VII — Papers Presented by the American Exchange Delegation in the USSR

The seven members of the United States delegation presented 11 papers at three seminars during the visit to the Soviet Union, May 31-June 21, 1959. Some of these were essentially reviews of work previously presented or published but five of the papers had not been presented in America. They are:

"Floating Caisson Foundations. Steel Piles with Attachments," by John Lowe, III.

"Current Practice in Soil Sampling in the United States," by John Lowe, III.

"Certain Features of Lateral Pressures of Soils," by Gregory B. Tschebotarioff.

"Critical Elements of Design and Construction of Heavy-Duty Flexible Pavements," by W. J. Turnbull.

"A Summary of Rotary Cone Penetrometer Investigations," by W. J. Turnbull, et al.

Brief resumés are included of the subjects presented by Prof. Kersten, "Studies of Frost Problems in a Northern State"; Prof. Lambe, "Soil Stabilization" and "Soil Structure"; Prof. Leonards, "Analysis of Design of Concrete Slabs on Ground"; and Prof. Seed, "Recent Researches in the USA on Soil Strength and Deformation Characteristics Under Dynamic Loading Conditions"; and "Foundations for Large Bridges Across the San Francisco Bay."

It was understood that all of these papers were to be published in the USSR in Russian language translations.

Floating Caisson Foundations

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In recent years two important structures in the New York City area, Pier 57 and the Nyack-Tarrytown Bridge, have been founded upon a new floating caisson type foundation. About 85 per cent of the load of the pier and 65 per cent of the load of the bridge are carried by buoyancy.

The development of this type of foundation and the design of the structures were carried out by Capt. E. H. Praeger, U. S. Navy Retired, who has kindly made available photographs and other material to the author for the preparation of this paper. The designs of the foundations of the two structures are described below starting with the bridge.

NYACK-TARRYTOWN BRIDGE

The Nyack-Tarrytown bridge across the Hudson River is an important link in the New York State Thruway system which extends 400 miles from New York City to Buffalo, and is presently being extended an additional 100 miles westward from Buffalo along the shores of Lake Erie. The minimum roadway design consists of two north-bound lanes and two south-bound lanes with a wide median strip. In the vicinity of New York City and other areas of heavy traffic, three or more lanes are provided in either direction. Not a single grade intersection or toll booth interrupts the flow of high speed traffic from the bridge for the full length of the Thruway.

The crossing of the Hudson River was chosen at Nyack-Tarrytown on the basis of traffic patterns and topography on both sides of the river as well as a territorial franchise held by The Port of New York Authority, operators of the George Washington Bridge across the Hudson River at New York City, which prohibits the construction of a toll bridge south of the

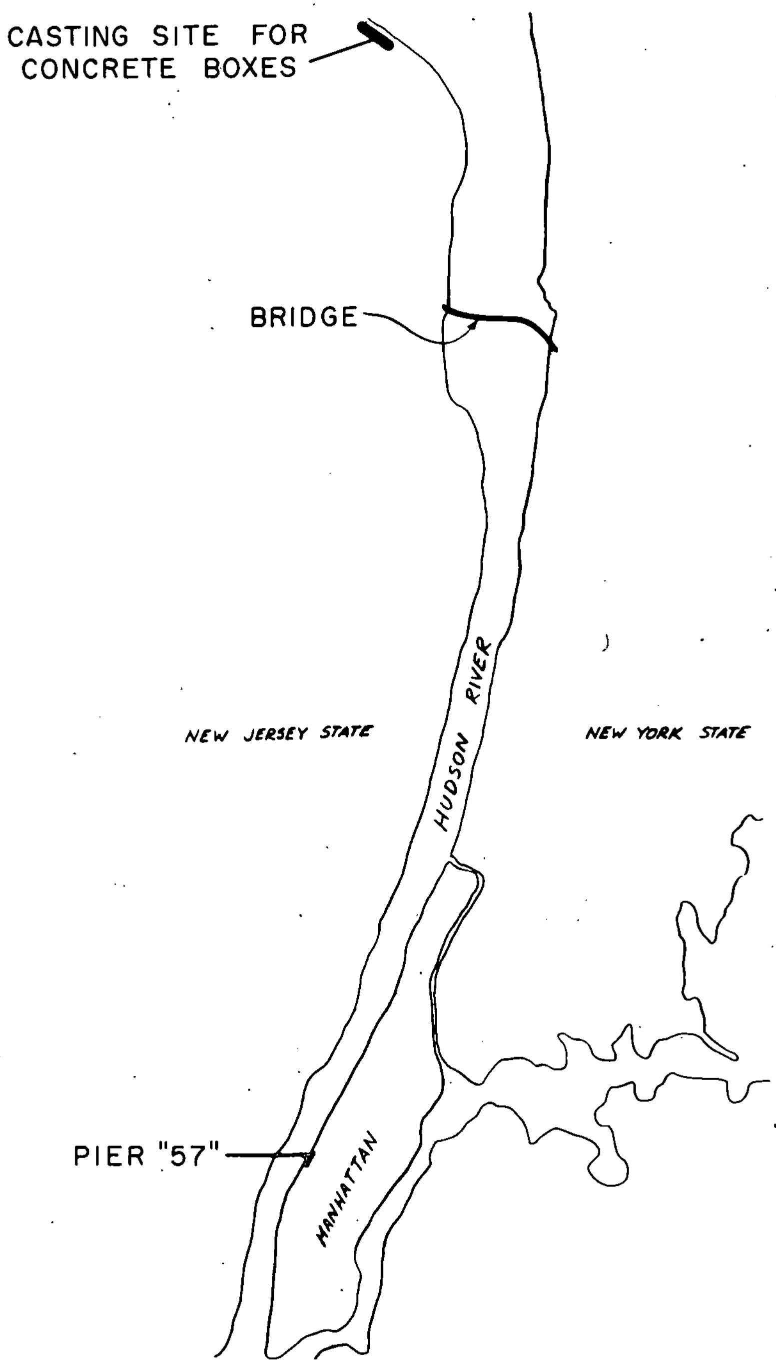


FIGURE 1
Location plan of Tappan Zee Bridge.

Nyack-Tarrytown location. The location of the Nyack-Tarrytown bridge is shown *(Fig. 1)*. The location of Pier 57 is also shown on this figure.

The bridge consists of three types of superstructure construction as may be seen in the aerial view of the bridge *(Fig. 2)*. The total width of the Hudson River at the site is three miles. Across the main river channel is a 1,212-ft cantilever span with two 602-ft anchor spans; on either side of the main spans are a total of 19 deck truss approach spans each of which is approximately 250 ft in length; from Nyack on the west shore, toward the center of the river is a trestle type structure approximately 8,000 feet in length and having 50-ft spans. A closer view of the main spans is given *(Fig. 3)*. The 1,212-ft main span was dictated by clearance requirements. The 602-ft anchor spans are structural adjuncts of the main span; the trestle spans were dictated by foundation conditions.

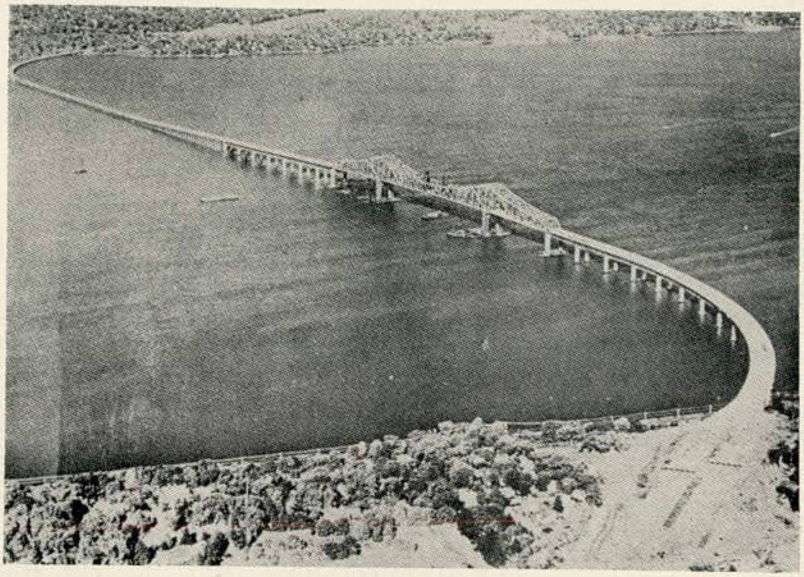


FIGURE 2
Tappan Zee Bridge, aerial view.

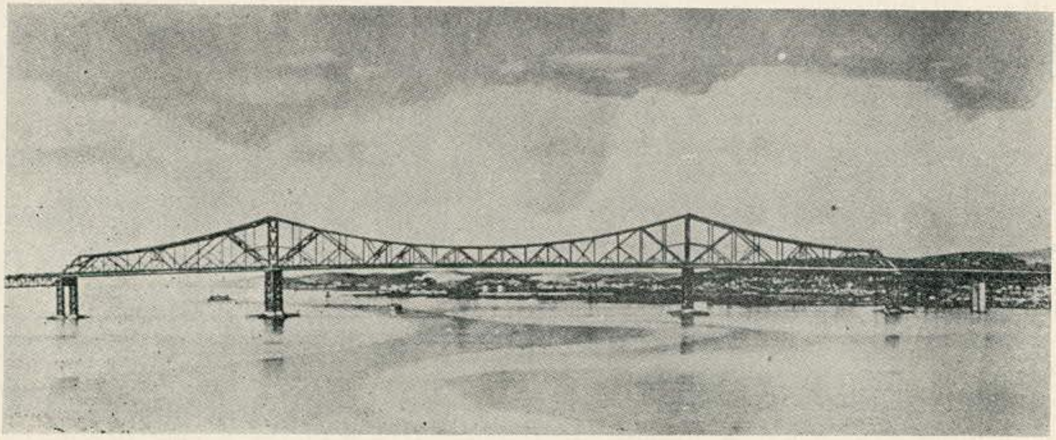


FIGURE 3
Main spans of Tappan Zee Bridge.

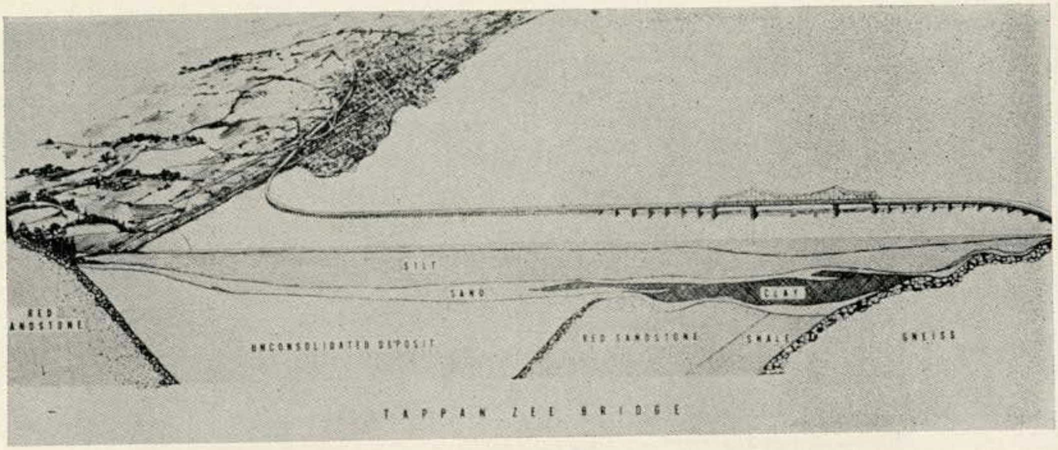


FIGURE 4
Geologic profile.

A geologic profile at the site is shown *(Fig. 4)*. Borings were made by barge mounted drill rigs. Testing of undisturbed samples was carried out in the soils laboratory of Columbia University. Gneiss bedrock was found to outcrop at Tarrytown on the east shore and dip gradually to the west. Overlying this basement rock, sandstone and a shale hardpan were encountered. These strata also dip to the west and at the center of the river occur at depths exceeding 300 ft. Water depths are 10 to 12 ft over the westerly half of the river and soft to medium stiff silt extends for depths of several hundred feet below river bottom. The strength of the upper silt is too low to permit construction of an approach embankment. The solution adopted utilizes concrete bents spaced 50 ft on centers and supported on 80-90 ft long untreated peeled Douglas Fir timber piles. Design loads are 8.5 tons dead plus 4 tons live.

Fifteen of the 19 approach spans are supported on concrete piers each of which consists of a two-legged bent founded on steel H-piles driven to rock.

The four towers supporting the main span and the anchor spans as well as four piers supporting approach spans on the westerly side are founded upon the floating caisson type foundation. Here the river bottom is 40 ft deep and sandstone bedrock occurs at a depth of as much as 260 ft below river bottom.

The floating caisson type foundations consist of pre-test hollow concrete boxes. The boxes were designed to carry 65 per cent of the dead load of the structure by buoyancy. The remaining dead load and all live load is carried by steel piles driven to rock. Fourteen-in. steel H-bearing piles weighing 89 lbs per ft were used in the case of the approach span piers and 30-in. diameter steel pipe piles, with a wall thickness of ½ in. were driven for the main span and anchor span piers. The pipe piles were driven open ended, cleaned out, and filled with concrete. The piles are connected to the caisson boxes by driving them through holes cast in the walls of the boxes and concreting the holes. The foundation is illustrated *(Fig. 5)*.

The eight boxes, one for each of the above mentioned piers, were prefabricated in a construction basin at West Haverstraw, approximately 12 miles north of the bridge site. An aerial view of the basin filled with water is shown *(Fig. 6)*. Equipment used for dewatering the basin is shown *(Fig. 7)*. Later views of the basin in the dry condition and of the eight boxes under construction are shown *(Figs. 8, 9)*. All boxes are 40 ft deep; the two boxes supporting the main span are 100 ft wide by 190 ft long; those supporting the anchor spans are 77 ft wide by 124 ft long; and those supporting the approach spans are 56 ft wide by 110 ft long. All boxes have interior longitudinal and transverse walls which divide them into

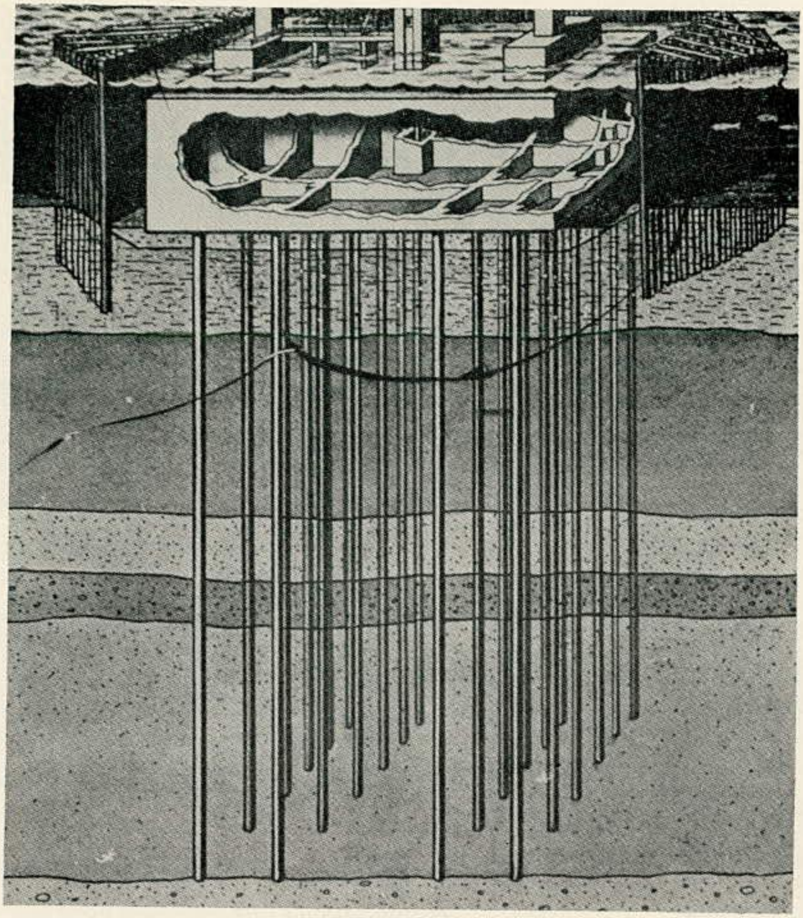


FIGURE 5
Floating caisson foundation.

compartments, and an intermediate floor under the top deck. The largest boxes weigh 23,000 tons each and have a draft of 30 ft. The top sections of the walls and the tops of the boxes were cast at the bridge site.

A view of the reinforcement for the boxes and the corrugated metal pipes forming the holes for the piles is shown *(Fig. 10)*. Reinforcing steel up to 2-in. square bars were spliced by the Thermit Weld process. A view of the splicing is shown *(Fig. 11)*. Another view of the details of the boxes is shown *(Fig. 12)* and a closeup of the completed boxes *(Fig. 13)*.

Concrete was made with carefully selected and proportioned aggregate consisting of both crushed basalt and quartz sand and gravel. Six bags of cement were used per cu yd. The concrete was placed at low slump, vibrated, cured first with water, and then cured with a coating consisting of a mixture of asphalt and aluminum powder. The aluminum was incorporated into the compound to reflect the heat of the sun. Strengths up to 4,000 lbs per sq in. at seven days and up to 7,000 lbs per sq in. at 28 days were obtained. Both vertical and horizontal pours terminated in cold joints and these as well as shrinkage cracks were sealed with a Neoprene process developed for this job. The testing of a joint for water tightness is shown *(Fig. 14)*. A special chamber was clamped to the wall, and pressure built up against the wall.

After completion of the casting of the eight boxes, the basin was flooded, the dike along the Hudson River breached and the boxes towed one by one to the site. Shown

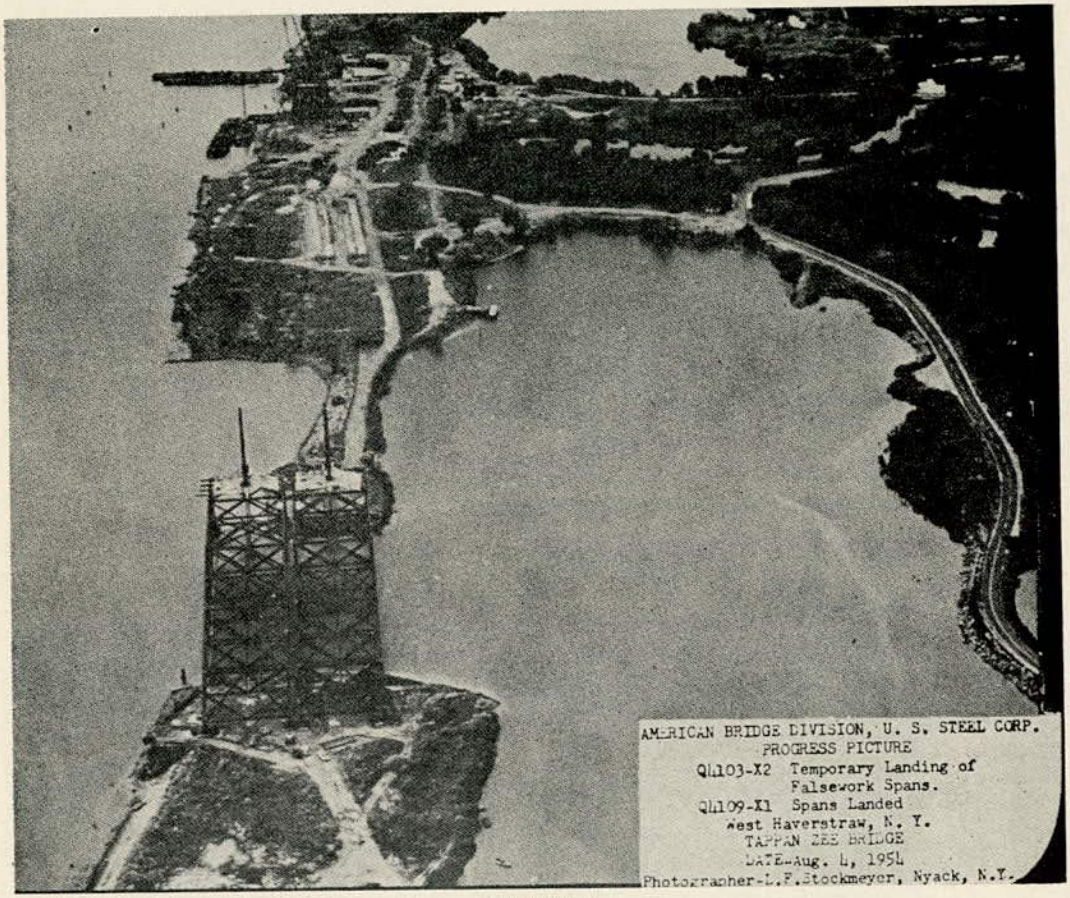


FIGURE 6
Construction basin for prefabricated foundation caissons.

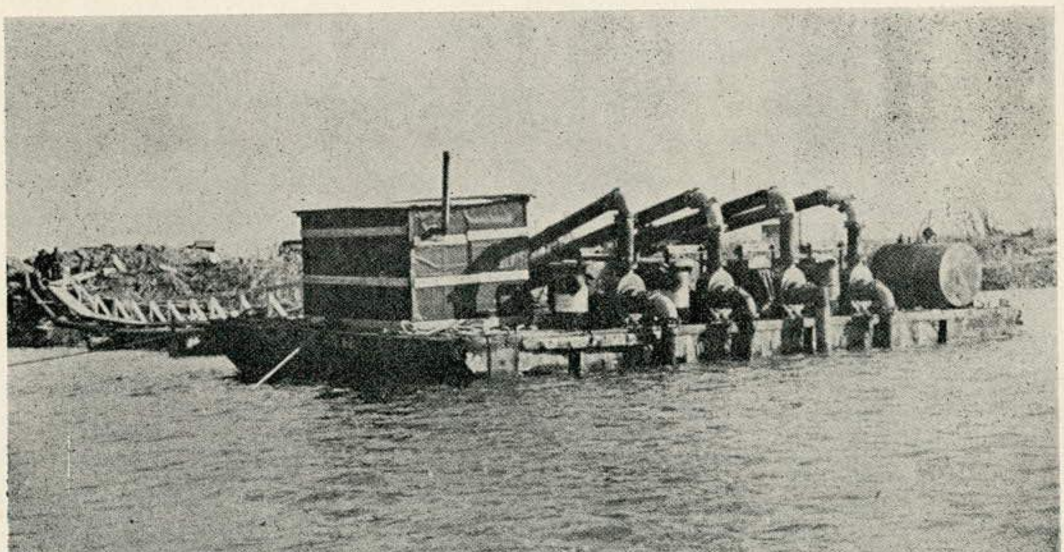


FIGURE 7
Dewatering equipment for construction basin.

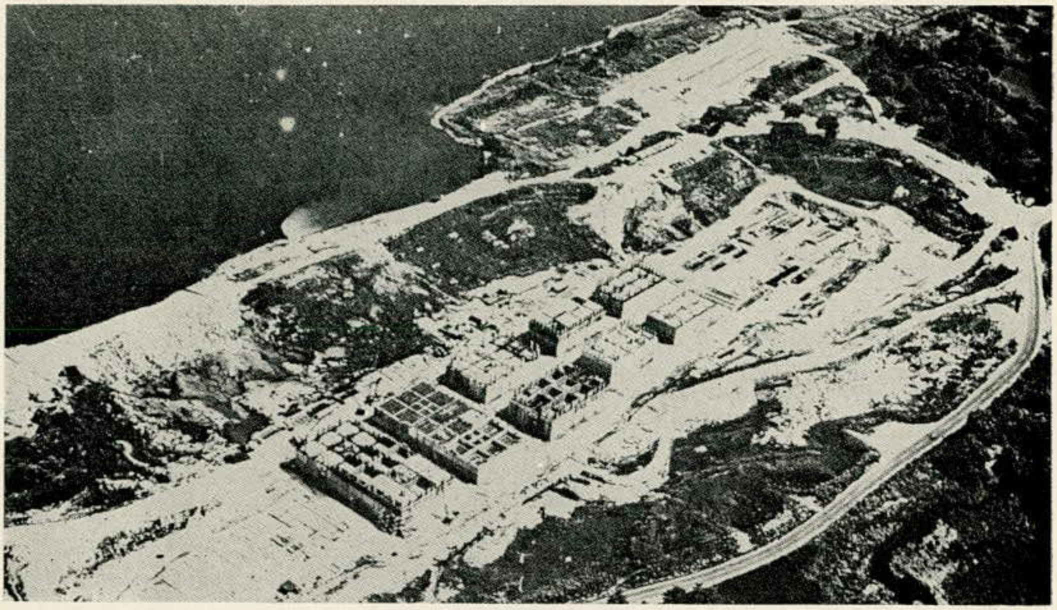


FIGURE 8
Foundation caissons under construction in the dry.

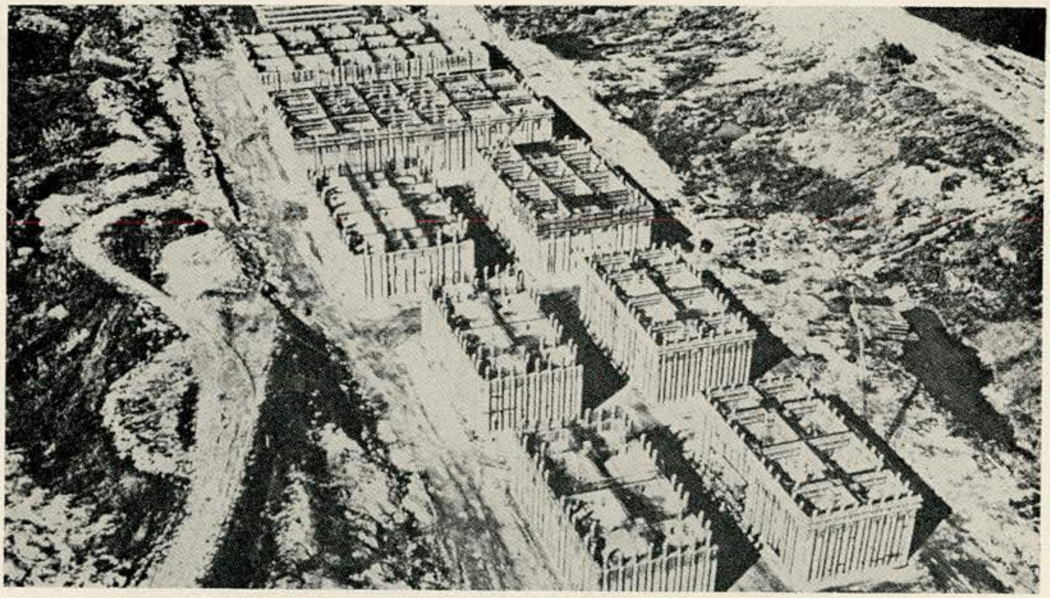


FIGURE 9
Foundation caissons under construction in the dry.

(Fig. 15) is the first box being towed out of the basin. Shown *(Fig. 16)* is the box en route to the bridge site and illustrated *(Fig. 17)* is the box held in position by clusters of timber piles.

When the walls and tops of the boxes were completed at the site the boxes were sunk by partially filling them with water so that their top was below extreme low water level. The piles were then driven and cast into the

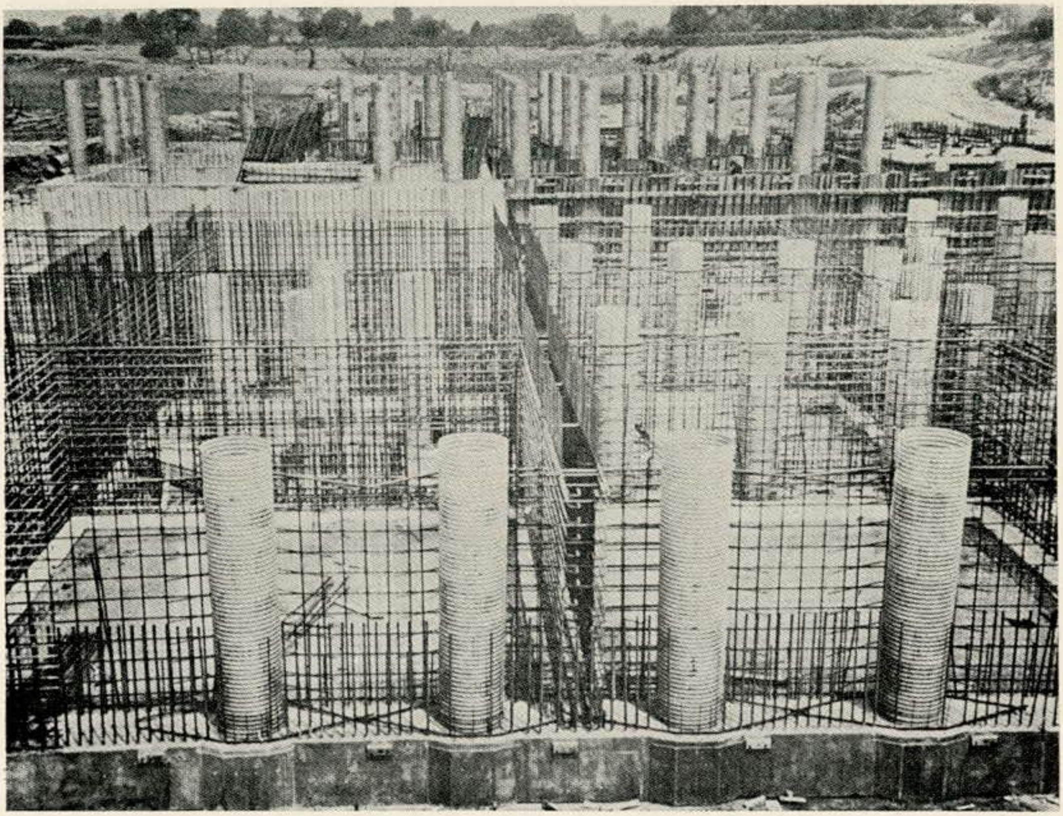


FIGURE 10
Caisson reinforcing bars and corrugated metal forms for holes in piles.

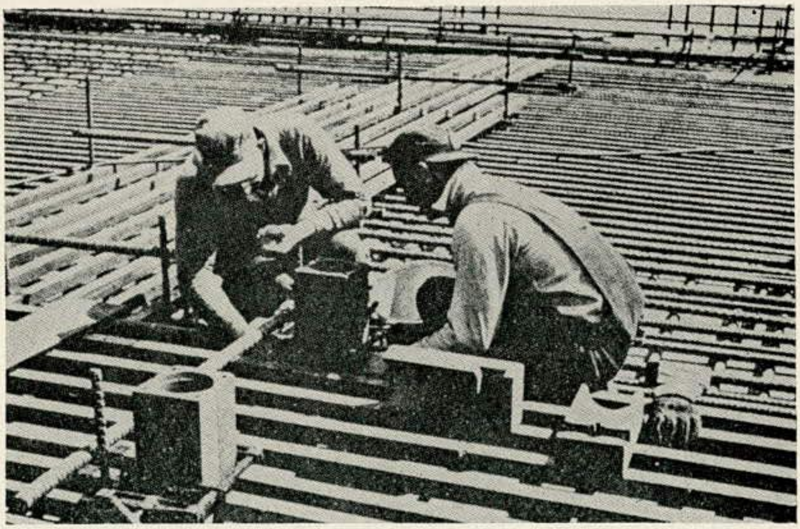


FIGURE 11
Splicing of caisson reinforcing bars by Thermit weld process.

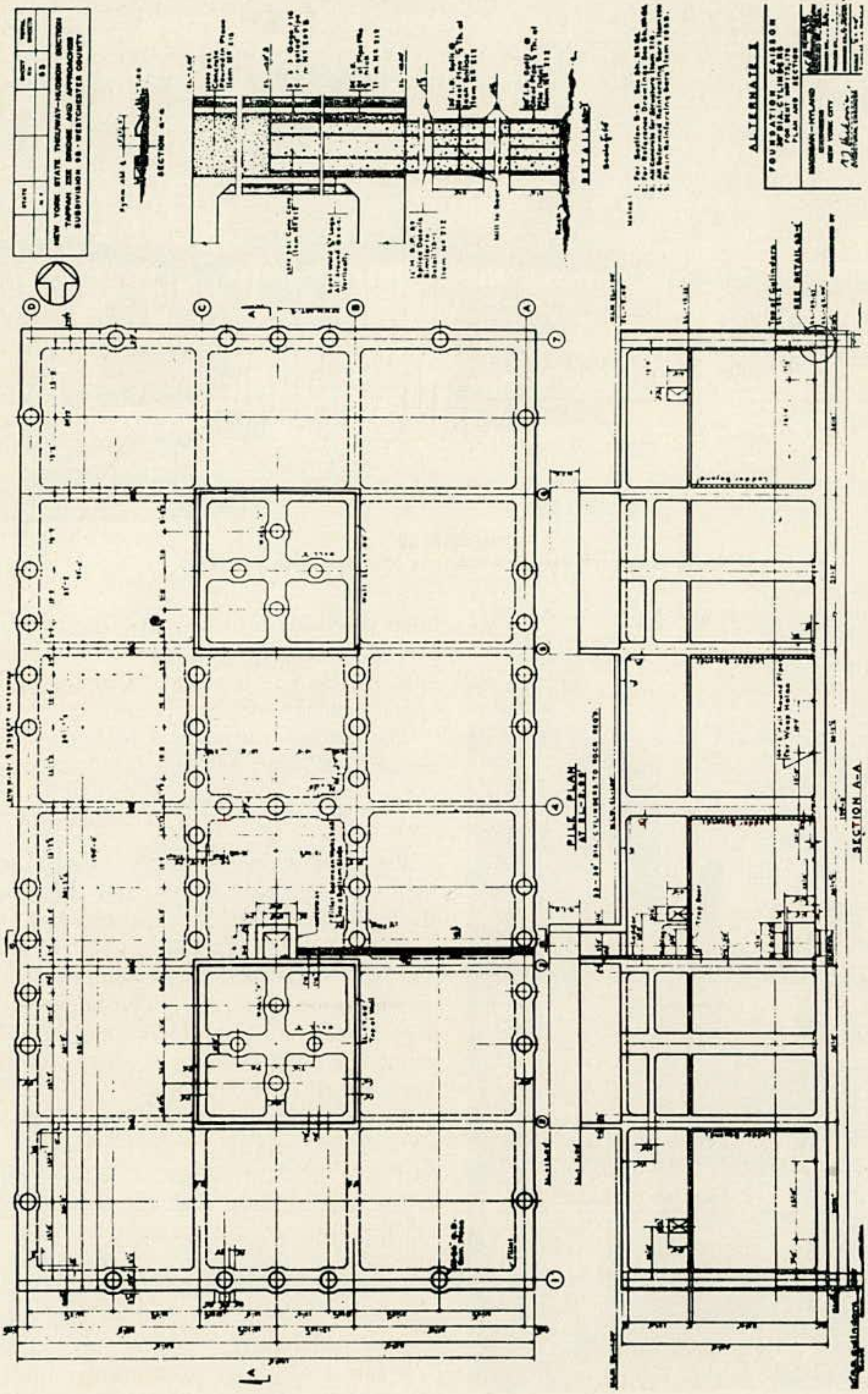


FIGURE 12
Foundation caisson plan and section.

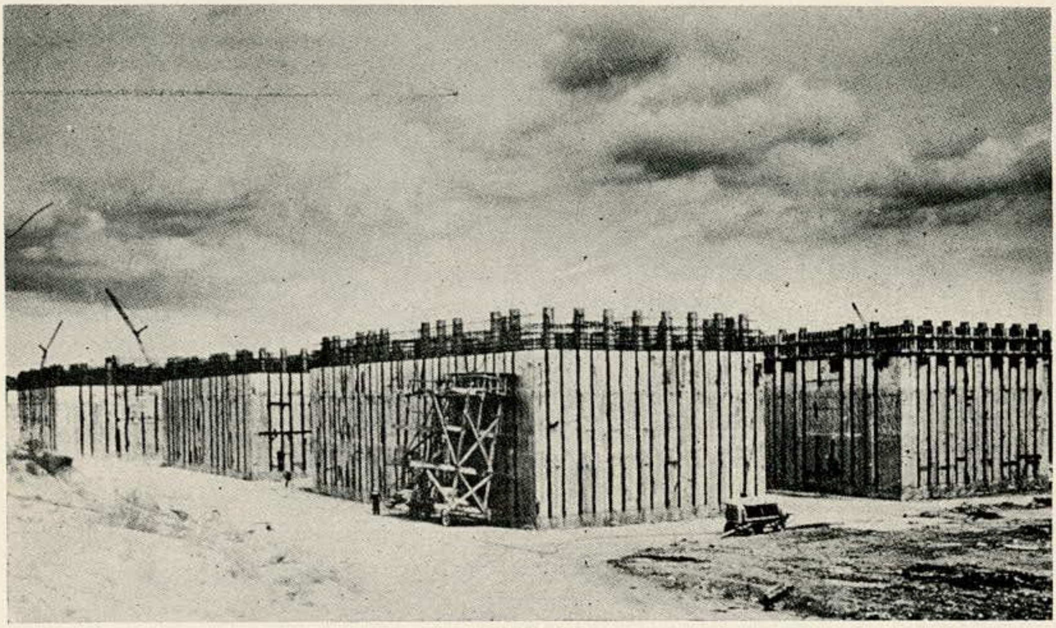


FIGURE 13
Completed prefabricated sections of foundation caissons.

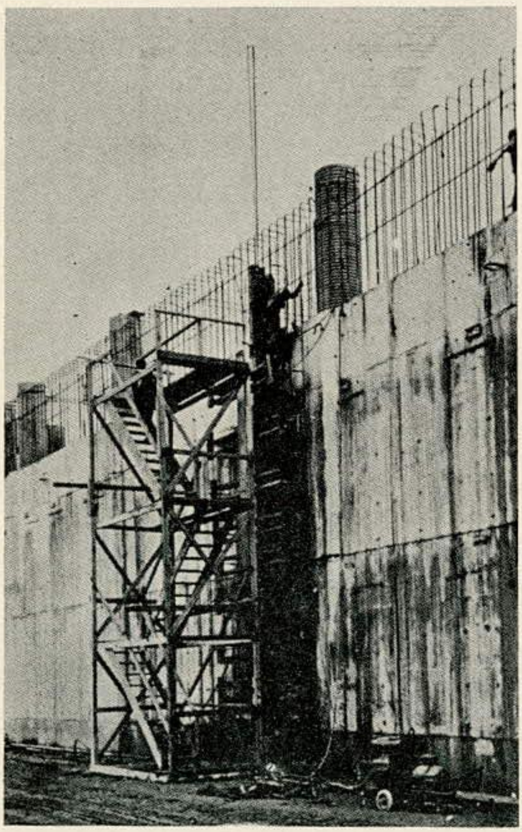


FIGURE 14
Pressure testing caisson joint for watertightness.

holes provided for them. As the superstructure was completed the boxes were pumped out. Access has been provided for periodic inspection of their watertightness.

The bridge was opened to traffic in December 1957 and has operated perfectly.

PIER 57, NEW YORK HARBOR

The pier which formerly existed at the foot of West 15th Street and the Hudson River, New York City, was destroyed by fire in 1947 *(Fig. 18)*. In 1952 a new "T" shaped pier was constructed. A plan of the new pier is shown *(Fig. 19)*. The finger extending into the river is 725 ft long and 150 ft wide; the portion along the shore is 375 ft long and also 150 ft wide.

Wharfage space on the New York City waterfront is at a premium, thus the Department of Marine and Aviation, owners of the pier, desired that the new pier have much more capacity than the old pier. This indicated a pier with several levels and much higher unit loadings than the old structure. Also, it was desired that the new pier be fireproof.

A forest of about 3,000 timber piles was all that remained of the old pier. Subsurface explorations at the site indicate bedrock

FIGURE 15
Towing first foundation caisson from construction basin.

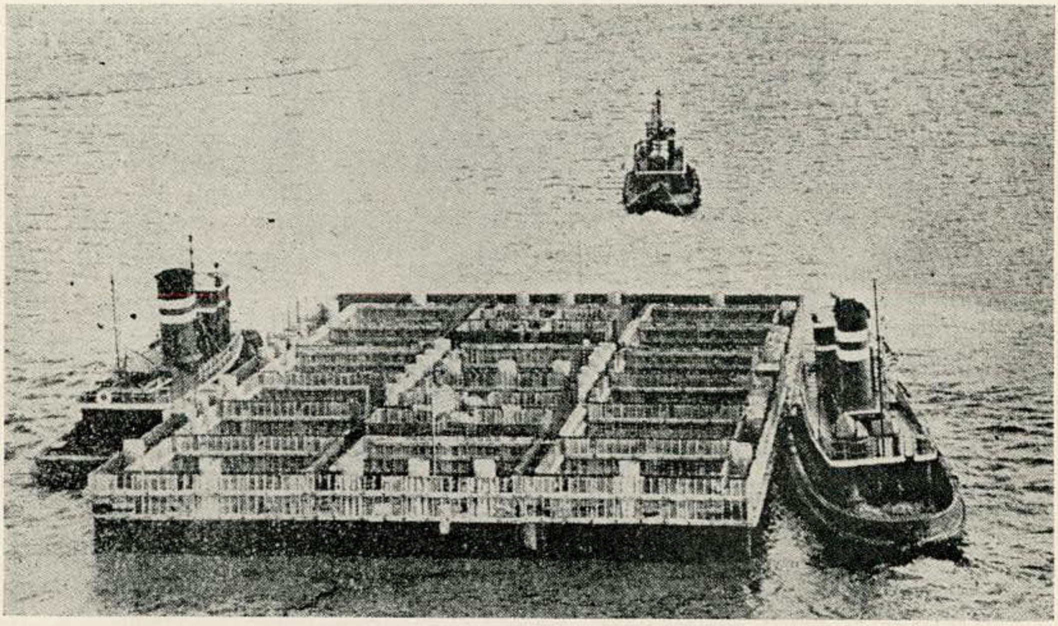


FIGURE 16
Foundation caisson en route to site.

at a depth of about 100 ft along the shore and about 400 ft at the offshore end of the pier. The stratum immediately below harbor bottom is composed of soft to medium stiff Hudson River silt; it varies from 60-ft thickness at the shoreline to 200-ft thickness at the offshore end of the pier.

To carry a new, heavier pier on a convential pile foundation, exceedingly long piles, particularly at the offshore end, would be required. These piles would have to penetrate through the silt and some distance into firm sand. The new piles would have to be located between the many timber piles or

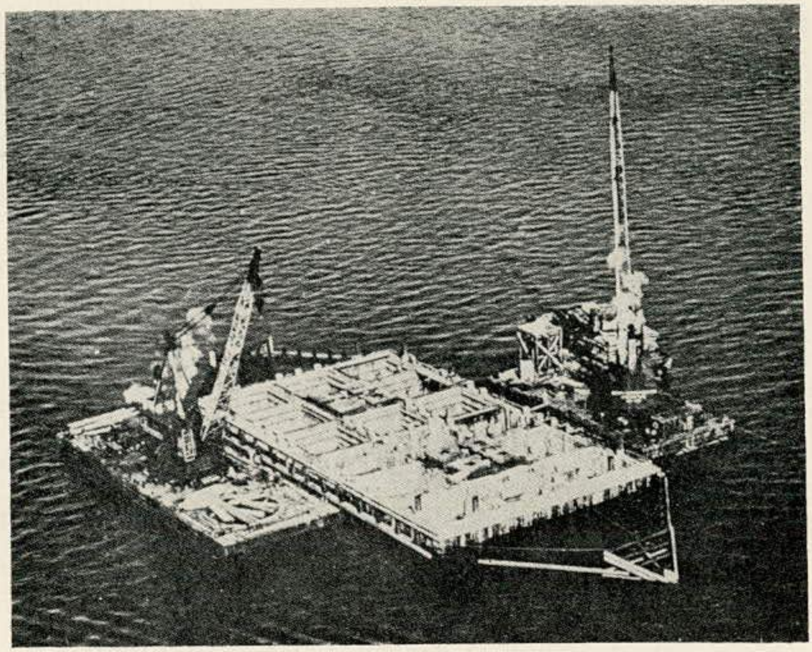


FIGURE 17
Foundation caisson held in place by clusters of timber piles.

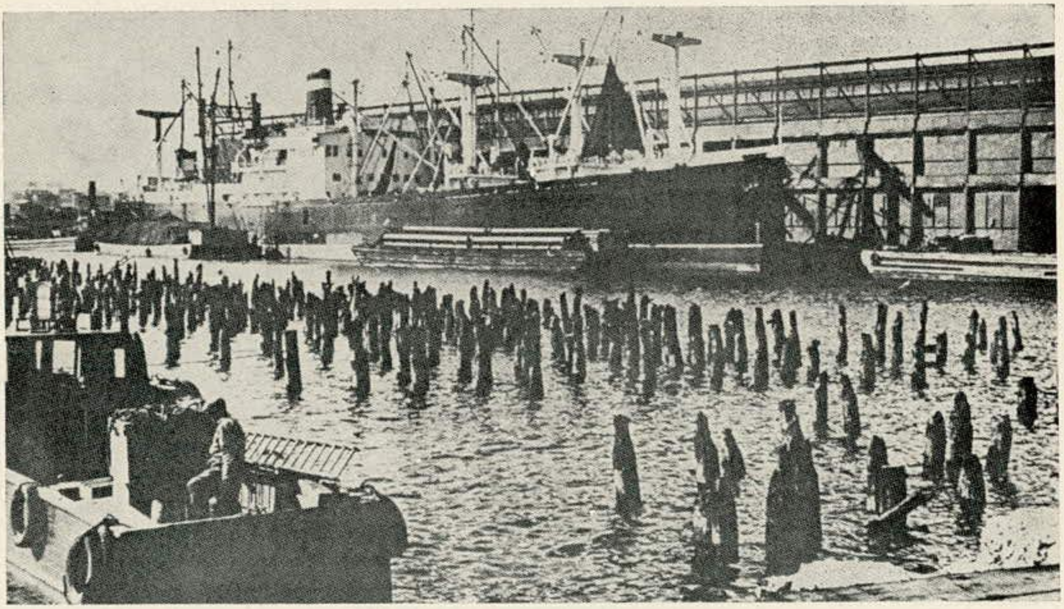


FIGURE 18
Site of Pier 57, New York Harbor, after 1947 fire.

the timber piles would have to be pulled. The timber piles could not be used in conjunction with the new piles due to the difference in stiffness between the silt and the underlying firm material supporting the two different types of piles.

It was decided that by using a floating caisson-type foundation the need for long expensive piles for the finger portion would be eliminated; the existing timber piles could be utilized to carry a load proven by their past performance and the difference

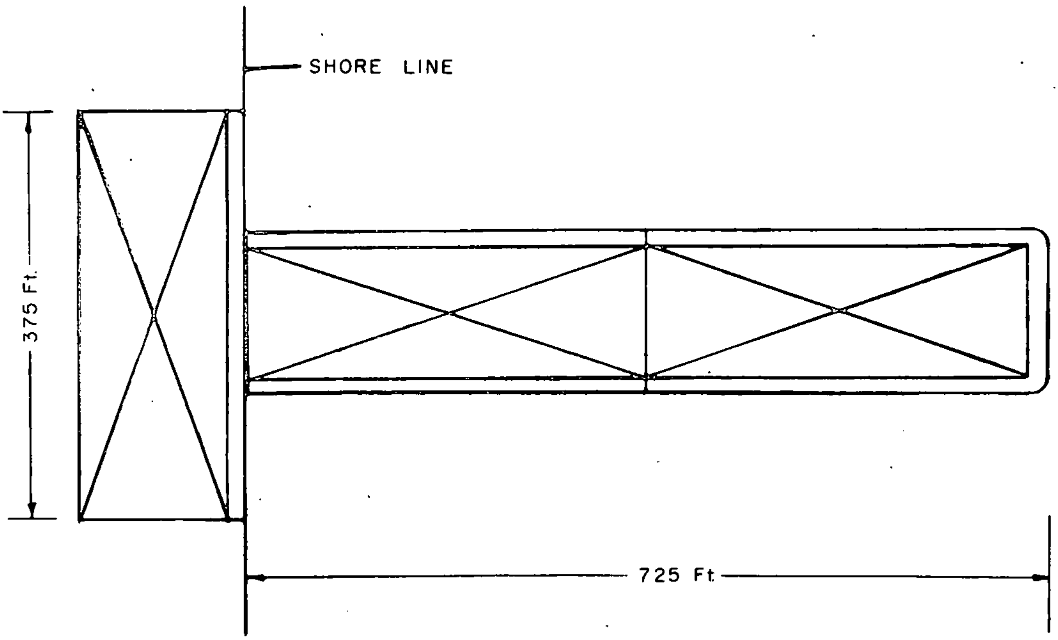


FIGURE 19
Pier 57 plan.

between the capacity of the wooden piles and the load imposed by the new pier would be carried by buoyance of the caissons.

Three caissons were cast, two to form the finger and one for the portion along the shore. The caissons were cast in the basin at Haverstraw in the same manner as described for the Nyack-Tarrytown bridge. The caissons were much larger, however, than any used for the bridge, being 350 ft long by 82 ft wide and 34 ft deep. For the caisson along the shore, piles were driven through the holes cast in the caisson walls to bear on bedrock. For the two caissons forming the finger, the piles driven through the holes in the caisson walls did not penetrate through the silt. These piles are considered to pin the caissons to the foundation.

Before the caissons were brought to the site, the silt and debris between the existing timber piles was removed by clamshell bucket. When the excavation reached about 35 ft below mean low water, the timber piles were cut off. In order to insure that the timber piles would carry their former load without causing settlement of the silt, sand drains were installed between them. The caissons were then brought to the site and the full height of walls and deck constructed. The deck cantilevered out about 35 ft on either side of the caisson. The bottom of the caisson cantilevered out in a similar manner in order to engage more of the existing timber piles.

The caissons were then sunk to rest on the piles by filling them with water. The filling was adjusted to impose full design load on the old piles. After allowing seven months for the foundation to adjust under this loading whatever voids existed between the bottom of the caisson and the foundation were grouted. As the superstructure was erected, water was pumped out of the caissons to maintain the proposed design load on the timber piles.

The finished pier consists of a ground level plus two upper floors as shown *(Fig. 20)*. Also, the roof was designed for storage of bulky cargo, such as automobiles. Further, the space below ground level in the hollow caisson is used for storage. The caissons are divided into various compartments by the walls and elevator access is provided

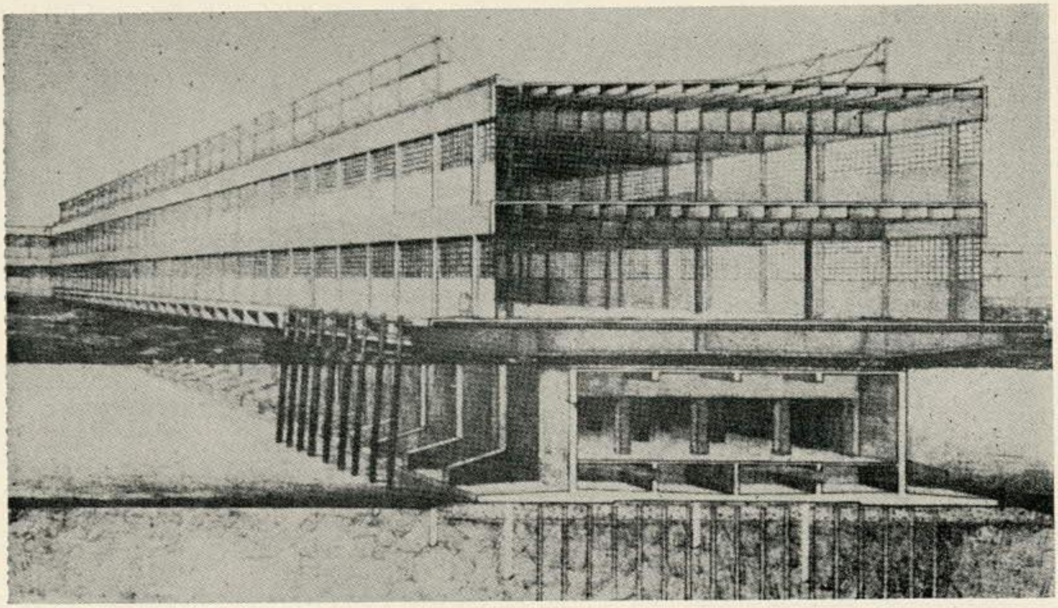


FIGURE 20

New Pier 57 is a complete departure from the conventional. Built of concrete and steel, it will not burn; marine borers will not touch it, and maintenance will be minor. Moreover, its foundation floats, its soil base will be sand-drain stabilized, and its main floor will be framed in prestressed concrete.

to the different compartments. The result is an efficient, high capacity, modern pier constructed on poor foundation materials for about the same amount of money as would be required for a similar pier on reasonably good foundation materials.

Steel Piles with Attachments

In recent years the author has been connected with the design of several projects on which it has been found economical to put attachments near the foot of steel piles in order to increase their bearing capacity. This paper describes the design of attachments for two such projects.

ORE TERMINAL, MOBILE, ALABAMA

In 1953 the Tennessee Coal and Iron Division of the United States Steel Corporation constructed an ore terminal at Mobile, Alabama, for the transshipment of iron ore from ships to railroad cars. A major feature of the terminal was a marginal wharf 1,000 ft long and 80 ft wide. A plan and cross-section of this wharf is shown *(Fig. 21)*.

The owners of the project, being a steel company, desired to use steel H-piles and steel box piles for the pier if at all feasible. Borings made at the site indicated that the subsurface conditions consisted essentially of a deep deposit of sand whose density varied in a consistent pattern with depth. For instance, from about Elevation —40 ft to —85 ft the sand was found to be medium dense, whereas below it, from Elevation —85 ft to —100 ft there exists stiff clay and below 100 ft the sand becomes dense.

Upon driving the test piles for the job, it was found that the simple H-pile and the box pile penetrated through the upper medium dense layer and into the lower dense layer requiring a rather low number of blows per foot for driving. Load tests performed on these piles confirmed that the

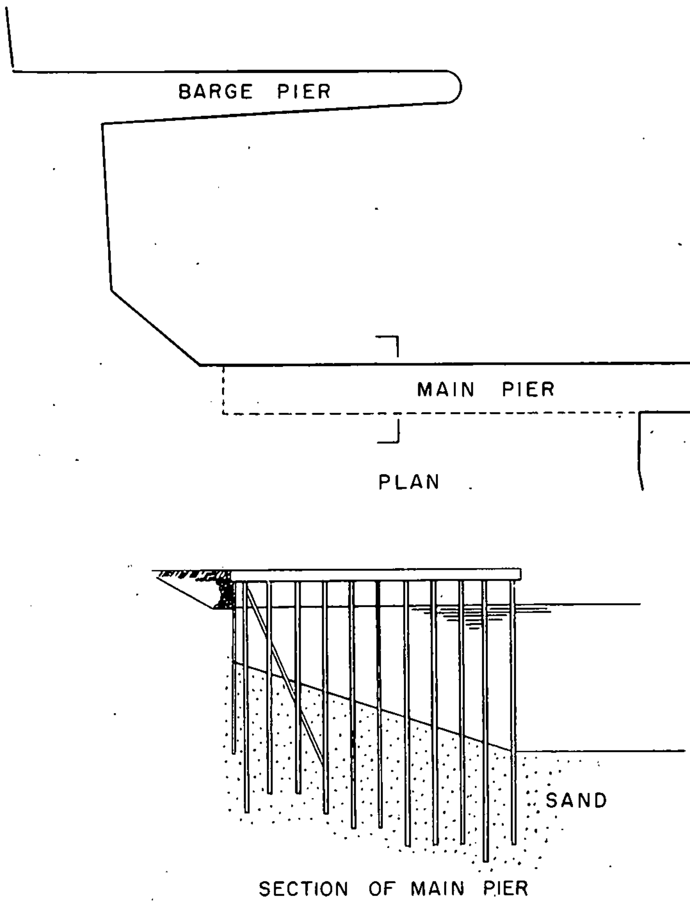


FIGURE 21
Ore terminal, Mobile, Alabama.

bearing capacity was inadequate. The driving records for two typical piles are shown *(Fig. 22)*. During a load test on the steel box pile a 1-in. movement was observed with 138 tons applied to the test pile.

In order to develop the bearing capacity of the piles within a reasonable depth, S. S. Cooke-Yarborough, who was working for the author on this project, suggested that the cut-offs from the H-piles be cut on a diagonal and be welded to the flanges of the H-piles as attachments to increase the point bearing capacity of these piles. The details of the attachment are shown *(Fig. 23)*.

When this was done it was found that piles with attachments at about 6 ft from the bottom of the pile developed high driving resistance in the upper medium dense sand layer. Load tests on these piles confirmed that their capacity was entirely adequate for the proposed design loads which had been based on 10,000 lbs psi, the allowable compressive stress for steel.

Somewhat similar attachments were developed for the box piles. Shown *(Fig. 24)* are typical driving records of piles with such attachments. Superimposed on the above driving records of piles with attachments are the driving records of similar piles without attachments. The marked increase in driving resistance developed by the attachments is evident. For comparison a 1-in. movement was observed when a 250-ton load was applied to the box pile with attachment.

The experience gained from this particular job is that although H-piles are not suitable as displacement piles and are not well

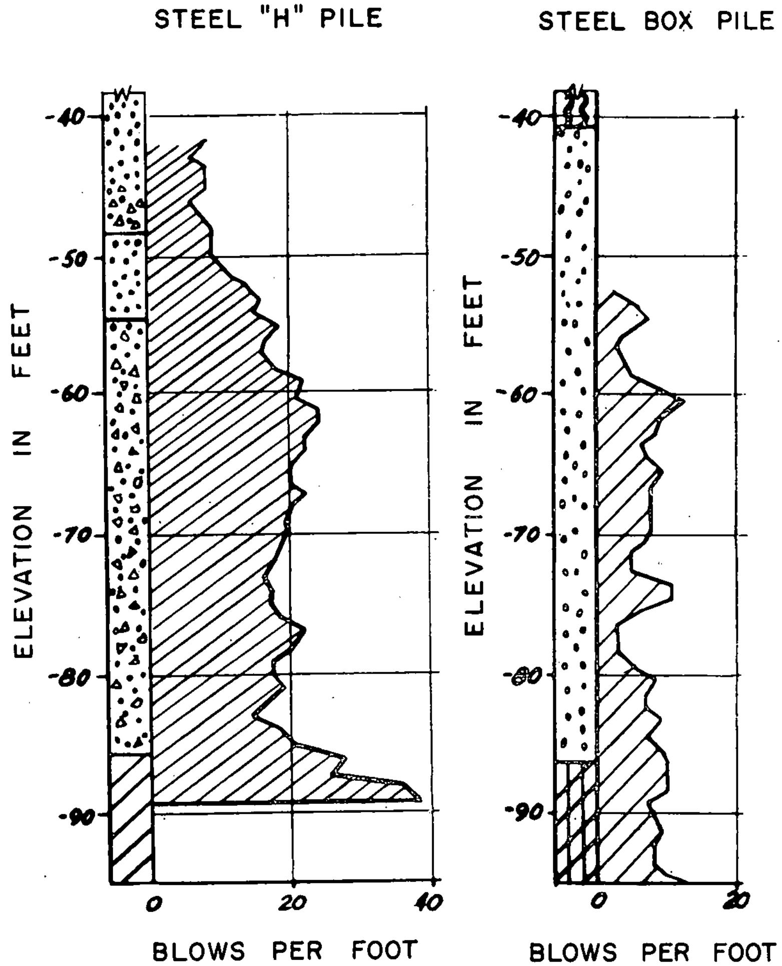


FIGURE 22
Pile driving graphs for steel H and box piles.

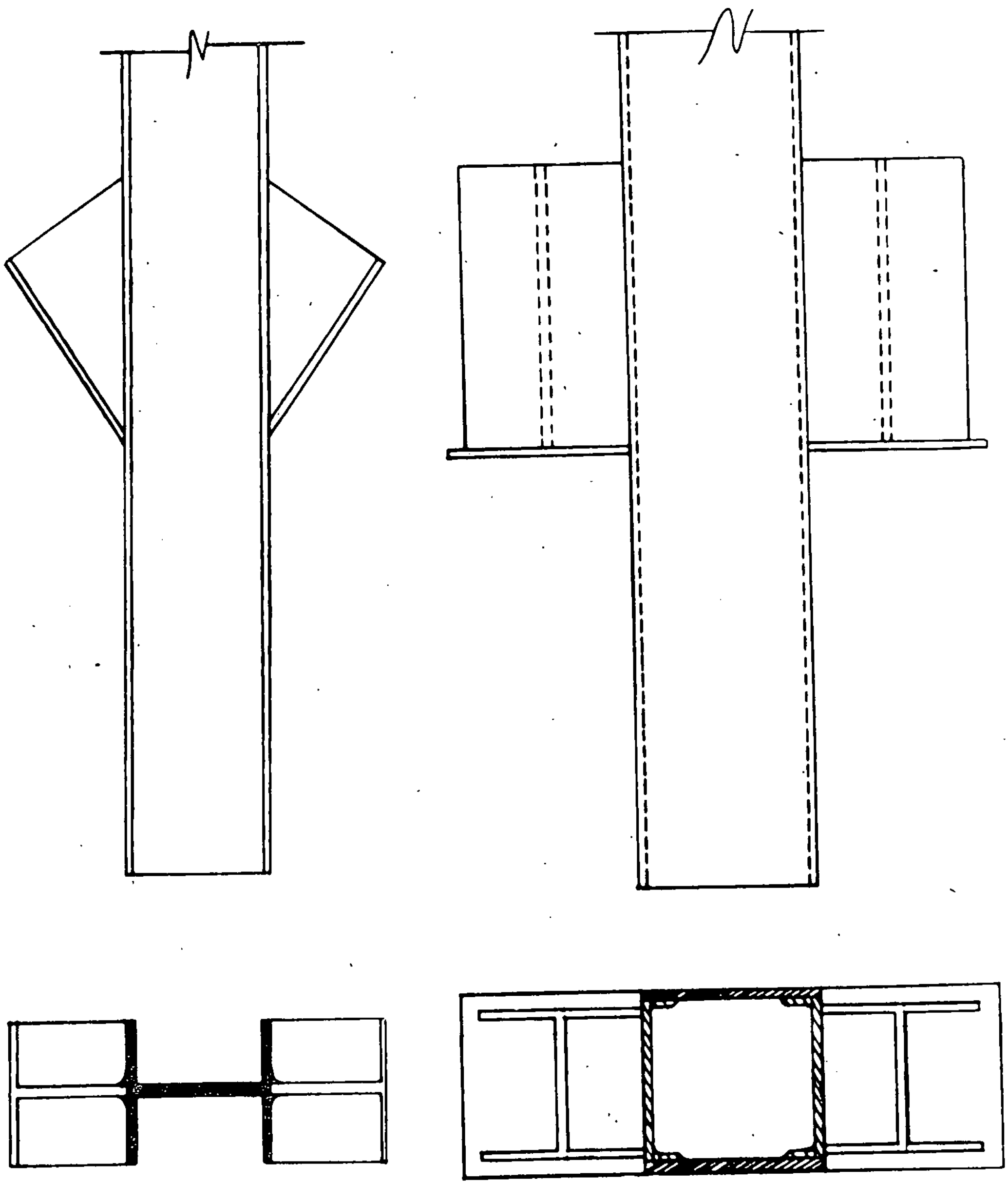
suited for developing point bearing capacity in soils, nevertheless, by means of simple, inexpensive attachments applied near the foot of the pile, their point bearing capacity can be greatly increased so that their full structural load capacity can be developed with minimum lengths.

MARINE TERMINAL FOR THE CITY OF ANCHORAGE, ALASKA

At the present time (Sept. 1959) construction is under way on the first deep water pier for the City of Anchorage, Alaska *(Fig. 25)*. Both the soil conditions and the harbor conditions present unusual problems for the design of the piles for the pier. Soil conditions at the site were investigated by a detailed program of undisturbed sample borings. The findings of these borings indicated the general soil profile shown in Figure 25.

Offshore, where the pier is to be located, there is a shallow surficial layer of loose silt and sand. Under this is a 20-ft to 40-ft thick layer of dense silt and sand which is somewhat stratified and below this is a deep layer of stiff clay.

Tide conditions at the site are severe, there being a total variation of about 40 ft from extreme low water to extreme high water and an average daily variation of 30 ft.



"H" - PILE WITH SPEAR

STEEL BOX PILE WITH BEARING PLATE

FIGURE 23
Modified steel H and box piles.

In the winter ice floes choke the harbor; the ice shifts both with the tide and wind. The pier must be sufficiently strong to resist the impact of large accumulations of this ice. Also, it is expected that during the winter ice will form on the piles in a manner similar to that which has been observed on adjacent small piers and the area under-

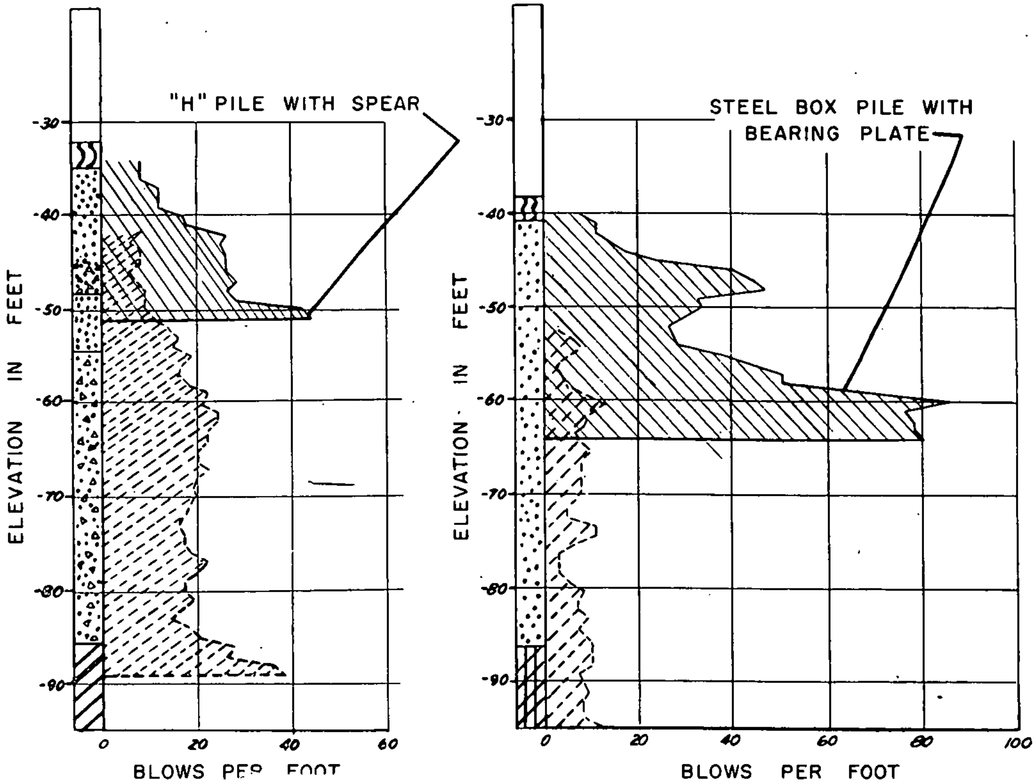


FIGURE 24
Pile driving graphs for modified steel H and box piles.

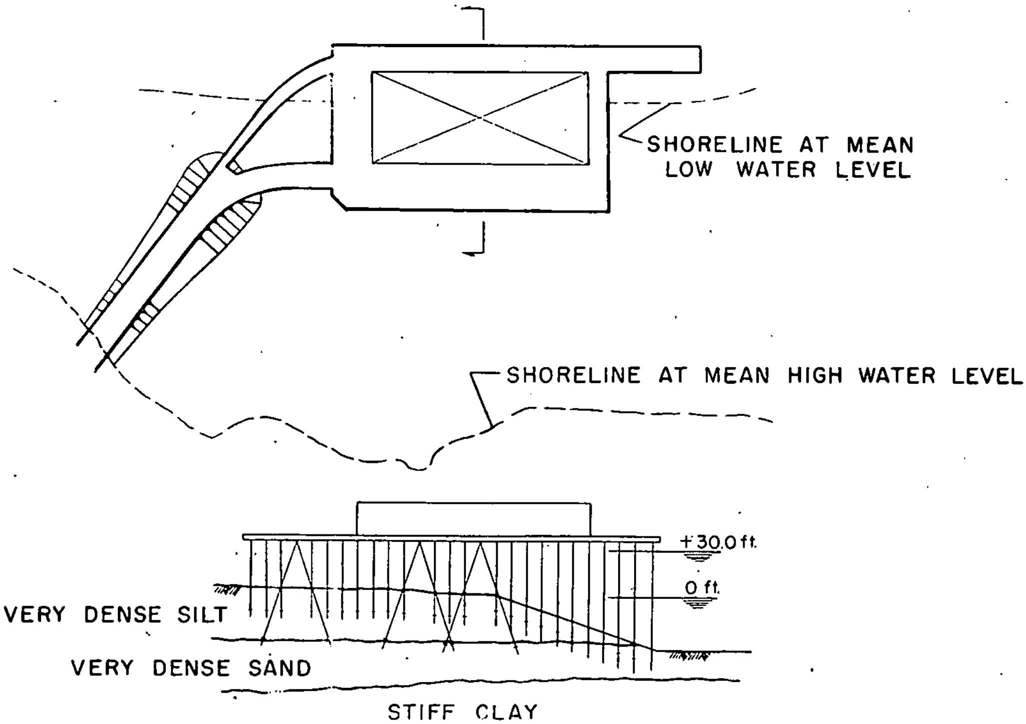
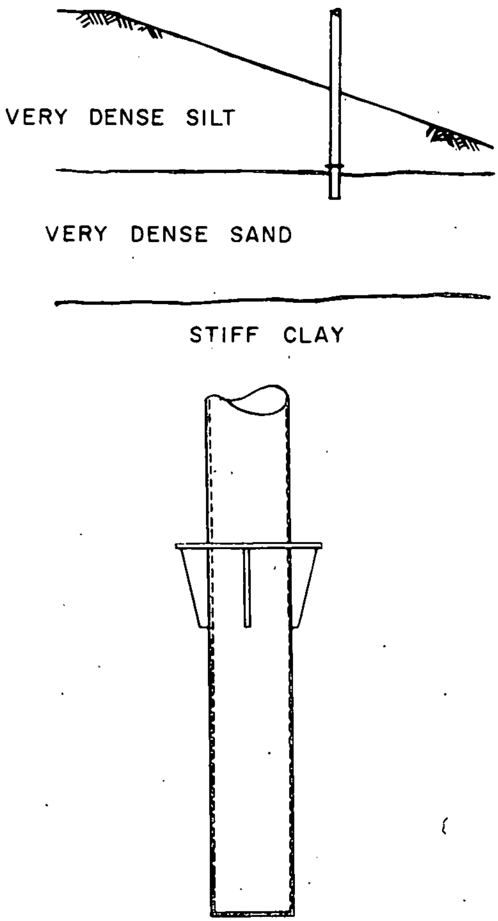


FIGURE 25
Marine Terminal, Anchorage, Alaska.

neath the deck of the pier may become more or less a solid mass of porous ice.

A cellular type of pier construction was considered but rejected because of stability and settlement considerations. The design chosen consists of various size steel pipe piles. At the offshore face of the pier is a row of 3.5-ft diameter caisson type piles. Behind this are four rows of 24-in. diameter and four rows of 20-in. pipe piles and at the inshore area are 17 rows of 16-in. piles.

The cost of shipment of steel to Anchorage is expensive so that the length of piles had to be kept to a minimum. In order to do this, it was necessary that the piles develop their bearing capacity in the stratified silt and sand layer overlying the clay. If the piles penetrated through this layer into the clay, the length of piles would have to be appreciably greater to develop the design bearing capacity. The design provided that attachments be placed on the sides of the piles in sizes necessary to develop the capacity of the piles in the stratified silt and sand layer. The attachments consisted of annular steel plates which were threaded on the pipe piles, welded to the walls of the pipe and stiffened by brackets as shown *(Fig. 26)*.



STEEL PIPE PILE WITH
BEARING PLATE

FIGURE 26
Modified steel pipe pile at Anchorage Marine Terminal.

The steel cut out to form the hole in the center of the annular ring was used as a cover plate at the bottom of the pipe. The diameter of the annular rings had to be determined very carefully by testing in the field. Although it was desired they be large enough to develop adequately the bearing capacity of the piles, they could not be so large as to prevent pile penetration to the minimum embedment required for fixity.

To date approximately half of the number of piles required for the pier have been driven and they have been meeting both the minimum and maximum penetration requirements as well as the criteria of driving resistance to develop their bearing capacity. It has been found necessary to make only one small change in size of plate to adjust the piles to variations in conditions encountered so far at the site. The correlation between driving resistance and static load bearing capacity was developed by a series of pile load tests performed at the site.

A very large saving in cost of the project was effected by designing the piles to develop their bearing capacity in the stratified silt and sand layer overlying the deep clay deposit.