Cofferdams for the Town Creek Piers of the Silas N. Pearman Bridge Over the Cooper River, Charleston, South Carolina

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This paper discusses the cofferdam design for Pier T-2 of the Town Creek spans of the Silas N. Pearman Bridge at Charleston, S. C. The discussion covers the adverse soil conditions at the site as well as the design assumptions and procedure. A large portion of the paper is devoted to the installation procedure in which the contractor employed various expedients to reduce the cost of cofferdam construction. These reduced costs were reflected in the bid price which was below the engineer's estimate.

•THE present paper is limited to a discussion of Pier T-2. This pier was the first of the two main piers to be constructed by the contractor, James T. Triplett, Inc., and L. R. Ryan, Inc., who built the four piers at Town Creek under one contract. In addition, Pier T-2 was located in the most adverse soil conditions.

In order to present a complete picture of the cofferdam, it is necessary to discuss the unusual soil conditions, the design assumptions, and the installation procedure.

SOIL CONDITIONS

River pier cofferdams have been built in poor soil conditions for years. However, when the soil is very poor and the cofferdam is very large, the problems are naturally compounded. The soil at this particular location can best be described as deep mud underlain by marl. The borings for Pier T-2, as shown in Figure 1, using mean sea level as zero datum, indicate 21 feet of water, 12 feet of dark gray silty clay of "push" quality, 21 feet of bluish gray organic clay with a trace of sand and shell fragments and still of "push" consistency, 23 feet of bluish gray organic clay with high sand content (note that even with a high percentage of sand, the soil offered no resistance to the sampling spoon). The next 4-ft stratum from -77 to -81 ft, consisting of soft bluish gray organic clay with little sand, mica, and shells, offered a resistance of 2 blows per foot. Below elevation -81, the soil was stiff brownish green calcareous clay with a slight sand content and the penetration resistance was about 8 blows per foot. This last stratum is typical Cooper marl. The soil boring was not carried to a greater depth since it was made to provide information for the cofferdam construction and not for the foundation of the bridge pier. All sampling was done with a 2-in. diameter split spoon driven by a 140-lb hammer falling 30 in.

From the description of the soil conditions it is obvious that the bridge design engineers were faced with the problem of a very deep foundation, and they chose a concrete pier on H-piles as the most economical solution. The contractor, however, had the responsiblity of designing the cofferdams and constructing the piers under these adverse soil conditions.

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Elevation in feet	Penetration Resistance in blows per foot	Soil Description
0 to -21.0		Water
-33.0	PUSH	Dark Gray Silty Clay
-54.0	PUSH	Bluish-Gray Organic Clay With Trace Of Sand And Shell Fragments
-77.0	PUSH	Bluish-Gray Organic Clay With High Sand Content
-81.0	2	Bluish-Gray Organic Clay With Little Sand, Mica, And Shells
-84.0	8	Brownish-Green Calcareous Clay With Slight Sand Content

Figure 1. Soil boring for Pier T-2.

COFFERDAM DESIGN

The design of the cofferdam was complicated principally by two features. First, the deep mud had to be retained during the pile driving operation, and second, 228 vertical and battered H-piles had to be driven to support the pier. In addition, the batter piles prevented the sheet piling from penetrating beyond elevation -62.

As in all work with soils, certain assumptions had to be made about the fluid properties of the mud. Although several analyses were attempted, the final soil properties used in the design were a submerged weight of 40 lb per cubic foot and an active pressure coefficient of 0.75. With the great number of bearing piles required to support each pier, the installation would have been simplified if the cofferdam could have been constructed without interior bracing. However, the alternatives to a braced rectangular cofferdam, such as a cellular structure or a circular ring-braced cofferdam, appeared to be either too expensive or impractical from a design standpoint. Therefore it was decided that a braced cofferdam, as indicated on the contract drawings, would be used.

The contractor proposed to use one bracing cage for three purposes: (a) a driving frame for the sheet piling, (b) bracing for the water-filled cofferdam during the pile driving operations, and (c) bracing for the unwatered cofferdam. The two principal loading conditions to which the cofferdam was subjected were (a) soil pressure while

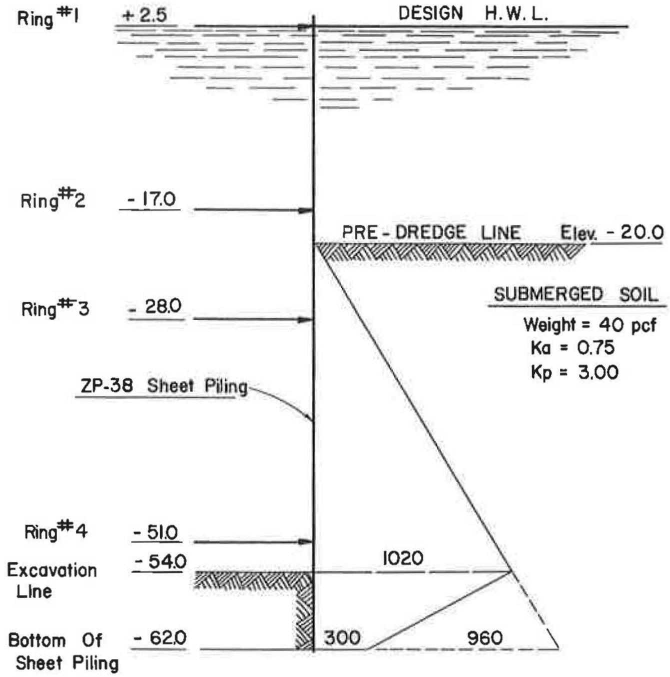

Figure 2. Loading diagram for submerged soil.

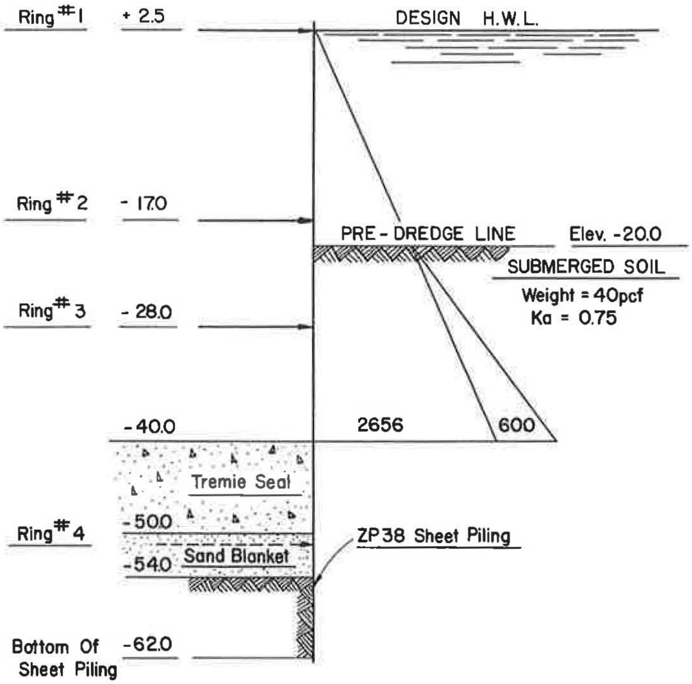

Figure 3. Loading diagram for unwatered cofferdam.

the bearing piles were installed and while the tremie seal was poured, and (b) soil and water pressures during the pier construction. Although the latter condition involved the heavier loads, the critical design was during the early phase of construction. Since the batter piles prevented the sheet piling from being driven into the firm soil, the sheeting had little toe support and most of the differential soil pressure had to be resisted by the internal cross-bracing. The loading diagram for this condition is shown in Figure 2. Note that a net passive pressure was assumed to exist in the zero blow count material below elevation -54 because of the high sand and shell content.

As noted previously, the second loading condition was during the unwatered stage of construction. The toe of the sheet piling, however, then had the 10-ft thick tremie seal as a firm support. The loading diagram for the unwatered cofferdam is shown in Figure 3.

With the two loading conditions established, it was necessary to design bracing which could be used for both phases of construction in accordance with the proposed installation procedure. The unwatered cofferdam was subjected to relatively high loading since it retained 42 feet of water and 20 feet of submerged soil. With these high external loads plus construction loads, cross struts and X-bracing were required to provide stability. The interior X-bracing, however, could not be installed during the initial erection of the bracing cage since it would interfere with the driving of the batter piles. The location and batter of the H-piles also dictated the location of the cross struts during the first phase of construction. Figure 4 illustrates the large number of bearing piles as well as the problem of fitting them through the bracing. It was necessary to design a system of bracing which would sustain the pressure of the submerged soil during the driving operation and, with all cross struts and X-bracing installed, would support the pressures exerted on the unwatered cofferdam. Figure 5

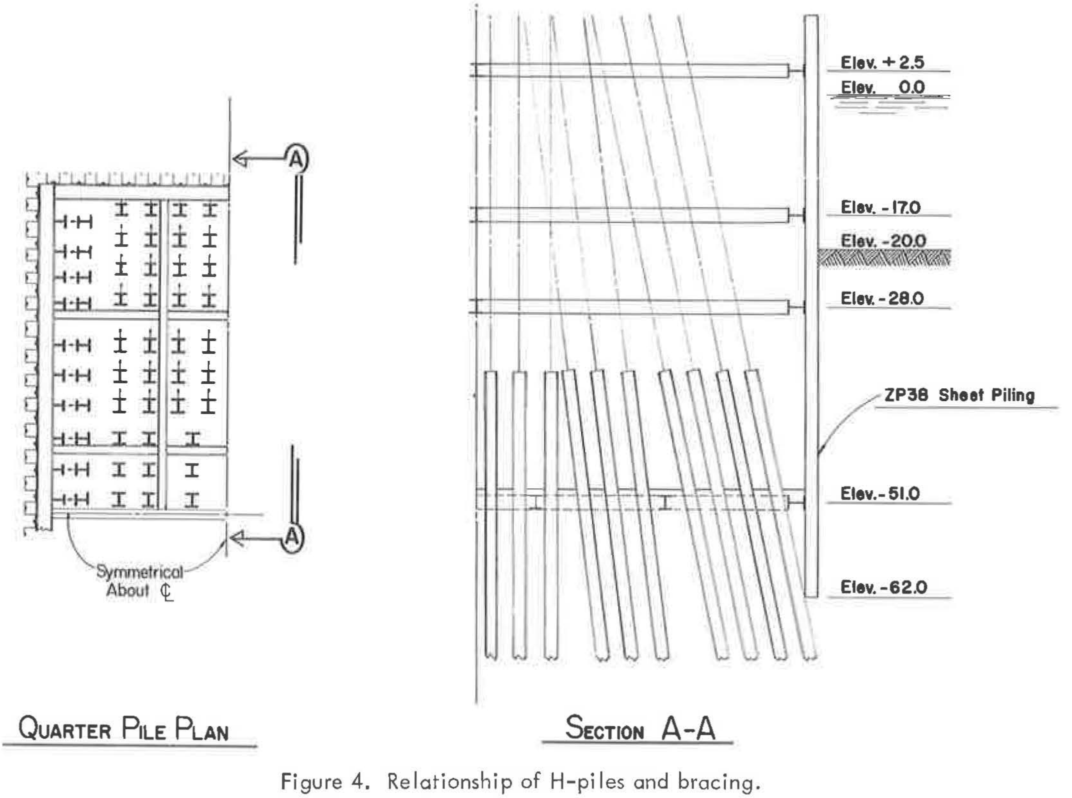

Figure 4. Relationship of H-piles and bracing.

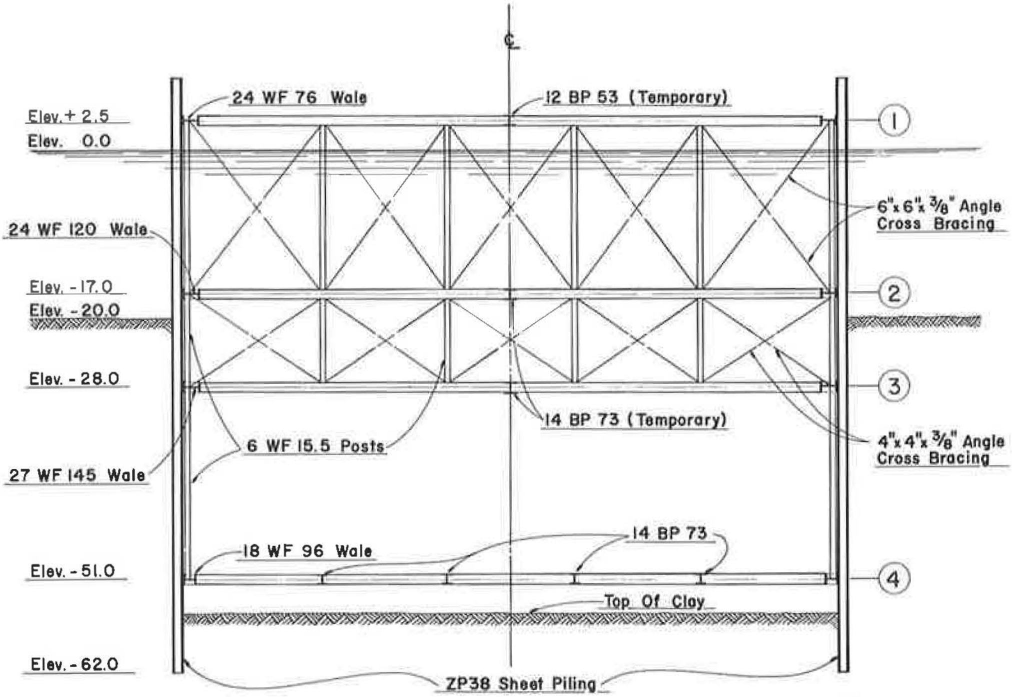

Figure 5. Bracing before unwatering.

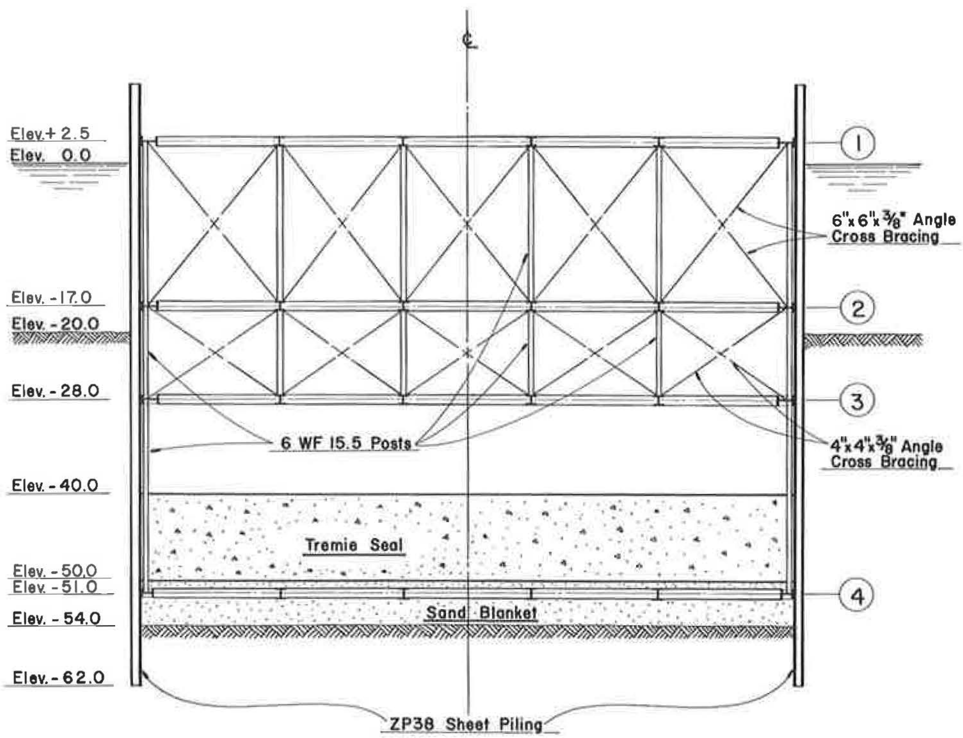

Figure 6. Bracing after unwatering.

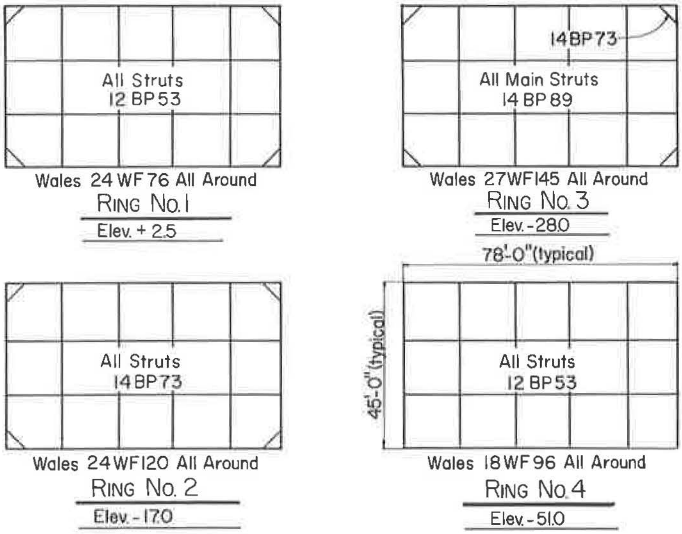

Figure 7. Plan view of bracing rings.

shows the bracing for the cofferdam prior to unwatering. Note that there are temporary struts at the centerline of the cofferdam for each of the three upper tiers of wales. One strut was all that was required in those tiers to sustain the loads from the submerged soil. The lowest ring, however, required four struts. Figure 6 shows the same view of the bracing for the unwatered phase. Note that the temporary struts have been removed and that each ring of bracing has four struts. The bracing can be seen more clearly in Figure 7 which shows a plan view of each tier. With a bracing system as shown in the preceding figures the contractor was able to modify it from one phase of construction to the next with a minimum of lost time. The methods employed by the contractor to accomplish this change are discussed later.

Most river pier cofferdams are similar; however, the two features which make this one somewhat unique are (a) the installation procedure, and (b) the use of a sacrificial wale below the tremie seal which permitted the use of standard sheet piling sections. If the soil adjacent to the toe of the sheet piling did not exhibit passive pressure as assumed, the ZP-38 piling had sufficient reserve strength to prevent failure. The wales were designed continuously since they were prefabricated and were installed in complete rings. The strut connections, however, were designed as simple supports so that they might be more easily installed and removed.

INSTALLATION PROCEDURE

The design was based on pre-excavating to elevation -20. The contractor was able to accomplish this on the outboard side of the pier but found that excavation below elevation -16 on the inboard side caused continual sloughing of the shore line and subsidence of the small structures which were supported on footings. Pre-excavation was therefore stopped at elevation -16 on the inboard side, and the resultant 4-ft differential in soil height was assumed to be insignificant in its effect on the cofferdam.

The contractor first assembled on a pontoon barge the bottom ring of bracing, Ring No. 4, with all permanent struts in place and Ring No. 3 above it with only a temporary strut in place. The barge was floated into place and the assembled bracing system was precisely positioned and leveled. Because of the massive size of the bracing system the contractor elected to drive six spud piles, section BP14 × 89, around the

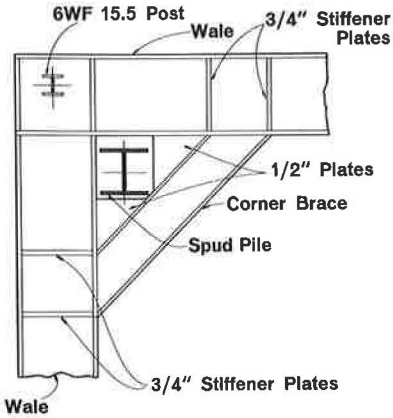

Figure 8. Spud pile guide.

periphery. When the spuds were dropped through the prefabricated slots along the wale (see Fig. 8), they penetrated the soil under their own weight to elevation -60, which was just 2 ft above the proposed sheet piling tip elevation. The six piles, each 120 ft in length, were then driven into the Cooper marl to elevation -87. The two tiers of bracing were hung from hand winches located at the top of the spud piles. Figure 9 shows the winch installation. The winch lines were taut at high tide so that, as the tide went out, the pontoon barge was floated out from under the bracing cage. Figure 10 shows the pontoon-mounted bracing.

The two-tier cage was lowered by use of the winches to the pre-excavation elevation and Ring No. 2 was added. The peripheral X-bracing was then placed between Ring No. 3 and Ring No. 2. The ZP-38 steel sheet piling was assembled around the three-tier cage and was driven in stages which were coordinated with the excavating operation. This multi-stage driving procedure, which consisted of driving the sheeting about 12 ft beyond the bottom brace and then excavating and lowering the bracing cage, helped to maintain the verticality of the sheet piling and prevented it from bowing in under the bottom brace. As the bracing was lowered to a point where Ring No. 2 was just above the water line, Ring No. 1 was added.

It is interesting to note that the borings indicated a "push" quality material all the way to elevation -77; however, the contractor found that excavating this material was considerably more difficult than anticipated. The soil exhibited some notable characteristics. For instance, it could be excavated by clam shell and would retain the general shape of the bucket when placed on the ground, but after a period of several days it would creep into an extremely flat angle of repose and would spread over the entire

Figure 9. Hand winch on spud pile.

Figure 10. Bracing on pontoons (photo taken at Pier T-3).

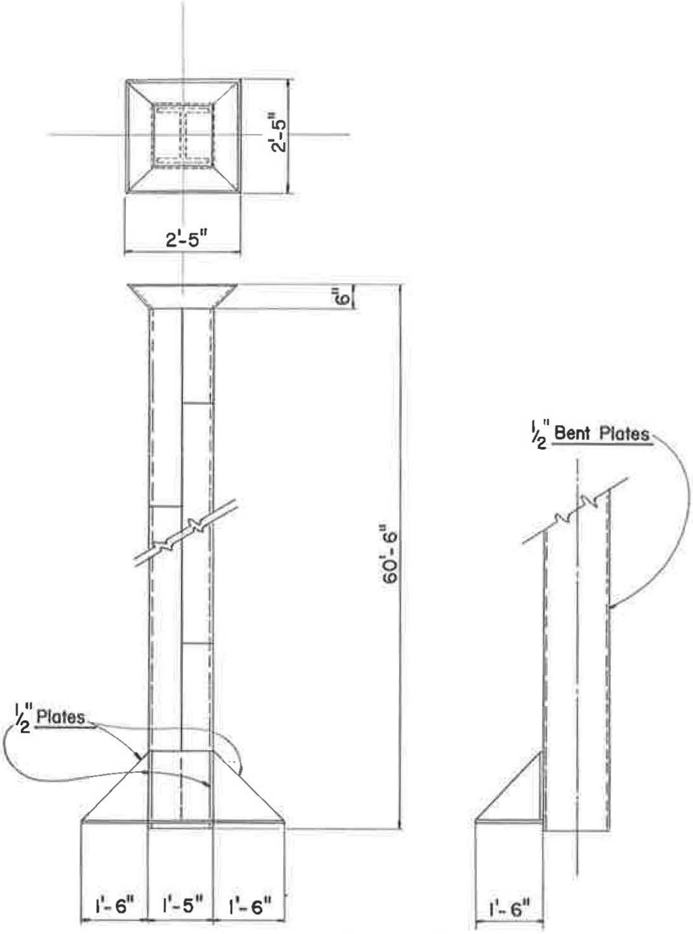

Figure 11. Pile guide box.

spoil area. On the other hand, the undisturbed strength of the soil was sufficient to maintain a vertical face and would do so under every wale and cross strut. This characteristic seemed to justify the design assumption that the soil could exhibit a net passive pressure, particularly since a high pressure jet was required to remove the walls of soil which remained standing under the cross struts.

The sheet piling was driven to a final tip elevation of -62 and the excavation was completed to elevation -54, which was 4 ft below the bottom of the tremie seal. At this point in the construction sequence all wales were in their final position, but only Ring No. 4 contained the permanent cross struts. All other rings had temporary struts which were required to sustain the soil pressure. As noted previously, the excavation was carried 4 ft below the bottom of the tremie seal. This overexcavation was made to accommodate a sand blanket which acted as a bed for the seal concrete. In addition, the sand provided a stable platform on which to rest the pile box that was used as a guide for the bearing piles.

The pile box is an interesting innovation which allowed the pile tips to be precisely spotted from above the water surface. The box, shown in Figure 11, was fabricated from ½-in. steel plate, was 60 ft long, and had a bearing plate on one side of the lower end to support the box on the sand blanket. When preparing to drive a line of piles, the contractor would place a guide beam across Ring No. 1 which was above the water line.

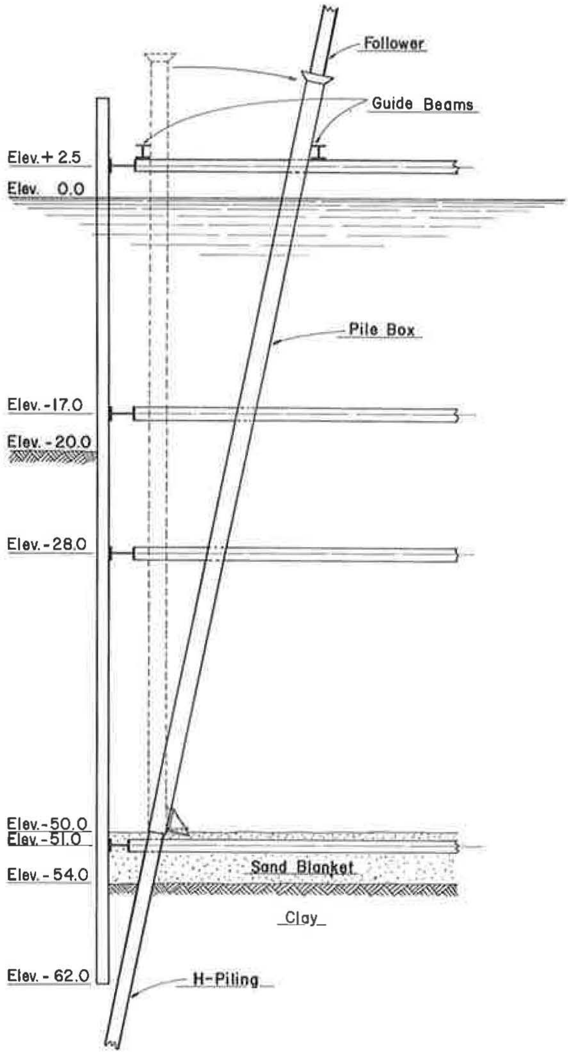

Figure 12. Setting H-piling.

In the case of vertical piles the box could be positioned plumb against this guide beam. For batter piles two guide beams were used, one located adjacent to a vertical plane containing the pile tip and the other a predetermined distance away. The pile box was accurately placed over the pile tip location and then tilted back against the second guide beam. This operation, shown in Figure 12, provided accurate pile location as well as the required slope for the batter piles.

As the H-piles were dropped in the box they hit the sand blanket and hesitated briefly until the sand was penetrated by the pile tip. The earlier piles would again plunge downward under their own weight until they rested on the firm layer of marl. As the driving progressed, the sand blanket became more dense from vibration and displacement, and the piles no longer penetrated the sand under their own weight, but required a few taps from the hammer in order to break through. A total of 228 BP14 × 73 H-piles were driven under Pier T-2 with a McKiernan Terry C-5 hammer and a BP14 × 117 follower. The follower permitted the piles to be driven continuously to the required cut-off grade which was 4 ft above the top of the tremie seal at elevation -40.

To reduce the number of handling operations involved with pouring the tremie seal, the contractor drove overlength piles at eight of the permanent vertical pile locations. These eight piles extended above the water surface and served as supports for an extension of the temporary construction trestle. With the trestle extended over the cofferdam the tremie seal was poured directly from ready-mix trucks through two 14-in. diameter steel tubes. This direct pour reduced handling costs of the tremie concrete.

After the concrete seal had set, a diver was lowered into the cofferdam to disconnect Ring No. 4 from the three upper tiers of bracing. The cofferdam was then overflooded with an additional head of water that spread the sheet piling away from the wales, thereby permitting the upper tiers of bracing to be raised above the water line. Diver work was minimized since all permanent struts and cross braces could then be installed above water. The bracing cage was then lowered to its final position inside the sheeting.

With the bracing all in place the cofferdam was dewatered and construction of the concrete pier was started. The footing was poured from the top of the seal up to elevation -30 just below Ring No. 3. Rather than strut to the footing in order to remove

the cross braces from Ring No. 3, the contractor filled the void between the footing and the sheet piling with sand except for the top 18 in. in which he poured a slab of concrete. After the concrete had set, the bracing was removed from Ring No. 3 and the wales were raised to elevation -19 just below Ring No. 2. The pedestal concrete was then poured up to elevation -20. Struts were placed from the pedestal to Ring No. 3 at its new location and Ring No. 2 was raised to elevation -2. The remaining lifts of pedestal concrete were poured and braced similarly until the concrete was well above the water line.

Backfill was placed inside the cofferdam to prevent a mud slide which might undermine existing surface structures upon removal of the sheet piling. The cofferdam was then flooded and the sheet piling and bracing were removed.

SUMMARY

The preceding discussion of cofferdam design and installation illustrates the various methods that contractors will employ to reduce construction costs. In this instance the reduction was substantial. The engineer's estimate for the overall contract for the four bridge piers was $1,527,000 and the contractor's bid was $1,442,000. It is evident that this difference of $85,000 was a result of finding economical and sometimes unusual methods of construction as outlined here.