INNOVATIONS IN THE DESIGN AND CONSTRUCTION OF BRIDGE FOUNDATIONS

By: Robert Bittner, P.E.

BITTNER-SHEN CONSULTING ENGINEERS, INC

INTRODUCTION

Large diameter tubular piles offer significant advantages in terms of design performance and economy for major marine structures. This is particularly true for foundations of major overwater bridges in regions of high seismicity such as the west coast of North America. The use of large diameter piles in the range of 2 to 3 m diameter means that fewer piles are required, and the size and mass of the pile caps can be significantly reduced, thereby greatly reducing lateral load demand during seismic events. Another advantage of using larger diameter piles is that the increased pile stiffness allows the pile caps to be located at or near the waterline, up off the bottom of the water, rather than being positioned at or below the bottom of the waterway. This elevating of the pile cap from the bottom of the waterway offers significant cost savings by eliminating the need for deep cofferdams. However, relocation of the pile cap creates new problems and opportunities in regard to the means and methods used to construct the pile cap. With the pile cap at depth, conventional steel sheet pile cofferdams with thick tremie seals are typically used to isolated the cap location and allow dewatering for construction of the cap. However with pile caps positioned up off the bottom, conventional bottom founded cofferdams are no longer cost efficient.

Float-in or lift-in cofferdams have been developed to address the need for a more cost efficient method for cap construction. This paper describes the key features of the concept and provides three case history examples of its successful implementation. In addition, examples are presented that illustrates the applicability of these means and methods to other major marine structure including dams, power houses, and airports.

PIER CONSTRUCTION USING CONVENTIONAL SHEET-PILE COFFERDAMS

Cofferdams are temporary structures used in the construction of bridge piers and other marine structures. Their primary purpose is to hold out water and unstable soil from the construction area, and thereby, allow in-the-dry construction of the permanent structure below the water line and quite often below the mud line. Typically, cofferdams consist of long interlocking steel sheet piles driven through water into the bottom of the waterway. The sheet piles form temporary exterior walls which are typically braced internally with wales and struts. The bottom of the cofferdam is typically sealed with tremie concrete or an impervious clay layer if one exists. The depth at which this type of cofferdam can be used is...
typically limited by the length of steel sheet pile that can be transported and handled by conventional marine equipment.

FLOAT-IN COFFERDAM CONCEPT

If the pile cap can be located up off the bottom of the waterway and near the waterline, the cofferdam must address different issues, some positive and some negative from a risk and cost standpoint. The positive aspects of the higher cap mean that the dewatered head acting on the cofferdam is significantly reduced and lower capacity sheet piles, internal bracing, and tremie seals are required. The primary negative aspect is that there is a vertical gap between the bottom of the waterway and the bottom of the cap that must be filled with some sort of supporting material or separate structural system. The float-in or lift-in cofferdam system was developed to address these issue.

This concept is perhaps best described by its sequence of construction (See Figs. 2 through 8 for a plan view of a typical float-in cofferdam and construction sequence):

1. Piles or drilled shaft casings are installed through the water. Following completion of the pile installation, they are cut off underwater.

2. Concurrent with pile installation, a thin shell of the structure is pre-fabricated at an off-site location. This thin shell (and follower cofferdam if needed) is referred to as the float-in cofferdam.

3. Following completion of fabrication, the float-in cofferdam is launched and towed to the installation site.

4. Once at site, the float-in cofferdam is lowered down on to the pre-installed piles.

5. A seal is then formed between the piles and the bottom of the float-in cofferdam, and grout (or tremie concrete) is placed to lock the float-in cofferdam to the piles.

6. The float-in cofferdam is then dewatered.

7. All remaining work is then completed in-the-dry similar to a conventional cofferdam. This includes completion of reinforcement and concrete placement in the piles or drilled shafts, and placing reinforcement and concrete for the cap and pier shaft.

8. After completion of the shaft, if a follower cofferdam was used, the cofferdam area is re-flooded and the follower cofferdam is removed for re-use on the next pier.

Fig. 2 Plan at top of precast pile shell
Fig. 3  Stage 1 – Pile installation and cut off

Fig. 4  Stage 2 – Launch and position float-in

Fig. 5  Stage 3 – Land cofferdam, seal pile connection and place tremie concrete

Fig. 6  Stage 4 – Dewater cofferdam and construct pile cap

Fig. 7  Stage 5 – Construct pier shaft

Fig. 8  Stage 6 – Strip follower
Float-in cofferdams are characterized by the following features:

- Use of a precast shell in the shape of a footing to act as both a cofferdam and a form for the pile cap concrete placement.
- Use of a steel follower cofferdam to extend the height of the cofferdam to allow submergence of the precast pile cap shell.
- Use of corrugated pipes or thin-walled steel pipe as pile-top bulkheads for sealing the pile holes in the bottom of the precast pile cap shell.
- Use of a guide system to ensure underwater mating of the floating cofferdam on to the pre-driven piles.
- Pre-installation and underwater cut-off of all foundation piles.
- Offsite casting of the precast shell concurrently with pile installation. Casting is typically performed in a drydock or on the deck of submersible barges.
- Floating the precast shell to the job site and positioning over the top of the cutoff piles.
- Ballasting the cofferdam to a pre-set elevation and locking the pre-cast pile cap shell to the piles by underwater placement of grout in annulus between the precast shell and the pile casing.
- Use of air pockets in the pile top bulkheads to maintain sufficient free-surface to ensure transverse floating stability of the cofferdam during launch, transport, and immersion onto the foundation piles. This allows the use of an open-flooding ballast system for adjusting draft and controlled sinking of the floating cofferdam without the need for added transverse bulkheads.
- Pile to cofferdam sealing system to allow the annulus between the precast bottom shell and the pile casing to be closed off for grouting or tremie concrete placement.
- Use of the bond-shear capacity of the grout closure between the precast bottom slab and the pile casing to resist hydrostatic uplift during the cofferdam dewatering stage prior to placement of the reinforcing steel and pile cap concrete in the dry.

**ADVANTAGES OF THE FLOAT-IN COFFERDAM CONCEPT**
Advantages of this type of cofferdam include:

1. Elimination of the need for extremely long sheet piles typically required for cofferdams in deep water.
2. Elimination of tremie concrete seal by transferring cofferdam uplift loads directly into the large foundation elements.
3. With elimination of the tremie seal, there is significant reduction in pile cap mass. This has very significant design and cost benefits by reducing lateral design loads in foundations for structures located in zones of high seismic activity.
4. Offers the potential for significant schedule reduction by starting pile cap construction concurrently with pile installation.
Case Histories of Float-In Cofferdams

Case History No.1 - Bath-Woolwich Bridge, Kennebec River, Bath, Maine

The floating cofferdam concept was first used on the Bath-Woolwich Bridge across the Kennebec River at Bath, Maine in 1998. The Maine Department of Transportation awarded the design-construct contract for the 1,000m long four lane bridge to Flatiron Structures Company (FSC) and Figg Engineers in August of 1997.

Figg’s design for the six main river piers called for two size footings: 10.9m by 9.9m by 3.6m deep supported by four 2.44m diameter steel cylinder piles socketed into rock and 9.6m by 9.6m by 3.6m deep supported by three piles of the same size. The typical footings had a 2.7m deep tremie seal and was located in 13m of water with the top of the footing about 1.4m below mean tide. This cap design significantly reduced the water depth at which the footing were to be constructed, however, it created the problem of how to construct an underwater footing suspended 6m above the river bottom.

In order to address this problem and reduce the amount of work in the river, a floating cofferdam system was proposed by the contractor. (See plan view in Fig. 2 and sequence of construction shown in Figs. 3 through 8):

• Pre-install the drilled shafts using a two stage template.

• Construct a precast pile-cap shell on shore and attach a reusable temporary steel follower cofferdam.

• Launch the cofferdam and tow it to the bridge site.

• Position the cofferdam over the drilled shafts and fix it in position with four spud piles.

• Lower the cofferdam down over the pre-installed drilled shafts with jacks located on top of the spud piles.

• Lock the pile-cap shell to the drilled shafts by placing a 1.3m deep tremie seal.

• Dewater the cofferdam and complete the pile cap and pier shaft in the dry.

• Flood and remove the follower cofferdam for reuse on the next pier.

The floating cofferdam system and the above sequence substantially reduced in-water work and improved quality control by transferring the work to an offsite dock location. The system also saved 4 months in the construction of the foundations by allowing the drilled shaft installation to start immediately after contract award and to proceed concurrently with fabrication of the floating cofferdams.

The final design for the precast pile-cap shell was a 5.182m deep box with 288mm thick walls and bottom slab.

The typical precast box weighed 340 tons and was cast on the deck of a barge at dock side. (See Fig. 9.)
Two 4.267m high steel followers were constructed of interlocking steel sheet piles and assembled on the deck of another barge. Two local shipyard cranes were used to lift the precast pile-cap shells off the deck of the casting barge and into the water. (See Fig. 11.) The precast shell was then leveled by using compressed air to vary the water level in the open bottomed 3m diameter pile top block outs. (See Fig. 12.) Once the segment was afloat and trimmed, one of the steel follower was lifted into position on top of the precast pile-cap shell and bolted to the top lip of the shell. (See Fig. 14.) The floating cofferdam was then towed into position over the top of the preinstalled piles (See Fig. 13.) and locked in horizontal position with four 1m diameter spud piles driven through the pockets, attached to the corners of the cofferdam. (See Fig. 15.)
A second key to the successful installation was the initial temporary underwater connection between the float-in cofferdam and the pre-installed drilled shafts. This connection was made with a 1.22m deep tremie seal, and the vertical load was transferred through bond of the tremie seal to the sides of the drilled shaft casing. This bond was sufficient to resist the buoyancy of the dewatered float-in cofferdam at high tide and allowed placement of the pile cap reinforcing steel and first concrete lift in the dry. (See Figs. 16 and 17.) When the footing concrete was being placed in the pile-cap shell, and the tide was out, the vertical loading was reversed, and the bond supported the dead load of the fresh concrete.
Following completion of the pile cap, the pier shaft was completed to a point above the high water line and the follower cofferdam was stripped. (See Fig. 18.)

Case History No. 2 - New Carquinez Bridge (North End of San Francisco Bay, California)

The contract for the new bridge was awarded in early 2000 to a joint venture between FCI Constructors and Cleveland Bridge California, (FCI/CBC). The float-in cofferdam system was selected by FCI/CBC to modify Caltran’s float-in cofferdam concept and further reduce the amount of onsite work.

The foundation design for the new bridge required six 3m diameter drilled shafts under each of the four main tower legs. Because of the very high lateral stiffness of these piles, it was possible to elevate the four tower footing blocks - 5.4m deep and 18m by 20m in plan - up off the bottom of the strait. At the north tower, the footing block is positioned 30m off the bottom with the lower edge of the footing at 2.34m below mean sea level. The original design by Caltrans was to cast the footing shells offsite, float them to the bridge and land them on pre-installed erection frames, or falsework. The footing shells were to be used as temporary cofferdams and as templates for the installation of drill shaft casings.

A unique feature of this float-in cofferdam was the temporary support system that allowed the float-in cofferdams to be landed directly on the pre-installed drill shaft casings. (See Figures 19 and 20 for details of the temporary support system, shown in yellow.) This modified design offered significant advantages. First, it allowed drilled shaft installation, an activity which was on the critical path, to begin sooner, by allowing casing installation to start before the float-in cofferdams had been completed. Secondly, the modifications meant that the construction schedule was shortened by allowing casing installation to run independently of and concurrently with cofferdam fabrication.
Cofferdams and connecting ties were cast on a flat-deck barge, 22m wide by 76m long and 4.6m deep, which was positioned at the dockside within reach of a shore-based crane. (See Figs. 21, 22 and 23.) For the launch operation, a site was selected in the Carquinez Strait approximately 2-km upstream from the bridge. The primary criteria were that the site should have a level bottom and a water depth at low tide of about 6.7m. The barge was towed to the site and positioned alongside a derrick barge on anchors. In order to maintain sufficient stability of the casting barge and cofferdams during the launch, the barge was ballasted down one end at a time. The stern of the barge was first set down on to the bottom during low tide, and then, as the tide turned, the bow was gradually ballasted down allowing the cofferdams to float.
free one at a time. Following launch, each cofferdam was towed to the dockside and stored afloat until the drilled shafts at the first tower were completed. (See Fig. 24.)

Caltrans set the horizontal positioning tolerance for the 3m diameter drill shaft casings at +/- 150mm, but site conditions including water depths of up to 32m and tidal currents up to 3m/s made meeting these criteria difficult. In order to meet the specified tolerances under these conditions, a steel guide template with four spud piles was installed at the south tower and a template attached to the pile driving barge was used on the north tower. As an added precaution against fit-up problems during landing of the cofferdams, the size of the blockouts in the bottom of the float-in cofferdams was increased to 3.66m. This provided a theoretical clearance around the pre-installed drilled shafts of 330mm.

The system for cutting off the drilled shafts under water included a 4.5m diameter cylindrical cofferdam that fitted over the top of the drilled shaft. The cofferdam sealed to the casing at about 1m below cut-off elevation by means of an inflatable rubber O-ring that fit inside a circular recess at the bottom of the cofferdam. Following inflation of the seal, the cofferdam was dewatered and the drilled shaft casing was cut off in the dry to a precise elevation. This cut-off cofferdam also allowed access for welding a steel ring to the exterior of the casing; the ring was used to provide the watertight seal between the float-in cofferdam and the drilled shaft casings. It also provided the necessary shear resistance for the support of the cofferdam during the concrete infill operation. (See Figs. 19 and 20 for details of the bottom seal.)

The high tidal currents in the Carquinez Strait allowed only about an hour of slack tide during which the cofferdams could be landed. Hence it was necessary to install a guide system to allow rapid positioning and landing. Because the water was more than 32m deep, the guide system was attached directly to the 3m diameter drilled shaft casings. The guides were L-shaped wide flange brackets attached to the casing just below the support bracket and extended above high tide. (See Fig. 25)
Landing support was provided by steel beams spanning across four of the bottom slab openings and bolted flush to the floor of the precast cofferdam. The actual bearing surface was the top edge of the drilled shaft casing and the underside of the steel beam. (See Figs. 19 and 20 for details of the bearing surface.)

In order to dewater the cofferdam it was necessary to create a watertight seal between the six drilled shaft casings and oversized holes in the bottom of the cofferdam. This was accomplished by pre-welding doughnut-shaped steel rings - 12mm thick by 700mm wide - to the casings just below the cut-off elevation, at the finished bottom elevation of the cofferdam. These plates were attached in-the-dry by using the same 4.5m diameter cofferdam used for casing cut-off.

At the outer edge of the steel doughnut, an inflatable O-ring was bonded to the plate. As the void space around each casing was dewatered the hydrostatic pressure pushed up on the bottom of the steel plates and the rubber O-rings provided the necessary seal. In areas where the seal did not seal properly, vertical
hanger bolts were available to pull the 12mm plate tight to the bottom of the cofferdam and compress the seal. (See Fig. 20 for details of the seal.)

The initial support system for landing the cofferdams was simple and allowed a quick landing, but it was directly in the path of both the vertical extension of the drilled shaft reinforcement and the horizontal reinforcement in the bottom footing mat, hence it had to be replaced before the reinforcement could be installed. Final support was provided by placing infill concrete in the void spaces between the 3m diameter drilled shaft casings and the oversized 3.6m diameter holes in the bottom of the float-in cofferdam. (See Fig. 20 for a detail of the void space.) The 3.8m diameter cans positioned over the top of each hole provided access to this area and the seal system described above allowed the cans to be dewatered and the infill concrete to be placed in-the-dry. (See Fig. 27, showing removal of the lids of the pile top bulkheads.) Hanger rods 25mm in diameter and spaced at 600mm along the center of the void space provided shear reinforcement and the fillet weld connecting the 12mm plate to the drilled shaft casing provided the necessary shear resistance.

After the concrete in the void space had reached the specified strength, the entire cofferdam was dewatered, the 3.8m diameter cans were removed from each drilled shaft location, and the initial support beams were removed. At this point the area over the top of each drilled shaft casing was free and open for extending the drilled shaft vertical reinforcement up into the footing, and installation of the footing bottom reinforcement mat. (See Fig. 28 for a view of the cleared area inside the float-in cofferdam.)

Case History No. 3 – Port Mann Bridge across the Fraser River, British Columbia, Canada

The float-in cofferdam for the Port Mann Bridge had the following distinctive features:

- **Size** - It is the largest float-in cofferdam ever installed – 109-ft by 138-ft by 32-ft high and weighing in excess of 6,000 tons.
- **Number of Foundation Piles** - It was installed over the top of 63 each 6-ft diameter pipe piles cut-off 10-ft under water.
- **Use of Pre-Installed Pre-Cast Support Collars for Initial Landing of Cofferdam** – Prior to underwater cut-off, each pile was outfitted with a precast collar that was hung from the top of the pile driving template and grouted to the outside of the pile.
- **Load Distribution System During Cofferdam Landing on 63 Piles** - In order to ensure anyone collar was not overloaded by the 6,000 ton cofferdam during landing, a 3”-pipe ring with an outside diameter of 9.43-ft was prepositioned underwater on top of each precast support collar.

The new bridge across the Fraser River at Coquitlam, British Columbia was started in early 2009. The main in-water tower of the cable-stayed bridge is located mid channel in 45-ft of water. Tidal variation at the site is approximately 14 feet. The final foundation design for N-1 Pier consisted of 63 each 6-ft diameter driven pipe piles. The tops of the piles are embedded in a 24.6-ft deep (138.13-ft by 108.60-ft) pile cap positioned just below the High Water Elevation of +9.25.
The original pre-bid cofferdam design was a conventional cofferdam using AZ-36 sheet piles up to 107-ft in length. The gap between the bottom of the pile cap and the slopping river bottom (which varied from 21-ft on the upslope side to 45-ft at mid channel) was to be filled with clean granular fill and topped with 6.56 feet of gravel.

Following contract award, the Joint Venture evaluated the float-in cofferdam concept for the N-1 Pier and concluded that it offered significant cost savings over the pre-bid conventional cofferdam.

**UNIQUE FEATURES OF THE PORT MANN FLOAT-IN COFFERDAM**

While the Port Mann cofferdam has the common features of other float-in cofferdams, it has several unique features including:

- **Cast & Launch Method** – The Port Mann cofferdam was cast on the deck of two barges in North Vancouver Harbor and launched from the deck of a floating drydock. See Figure 1.0 – Cofferdam on two-barge casting bed under tow to floating drydock.

- **Cofferdam Size** - It is the largest float-in cofferdam ever installed – 109-ft by 138-ft by 32-ft high and weighing in excess of 6,000 tons. See Figure 2.0 – Pier N-1 plan view at top of footing form, Figure 3.0 – Plan view at top of follower cofferdam, and Figure 4.0 - Transverse and longitudinal sections of floating cofferdam.

- **Number of Foundation Piles** – The pre-cast cofferdam was installed over the top of 63, 6-ft diameter pipe piles cut-off 10-ft under water.

- **Use of Pre-Installed Pre-Cast Support Collars for Initial Landing of Cofferdam** – Initial support for the cofferdam was provided by a support collar grouted to each of the 63 foundation piles. See Figure 5.0 – Detail of Precast Support Collar.

- **Load Distribution System During Cofferdam Landing on 63 Piles** - In order to prevent overloading of any single support collar by the 6,000 ton cofferdam during landing, a pipe ring positioned on each support collar was used to cushion the cofferdam landing.

- **Pile Cap Size** – 24.6 feet deep with a total volume of 10, 400 CY.
Figure 29 – Cofferdam on two-barge casting bed under tow floating to drydock

Figure 30 – Pier N-1 plan view at top of footing form
Figure 31 – Plan view at top of follower cofferdam

Figure 32 – Transverse and longitudinal sections of floating cofferdam
CAST AND LAUNCH OF COFFERDAM

The cofferdam was cast on the deck of two cargo barges that had been welded together to form a single casting bed. The two-barge casting bed was positioned dock side in the North Vancouver Harbor where it was serviced by land-based cranes for forming, rebar and concrete placement. Following casting operations, a 16-ft high steel follower cofferdam constructed of steel plate concrete form panels and internal steel bracing was installed on top of the precast cofferdam shell.

After installation of towing attachments and ballast system, the two barges with the completed cofferdam were positioned inside a floating drydock. The drydock was deballasted up, lifting the two barges and cofferdam clear of the water. Once the hulls of the two barges were clear of the water, pre-installed valves in the external hulls of the barges were opened to allow internal flooding of the barge hulls, and the drydock was ballasted back down. As the drydock was lowered and the two-barge casting deck went underwater, the cofferdam floated free of the drydock and was towed back to dockside for final outfitting.

TRANSPORT AND OUTFITTING

16
Following completion of outfitting, the cofferdam was towed out under the Lions Gate Bridge at the harbor entrance and into Strait of Georgia. It was then towed south 15 miles to the mouth of the Fraser River and towed approximately 36 miles up the Fraser River to the bridge site. See Figure 6.0 – Floating cofferdam positioned over 63 foundation piles.

**Figure 34** – Floating cofferdam positioned over 63 foundation piles

**SUPPORT COLLARS FOR LANDING OF COFFERDAM**
Prior to underwater cut-off, each pile was outfitted with a 10.27-ft O.D. precast support collar. The collars were suspended from the top of the pile driving template with calibrated rigging and threaded rods to provide exact elevation adjustment. The inside diameter of the collars was 6.33ft which provided a theoretical annulus of 2 inches. Once a collar was at the required elevation, the top and bottom of this annulus was sealed by inflating rubber tubing embedded in the inside wall of the pre-cast collar. The sealed annulus was then filled with grout through pre-installed grout and vent hoses. Return of high quality grout through the vent hoses ensured complete grouting of the collar and prevented any contaminated discharge from entering the river.

In order to ensure that no single support collar was overloaded by the 6,000 ton cofferdam during the initial landing stage, the top of each collar was equipped with a 3 inch diameter tube ring with an outside diameter of 9.43-ft. These rings were pre-attached to each collar prior to installation of the collars.

**GUIDE SYSTEM FOR MATING COFFERDAM TO FOUNDATION PILES**
The landing of the cofferdam onto the support collars necessitated ballasting the cofferdam from an initial draft of 6.1-ft to a final draft of about 25ft. At a draft of about 12ft, the tops of the foundation piles began to thread up into the bottom of the cofferdam. The inside diameter of the corrugated pile top bulkheads through which the sixty-three 6-ft diameter foundation piles were threaded during the landing operation
was 8 ft. See Figure 7.0 – 8-ft Diameter Pile Top Bulkheads. This provided a theoretical annulus clearance of 12 inches between the inside face of the pile top bulkheads and the 6-ft diameter foundation piles.

![Figure 35 – 8-ft diameter pile top bulkheads](image)

In order to ensure that the foundation piles cleared the bottom openings and did not punch a hole in the bottom of the cofferdam, 4 guide dolphins were installed and equipped with guide shims for precise alignment of the floating cofferdam over the tops of the pre-driven piles. The cofferdam was positioned against the four dolphins on a falling tide and ballasting was commenced by opening four 12in diameter gate valves built into the exterior walls of the precast cofferdam. The four valves were left open until the bottom of the cofferdam reached a position about 4 inches above the tops of the 63 cut-off piles. Divers were then used to check the clearance at each pile location to ensure adequate clearance between piles and the 63 openings in the bottom of the cofferdam. After confirmation, the tide was then used to thread the cofferdam onto the piles.

Stability of the cofferdam was maintained during the ballasting operation by ensuring that each of the 63 pile top bulkheads contained a minimum volume of air at all times during the cofferdam lowering operation. Once the cofferdam was landed on the 63 support collars, the air in each compartment was bleed off through vent valves at the top of each corrugated pile top bulkhead.

**LOCKING COFFERDAM TO FOUNDATION PILES**

Once the cofferdam had landed, divers were used to remove the pile top bulkheads and tremie concrete was pumped into the 36-in deep 12-in annuluses between the piles and the 8-ft diameter holes in the bottom of the cofferdam. After the tremie concrete attained a minimum strength of 3,000 psi, the interior of the cofferdam was dewatered with 2 each 10-in and 3 each 4-in electrical submersible pumps.
Following dewatering, the interior of the 63 piles were excavated to the specified depth and filled with concrete placed in the dry. See Figure 8.0 – Dewatered cofferdam locked to 6-ft diameter piles.

While the grout was curing, the cofferdam remained vented to the river in order to maintain a relatively constant load on the collars as the river elevation changed with the tides. However, this created an environmental challenge, because the pH of the water within the cofferdam had to be maintained within tight tolerances stipulated by the project’s water discharge permits. After extensive analysis, a specialty designed pH monitoring and treatment system was installed to continually remove, test, treat and then return the water to the inside of the cofferdam such that the water inside the cofferdam never exceeded the allowable discharge levels.

CASTING OF PILE CAP
The 24.6-ft depth of pile cap concrete was placed in two lifts, with the bottom mat, walls, half length hairpins, and pylon rebar installed prior to the 1st lift. See Figure 9.0 – Placing first lift of the N-1 pile cap. The other half of the hairpins, top mat support frame and top mat rebar were installed prior to pouring the 2nd lift. Both lifts were considered mass concrete and required a thermal control plan, cooling tubes, thermal sensors, and monitors throughout the entire placement. Total volume of concrete placed was 10,400 CY. For a complete construction sequence of the float-in installation of the cofferdam and casting of the pile cap, see Figure 10.0 – Construction sequence.
Figure 37 – Placing first lift of N-1 pile cap
Figure 38 – Construction sequence
APPLICATION OF CONCEPT TO OTHER STRUCTURES

Dams – Braddock Dam

The new gated dam on the Monongahela River at Braddock, Pennsylvania, was built for the US Army Corps of Engineers using an innovative “In-the-Wet” technology. A key feature of this technology involves construction of the foundations through the water and later mating the completed structure or a shell of the structure to the pre-installed foundation. This paper presents a description of the structure, its foundation, construction sequence and design details required to successfully implement this technology.

Locks and dams in the United States have historically been built by blocking off large sections of the river with circular-sheet-pile cells; dewatering the area surrounded by the cells; and then proceeding with foundations followed by the superstructure. After completing one segment of structure, the area enclosed by cells is again flooded, the circular cells are moved and another section of the river is dewatered to allow a continuation of construction on the adjacent sections. This process is continued for multiple cycles until the entire lock or dam is completed. (See Fig. 1 for an example of this type of construction.)

Figure 1 - Conventional Cofferdam Construction.

This method has been successful in the past, but suffers from a number of drawbacks:

- The large temporary cofferdams are expensive to build and remove;
- They restrict river flow and impact navigation (This is especially true when replacing an existing navigation structure.)
• Drying up extensive areas of the river bottom requires construction and operation of large expensive dewatering systems;
• If the structure is located on ground that contains contaminates, dewatering can spread contaminates in the ground and create a need for treatment of the water once collected by the dewatering system.
• The large temporary cofferdams are typically in place for 2 or more years and are subject to overtopping and flooding of the work area during periods of high water.

An alternative to this approach is a method referred to as “float-in” or “in-the-wet” construction that involves constructing the foundation for the structure through the water and then floating in or lifting in a shell of the structure and mating it to the pre-installed foundation. This technique has been in use for centuries. Early examples include float-in caissons used to build breakwaters and quay walls. Over the last 100 years, 125 immersed tube tunnels have been built around the world using this technique. It has been used to build paper mills up the Amazon River, power plants on the Mississippi River and large off-shore gravity base structures in the North Sea and Arctic Ocean.

The Pittsburgh District of the US Army Corps of Engineers in an effort to develop alternate and more cost effective construction methods for their navigation structure on the inland waterways of the US selected this method of construction for replacement of Dam 2 on the Monongahela River, eleven miles upstream from Pittsburgh at Braddock, Pennsylvania. See Fig. 2 for an aerial view of the dam site. The Corps developed the initial concepts and then selected a team consisting of Ben C. Gerwick, Inc of San Francisco, Bergmann Associates of Rochester, New York, and D'Appolonia of Pittsburgh, Pennsylvania, to perform the detailed design of the new 720-foot long navigation dam.

![Planned Axis of New Gated Dam](image)

Figure 2 - Braddock Dam Site.

See Fig. 3 for a location map of Braddock Dam.
Figure 3 - Location of Braddock Dam.

The Corps initial "In-the-Wet" concepts envisioned using off-site prefabrication of thin concrete shells (or segments). The segments were to be built on a large barge in a two-level casting basin, or in a drydock, floated into final position over pre-installed foundations, and then locked onto the foundations by underbase grouting and infilling of the segments with tremie concrete. These concepts offered several advantages over conventional "In-the-Dry" construction:

- Less disruption to river navigation and river flow;
- Lower cost of construction through the elimination of conventional large sheet pile cofferdams and site dewatering;
- Shorter construction time by allowing concurrent construction of dam segments and dam foundations;
- Less environmental impact by reducing dredging and eliminating site dewatering;
- Higher quality by allowing the use of precast concrete produced in a controlled on-shore environment.

The final design developed by the Gerwick-Bergmann-D’Appolonia team was constructed by a joint venture of JA Jones and Traylor Bros. The final structure consisted of two large prefabricated concrete segments that were floated into place, and set-down on a pre-constructed foundation. The float-in section of the new dam was a 600-ft long by 104.5-ft by 42-ft high structure with four gate bays and one fixed-crest weir bay. The final "In-the-Wet" design called for breaking the dam into two segments of 333-ft and 265-ft. The segments were constructed as closed bottom boxes in a two level casting basin. The exterior walls, bottom and top slabs of the boxes were 12-in thick and the interior walls were 10 and 12-in thick. The bottom slabs were post-tensioned and contained recesses to allow the pre-installed foundation to penetrate the bottom of the slab during set-down of the segments.

As each segment was completed, it was launched by flooding the basin and then towed to the outfitting pier two miles upstream of the dam site for final outfitting. The dam segments were then floated downstream to the dam site and positioned over the foundation caissons with a mooring system mounted on top of each dam segment. The foundation system consisted of sheet-pile cut-off walls upstream and downstream, H-piles below the tailrace area and 72 inch diameter drilled shafts installed in a pre-
excavated area upstream of the existing dam. Each segment was ballasted down onto 6 landing shafts and leveled with flat jacks installed within the piers of the float-in dam segments. The under-base was then grouted, and 8-ft of tremie concrete was placed in the segment compartments. Each compartment within each segment was then dewatered and the remainder of the dam including tainter gates was completed in the dry. After all cells within a given segment had been filled with concrete, the dam segment was locked on to the tops of the drilled shafts by grouting the pile-top annulus.

Sequence of Construction

Construction of the dam was carried out in seven stages. See construction sequence stages 1 through 7 below:

Stage 1 – Cast segment in two level casing basin.

Stage 2 – Launch and tow segment to outfitting pier.

Stage 3 – Position segment over pre-installed foundations.

Stage 4 – Ballast and land segment.
First Stage Excavation and Bottom Preparation

Dredging was performed in two stages. The first stage of pre-excavation for the dam foundation was performed below the footprint of the dam and consisted of excavation the river bed to El. 690, from the existing lock river wall to the left bank abutment toe in a strip about 140 feet wide. The depth of the excavation ranged from 14 feet to 32 feet across this reach. The excavation was inclined up the existing river bed grade at a slope of 1 on 10 upstream and 1 on 5 downstream to El. 702. Dredged materials were transported by barge to the upstream disposal site.

Cut-Off Walls

After pre-excavation, steel sheet piling were installed to provide both upstream and downstream cut-off walls and to serve as retaining walls for various stages of work on the dam. A prerequisite pile driving program was used to determine the elevation of rock to which sheets were to be driven. Sheets were ordered to accurate lengths once these elevations were determined. The upstream cut-off wall, located 3-ft from the upstream face of the future dam, was installed first. This 600-ft long wall was installed in two stages using a barge mounted pile driver. The first stage involved installation of 75-ft long H-piles on a
19-ft spacing. These H-piles were driven to the top of rock and were outfitted with pre-installed interlocks welded to one flange. At completion of driving, the top of the H-piles and the interlocks extended above water and provided a guide for installation of 18-ft wide sheet pile wall panels between the H-piles. The sheets were driven to final grade, approximately 29-ft under water, and thereby eliminated the need for divers to cut the sheets off under water.

The downstream cut-off wall was then installed in a similar manner. The downstream cut-off wall consists of a structural system of 24-inch diameter pipe piles and sheet piles. Pipe piles were used rather than H-piles because the pipes supported the downstream end of the precast tailrace panels, and this wall system was required to resist the loads imposed by the retained alluvium when the downstream face of the wall was excavated to rock for installation of scour protection. The pipe piles were first driven and a 6-ft deep reinforced concrete rock socket was installed in each pile tip. The sheet piles were connected to the pipe piles by a special interlock section that was pre-welded to each side of the pipe pile prior to driving.

Second Stage Excavation, Bottom Preparation, Dam Foundation System

With the cut-off walls in place, the area between the walls was excavated an additional 8 ft to El. 682, and a 12 inch layer of 1.5 inch stone placed and levelled. Eighty nine drilled shafts were used to support the dam piers and gate sills. Two rows of H-piles on 8-ft spacing driven to the claystone rock layer were used to support the dam tailrace. The tops of the H-piles extended up into the first tremie concrete poor below the tailrace slabs.

Drilled Shaft Foundations

The base of the dam is at El. 683.7 ft which is about 38 ft below the normal pool level and 15 ft below the existing riverbed. The stratigraphy below the dam consists of approximately 16 ft of alluvium comprised of sandy gravel (GM) to silty sand with gravel (SM). Below this layer is soft to medium hard clay shale and claystone down to El. 658 ft. Medium hard to hard siltstone is encountered between El. 658 ft and 626 ft. The drilled shafts for the dam were founded in this layer. The average unconfined compressive strength of the clayshale is 840 psi and the siltstone 3600 psi. Two test shafts where installed and loaded both laterally and vertically. See Fig. 4 for a stratigraphic section of the river bottom relative to the river bottom and the drilled shafts.

Figure 4 - General Geologic Section.

Figure 5 - Layout of pre-installed drilled shafts.
See Fig. 5 for a layout of the 89 drilled shafts under the dam.

Two types of drilled shafts were installed under the main body of the dam:

- Six set-down drilled shafts under each of the floating-in dam segments provided initial support of the float-in dam segments at set-down;
- Seventy seven foundation drilled shafts provided long-term support of the completed dam.

See Fig. 6 for a schematic of the two drill shaft types.

![Two Types of Drilled Shafts](image)

Figure 6 – Two types of drilled shafts

All drilled shafts were step-tapered with a 72” diameter rock socket drilled through a 78” diameter permanent steel casing. Permanent casings were driven and seated into the top of the claystone rock layer. Drilling removed all material from within the casing and the 72” diameter rock socket were then drilled roughly 6 feet into the lower siltstone rock layer.

All drilled shafts were positioned using a four-legged two-level template. The fixed lower template was installed first to a tolerance of +/- 12 inches, and the sliding upper level template was set and fixed into position to a tolerance of +/- ¼ inch. Casings were then installed and barge mounted equipment performed the drilling. After the drilled shafts were thoroughly cleaned out, a steel reinforcing cage was installed, and the shaft was filled with tremie concrete to an elevation approximately 6 ft below the bottom of the future dam.

After the tremie concrete attained sufficient strength, the casings were dewatered and latent concrete was manually removed in-the dry. A tension-anchor/shear-pin assembly consisting of a 14” steel wide flange tension anchor and 36 inch diameter pipe shear pin was then installed and fixed in position by concrete placed in the dry. The top of each casing was then cut-off to a vertical tolerance of +/- ¼ inch using a temporary circular cofferdam that was installed over the top of the drilled shaft casing and sealed to the outside face of the casing below the cut-off point using inflatable rubber seals. The annulus between the casing and the temporary cofferdam was then dewatered and the drill casing was cut off in-the-dry from the inside the casing.

The 12 Set-down drilled shafts were generally constructed in a similar manner to the foundation drilled shafts. However, instead of installing a tension-anchor/shear-pin assembly, a levelled grout surface was constructed at the bottom elevation of the dam. A tolerance of +/- 1/8th inch was obtained by milling the
top surface of the drilled shaft concrete. The casing was then cut-off underwater using the same technique described above.

**Precast Shell Fabrication, Launch Transport and Outfitting**

While the dam foundations were under construction, the two float-in dam segments were constructed at a two level casting basin located at a 15-acre site 27 miles downstream from Braddock on the Ohio River. The two level casting basin allowed the segments to be cast on an upper level, a few feet above normal pool elevation in a protected area behind an earthen berm. See Fig. 7 for an aerial view of the casting basin and launch site at Leetsdale, PA.

![Figure 7 - Cast and launch site.](image)

The total 600-ft length of float-in dam was divided into two segments. The 333-ft long Segment 1 weighed approximately 12,000 tons and the 265-ft long Segment 2 weighed approximately 10,000 tons. The float-in segment included: the gate sills, a portion of the stilling basin, and the pier bases up to El. 726.0. Segment No. 1 included the fixed weir bay, the water quality bay and one of the standard gate bays while Segment No. 2 included two standard gate bays. All bays were 110’ wide. The joint between Segments 1 and 2 occurred at Pier 3, which made it 11 feet wider than adjoining piers to facilitate this connection.

The walls and diaphragms of the float-in segments were precast panels. The top and bottom slabs were cast-in-place concrete. The largest individual precast wall panels were 21 feet by 30 feet and weighed approximately 80 tons. All wall panels were tied together with cast-in-place closure pours positioned at the intersection points of the walls. Sixty-inch diameter corrugated sleeves were cast into the bottom of the dam segments over the connection points to the tension-anchor/shear-pin assemblies installed in the tops of the drilled shafts. After erection of the wall panels, installation of post-tensioning ducts and bottom recess sleeves, the bottom slab was cast in place. See Fig. 8 for a view of the erected precast wall panels and corrugated recess sleeves used to provide connection to the drilled shaft tension-anchor/shear-pin assemblies.
When the precast dam segment was ready for launch, the upper and lower basins were flooded to a common water level and the segments were floated one at a time over the lower basin. The water level in the lower basin was then returned to the outside river elevation, the exit sheet-pile wall was removed and the segment was towed out of the casting facility. The segments were completed to a partial stage such that the maximum draft of the segments did not exceed 11 feet. Segments were transported from the off-site assembly location to the project site for additional outfitting prior to set-down. All handling and transport were by towboats.

Each segment with a draft of approximately 11 feet was then towed to the outfitting site upstream of the dam site. Transport of the segment was performed using a primary tow boat and guided as necessary by a snubbing tow. The segments were transported through two locks on the Ohio River, past the city of Pittsburgh, and up the Monongahela to a site upriver of the Braddock Dam site. See Fig. 9 for a view of the Segment No. 1 under tow.

Once a segment arrived at the outfitting site, it was moored to the outfitting pier. The outfitting pier consisted of a system of circular sheet pile cells and arcs, and included a 15-ft high braced fendering system to keep the segment on the face of the pier during high water.
Each segment was completed and readied for set-down at the outfitting pier. The piers of each delivered segment were extended approximately 21 feet. Temporary bulkheads for immersion were added both upstream and downstream on each gate and weir bay. Additional permanent concrete ballast, ballast piping and portions of the mooring and alignment equipment were also added. Work platforms and vertical tremie pipes for post-set-down underbase and concrete infilling operations were installed. Once completely outfitted, each segment had a draft of approximately 14 feet. Additional water ballast was added as necessary to trim the segment before transport to the project site.

**Positioning and Guidance Systems**

After completion of outfitting on a given segment, it was maneuvered down river by a primary tow boat and guided by a snubbing tow boat. After positioning close to the set-down site, 8 mooring lines from winches mounted on top of the segments were connected to the drilled-shaft anchors pre-installed upstream of the dam site. The segment was then rotated transverse to the current, winched downstream, and then positioned directly over the top of the prepared foundation. The segment was then aligned with two horn guides located on the face of the river wall. See Fig. 10 for an aerial view of Segment No. 1 being positioned at the dam site.

![Figure 10 - Segment No. 1 being positioned at site.](image)

**Ballast Systems**

After each outfitted segment was maneuvered into position, it was ballasted onto its 6 set-down drilled shafts. Water was first added to the interior compartments of the segment and as the segment sank deeper into the water, ballast water was added to the individual bays. As ballasting progressed, all interior tanks were continuously sounded to monitor free surface effects and maintain hydrodynamic stability of the segment at all times.

By adding ballast water to the gate bays and using the horn guides, mooring lines and land based survey control for guidance, each segment was accurately lowered into position and the tension-anchor/shear-pin assemblies were threaded into the hollow recesses built into the bottom floor of the dam segments. Ballast water was then fed into the dry piers until the hydraulic jacks register the required initial loading. See Fig. 11 for a view of the ballast system positioned on top of Segment No. 1 during float-in.
Figure 11 - Segment No. 1 with ballast system on the top deck.

A hydraulic ram on the downstream corner of the Segment 1, adjacent to the lock wall and in conjunction with the mooring lines, was used to align the segment on its longitudinal axis.

Segment Leveling System

Each segment was landed on 6 set-down drilled shafts. These drilled shafts were laid out to provide two support points on the longitudinal centerline of each dam pier. In order to provide even support and level the segment to the specified tolerances, two hydraulic pistons were built into each pier. When a given dam segment was properly aligned and landed, the 6 pistons were directly aligned over the top of the 6 set-down drilled shafts. The pistons were 36” diameter steel pipe sections with end caps and were built into the floor of the pier, flush with the bottom of the dam segment. The pistons were fitted with rubber seals to prevent water leakage and their sides were greased prior to placing the bottom slab concrete. A pair of 36” diameter, 1000 ton capacity flat-jack was stacked on top of each piston and the upper surface of the flat-jack was in direct contact with a concrete reaction beam spanning the 12-ft width of the pier shell. The 6 flat-jacks were connected into three hydraulic circuits to provide a three-point support system for each segment.

Once a segment was levelled to the specified tolerance, the flat jacks were “locked-off” and the hydraulic fluid in the jacks was replaced with a high strength 2-part resin.

Under-Base Grouting

Following set-down, the 12-inch deep void area between the underside of the dam segment and the preleveled, stone-covered river bottom was filled with grout to eliminate flow below the dam. To make the underbase grout placement more manageable, the area under the segment were divided into five 70-foot wide transverse strips by using inflatable grout bags pre-attached to the underside of the float-in segment prior to set-down. Both transverse and downstream bags were used. First, the space between the upstream cut-off wall and the dam was sealed to stop flow beneath the segment. Then each 70 foot wide strip was filled with grout using the vertical grout pipes installed at the outfitting pier. Typically, each grouting area contained six rows of 8-inch diameter grout pipes spaced at a maximum of 21-ft on center. Underbase grouting was started at the downstream row of grout pipes and continue upstream until the
space below the dam was completely filled. Grouting was stopped when grout filled the 8-ft high by 3-ft wide vertical gap between the dam shell and the upstream cut-off wall.

**Structure Infill**

Once all the underbase grouted had attained minimum strength, the dam was infilled with concrete which acted compositely with the hollow segment. Thirty two compartments in Segment 1 were filled with concrete in a two stage operation. The first stage placement consisted of filling the bottom 8 feet of each compartment continuously with tremie concrete while the segment was fully flooded. The second stage placement was placed in-the-dry after the first stage tremie concrete had cured and the compartment had been dewatered. Typically, each compartment had one or two 10 inch tremie pipes. The downstream compartments also had 8 inch evacuation pipes in each corner.

After infilling was completed, the dam segment was “locked” onto the foundation drilled shafts by pumping a sand/cement grout into the hollow recesses surrounding the tension-anchor/shear-pin assemblies.

**Pier Completion Tailrace Construction and Gate Installation**

The upper portion of the dam piers between EI 726 and EI 765 was placed in-the-dry after the float-in dam segment were set-down and filled with concrete. This portion of the dam piers was completed with a combination of precast concrete and conventional cast-in-place concrete.

Trunnion girders were attached to the downstream face of each of the five piers for support of the four tainter gates, the trunnion girders were constructed of cast-in-place concrete.

While the piers were being constructed, the dam’s tailrace was constructed in-the-wet. The tailrace was built utilizing precast panels that were 30’-6” wide by 20’-0” long by 15” thick. The upstream end of the panels was supported on a ledge cast in to the downstream edge of the dam segments and the downstream end of the panels was supported on the row of previously installed 24” diameter pipe piles, which were integral with the downstream cut-off wall. The area below the tailrace panels was filled with tremie concrete to create a mass concrete tailrace section supported by the previously installed H-pile foundation system.

Following the completion of the pier and tailrace structures and prior to installation of the left closure weir, the 220-t tainter gates were installed by floating in the assembled gates one at a time on the deck of a barge. Once the gate was located in its final position, the two hydraulic operation cylinders were installed. The cylinders were connected to each end of the tainter gate and to the cylinder girders which are anchored to the upper pier structure.

See Fig. 12 for a schematic of the completed dam.
Figure 12 - Schematic diagram of the complete dam and closure cells.

- Power Houses
- Airports – San Francisco International Expansion

Conclusion

Bibliography

10. BERGMANN ASSOCIATES., 3D Animated Video, 44 minutes, January 1999, (also part of the Construction Contract Documents).


12. MILES, W.R., and KARAFFA, W. In-the-Wet Construction of a New Gated Dam Braddock Locks and Dam, Monongahela River, PA.