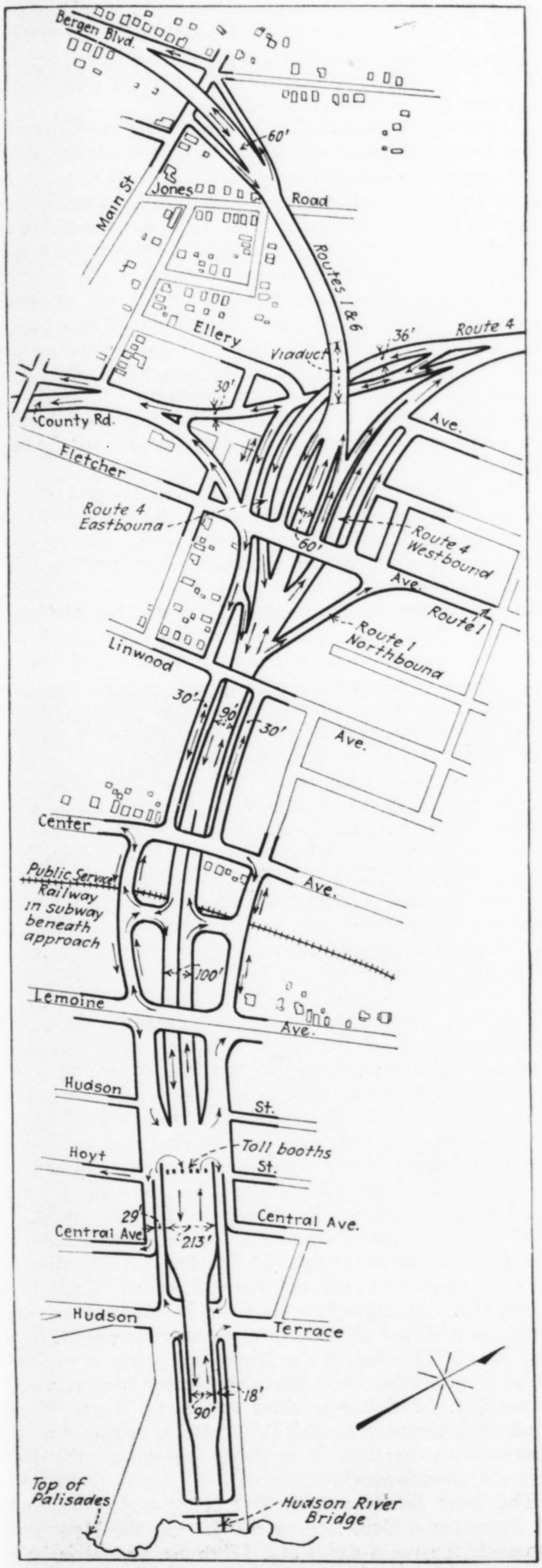


Fig. 11—New Jersey approach to Hudson River bridge is a vast collecting and distributing agency for traffic, in which all grade crossings have been eliminated

state highways. The Port of New York Authority designed the approach as far west as Lemoine Ave. but is only building the section from the end of the bridge to Hoyt St. The Port Authority is paying the New Jersey highway commission $500,000 for building the portion between Hoyt St. and Lemoine Ave., and has contributed $2,500,000 to the cost of the approach work beyond that point, which has been designed and is being built under state highway contracts.

New Jersey Toll Plaza—All tolls will be collected at the New Jersey end of the bridge. To facilitate this operation the 100-ft. roadway at the beginning of the approach, which corresponds to the total roadway width on the bridge, is flared out to nearly 200 ft. after it crosses over Hudson Terrace. This 200-ft. width provides space for eight toll-collection booths and side ramps, as shown in Fig. 11.

Hudson Terrace is the only cross-street that passes under the approach, the other intersections being overcrossings. Traffic to and from Hudson Terrace utilizes ramps at either side of the main approach roadway. The three streets paralleling Hudson Terrace to the west do not cross the approach but are dead-ended into so-called marginal roadways outside the ramps. These marginal streets are readily accessible from the main approach roadway immediately west of the toll booths.

Cross-Street Connections—From the toll booths west to the state highway distributing points (Fig. 11) the approach is a depressed roadway 100 ft. wide. All cross-streets are carried over on concrete-incased steel-girder bridges of one or two spans. In the case of two-span structures the center pier with islands divides the east and westbound traffic. There are three of these cross-streets—Lemoine, Center and Linwood Aves. Local traffic facilities are provided between Lemoine and Center Aves. by means of a cloverleaf layout of ramps, and between Center and Linwood Aves. by means of marginal streets connecting with Center Ave. and the cloverleaf system. The layout and construction of the cloverleaf were somewhat complicated by the presence of a trolley line just east of Center Ave. that crosses the approach in a subway below.

Distributing Roads—West of Linwood Ave. begin the facilities for distributing the traffic to the various state highways, all of which are entirely new routes being built in large measure as an integral part of the approach project. Reference to Fig. 11 with careful attention to the arrows denoting direction of traffic flow will be necessary to understand this distribution layout, which is probably as complicated a bit of grade-separation work as has ever been undertaken.

Routes 4 (from the north) and 6 (from the south) have the bridge as their terminus. Route 1 bypasses the bridge, coming in from the south over route 6 and continuing in a northeast direction from the distribution layout.

Bridge traffic moving west under Linwood Ave. and entering the distribution road layout first encounters a branch to the right, which leads to route 1 northbound. The next branch to the right leads under Fletcher Ave. to route 4 northbound. Traffic keeping straight ahead on the main approach roadway runs into routes 1 and 6 southbound. These are the three principal alternatives for bridge traffic bound west.

Similar facilities, with no intersections at grade, are provided for traffic originating on any of the three main highways and bound for the bridge. Thus traffic from the north on route 1 crosses over all of the ap-

proach roadways on the Fletcher Ave. viaduct and makes a left turn onto a ramp that leads eastbound down to the main approach. (If this traffic wishes to continue south on route 1, it turns right on a westbound ramp from Fletcher Ave. that leads down to the main approach roadway.) Traffic from the north on route 4 passes under the main approach (and incidentally over a county road, making a three-level grade separation at this point), thence under Fletcher Ave. and into the main approach. Traffic from the south on routes 1 and 6 enters directly onto the main approach west of Fletcher Ave. if it is bound for the bridge. If it is bound north on route 1, it leaves the main approach on an eastbound ramp leading up to Fletcher Ave., where it makes a left turn onto route 1.

Structures in the New Jersey Approach

For the most part the grade-separation structures in the New Jersey approach are of simple design and construction. Standard express highway design was utilized for the slab of the main approach road, which is built in a cut whose rock sides are self-supporting. The viaducts that carry Lemoine, Central, Linwood and Fletcher Aves. over the main approach cut are all of similar design—concrete-incased rolled girder beams on concrete abutments with wide wing walls. The steel girder beams are equipped with a fixed shoe at one end, while the opposite shoe is fitted with slotted anchor-bolt holes that permit expansion. The 8-in. concrete roadway slabs are poured integral with the beam incasement. Parapet walls, abutment faces and pier faces (where used) are bush-hammered for architectural effect. Ornamental concrete railings complete the structures.

At Hudson Terrace, where the approach roadway is carried overhead, the viaduct structure, 135 ft. long, consists of three transverse steel bents, each six columns wide, that carry the 90-ft. approach roadway. East of Center Ave. the main approach is carried over a street railway in a concrete box subway.

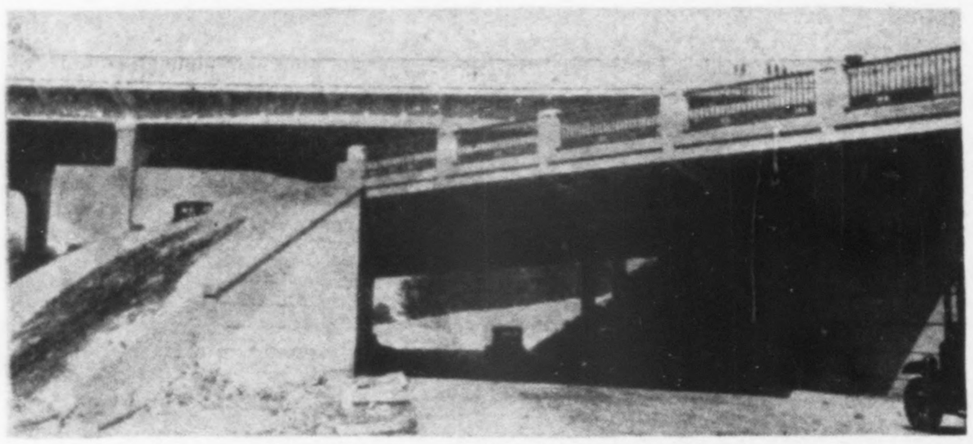


Fig. 12—Three-level grade separation in which route 4 eastbound passes beneath main approach viaduct and over county road

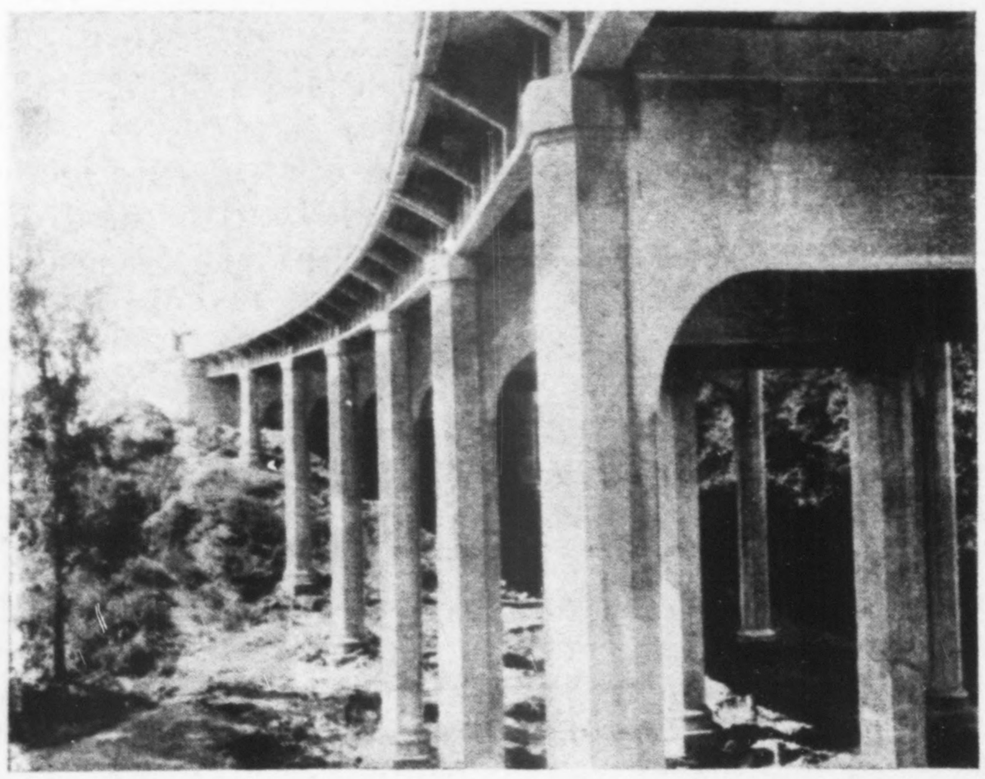


Fig. 13—Main approach viaduct consists of concrete-incased steel girders on three-column reinforced-concrete bents

The three-level grade-separation point near the western limits of the approach presents the most interesting structural layout of the project. The main approach roadway is carried overhead on a high-level viaduct that crosses a deep wide valley. At its eastern end this viaduct crosses over route 4 eastbound, which in turn spans a county road running north and south.

In general, all structures are of steel girder design incased in concrete. The three-column bents that support the high-level viaduct are of reinforced concrete except for two skew bents in the vicinity of the grade separation, which utilize rolled steel columns. Steel is used for these columns to facilitate the viaduct floor framing connections, which are complicated because of the skew layout.

The structure that carries route 4 eastbound over the county road is on a 33 deg. skew and consists of two long plate girders 8 ft. deep, spaced 42 ft. apart by 36-in. plate-girder floor beams on about 7-ft. centers. The bottom flanges of the floor beams frame into the bottom flanges of the longitudinal girders, so that the upper parts of these girders serve as railings for the bridge. The roadway is 36 ft. wide between curbs and consists of a $9\frac{1}{2}$-in. concrete slab.

The main viaduct structure is 62 ft. 4 in. wide between outside columns, the main framing of the roadway structure on top of the three-column bents consisting of longitudinal lines of girders. The two plate girders over the outside columns are 66 in. deep, while 36-in. rolled girder beams on 6 ft. 9 in.-centers constitute the interior floor framing. These beams are fitted with $\frac{3}{4}$-in. bearing plates at either end. Slotted holes in the bearing plates at one end permit expansion. The beams are braced transversely by the $10\frac{1}{2}$-in. concrete roadway slab. Sidewalks 5 ft. wide are cantilevered on brackets on either side of the structure.

The tallest concrete bent in this main viaduct is about 45 ft. from base of column to bridge seat. Columns are octagonal in plan, from 3 ft. 9 in. to 4 ft. 3 in. across, and are reinforced by $1\frac{1}{4}$-in. vertical square bars and $\frac{1}{2}$-in. square hooping. The cross-girders of the bents are 8 ft. deep and 3 ft. 4 in. wide, reinforced top and bottom by $1\frac{1}{4}$-in. square rods and with $1\frac{1}{4}$-in. square stirrups. No transverse bracing is used between the columns of a bent, and there is no longitudinal bracing between the bents.

Personnel—The development of the approaches for the Hudson River bridge was under the direction of J. C. Evans, terminal engineer, Port of New York Authority. Montgomery B. Case is engineer of construction and had as resident engineers Homer R. Seely and R. F. Wheadon at the New Jersey and New York ends, respectively. For the New Jersey highway commission, J. L. Bauer is chief engineer; Morris Goodkind, bridge engineer; and R. M. Beck, engineer of the metropolitan division, is in direct charge of the New Jersey approach work.

The New Jersey approach is being built by George M. Brewster & Son, Bogota, N. J. On the New York approach the main ramp and 178th St. tunnel is being constructed by the Cornell Contracting Corp., New York. The Riverside Drive connections have been built by Wm. P. McGarry Co., New York.

Well Points Drain Swamp Under Bridge Approach Structure

Peat bog dried up and removed to provide foundations for street-car subway crossing New Jersey approach to Hudson River bridge after the piling proves unstable

A DIFFICULT foundation problem in building a concrete box subway to carry the Public Service electric railway line through a peat bog across the New Jersey approach to the Hudson River bridge was solved by the effective use of well points. Original designs for the subway foundations called for creosoted piling driven through 25 ft. of swamp to rock. After driving the first few piles it was found that the muck offered little lateral stability to the piles, and therefore their use was impossible. Many schemes were then proposed to provide suitable foundations. It was finally decided to install a well-point system to dry up that part of the bog underlying the proposed subway, then to remove the swampy material to rock, backfill with good material and place the subway structure directly on the new fill. This scheme was entirely successful.

The Public Service street-car line runs parallel to



Fig. 1—Why pile foundations were not suitable for subway foundations in New Jersey swamp

and about one half mile west of the Hudson River, across the west end of an extensive swamp. For years a single track has been carried across this swamp on a stable fill. Three main roadway approaches to the Hudson River bridge cross the street-car line at almost right angles (Fig. 12, p. 664). As the new roadways are carried across the swamp on fills at a considerably higher level than the street-car grade, a series of four-track concrete subway boxes, totaling 660 ft. in length, with connecting retaining walls and wings, were designed to separate the highways and railway. All of the structures were designed to be carried on wood piling.

Street-car service was diverted during construction

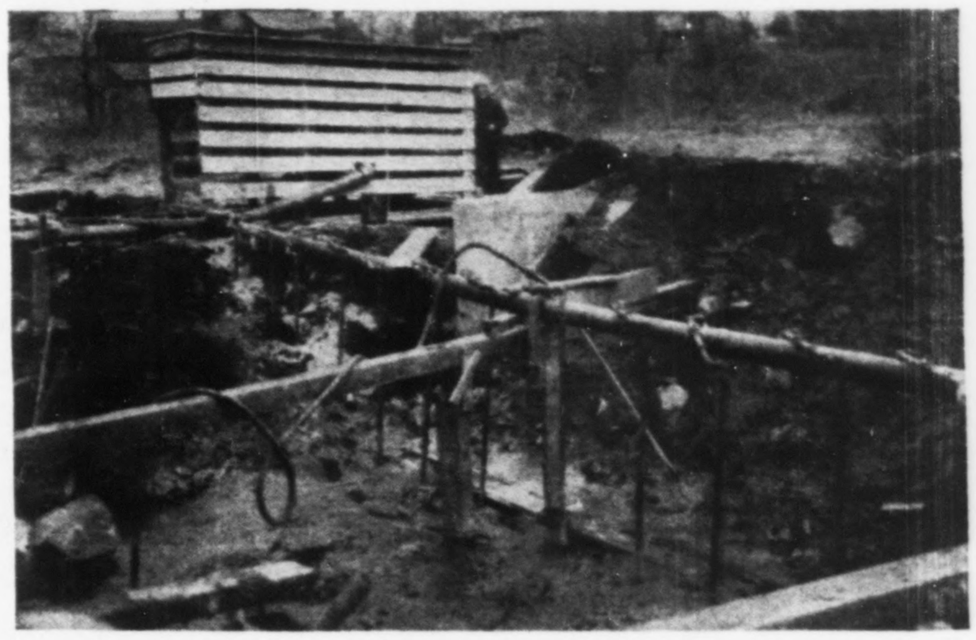


Fig. 2—End of one of two headers of the well-point system that drained an area 65x600 ft.
One of the five centrifugal pumps is within shanty.

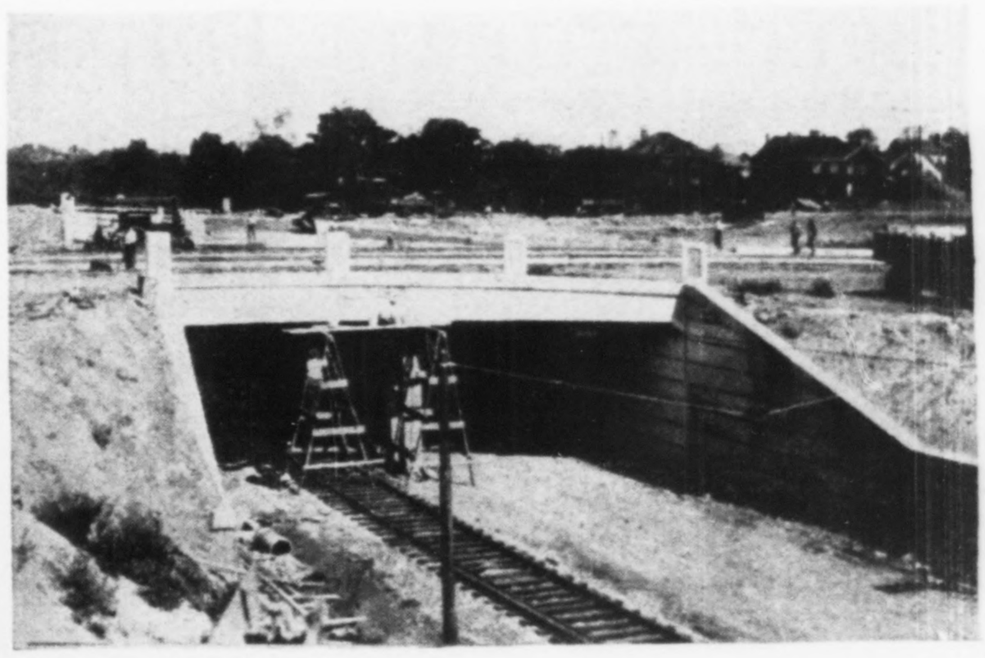


Fig. 3—One of the new subway sections about completed
The main center approach road will cross this structure.

operations. Piledriving was started at the south end of the job by a gas crane working from the old railroad grade. It was soon apparent that pile foundations would be unsuitable. The piles would not stay down, and after driving several the surrounding ground became a soupy muck that gave the piles no lateral support. Some other type of foundation was necessary.

In as much as ledge rock was about 25 ft. below grade, a proposed scheme to dry up the muck, dig it out and replace it with good material seemed feasible. A well-point system was installed for the purpose of drying out that part of the bog. Two header pipes were placed 65 ft. apart, extending for 600 ft. along the outside lines of excavation. Well points were driven to rock every 3 ft. along these header lines, each point being connected directly to the header. The whole assembly was hooked up to five centrifugal pumping units.

Pumping was started at the south end of the job. The ground was soon dry enough to permit excavating. Digging was done with a clamshell bucket handled by a gas crane, the crane at first operating on the old railroad grade, backing away from the hole as excavation progressed. Excavated material was loaded into trucks backed down the railroad grade from a ramp at the center of the job.

The well points proved very effective in drying up the material, not only between header lines but far enough outside to permit the excavated banks to stand on a reasonable angle of slope. As the excavating progressed northward, filling-in followed from the south end, leaving but a comparatively small area open at any one time.

After the new fill was started, the crane was transferred from the old railroad grade to the new fill. Another crane was then brought into use, the machines working side by side but swinging in opposite directions. Well points were removed with the excavation. A total of 58,000 cu.yd. of muck was removed. Backfilling required 40,000 cu.yd. of material, the backfilled grade being below original ground level. The new fill was thoroughly rolled before the invert slabs of the subway structures were poured.

The subway was built by the bridge department of the New Jersey state highway commission, under the direction of Morris Goodkind, bridge engineer, and M. L. Vail, resident engineer. George M. Brewster & Son, Bogota, N. J., was contractor, with the Moretrench Corp., of Rockaway, N. J., doing the drainage work.

LETTERS TO THE EDITOR

Determining Solar Meridian by Stopwatch

Sir—In *Engineering News-Record* of Sept. 10, 1931, p. 420, there appeared an article by James Girand, Jr., entitled "Solar Meridian by Stopwatch." I read this article with considerable interest, as I once made an unsuccessful attempt to derive a short method by similar reasoning.

Mr. Girand arrives at a simple method in formula No. 4, but unfortunately the derivation contains one of those errors that are difficult to avoid in new and untried methods.

In differentiating equation (2) it was assumed that the declination D is constant, which is practically correct for polaris and similar bodies and for the sun when it is near either solstice, but which is considerably in error for the sun when it is near the equator. Declination D is a function of t and should be considered as such in equation (2). If it were so considered, the resulting equation (4) would be more complicated than equation (1). R. R. ROBERTSON.

Priest River, Idaho.
Oct. 1, 1931.

Defends Public Engineering Work

Sir:—Your issue of Sept. 17 contained an interesting letter from a Los Angeles engineer, on the subject "Public Work by Private Engineers." It stated that if unemployment relief is to be secured by increased construction of public works the only way that a large enough volume of such work could be handled with sufficient dispatch would be by turning over the projects to private engineers and architects with full power to design, let contracts and supervise construction. The statement is also made that these suggestions apply to city, county and state work, and that in the past city and county engineering departments have taken on a large amount of work more properly belonging to the private engineers.

A casual reader of this letter might think that in Los Angeles the city and county engineering departments are too small to handle an increased volume of work. As a matter of fact, hundreds of engineers in these departments are being laid off and added to the list of unemployed because of the claim put forth by the Los Angeles realty board that there is not enough work to keep them busy. The realty board is interested only in reducing taxes on real estate, and as usual the line of least resistance is to slash engineering departments. A letter such as that mentioned will arouse the suspicion that private engineers are backing the demand for reducing public engineering departments, in the hope that after such departments are sufficiently reduced the authorities may be persuaded to turn over new work to private engineers on the plea that the public departments are too small to handle efficiently a sudden increase in volume of work.

It would be interesting to know what "large amounts of work more properly belonging to the private engineer" have been taken on by city and county engineering departments in the past. There may be difference of opinion as to where such work properly belongs.

Public improvement work in thickly populated communities is a complex problem. Streets, sewers, bridges, water supply, storm drains, grade separations, etc., are all closely related and can not be designed independently of each other. Governmental legislative and administrative bodies constantly call for immediate information or preliminary estimates of cost of some detail of a project under study or design. Interested property owners continually inquire for similar information, all of which is concentrated in a single public engineering department in order best to serve the public.

When and if it can be demonstrated that this work will be handled as efficiently by private engineers as by public engineering departments, then it will be soon enough to discuss whether such a change is desirable. If it is determined that our public improvement engineering would be better handled by private engineers, it follows logically that all corporations that now maintain their own engineering department should disband them as a measure of economy and turn their work over to private engineers also.

Los Angeles, Calif.
Oct. 8, 1931. W. C. HOGOBOOM.

[*Engineering News-Record* agrees with our previous correspondent in believing that public engineering staffs are not adequately manned to plan the coming winter's work program without extensive help from private engineers.—EDITOR.]

Lowered Groundwater Endangers Foundations

Sir:—The lower section of Manhattan, bounded by Fulton St. on the north, Greenwich St. on the west and William St. on the east, has been practically drained of subsurface water by the several parallel lines of subways, the lines in William, Nassau, Broadway, Church and Greenwich Sts. running north and south, and the cross-lines at Stone, Wall and Fulton Sts. These subways, being at considerable depth below the surface, have honeycombed this area. Naturally any flow of water toward them would cause considerable loss of ground.

After construction there is always ample space for free passage of water along the sides and bottoms due to decay of organic matter incorporated in the fill and to improper tamping. There is also considerable loss of subsurface water by reason of continuous pumping from subway sumps fed by inflow through porous concrete, and further loss through the constant pumping from sumps in subcellars, of which a great many in this area are deep.

Many of the older buildings in this section have wooden pile and timber foundations. This type of foundation has proven its worth, being economical and durable if at all times submerged. If by any chance this preserving bath is drawn off and the foundation is exposed to air, decay soon results. The capillarity of the soil holds a certain amount of moisture in suspension, but also provides a fertile field for spores of decay. It is a question how long it will be before the bearing value of these timber foundations has been so reduced that they can no longer hold the loads for which they were originally designed.

While the front walls and bearing piers adjacent to the new subways have been underpinned, their rear and sides are still jeopardized. Unquestionably many buildings in this area should have their walls resupported by underpinning with either concrete or jacked piles. A survey would probably show an advanced state of deterioration. Some of these older foundations are now being exposed by adjacent deep excavations, affording a good opportunity for investigation. For safety sake this survey should be made and samples of timber taken for study by properly qualified engineers.

New York City.
Oct. 10, 1931

MAURICE BLUMENTHAL
and JAMES HAMILTON.
Contracting Engineers

NEWS OF THE WEEK

Federal-Aid Road Money for 1933 Apportioned Among States

Apportionment to the states of the $125,000,000 previously authorized for federal aid in road construction for the fiscal year ending June 30, 1933, has been announced by the secretary of agriculture. The new funds are available to the states for immediate obligation on projected construction. The apportionment has been made 2½ months in advance of the usual date in order that the states may get an early start on plans for next season's construction.

In making the apportionment 2½ per cent was first deducted for administration. The remainder was then apportioned on the basis of area, population, and mileage of rural post roads.

From the apportionment of each state there was deducted one-fifth of the emergency funds apportioned last December. This deduction amounted to $16,000,000, leaving $105,875,000 as the net apportionment available for new projects.

Massachusetts Highway Mixup Goes to Supreme Court

The fight over award of highway contracts in Massachusetts has reached the state supreme court, where a petition was filed on Oct. 15 for a writ of mandamus to compel the governor's executive council to approve or disapprove a contract entered into by the state department of public works and Arute Brothers, Inc., of New Britain, Conn., to construct a section of state highway in the town of Randolph for $51,092. The issue is whether Massachusetts highway contracts can be awarded to bidders outside the state.

The supreme court petition was filed by counsel for Arute Brothers, and asks that the executive council be ordered to act upon the merits of the case without regard to extraneous considerations. The Arute bid was $300 less than that of the next bidder.

Hudson River Bridge to Be Dedicated Oct. 24

Dedication of the new Hudson River Bridge, at New York, will take place on Oct. 24, with appropriate ceremonies. The program as published includes speeches by Secretary of the Navy Adams and the governors of New York and New Jersey, speaking from a point on the span midway between Manhattan and Fort Lee. After the ceremonies on the bridge, which will begin about 3 o'clock and last an hour, the bridge will be thrown open to the general public. The bridge will be opened to traffic on Oct. 25, which is several months in advance of the original schedule.

Another Pipe Line to Be Laid From Kettleman Hills

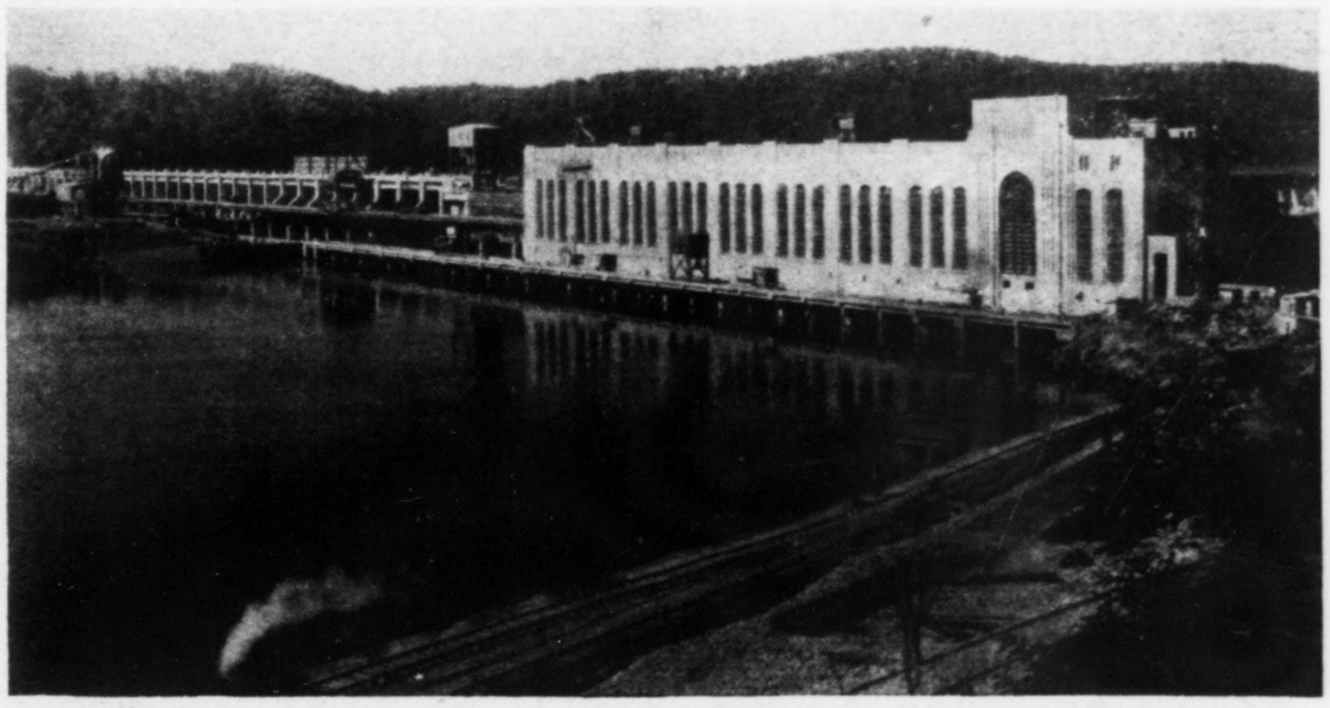
An additional pipe line 8 in. in diameter and 67 miles long will be constructed by the Standard Oil Company of California along the same route as the present pipe line from the Kettleman Hills oilfield to Estero Bay, near San Luis Obispo. The line will cost $500,000.

Employment Relief Plans Take More Definite Form

Plans for the relief of unemployed engineers are taking definite shape in Chicago and New York. In Chicago representatives of the Western Society of Engineers and of the local sections of the national civil, mechanical and electrical engineering societies met last week and set up the nucleus of a committee to develop a definite plan for the Chicago region. Similarly at New York the committee appointed by the metropolitan section of the American Society of Civil Engineers has been expanded to include all the metropolitan sections of the Founder societies. This larger group is now organizing its working committees and soon expects to send out a call for contributions.

The planning of both groups is centered around the proposals made to the board of direction of the Am.Soc.C.E. by its committee on salaries, headed by E. P. Goodrich. These proposals were outlined in our issue of Oct. 8, p. 586. More recently the board has sent to all local sections a plea for prompt action in developing a relief program of their own or in assisting in regional relief planning. The board also indorses the plan put out by American Engineering Council.

The Engineering Council plan is directed toward stimulating engineers to assist in general relief-work planning by seeking out work which may be undertaken immediately for relief this winter and in outlining other work that may be included in long-time planning for future emergencies.



SAFE HARBOR WATER POWER PROJECT NEARING COMPLETION

Announcement has just been made that the Pennsylvania Railroad has contracted with the Consolidated Gas, Electric Light & Power Co., of Baltimore, for power to be used on the line now being electrified between Philadelphia, Pa., and Washington, D. C. Much of this current will be supplied by the $30,000,000 hydroelectric development of the Safe Harbor Water Power Corporation on the Susquehanna River, now nearing completion. The initial capacity of this unit will be 250,000 hp., the ultimate capacity 510,000 hp.

Indiana Highway Department Spreads Work by Shortening Day

Hereafter eight hours instead of nine will measure the length of a day for all maintenance forces of the Indiana state highway department. As labor is paid on an hourly basis, the same amount of work will be spread among more men with no additional cost. A. H. Hinkle, maintenance superintendent, in his letter to district engineers and superintendents in 33 cities of the state stresses the importance of obtaining new employees from local relief agencies. The latter, he stated, are to be impressed with the fact that only ablebodied men are capable of doing highway work, since it is hard and subjects the men to severe climatic exposure at times. A few weeks ago the commission absorbed more than 400 miles of county roads into the state system, which has been the means of providing employment in many localities spread widely over the state.

Federal-aid collection by Indiana were far greater this year than ever