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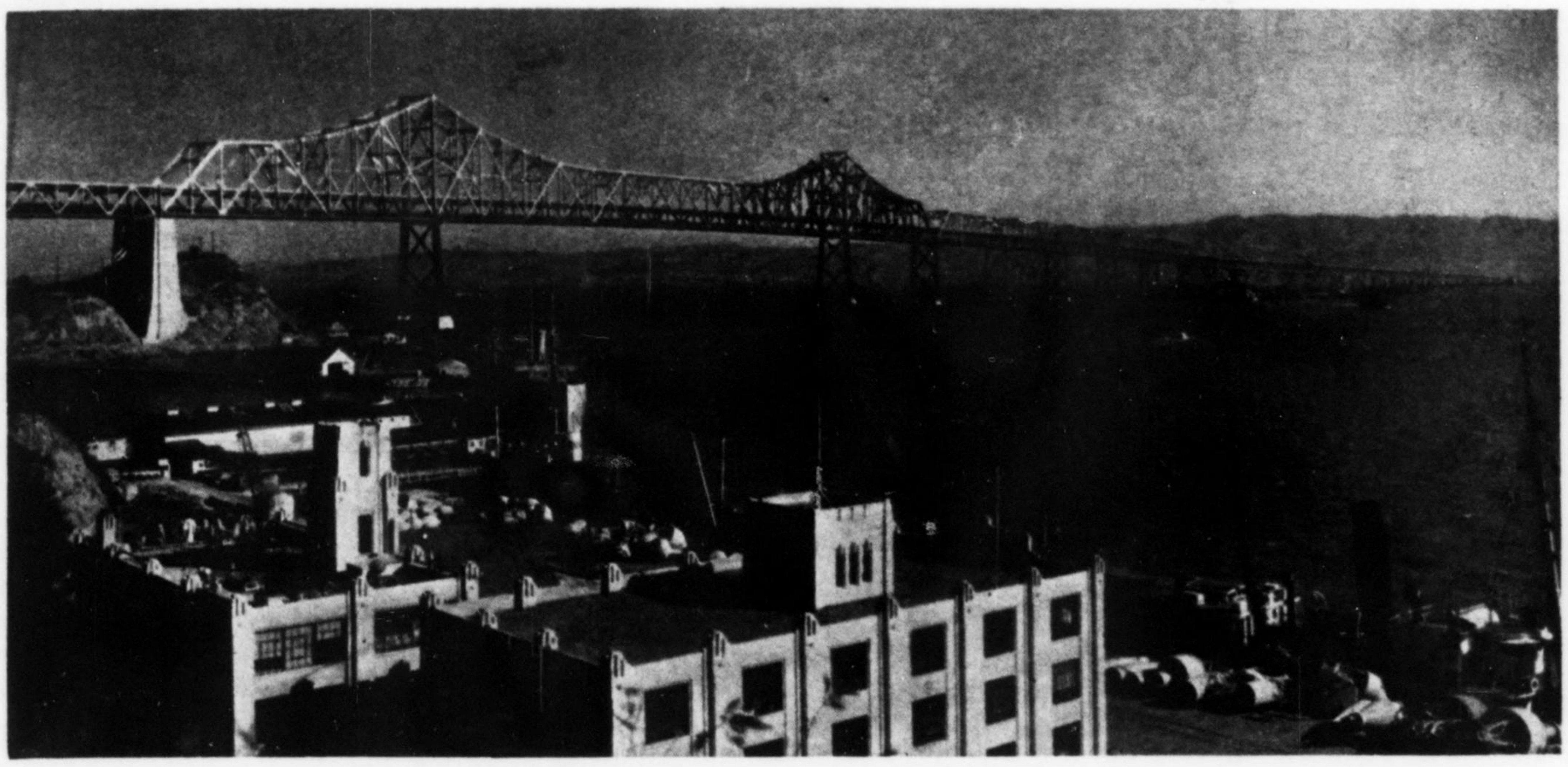

FIG. 1—THE 1,400-FT. CANTILEVER of the East Bay crossing. Beyond are the 504-ft. spans and the 288-ft. spans.

East Bay Crossing of the Bay Bridge

Steel structure includes longest and heaviest cantilever in the United States which is supported on tall towers and anchored at only one end—Erection by two-derrick travelers

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THE EAST BAY CROSSING of the San Francisco-Oakland Bay Bridge, spanning the waterway between Yerba Buena Island and Oakland is in every respect a different type of structure from that between San Francisco and the Island. Containing 60,500 tons of structural steel and having, as one of its parts, the longest and heaviest cantilever span in the United States, the East Bay Crossing would of itself rank among the major bridges of the world. Despite the fact that it is overshadowed by the more spectacular West Bay suspension spans, the cantilever presented the greater difficulties in design, fabrication and erection.

The span layout of the East Bay Crossing (for diagram see *EN-R*, **Mar.** 22, 1934, p. 375) consists of four 288-ft. spans crossing a valley on the island, the 1400-ft. cantilever span with its two anchor arms of 508 ft., five spans 504 ft. c. to c. of pins, a 50-ft. braced tower, 14 spans 288 ft. c. to c. of pins and a steel and concrete viaduct 1072 ft. long. Among the determinants of this layout were the following:

(1) The main navigation channel is at the edge of the Island. The East Bay pierhead line is only 2900 ft. from the Island. The 1400-ft. span with a vertical clearance of 185 ft. above mean higher high water and three additional spans of 500 ft. with a clearance of 165 ft. at the pierhead line were requirements of the War Department permit.

(2) East of the pierhead line the water is comparatively shallow and the problem for this part of the crossing was that of so balancing costs of superstructure and substructure as to secure the greatest economy.

(3) Foundation conditions (*EN-R*, Aug. 23, 1934, p. 227) were such that deep foundations were required; the one at the east end of the 1400-ft. span reached a record depth of 242 ft. These conditions were also such as to make it essential to keep the foundation loads to a minimum. For this reason steel towers rather than the usual masonry shafts were used to support the superstructure, except where the height of the masonry shaft would be less than 60 ft.

A masonry pier of the height required to support the cantilever and the other high level spans would have afforded little resistance to wind and earthquake forces in a longitudinal direction. The steel towers have even less longitudinal rigidity. To secure the required longitudinal stability the west end of the cantilever was anchored by a heavy masonry pier founded on the rocky tip of the Island. At the east end of the 504-ft. spans a 50-ft. braced tower was erected, on a pier 100 ft. wide, to furnish another point of longitudinal anchorage. The expansion for the distance of 5000 ft. between these two points is taken by a split column at the east end of the cantilever. This expansion involves a possible movement of 20 in. each way from normal. Similar provisions for anchorage and expansion were made at other points of the structure.

(4) Two horizontal curves were required in the structure, one on the Island truss spans to connect to the tangent of the West Bay Crossing, the other at the east of the 504 ft. spans

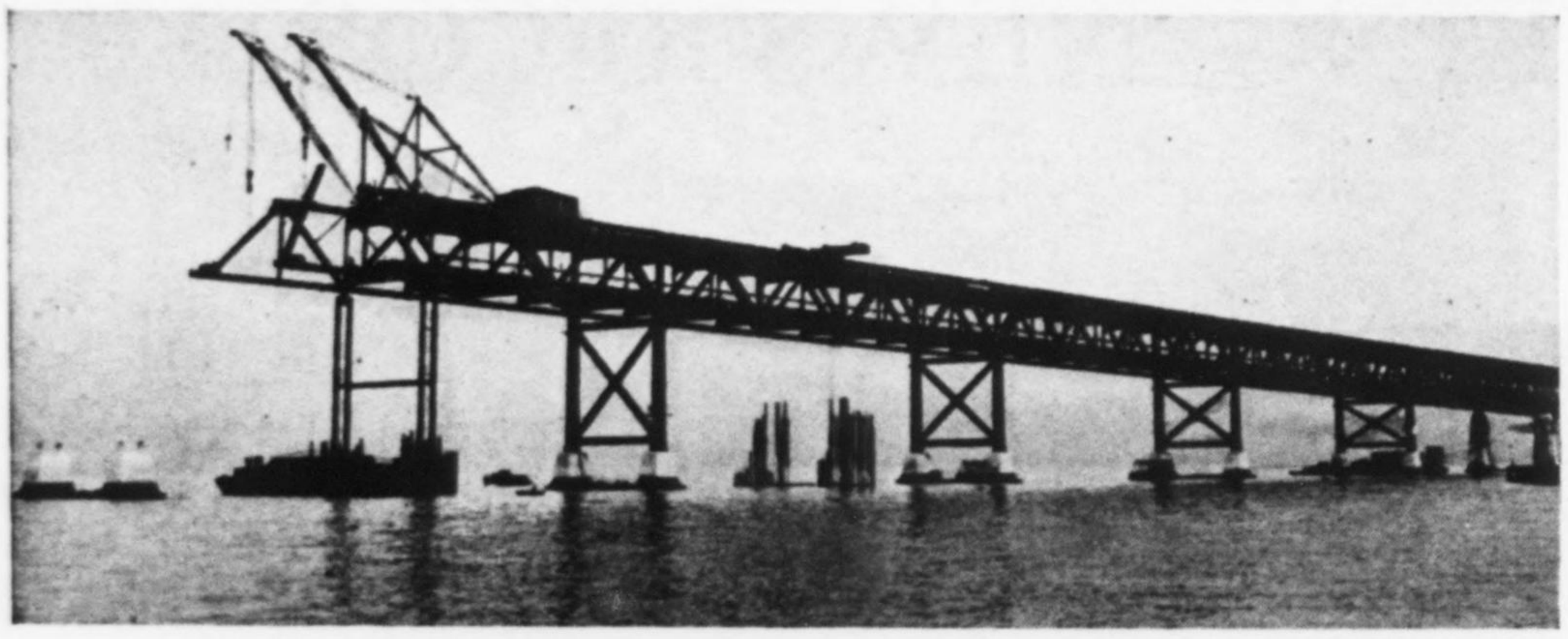


FIG. 2—TRAVELER erecting 288-FT. SPAN. Temporary bent on steel piles in foreground. Pile puller in span behind traveler is removing falsework piles.

which was introduced to avoid interference with the Key System ferry terminal.

(5) The usual procedure of selecting span lengths and then determining economical truss depths was to some extent reversed by the condition of the double-deck cross section. To provide satisfactory clearances over the upper-deck roadway and sufficient depth of portal and sway bracing, a truss depth of about 80 ft. was required. Where the top chord was braced by the upper-deck roadway, the truss depth became about 38 ft. For economical proportions these truss depths corresponded to the adopted span lengths of 504 ft. and 288 ft.

Design

With these span lengths and having adopted specifications for loads and unit stresses (*ENR*, Mar. 22, 1934, p. 371), the design of the simple truss spans presented no unusual problems. Silicon steel was used in all members where it resulted in a saving of weight, except that heat-treated eyebars were used for the bottom chords of the 504-ft. spans. The principal reason for the use of these eyebars was that the designers had more confidence in the pin joints than in the heavy riveted splices that otherwise would have been required. The higher unit stresses and the smaller amount of detail material resulted in a saving in weight, an advantage that was offset to some extent by the fact that the eyebar chords rendered erection by cantilever methods impracticable.

For architectural reasons, tied arch and self-anchored suspension designs were investigated for the 1400-ft. span. Even on this basis, the results were not as satisfactory as the adopted design. Both in weight of steel and in cost of erection, these other designs were much more expensive. Various profiles of the cantilever were investigated. A curved top chord in the suspended span of the cantilever (and in the 504-ft. spans) would have reduced the weight of steel slightly, but not sufficiently to balance the increased costs of fabrication and erection.

The anchoring of the cantilever structure in a longitudinal direction at only one end had several advantages. It permitted a continuity of the diagonal web system throughout the structure. It avoided the necessity for elaborate expansion details in both the floor and the lateral system at the ends of the suspended span, as well as traction and earthquake brakes at the same points. It reduced the number of points at which jacking devices were necessary during erection. The arrangement also avoided necessity for expansion rollers which always are a source of trouble.

There were accompanying disadvantages. The arrangement results in stress dissymmetry. Since all longitudinal forces are taken at the one anchorage point, the chord stresses from longitudinal earthquake and wind at the west end of the anchor arm are double what they would be in the case of two points of anchorage. This required some additional section in these chords. Difficulties in stress analysis of the lateral system were introduced, since this system became continuous over four supports, fixed at the west end and on slightly yielding supports at the steel towers.

There have been several cases of long-span bridges where the live load carrying capacity was reduced by reason of underestimating the weight of the steelwork and the dead load stresses caused thereby. In this case every precaution was taken to make ample provision for the dead load of the structure. Although layouts of panel points were made in the design department and the weights of steelwork calculated therefrom, the fabrication of the 504-ft. spans and of the cantilever was delayed until weights could be calculated from approved shop detail plans and the stresses rechecked using these weights. As a provision for future utilities not contemplated at present, an excess dead load allowance of 500 lb. per lineal ft. of bridge was included in the calculation of the dead load stresses.

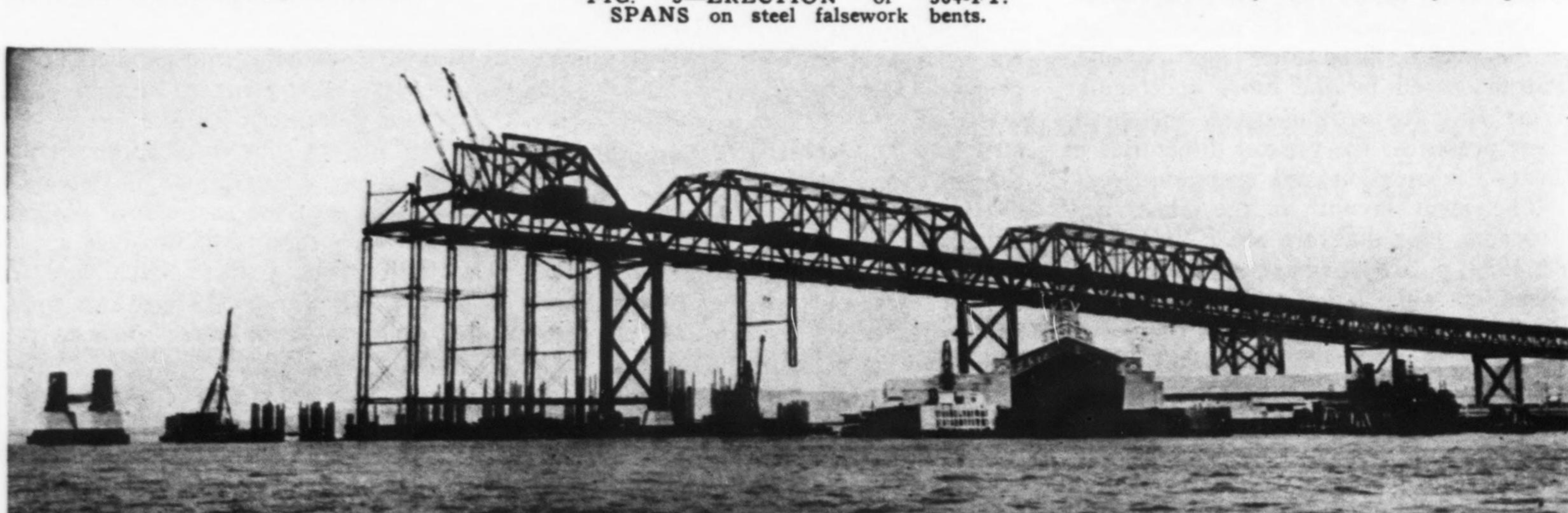
Earthquake stresses

The stresses in the superstructure were calculated for seismic forces equal to 10 per cent of gravity acting in any direction. The stresses set up by these horizontal forces, however, were not of sufficient magnitude to call for increase of section beyond that required by dead, live, temperature, and wind loads.

In a structure of this nature some deviations from usual methods of calculating earthquake stresses are necessary. Stresses in the lower chord of the cantilever due to an earthquake acting parallel to the bridge center line, vary from zero at the east end to a maximum at the west end. The stresses produce deformations in the chord so that the chord acts as a spring. By taking this action into consideration, the longitudinal earthquake stress in each lower chord at the west end is reduced from 3000 kips to 1600 kips.

While it is possible to derive theoretical rules for economic proportions of cantilevers, these rules are generally invalidated by the necessity of placing

FIG. 3—ERECTION of 504-FT. SPANS on steel falsework bents.



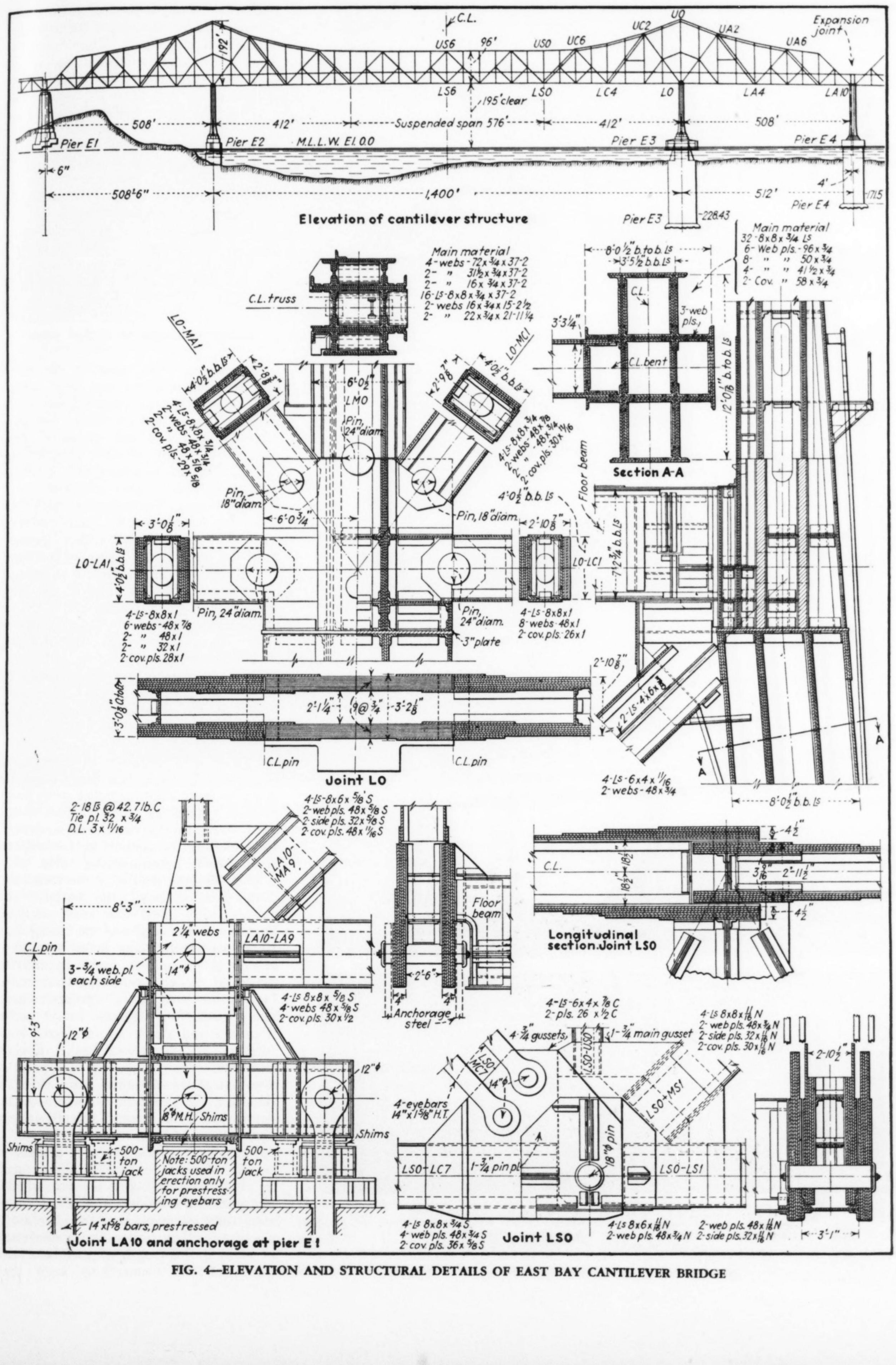


FIG. 4—ELEVATION AND STRUCTURAL DETAILS OF EAST BAY CANTILEVER BRIDGE

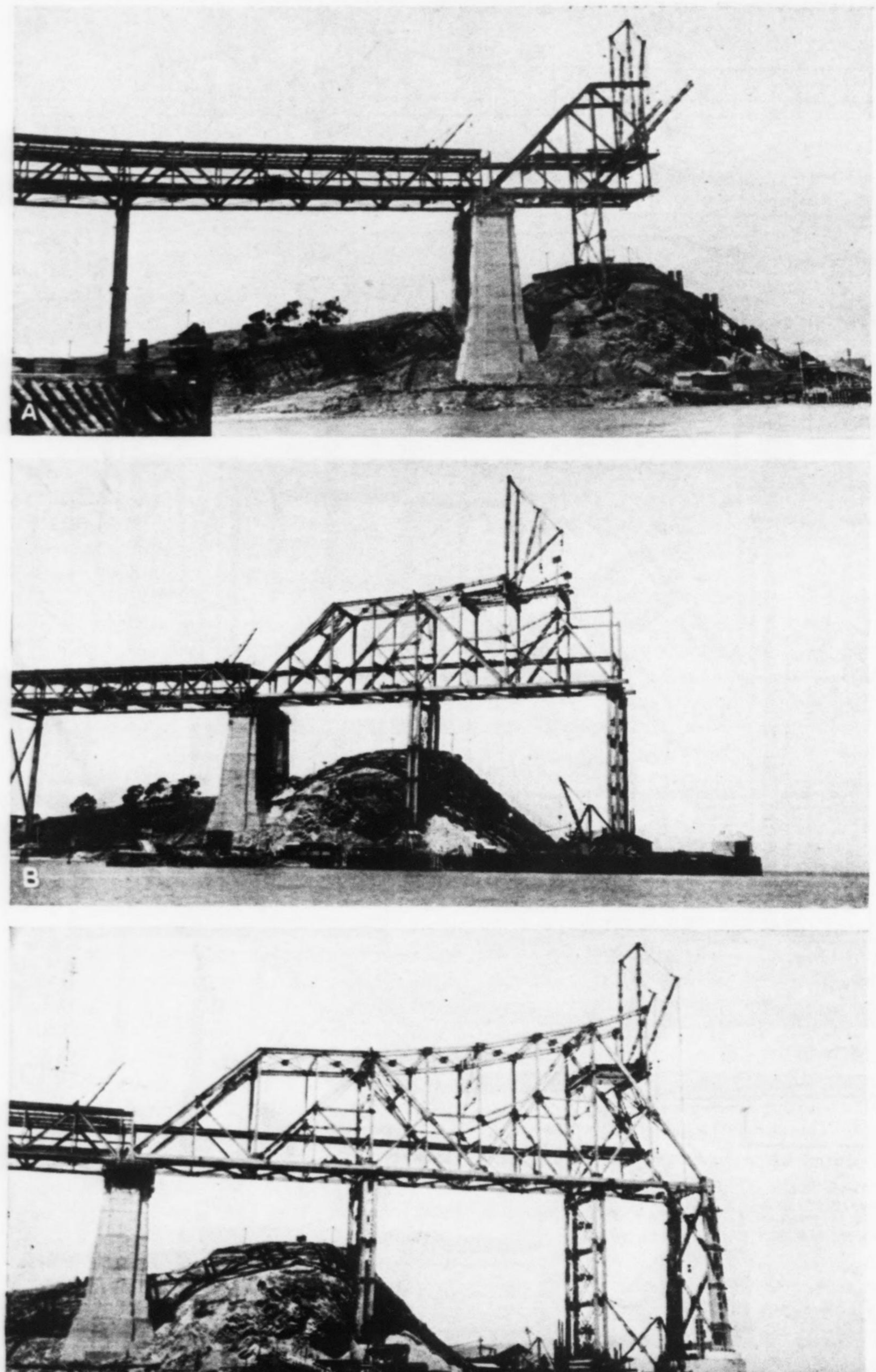


FIG. 5—SUCCESSIVE STAGES in erection of cantilever span: A—Start of west anchor arm. B—West anchor arm traveler at upper level. C—Permanent tower E-3 being erected by the traveler. D—Start of erection of suspended span.

additional metal to provide for erection stresses. These stresses in turn depend upon erection methods. The design was based on the assumption that the 1400-ft. span would be erected by the cantilever method using guy derricks, an assumption that fitted very closely the equipment later designed for the purpose. The proportions of the structure were made such that extra material to provide for erection stresses was required only in some of the web members, especially in the upper verticals which supported the traveler, in certain verticals and diagonals at the temporary erection bents and in the false top chords at the ends of the suspended span.

Large tonnage of nickel steel

Nickel steel was used in the main compression members and also, where rigid members were required, for erection, in the bottom chords of the suspended span. The use of some 3500 tons of this high strength material resulted in lighter members with a consequent reduction in dead load and earthquake stresses. Furthermore, with material of less strength, each member would have required either greater depth or greater thickness of material to secure the required area. The greater depth would have increased the secondary stresses; greater web thickness would have increased the grip of rivets beyond the maximum of $7\frac{1}{8}$ in. for $1\frac{1}{4}$-in. diameter rivets.

Heat-treated eyebars were used for main tension members, and silicon steel was used for secondary truss members and for most of the floor and lateral system. Carbon steel was used for details and minor members. Manganese rivets were used for the heavier joints, and pins were chrome-nickel steel, heat-treated. Quantities of the various structural steels in the cantilever structure, not including the supporting towers, are given in the accompanying table.

In developing details of the members, closed box sections with manhole entrances were used wherever possible. This saved the weight of the lacing that otherwise would have been necessary. Lacing is difficult to maintain and its structural action somewhat uncertain. Pins were used at all points where heavy secondary stresses would occur otherwise and also to avoid riveted connections of excessive length. Eyebar members were packed in two parallel rows one above the other in the plane of the truss. This reduced the width of the member, and the increased depth gave a more substantial appearance.

Secondary stresses were calculated for all members of the cantilever and the 504-ft. spans. On account of the long panels and moderate depth of members, these stresses are within reasonable limits. The trusses were cambered by making the length of members and angles of the connections such that

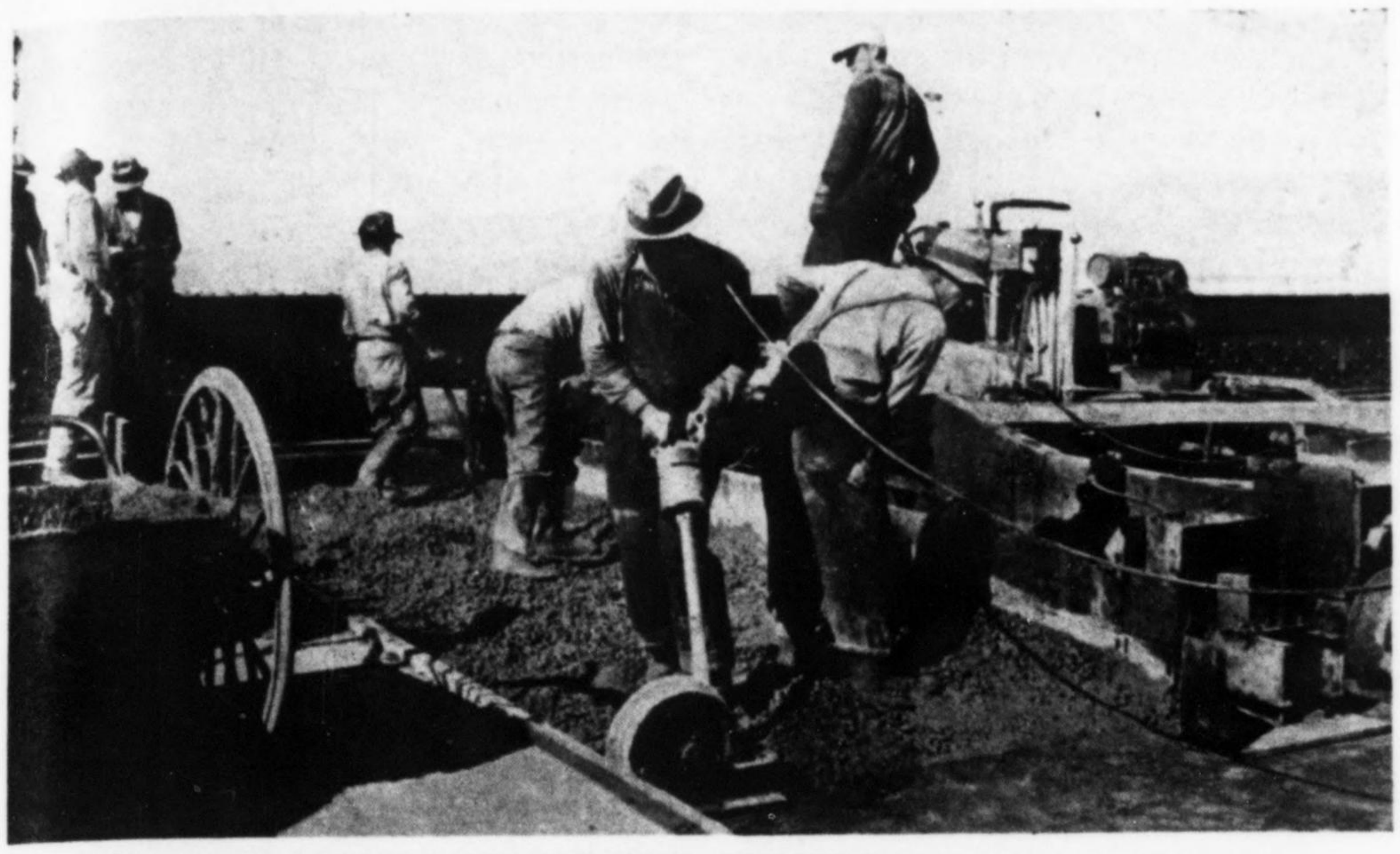


FIG. 6—PLACING LIGHT-WEIGHT CONCRETE on upper deck roadway. Hand vibrators were used in addition to mechanical vibrators mounted on the finishing machine in the background.

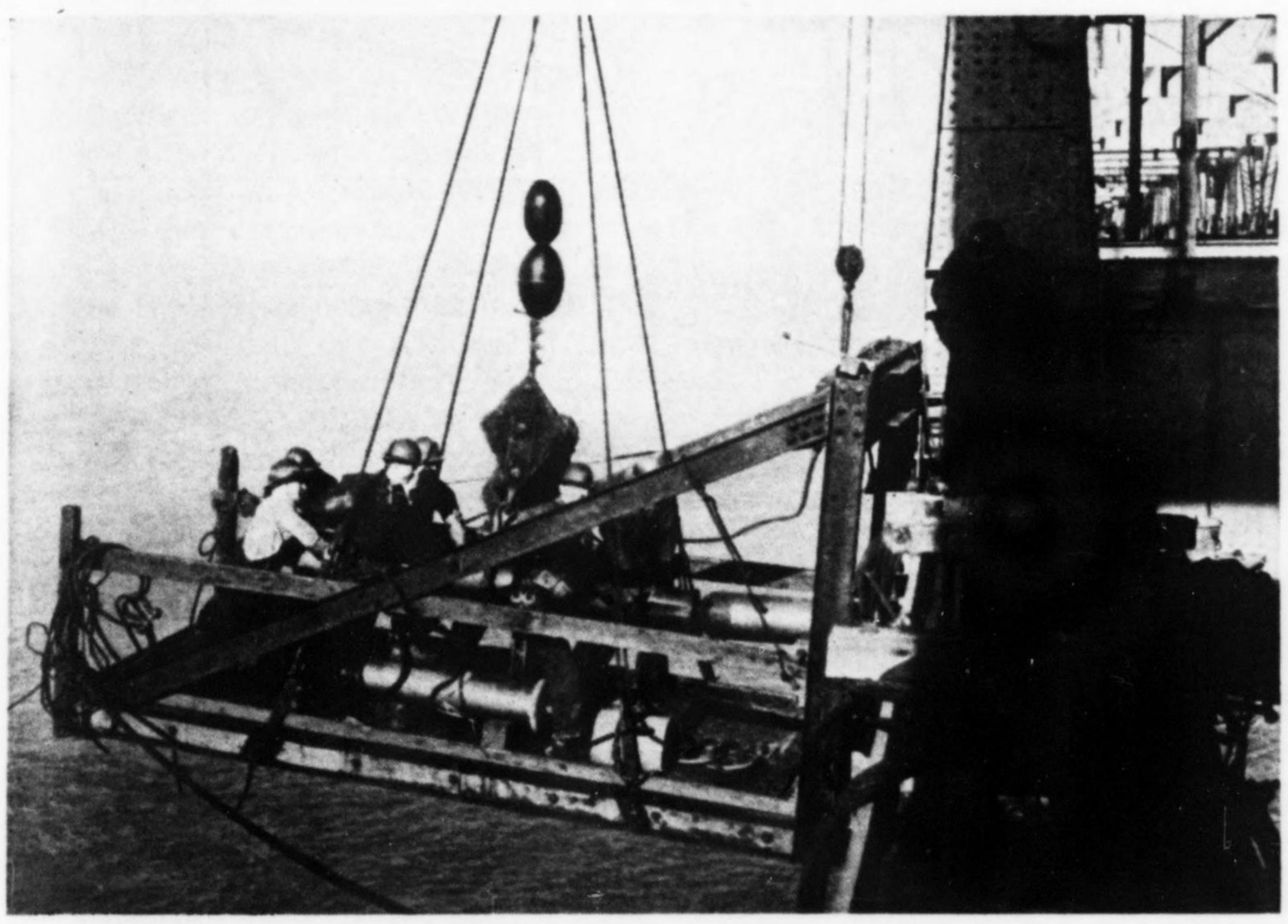


FIG. 7—DRIVING the closing pin in the suspended span of the East Bay Cantilever.

secondary stresses are theoretically zero under full dead and half live load. The end connections of the floorbeams were set at such a bevel as to relieve bending stresses in the verticals.

Before erection, strain gage points were established in a large number of members. Strain gage readings were taken during erection and in the completed structure. This data had not yet been reduced when this was written.

Fabrication

At all stages the shopwork was of a character to avoid damage to the material and to insure a good fit in the field. All rivet holes were subpunched or drilled, then reamed, and all sheared edges were planed. All compression chord joints were milled, and all rivet holes in chord joints were reamed while assembled and the joints for diagonals were reamed to steel templates. As a result of these precautions, there were practically no shop errors that required correction in the field.

Truss span erection

The 288-ft. spans were erected by a semi-cantilever method in which temporary upper chord ties and lower chord struts were placed over the piers. The falsework consisted of steel bents supported on steel H-piles. Since the eyebar bottom chord of the 504-ft. spans did not permit cantilevering, a steel falsework bent was placed under every main panel point. A horizontal wind truss at El. +25 was placed in this falsework system.

These spans were erected by a 194-ton traveler supported on twelve two-wheel trucks which ran on four 130-lb. rails placed at upper deck level. The frame of the traveler was 102 ft. long with a sill width of 60 ft. and a rear width of 44½ ft. The traveler carried two derricks, each with booms 87 ft. in length for the 288-ft. spans, subsequently lengthened to 100 ft. for the 504-ft. spans. These booms had 12-ft. trussed gaffs which carried the runner lines. The 44-ft. masts were spaced 42 ft. apart. The front stifflegs were in a plane at right angles to the bridge centerline and the rear stifflegs met at a common point at the rear of the derrick. This allowed the booms to swing to the rear while erecting the top bracing system of the 504-ft. spans. The derrick had a capacity of 20 tons at 100-ft. radius. The hoisting engines, which were mounted on the traveler, were triple-drum, three-speed, with a swinger attached and were operated by gasoline engines.

The erection of the Yerba Buena truss spans proceeded similarly to that of the east 288-ft. spans. The principal difficulty was that the rugged terrain made it difficult to get material to the traveler. The traveler used in erecting these spans had two 90-ft. booms at the front and one 50-ft. boom at the rear. Each of the front booms was rigged for a set of falls and a runner line. The rear boom was rigged for falls only. The derricks were operated by two 60-hp. tractor-hoist combinations located on the ground at the west end of these spans.

Guy derricks erect cantilever

Upon completion of the Yerba Buena spans this traveler erected a second traveler to be used for erection of the cantilever. This second traveler had two 50-ton guy derricks mounted on a mast beam. The masts were 118 ft. high and were spaced 44 ft. on centers; the booms were 100 ft. long and were rigged for main and auxiliary falls and a runner line. Power for operating the traveler was supplied by two 175-hp. gasoline engines and two auxiliary engines, all of which were placed on the bridge deck and did not move with the traveler. The traveler was supported and moved on skids made of the beams designed to be used later as railway stringers in the suspended span. In lower position, these skids were supported on the upper deck floorbeams which were in turn shored from the lower deck floorbeams. In its elevated position the skids were supported by floorbeams temporarily attached to the truss verticals.

The sequence of erection, which was the same for both halves of the cantilever is shown in the accompanying drawing. The traveler used in the

STEELS USED IN THE CANTILEVER STRUCTURE

	Tons	Per Cent	Cost Per Lb. Erected (cents)
Nickel	3,576	7	9.15
H. T. Eyebars and Pins	4,816	9	7.30
Silicon	25,312	48	6.75
Carbon	19,102	36	6.35
Manganese Rivets.	84	..	6.40

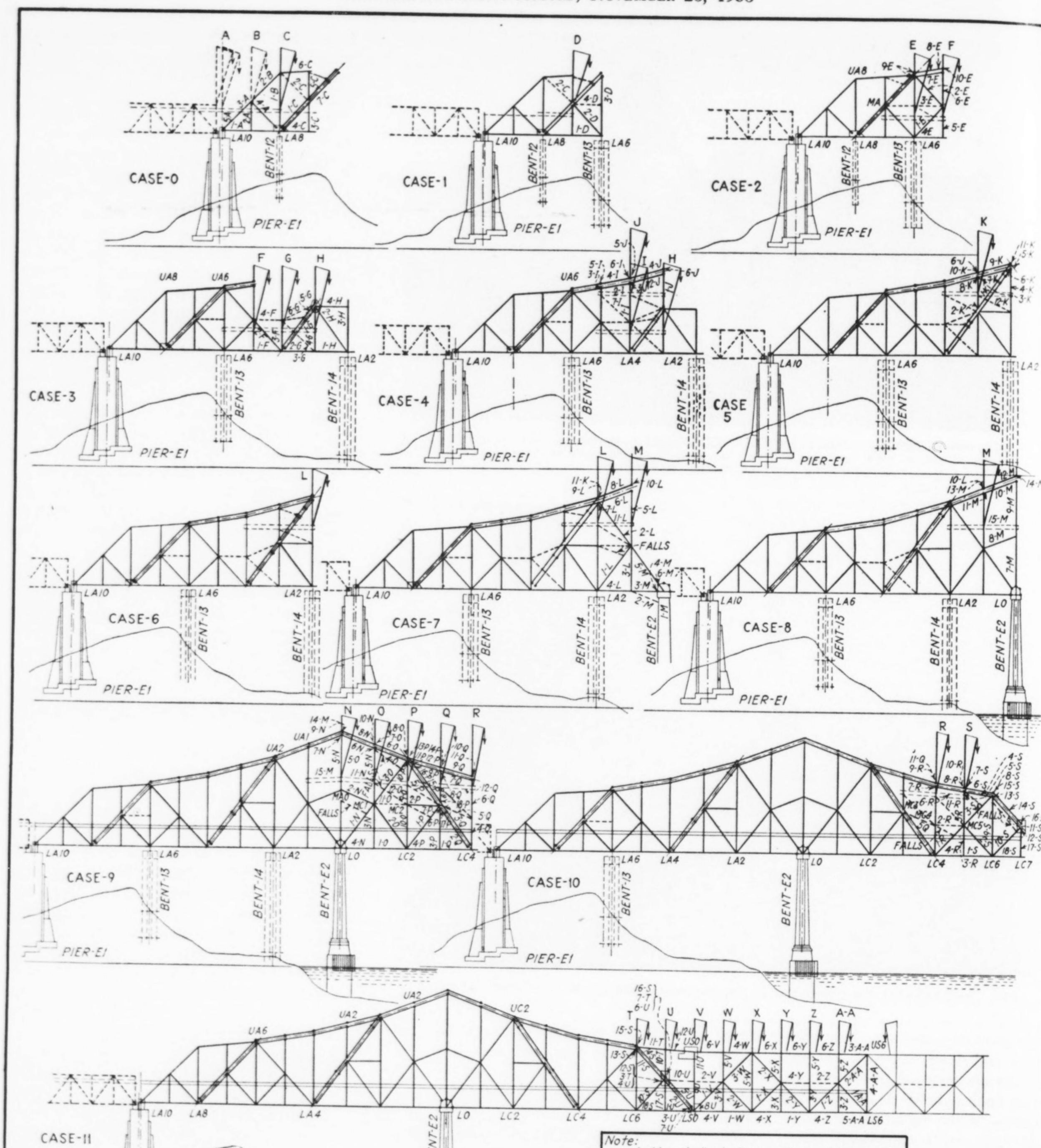


FIG. 8—SEQUENCE OF ERECTION OPERATIONS ON EAST BAY CANTILEVER BRIDGE

erection of the Yerba Buena spans had sufficient capacity to erect the members of the suspended span. Therefore when the guy derrick traveler reached the east end of the west cantilver arm, it was dismantled and re-erected at pier *E*-4. From this point this traveler worked through to the center of the suspended span; the west half of the suspended span was erected by the Yerba Buena traveler. This procedure saved an additional heavy traveler and inasmuch as the bridge construction schedule was governed by the West Bay Crossing, the additional time was of no consequence.

The movements of the truss necessary to effect the closure of the span were controlled by three sets of two 500-ton jacks per truss located at panel points *USO* east and west and at *LA* 10 east.

Paving

At an early stage of the design, it was decided to use a comparatively light-weight floor for the upper deck roadways, and an arbitrary weight limit of 60 lb. per square foot was established. After an investigation of the various types that would come within this limit, the designers concluded that a light-weight concrete floor would be more economical in first cost and in maintenance than any of the other possibilities.

The aggregate consists of a shale which is crushed, pugged, cut into pellets and then burned in a rotary kiln. Natural sand is used for the finer particles of the aggregate. When mixed with six sacks of cement, the resulting concrete has a wet weight of 100 lb. per cu.ft. and a 28-day strength of 3,000 lb.

The roadway slab on the upper deck is 6 in. thick and is reinforced in both directions. Longitudinal reinforcing consists of ½-in. rods on 6-in. centers. Reinforcing transverse to the bridge centerline consists of welded trusses spaced 6 to 9 in. apart. Top and bottom chords of the trusses are made of two ½-in. round bars; the truss bracing system is a 7/16-in. round bar bent to make 45-deg. angles between the two chords to which it is welded at each intersection. These welded trusses have an over-all depth of 4½ in. and a length equal to the roadway width. To hold them firmly in place during pouring operations they were welded to the supporting stringers, which are spaced on 6-ft. centers.

The upper roadway slab was placed in two operations. In the first operation the light-weight concrete was placed in the forms, compacted by a vibrator-equipped finishing machine and screeded ¼in. low by the same machine. Before the concrete had taken its initial set it was covered by a mortar topping and again vibrated, screeded and belted by the finishing machine. The results with this pavement have been extremely satisfactory in every particular.

The lower deck pavement is designed for heavy trucks. The designers considered the light-weight floor somewhat experimental and therefore the lower deck pavement was designed with a 6½-in. slab of standard concrete. The reinforcing steel and methods of placing were as above described for the upper deck except that the concrete was placed in one layer.

Personnel

The contract for the superstructure of the East Bay Crossing was held by the Columbia Steel Co. of which Ambrose Diehl is president. The late E. J. Schneider was succeeded by J. R. Fox as contracting manager. The steel west of pier *E*-4 was fabricated and erected by the American Bridge Co. of which C. S. Garner is general manager of erection and H. C. Hunter is western erecting manager. Steel for the spans east of pier *E*-4 was fabricated and erected by the bridge and fabrication department of Bethlehem Steel Co. for which company D. S. Gendel is general manager of erection, A. F. McLane, general manager of erection for the Bay Bridge, and H. A. Schirmer, resident engineer. The concrete paving was included in the general contract and was sublet to Bates & Rogers with R. Rasmussen as superintendent of construction. The personnel of the California Toll Bridge Authority was given in ENR March 22, 1934, p. 377.

Full Circle Tunnel Lining Placed in a Single Pour

Form traveler 160 ft. long aids completion of 70-ft. lengths of concrete lining as single operation in 10-ft. tunnel on Colorado River aqueduct distribution system

THE DISTRIBUTION SYSTEM of the Colorado River aqueduct includes a tunnel of 10-ft. inside diameter passing under the residential district of Pasadena. Requirements to be met in placing concrete lining in this tunnel made it desirable to use equipment and methods not common in tunneling practice. Primarily, these are: forms made up in completely circular panels with which a 70-ft. length of lining can be placed in a single pour; methods of bending the reinforcing steel to the required radius in the tunnel and close to the scene of operations and, finally, a working schedule controlled by the necessity of avoiding noise during hours of the night when the residential district was not to be disturbed. Incidentally, these conditions automatically limited the daily advance which the contractor could make and since there was no opportunity for profits resultant from speed, it was incumbent upon him to work out a construction schedule such that any bonus to be earned would accrue from a low cost of carrying on the work.

This tunnel originally was to have been driven as an open cut (the overburden is 40 to 80 ft.) but later it was decided that there would be less interference with street traffic and less disturbance of the residential district if the workings were kept entirely underground. This program has worked out well in several ways and record progress was claimed in the driving operations (*ENR*, Oct. 10, 1935, p. 513).

To the cast of an access portal in Arroyo Seco, which lies toward the westerly side of the city of Pasadena, this part of the aqueduct distribution system is in almost continuous tunnel for a length of about 5 miles. The amount of concrete required for the lining averages about 2¼ cu.yd. per foot. Reinforcing steel, consisting principally of 1¼-in. square bars, amounts to about 650 lb. per lin.ft. For purposes of contract award, the tunnel lining was divided into two sections and lining equipment of similar type has been used on both contracts. In some respects this equipment is similar to that used in the Lake Cushman hydro-electric power tunnel built by the city of Tacoma some six years ago but in many particulars it represents an advance developed recently for work of this type.

Essentially the form set-up consists of a series of 5-ft. panels completely circular when jacked into place, but hinged for convenience in collapse and movement. A 70-ft. length of these forms is assembled on a traveler in which there is a truss 160 ft. long carrying an elevated track for moving the forms. The truss is supported at each end on four-wheel trucks which move on tracks in the tunnel invert. This great length of the supporting truss makes it possible to place the forms for a completely circular pour in a 70-ft. length of the tunnel and later to move the same forms ahead to a new set-up without changing the traveler position. The traveler is moved ahead of the forms after each pour. The sequence of operation is based on the