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Engineer Technical Letter
No. 1110-2-565   30 September 2006

Engineering and Design
FOUNDATION ENGINEERING: IN-THE-WET DESIGN AND
CONSTRUCTION OF CIVIL WORKS PROJECTS

1. Purpose.

This engineer technical letter (ETL) provides guidance for planning, design and construction of foundations constructed in-the-wet for civil works structures. This ETL concentrates on successful methods and potential problems. Case histories are provided.

2. Applicability.

This ETL applies to HQUSACE elements, major subordinate commands, districts, laboratories, and separate field operating activities having responsibilities for the planning, design, and construction of civil works projects.

3. Distribution Statement.

Approved for public release; distribution is unlimited.

4. References.

References are listed in Appendix A.

5. Background/Discussion.

   a. Traditionally, marine structures have been constructed in-the-dry within a cofferdam using a dewatering system. An alternative method, with potential cost savings, is in-the-wet construction. To extend foundation preparation and construction technology to in-the-wet conditions introduces a new level of difficulty.

   b. This ETL will furnish guidance for the planning, design, and construction of foundations constructed in-the-wet for civil works structures. This document covers foundation types, subgrade preparation methods, test programs, positioning systems, installation techniques, quality control procedures, tolerances, case histories, and lessons learned.

6. Action.

The enclosed guidance should be used for the planning, design, and construction of in-the-wet foundations for civil works projects.

FOR THE DIRECTOR OF CIVIL WORKS:

1 Appendix   M.K. MILES, P.E.
(See Table of Contents)   Acting Chief, Engineering and Construction
                                    Directorate of Civil Works

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APPENDIX A
Foundation Engineering:
In-the-Wet Design and Construction of Civil Works Projects

A-1. Background

a. Constructing foundations in-the-wet has always presented challenges, uncertainties, and risks. Nevertheless, working in-the-wet presents not only difficulties, but also unique opportunities. Man has been dealing with both these difficulties and opportunities to install structural foundations in-the-wet since prehistoric times, and each new advance in foundation technology has resulted in the construction of ever more demanding foundations built in-the-wet, while keeping the level of risk at, or below, the threshold of acceptance for each new era of building.

b. The Neolithic lake-dweller culture, which peaked in 5000 B.C. in what is now Switzerland, utilized timber pile foundations driven in shallow water to support platforms for houses and other village buildings. When faced with difficult foundation conditions, this ancient culture often took advantage of marine transport to carry canoe loads of stone to dump around and to shore-up the timber piles.

c. By 1500 B.C., the people of both northern India and Mesopotamia were excavating deep foundations for bridges to provide year-round transportation across seasonally flooding rivers. These Bronze Age cultures utilized technology learned from well construction to shore-up the holes with bricks and stones, and utilized divers and buckets to assist with underwater excavation. One of the great challenges for these early, over-water, bridge foundations came from scour during floods, which these people partially addressed by the use of scour stone, frequently facilitated by marine transport.

d. By the Roman Era, engineers had developed crude crib-like cofferdams (two parallel walls of timber filled with clay) to enable the Romans to place pozzolanic concrete footings below water. Furthermore, the Romans made use of battened timber piles for the first time to resist lateral river forces. The Romans also used concrete placed underwater on a relatively massive scale in the construction of the breakwater for the Herodian artificial harbor in Israel. It appears that the Romans formed large concrete foundation blocks for the breakwater by sinking large timber forms with stone, and filling in the interstitial spaces between the stones with pozzolanic concrete placed by divers, with the forms and pozzolan being transported by water from Italy.

e. By the beginning of the Modern Era, in the latter half of the 18th Century, engineers such as Eads and Roebling were using the pneumatic caisson method to build major marine bridge foundations, with these large caissons being floated into position prior to sinking. Other marine foundation advances of the early Modern Era include the use of tremie concrete, beginning around the time of the Civil War, and the open caisson method, which was used extensively for building foundations on the Mississippi River.

f. In recent times engineers are minimizing risks: of delays, of cost over-runs, of claims, and of not being prepared to deal with changed subsurface conditions, by using advanced con-

struction equipment and techniques, by minimizing the use of personnel, and by maximizing the use of prefabrication. Availability of large floating equipment has encouraged modern in-the-wet engineers to use large driven piles and drilled shafts (often socketed into rock). Indeed, offshore equipment has been used to install large diameter steel, concrete, and composite cylinder piles for major foundations in deep water and in difficult soils, safely, rapidly, and economically.

g.   Other modern in-the-wet foundation methods include the use of sunken prefabricated steel (and concrete) box caissons, in-the-wet slurry wall cofferdams (such as for the Kawasaki Ventilation Structure in Japan), and gravity base foundations. In the future, in-the-wet foundations will likely rely more on new materials, such as very high strength concrete, advanced equipment, such as new underwater robotic equipment, and new techniques, such as advanced soil mixing and advanced soil consolidation and compaction.

h.   Not unlike these past cultures and engineers, the challenge facing the modern engineer designing in-the-wet foundations is to minimize uncertainties by adequate investigation, and to minimize the risk, and the consequence of potential failures, by establishing criteria that result in redundant, flexible, and adjustable foundation designs that have benefited from the lessons learned from the past. The following paragraphs are provided to assist the modern engineer in carrying on the tradition of past marine foundation success, to overcome the many challenges of in-the-wet foundation construction.

i.   Much of the information in this document is taken from ERDC/GSL TR-01-24. The Louisville, New Orleans, Pittsburgh, St. Louis, and Nashville Districts prepared the information in Paragraph A-11. Ben C. Gerwick, Inc., prepared the remainder of the information in this document.

A-2.   Design Considerations.

a. Introduction.

(1) In-the-wet foundations have special design considerations, including: i) the need for extensive subsurface investigation, ii) the need for extra contingencies, redundancy, and flexibility related to tolerances; iii) more difficult field quality assurance (QA); iv) specialized equipment and customized construction schedules; and v) different repair and mitigation techniques and measures. For example underwater rock excavation requires customized equipment and a carefully planned execution plan, and it can be difficult to inspect the exposed rock foundation for defects and damage caused by blasting, or other excavation methods. Such issues can require measures such as: i) limiting blasting charge sizes to minimize potential damage; ii) using geophysical and acoustical imaging equipment to locate any defects or damage; and iii) potentially using extra redundancy, or repair and mitigation measures to address potential consequences of unidentified foundation defects or damage. The following sub-paragraphs provide additional discussion of such issues.

(2) Also, as indicated in the Paragraph A-2, the foundation engineer should seek to learn from the past, including extensive use of the experience gained by experts on Peer/ITR review panels. Experts should be selected from both the USACE (or other Federal agencies), academia,

and the private industry, and could be selected from the Inland Waterways community and from other related groups, such as the offshore, bridge foundation, immersed tubes, and port and harbor works communities.

b. *Tolerances.* In-the-wet construction typically involves mating the structure to the foundation. The shell of the structure is floated-in or lifted-in over the top of the foundation. Final positioning and landing of these shells usually involves underwater mating to multiple foundation elements. This mating is usually done without visual confirmation of clearances during the mating process, and it is therefore essential that tolerances are adequate to meet these difficult conditions.

(1) *Horizontal.* The positional accuracy of installing piling and casings over water depends on the construction methods used and the working environment, including currents, water depths, and wave conditions. For example, driven piles from floating equipment on an inland waterway can typically be installed to a horizontal tolerance of ± 6 to 8 inches at the water surface. These tolerances can be cut in half by using a bottom-founded template or by installing the piles off of a work trestle, where the installation crane is not subject to barge motion.

(2) *Vertical.* Driving piles or casings to a specified vertical tolerance can generally be done to within ± 3 inches, and within 2 percent of vertical; however, there may be other factors that dictate when to stop driving, such as whether or not adequate end bearing has been obtained, or whether the casing has been seated well enough into the rock to create an adequate seal into sound rock at the tip of the drilled shaft casing. These considerations may dictate that the pile or casing be taken deeper than the planned tip elevation. Therefore, the final top elevation of piles and casings is usually made by underwater cut-off using robotic cut-off tools developed in the offshore industry or in-the-dry by personnel working with circular cofferdams that fit over the top of the pile and seal to the casing below the cut-off point (see Figure A-1). Divers have, however, achieved precision cut-off using guidance from the surface.

(*a*) These cofferdams allow dewatering of the annulus between the outside of the casing and the cofferdam down below the cut-off point. The interior of the casing is then dewatered and the casing is cut-off from its inside. With both of these methods, it is possible to cut the casing or piles to within ± 0.25 inch.

(*b*) An alternative method for pile cut-off is to precut the drill casing and to install a mono-directional disconnect that allows the driller to reverse the torque on the casing and disconnect under water. This technique was used on the drilled shaft casings on the Bath-Woolwich Bridge (Figure A-2).
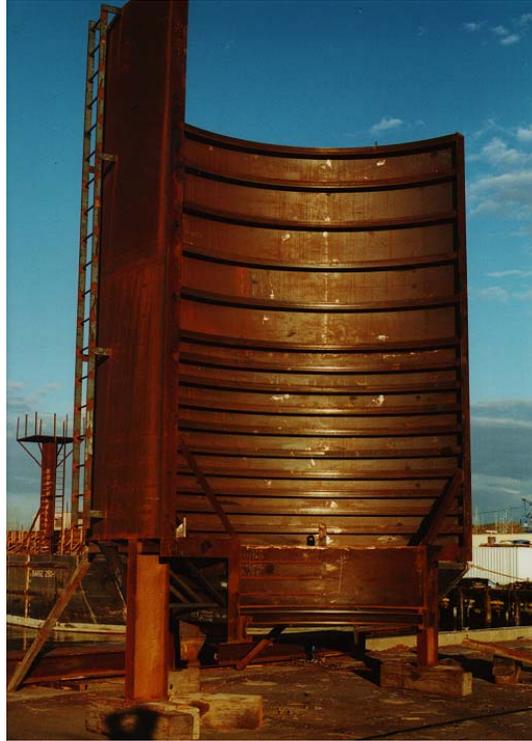
Figure A-1. One-half of temporary circular cofferdam used to cut off the 3-meter-diameter drill shaft casings underwater at the New Carquinez Bridge.

(3) *Interfaces*. There are two tolerance issues to consider: positioning tolerance of the pre-installed male foundation element and positioning tolerance of the opening in the precast element. Both of these tolerances need to be taken into consideration and be additive for the total clearance needed at a given location.

Figure A-2. Mono-directional casing disconnect for underwater termination of casing—Bath-Woolwich Bridge.

c.   Interfacing with Existing Structures. Existing structures can be used to advantage for the positioning of new float-in or lift-in structures. Fixed guides can be attached to the existing structures to force the new structure into the required alignment. This technique was used successfully at the new Braddock Dam on the Monongahela River to align the new dam segment to the outside face of the existing lock. Horn guides were mounted on top of the lock and an arm was extended out from the end of the float-in dam segment. As the segment was ballasted down, the arm engaged the horn-guide and the segment was locked into the correct alignment over the last 3 feet of landing. See Figure A-3 of the horn-guide used on Braddock Dam and Figure A-4 and for the Oresund Crossing.

Figure A-3. Looking down on guide-arm as it enters horn-guide during landing of first dam segment at Braddock Dam.

Figure A-4. Detail of horn-guide used on the Oresund Immersed Tube Tunnel.

d. Structure–Foundation Connections. The connection of the piles or drilled shafts to the float-in or lift-in structure can be pinned or fixed. The only limit on the degree of fixity is the stiffness of the structure above and the allowable embedment depth of the pile or casing into the structure at the point of connection.

(1) The connection is typically made by providing block-outs in the bottom of the structure to allow the piles, casings, or anchorage devices to protrude up into the bottom of the segment. See Figures A-5 through A-8.
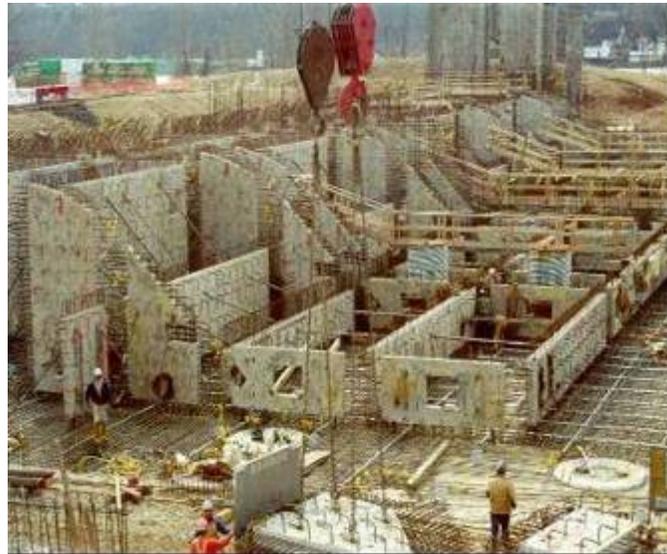
Figure A-5. Casting yard for Braddock Dam segments. See cylindrical block-outs in center.

Figure A-6. First dam segment under tow up the Monongahela River to Braddock Dam.
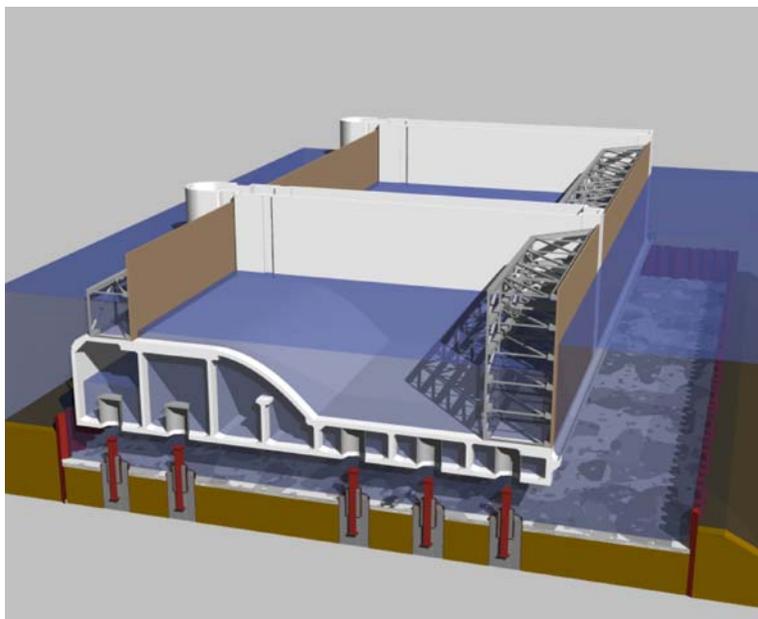
Figure A-7. Diagram of float-in dam segment fully engaged to pre-installed foundation elements—Braddock Dam.
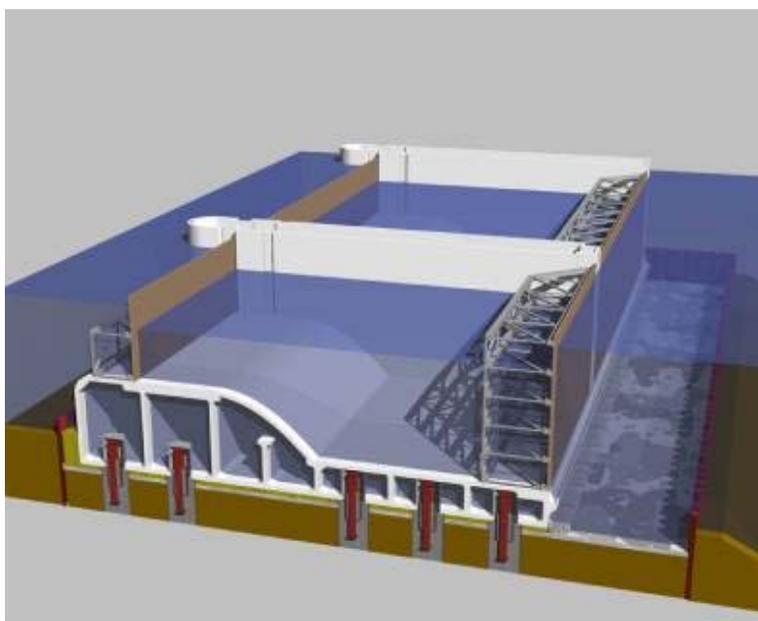
Figure A-8. Diagram of float-in dam segment at Braddock Dam about to engage tension anchors.

(2) After the lift-in or float-in structure has been mated to the foundation elements, the bottoms of the block-outs are sealed with a tremie or grout closure pour, and the connection is completed by placing grout or tremie concrete in the block outs.

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(3) Alternatively, after the seal concrete has attained sufficient strength at the bottom of the block-outs, the block-outs can be dewatered if necessary to allow access for placement of reinforcing steel and concrete in the dry, see Figures A-9 and A-10. For drilled shafts, this alternate method also allows access to the top of the drilled shafts to clean off any laitance from the tremie concrete placement.
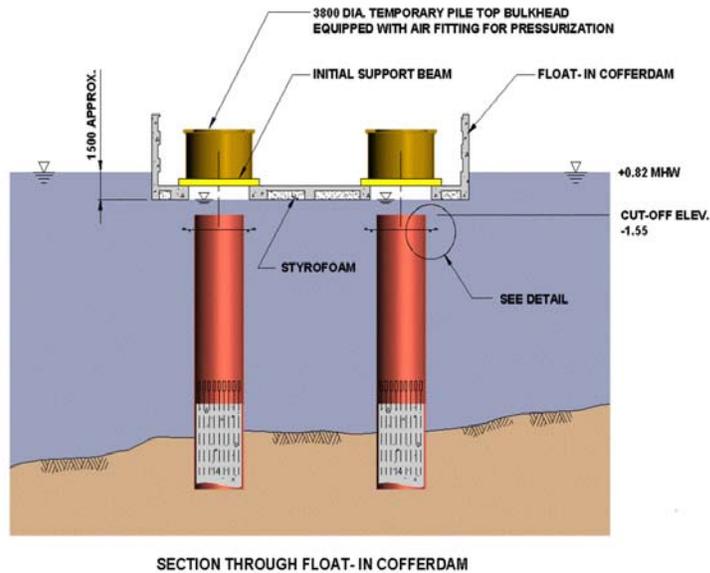
Figure A-9. Float-in cofferdam for New Carquinez Bridge.
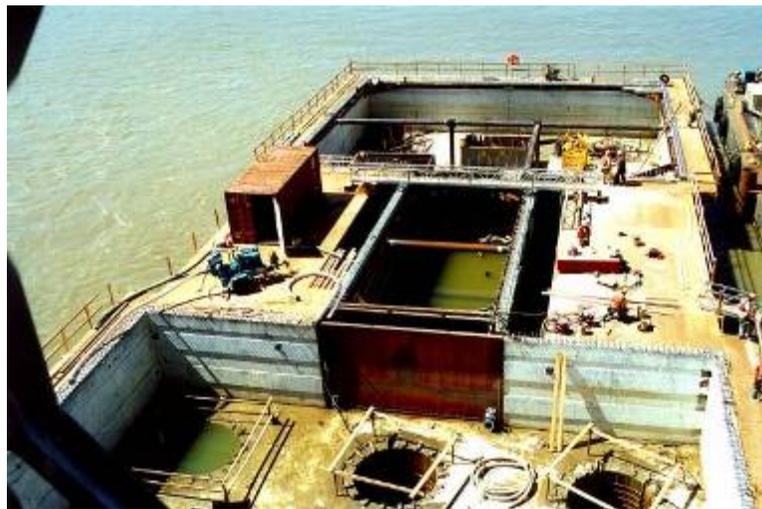
Figure A-10. Dewatered cofferdam and tops of drilled shafts at New Carquinez Bridge.

(4) The allowable bond strength between the infill concrete and the block out wall is limited to approximately 20 psi. This can be increased by adding shear studs or welding on shear

rings. (See API (2000), Section 7.4, *Grouted Pile to Structure Connections*). Alternatively, corrugated pipe can be used to create the block-outs, in which case the allowable shear strength of the infill concrete can be used. With this method, it is still necessary to attach shear connectors to the smooth skin of the pile or attach an anchorage device to the top of the pile. This approach was used to provide the tension connection of the drilled shafts to the float-in dam segment at Braddock Dam.

(5) A different pile connection detail was proposed (but not used) for both the Harvey Canal storm surge barrier, and the Inner Harbor Navigation Canal Replacement Lock, in New Orleans. This proposed alternate pile connection detail provided penetrations in the base of the concrete shells that were to be fitted with special seals during float-in operations. The seals were designed to be easily penetrated by driven piles after the shells were installed. Although this method has not been used for inland applications, it has been used for installing some steel jacket platforms in the Gulf of Mexico and offshore California.

e. Contingency—(Installation Considerations). In-the-wet foundation design requires careful planning of contingencies, redundancy, and flexibility related to construction tolerances. Contingency planning applies to both construction-stage, as well as in-service, operations. Contingency measures during construction include: i) specification and use of construction equipment that is sufficiently robust to handle different construction operations, and unexpected environmental conditions; ii) scheduling of construction activities with sufficient float time, and in a manner to allow work to progress in other areas, in the event of a work stoppage in the primary construction sequence; and iii) designing adjustable and forgiving construction joints/connections/seals that can accommodate in-the-wet construction tolerances, and which can readily be implemented underwater.

(1) Driving piles to their designed penetration lengths may not always be possible owing to the inadequacy of the pile-driving equipment in overcoming soil resistance. The analyses of pile drivability evolved from simply pounding the pile to refusal using the biggest available hammer to using mathematical models depicting the real-time pile-driving resistance.

(2) Wave equation analysis can provide a guide in the selection of equipment, and piling and penetration rate for impact hammers to determine when the pile has been driven to develop the required capacity (Engineer Manual 1110-2-2906). The mathematical model concept is based upon the solution of the 1-D wave propagation equation. It determines the pile-driver adequacy and the soil's resistance to driving. The original development of the model wave equation concept began in the 1950s. In the 1960s, computer solutions to the 1-D wave equation were further detailed and refined. It was applied to offshore piles (using above-water hammers) in the 1960s (Bender et al. 1969) and led toward the analyses of different elements of the pile-driving process. Besides the hammer and the pile, accessories were analyzed during the actual pile-driving process. The contributions of pile leads (cradle), cushions, anvil, follow block (helmet or pile cap), and followers (chasers) to driving resistance were analyzed. The application of the wave equation to long piles unsupported through the water column was analyzed and found to accurately predict the behavior on several projects (Lowery et al. 1969; Engeling 1974; Cunningham and Naughton 1977).

(3) By incorporating the wave equation analysis during the early stages of the pile design process, preliminary selection of the pile hammer can be ascertained. During the pile-driving process, the wave equation analysis can be used to predict the pile's ultimate axial capacity (Hirsch et al. 1975; Porter and Ingram 1989). The wave propagation method was further refined to include computerized instrumentation, data acquisition, and data analysis thanks to electronic technology developments (Rausche et al. 1971).

(4) Other wave propagation numerical solution techniques have also been proposed (Fischer 1975; Foo et al. 1977). During the 1980s, many case histories of pile-drivability analyses were recorded by the offshore industry, including hammer types (steam versus hydraulic) (Heerema 1980); hammer location (above-water versus submerged) (Aurora 1984); soil conditions (sands, clays, carbonate soils, weak rocks, layer conditions, etc.) (Agarwal et al. 1978; Tagaya et al. 1979; Aurora 1980; Stevens et al. 1982; Stockard 1979, 1986); pile size (Lang 1980); and analysis methods (Hollowayet al. 1978; Van Zandwijk et al. 1983). Additional computer codes that numerically integrate the wave equation were developed by offshore companies and universities. The university-developed codes include TTI, OCEANWAVE, TIDYWAVE, DIESEL1, WEAP, CAPWAP, DUKFOR, and PSI. The latest version of the WEAP series is GRLWEAP (Holloway et al. 1978; Porter and Ingram 1989). Recent literature suggests that GRLWEAP accurately predicts pile drivability (Dutt et al. 1995; Doyle 1999).

(5) Pile driving may encounter too much resistance to achieve desired penetration (cannot drive because of, for example, high friction, hard strata, boulders, cobbles, etc.) or too soft conditions (inadequate bearing capacity because of, for example, calcareous sands, or soft clays).

(*a*) *Too Stiff.* If the piles or casing encounter difficulty in driving to the specified tip elevation, penetration can be aided by any one or combination of the following techniques:

- If piles are closed-ended, allow the piles to be driven open-ended followed by cleanout and placement of a concrete plug at the bottom.
- Pre-drilling or pre-jetting to loosen the soil.
- After encountering refusal, cleaning out to tip of the pile or casing, and then continuing driving.
- Installing jet pipes in the pile or casing, and jetting during pile driving.
- Using larger capacity hammer or different type of hammer.
- Drilling ahead after cleaning out of pile.

The need for these techniques should be established before beginning to install piles or casings by specifying that the contractor make a drivability analysis (such as using GRLWEAP pROGRAM) to demonstrate the adequacy of the installation equipment.

(*b*) *Too Soft.* For driven piles, the expected driving resistance and capacity should be confirmed by the drivability analysis mentioned above. If this procedure shows inadequate bearing capacity, the piles can be taken deeper or redesigned to deal with the lower bearing capacity. For drilled shafts, the drilled hole can be taken deeper or the tips of the drilled shafts can be post-grouted to allow higher use of end bearing.

f. Heave. Heave is generally associated with driving of large, closely spaced displacement piles, such as solid concrete or closed-end pipe in plastic soils. If these conditions exist, then heave can be expected. The problem can be eliminated or minimized by driving the piles open ended and cleaning out during driving or after the piles have been driven, or by increasing pile spacing.

g. Settlement Around Piles. The ground around piles usually settles when piles are driven through relatively loose sands. The driving causes vibration and consolidation of the ground surrounding the piles. This can be avoided by densifying the ground before installing the pile or casing, although this will also increase driving resistance. Settlement can also occur if a drill casing fails to seat properly on the top of rock and loose sand runs in under the tip of the casing and into the drill hole. This can be detected by the build up of sand in the bottom of the drill hole after repeated cleanout. Settlement can be avoided by proper seating of the casing or grouting at the tip before starting to drill beyond the tip. The downward movement of soils relative to the pile will induce negative skin friction loads on the pile, which should be taken into account.

h. Liquefaction of Saturated Sands During Pile Installation. Loose sands that are not completely free draining, such as silty-sands, have a tendency to liquefy during pile driving. This can happen with both impact hammers and vibratory hammers, and can be a serious problem for existing structures in the immediate area, such as levees, buildings, cofferdams, or braced embankments. This problem can be minimized by installing vertical drains, such as stone columns or wick-drain, to prevent the build up of pore-water pressure in the ground.

i. Obtaining High Quality Concrete in Drilled Shafts.

(1) Proper mix design and tremie concrete placement procedures are essential to obtaining high quality concrete for drilled shafts. The mix should be a free flowing, highly workable mix, with anti-washout admixtures. The aggregates should be rounded gravel rather than crushed rock. The minimum spacing of the reinforcing steel in the drilled shaft should be at least 5 times the diameter of the largest aggregate. The tremie concrete placement procedures should include a controlled way for starting the process that prevents mixing of the tremie concrete with the water. It is also essential to monitor and record the elevation of tremie pipe tip relative to the rising level of concrete in the drilled shaft to ensure that the tremie pipe remains embedded in fresh concrete. Once the pour has commenced, the tip of the tremie pipe should remain embedded in the fluid concrete at least 3 to 5 feet at all times.

(2) In addition, the quality of the drilled shaft concrete should be confirmed by cross-hole sonic logging, gamma-gamma logging, or similar means. Coring can also be used to confirm the quality of the concrete. If defects are detected, their effect on the capacity of the pile should be evaluated, and if the cross section is found to be below the required capacity, the concrete should be removed and replaced by hydro-blasting and re-grouting of defective zones.

j. Buoyancy and Hydrostatic Pressure. In-the-wet structures by definition are structures that are partially or completely submerged. Float-in and lift-in elements typically have large voids that are filled with concrete or additional ballast to hold them down. The best examples of

this are immersed tube tunnels, where ballast concrete is added to the immersed tube tunnel elements to keep them on the bottom and prevent them from floating up. Most immersed tube tunnels are designed with a minimum negative buoyancy of 10 percent. The additional weight is provided in the form of ballast concrete in the roadway or backfill rock on top of the tunnel.

(1) Buoyancy can of course be used to offset the deadweight of the structure and thereby decrease the loading into the foundation. However, accidental flooding of the void space should be considered as an extreme-event load case.

(2) Buoyancy can also become a concern under conditions conducive to liquefaction. This includes submerged structures founded in loose sands or backfilled with loose granular material. In either situation, the loose material has the potential, when subjected to earthquake motion, to liquefy, turning the surrounding ground into a heavy liquid, and thereby causing the structure to float out of the ground. For a loose granular backfill with high porosity, this risk may not exist for years after placement of the backfill. However, over time this material has the risk of silting up and significantly decreasing its porosity and greatly increasing the potential for liquefaction during an earthquake. An example of this situation is the BART Immersed Tube Tunnel under San Francisco Bay, which was completed in 1970 and now requires ground stabilization of the backfill after 35 years of service. Hydrostatic pressure must also be adequately considered in design.

k. Need for Extensive Subsurface Investigation.

(1) Underwater foundations present numerous potential design issues that merit extensive subsurface investigation, including: unconsolidated mud, liquefiable granular materials, debris and wreckage, complicated geologic histories and formations, and a host of other issues. This subsurface investigation should include: i) advanced explorations, such as: drilling soil borings, using in-situ probes, and taking geophysical measurements; ii) preliminary foundation installation and monitoring, such as: the use of indicator piles, proof loading of the preliminary foundation elements, and the advanced use of techniques such as Osterberg cell, Statnamic, and CAPWAP for driven piles; and iii) investigation and monitoring during construction to identify problems that require modifications to the foundation design. Geotechnical investigations are discussed in EM 1110-1-1804.

(2) All of these investigations are also warranted for in-the-dry construction; however, in-the-wet investigations generally require more planning and expense than comparable investigations in-the-dry owing to logistics and environmental issues, such as wind, waves, currents, tides, sedimentation, ice, and floods. However, just as for in-the-dry construction, without adequate investigation, the underwater foundation design is required to carry significantly higher factors of safety, which provides incentive to bear the cost and effort of conducting adequate subsurface investigations. See Paragraph A-4 for a more detailed discussion of specific site exploration techniques and methodologies.

l. Environmental Impacts. In-the-wet foundations present both increased risks in some areas, as well opportunities to decrease risk in other areas with regard to environmental impact, as compared to in-the-dry construction. Typically, building marine foundations in-the-dry re-

quires, first ,the construction of some kind of cofferdam in-the-wet, followed by foundation construction in-the-dry, these two combined activities frequently can disturb the environment as much as, or more than, the single activity associated with an in-the-wet foundation. Nevertheless, environmental impacts of an in-the-wet foundation must be clearly identified, and frequently measures must be taken to minimize, or mitigate, these impacts. The following list identifies some of the environmental disturbances unique to in-the-wet construction, together with some of the common measures taken to minimize, or mitigate, these impacts:

(1) Driving piles in-the-wet can cause the transmission of shock waves through the water that can disturb, or even kill, fish. Common measures taken to minimize this problem include: i) driving piles during periods when the fish are not present, or are less susceptible to disturbance; ii) use of bubble curtains to dampen the transmission of the shock waves, and iii) restoration of fish habitats and spawning areas.

(2) Pile driving in-the-wet can disturb neighboring birds and common means to minimize this impact include: i) use of underwater hydraulic or vibratory hammers, ii) use of air-steam hammers as opposed to diesel hammers, iii) driving during periods when the birds are not present, or are not nesting, and iv) restoration of bird habitats and nesting areas.

(3) Cement particles can be leached into (and contaminate) the water during underwater tremie concrete placement. Common methods for minimizing this problem include: i) use of anti-washout admixtures in the concrete, ii) isolating the tremie concrete (or laitance) from the surrounding water; and iii) minimizing over-filling of underwater shells, forms, or other tremie concrete containment structures.

m. Non-Traditional Construction Methods. Non-traditional construction methods are usually only non-traditional for a given design and construction community, and typically have been used in other design and construction communities. It is strongly recommended that experts from communities with relevant experience be identified and invited to serve on both the Product Delivery Team (PDT) and the Peer/ITR (Independent Technical Review) review panel, as cited in the previous paragraph. Furthermore, contractors and vendors from communities with relevant experience should be identified, surveyed and, if qualified, invited to bid on the proposed work. It is essential that the design engineer should thoroughly investigate the state-of-the-art practices before engaging in the use of construction methods that are non-traditional in the community in which he or she works.

n. Adjustments to Design Criteria to Mitigate Risk. It may be practicable to adopt more conservative design criteria as partial compensation for reduced in-the-wet QC investigation. However, it is not possible to eliminate risk, thus risk should be mitigated and, when possible, shared with the contractor. Furthermore, the design criteria documents should be treated as living documenta, and should be regularly reviewed and updated from the beginning to the end of the project, with the objective of minimizing risk for in-the-wet foundation designs.

o. Environmental Considerations. The design basis developed for the project should address the relevant environmental considerations, such as wind, waves, current, tides, debris, ice, sedimentation, scour, floods, water density and salinity, water visibility, river gradients, ed-

dies, sand waves, debris, and water stage, depth, and bathymetry. Furthermore, this information should be made available to the contractor.

p. Limited Construction Periods Because o Environmental Issues. To limit risk it may be advisable to limit the in-the-wet construction to periods when the water and environmental conditions are less severe. Typically, construction engineering design parameters would be selected on a not-to-exceed 5 percnt of the time basis during the construction period; however, other percentiles may be selected, depending on the consequence of exceedance. In this regard, the design should evaluate considerations such as the following.

(1) When working afloat, an increase in river stage may, or may not, represent a construction problem, depending on the details of the construction plan.

(2) Scour and sedimentation problems associated with moveable riverbeds can frequently be avoided by avoiding construction during certain periods, or by the construction of physical features, such as sedimentation traps, or deflection vanes.

(3) Suitable construction, engineering, and environmental design parameters for a given construction period depend on the contractor's ways and means, and the designer should clearly indicate both environmental design parameters and the ways and means that were used and assumed for the in-the-wet design.

(4) Needless to say, construction will almost certainly need to be temporarily halted during warning periods for severe weather conditions, such as for hurricanes, thunder/lighting storms, tornados, flood events, etc.

q. Specialized Equipment. Specialized equipment is frequently required to do in-the-wet foundation work properly, which presents several risks including the following.

(1) The limited supply of specialized equipment may lead contractors, or sub-contractors, to charge inappropriately high costs for this equipment.

(2) The specialized equipment may not be available at bid or construction time.

(3) The specifications (or bid documents) need to be written to require the use of appropriate equipment, and personnel, otherwise the contractor may use inappropriate equipment and personnel, thus increasing risk (fiscal, environmental and personnel safety).

r. Decreased Ability for QC. In-the-wet foundation work can decrease the accessibility, or cost, both of quality control, QC, and quality assurance, QA. This requires a plan for the QA/QC means and methods to minimize the risk of accepting sub-standard work. Means and methods to minimize this risk include the following (see also Paragraph A-9).

(1) Using advanced technology and equipment, frequently associated with the offshore industry, such as: i) advanced acoustic imaging, ii) robotics (Remotely Operated Vehicles, ROV's, such as those used for side scanning, or Autonomous Underwater Vehicles, AUV's, more common for deep water work), and iii) geophysical investigations.

(2) Using mock-up tests onshore, or nearshore, to verify the contractors' ways and means for such operations as tremie concrete mix design and placement, drilled shaft construction, construction joint construction, or underwater repair methods.

(3) Specifying that selected foundation element either be constructed using templates, or be pre-assembled in-the-dry and then placed underwater (such could be the case for pile cap assemblies).

(4) Prefabricating foundation elements.

s. Risk Mitigation, Repair. The previous sub-paragraphs identify many of the measures associated with risk mitigation and repair of in-the-wet foundation construction; and the following list expands on the points already presented.

(1) In-the-wet foundation work requires considerable planning, and this planning can substantially mitigate risks.

(2) Risks can be mitigated by arranging for input from in-the-wet experts, including the incorporation of Independent Technical Review (ITR) teams, and expert consultants, during the design process.

(3) Risk can also be mitigated by better project-wide: i) communications, ii) training and education, iii) site investigation, and iv) mock-up tests.

(4) Anything built in-the-wet can either be repaired in-the-wet, or removed from the wet, repaired in-the-dry and then returned to the wet. The details and cost of the repair depend highly on the nature, and logistics, of the problem. Common underwater repair techniques include: i) using high-pressure water jets (with, or without, grit) to demolish either a concrete or steel element, followed by replacement; ii) pulling, or replacing, piles inappropriately driven; and iii) underwater epoxy injection of cracks and construction joints.

(5) Underwater cutting and welding.

A-3. Summary of In-the-Wet Foundation Types.

a. Introduction.

(1) Numerous foundation types are constructed onshore (as land-based construction), and they broadly fall into either the "shallow" or "deep" foundation category. By definition, the shallow foundation embedment depth is less than its minimum lateral dimension and includes spread footings and mats. The deep foundation embedment depth is much greater than its minimum lateral dimension. Examples include piles and shafts (Herrmann et al. 1972).

(2) Underwater foundations may be classified into one or more of the following general categories: *improved-site*, *gravity-based*, and *pinned*. Improved-site foundations include those that are dredged, excavated, backfilled, leveled, or site-modified. Gravity (or gravity-based)

foundations include those that distribute the structure's loads over a soil or rock area wide enough to adequately resist the imposed loads. Pinned foundations distribute the structure's loads deeper into the soil and rock materials.

 b. *Driven Piles*.

 (1) *Displaced Soil.* These methods include driving piles into the soil or weak rock by impact, vibratory oscillation, hydraulic pushing or jacking methods, rotating, jetting, or suction techniques and equipment. Hydraulic pile-driving is virtually noiseless and vibration-free, while pile jetting is a time-saving pile-driving method (Tsinker 1988). A brief discussion of several types of displaced-soil piles follows.
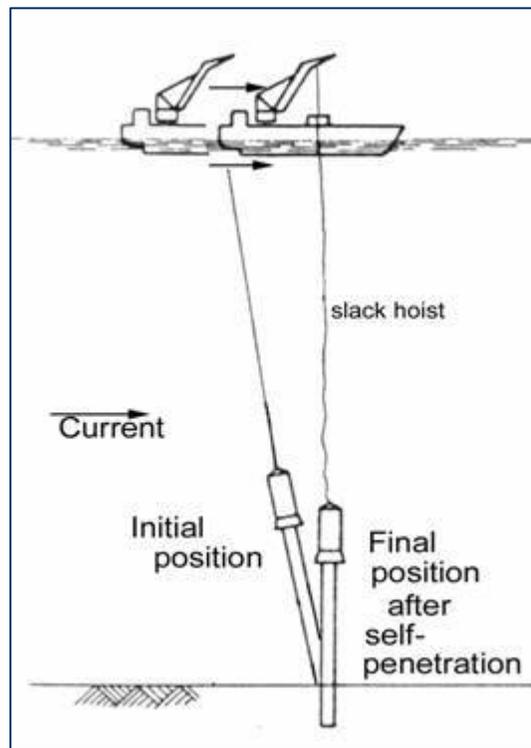
Figure A-11. Underwater impact-driven pipe pile installation without a template (after Jansz and Brockhoff 1979).

 (2) *Driven Preformed Piles.*

 (*a*) Examples of driven preformed piles include tubular steel (open or closed end pipe) piles, solid steel (H-, X-, A-, or sheet) piles, timber piles, precast (prestressed or reinforced, or both) concrete piles, and polymer piles. Figure A-11 illustrates an underwater impact-driven pile installation procedure. The advantages of driven preformed piles include the following:

- The pile materials are preformed and subject to fabrication and site (quality control/quality assurance) inspection.
- The pile materials can withstand high bending and tensile stresses.
- Numerous choices for pile materials, equipment, and installation procedures are available.
- Waterborne delivery and onsite fabrication permit very long lengths of piles for site-specific conditions.

(*b*) The disadvantages of driven preformed piles include:

- Above-water noise may affect local communities. Underwater noise affects sea mammals and fish.
- Excessive vibration may cause local liquefaction.
- Soil disturbance may affect surrounding structures.

(3) *Driven Cast In-situ Piles*.

(*a*) An example of the driven cast in-situ pile includes an open- or closed-ended steel casing or pipe pile that is driven and then backfilled with concrete. The advantages of this type of pile are as follows:

- The length can be easily adjusted to match the bearing stratum.
- Site-specific modifications, such as an enlarged base or bells, are possible.
- Noise and vibration during driving may be reduced.
- The internal pile material, such as concrete or grout, will not be affected by handling or driving stresses.

(*b*) The disadvantages of driven cast in-situ piles include these:

- Mixing and placement of the concrete or grout may require special equipment and inspection.
- The inspection and placement of the reinforcing bar cage may require special considerations or techniques both in design and construction.

(4) *Pipe Piles*. Pipe piles come in a variety of materials but most consist of either steel, or prestressed concrete and come in a variety of diameters ranging from a few inches (tens of millimeters) to over 33 feet (10 meters) (note that the Yokohama Bay Bridge in Japan used 10-meter-diameter prestressed concrete piles, and the Ohnaruto Strait Bridge also in Japan used 6-meter-diameter cylinder piles). Steel pipe piles, available from the offshore industry, commonly range from 3 up to 13 feet (1 up to 4 meters), with ultimate axial compressive capacities of over 10,000 tons, far exceeding conventional onshore piles. API provides standards for the design of such large offshore piles.

(*a*) Larger diameter pipe piles are more commonly used for in-the-wet construction for a variety of reasons including: i) large diameter vertical pipes have more lateral stiffness than an equal area of smaller piles; ii) the expense of mobilizing and using marine pile driving equip-

ment warrants driving fewer large diameter pipe piles, rather than more smaller diameter pipe piles; iii) if the pipe piles are to be socketed, it may be more practicable to clean-out and socket a few large diameter piles rather than many smaller diameter piles, iv) in hard driving conditions it may be more practicable to drive large diameter piles, rather than small diameter piles; and v) it may be more practicable to transform a larger diameter driven steel pile into a composite steel and concrete pile (by the addition of tremie concrete) than for smaller diameter piles.

(*b*) Nevertheless, smaller diameter pipe piles offer their own unique advantages. Indeed, one of the largest uses of pipe piles for in-the-wet foundation construction for the inland waterways is the Olmsted Dam, which uses over 3250 steel pipe piles having a 2-foot (0.61-meter) diameter (with 12- to 16-inch [305- to 406-millimeter] top pipe extensions to increase structural flexibility), for reasons including: i) the smaller diameters increased the period of the structure, which decreased the seismic loading on the structure; ii) the unit price of the pipe was lower than for larger diameter pipes; and iii) the smaller pipes introduced lower local bending moments into the concrete portions of the dam, thus allowing the concrete portions of the dam to be thinner.

(*c*) Aside from the tendency toward the use of larger diameter pipe piles, other unique considerations of using pipe piles for in-the-wet construction include: i) availability of pile driving procedures and equipment; ii) convenient jetting procedures; iii) common splicing procedures and equipment; iv) potentially reduced environmental impact; v) ease of driving potential with increased positional accuracy, and vi) suitability for tie-in with other sub-structure elements and tremie concrete, as briefly discussed in the following:

- *Driving Pipe Piles In-The-Wet.* Considerations include: i) providing vents in the pile, or in the driving head, to relieve water pressure built up on the interior of the pile during driving, ii) using hydraulic, or vibratory hammers, for underwater pile driving; iii) fitting rubber diaphragms to the ends of pipe piles to provide buoyancy to facilitate transport and handling of submerged piles; and iv) using the very large hammers that are available.

- *Marine Splicing.* Considerations include: i) field splicing of large diameter marine piles requires careful planning and quality control, and the number of splices required should be minimized by using pipe segments as large as can be transported and handled economically, and ii) field splicing of large cylindrical pipe piles must be strong enough to resist repeated hammering during driving, and the splice itself can be made using any one, or combination, or means, including: welded steel end fittings, steel dowels epoxy grouted into corrugated ducts, mechanical connectors, and friction sleeve connectors. In the Netherlands prestressed splices have been made on 12-foot (3.5-meter) diameter concrete cylinder piles.

- *In-The-Wet Connections.* Considerations include: i) when tremie concrete is used to connect pipe piles to other elements, underwater care should be taken to ensure that the tremie concrete does not inadvertently fall inside the pipe pile and flush water from inside the pile into the fresh tremie outside the pile. One way to address this is to attach a screen near the top of the pile prior to driving. ii) Grouted connections to pipe piles can be made either using standard offshore technology used on steel jacket platforms, or by grouting

the pile into a socket to another underwater element. And iii) concrete plugs may be installed at top of piles after driving.

(5) *H-Piles.* H-piles are frequently used onshore owing to their economy and availability, and they should be considered for use on in-the-wet foundations for the same reasons. H-piles have both a strong and a weak bending axis and thus must be aligned accordingly, and frequently must be battened to resist large bending moments. Where alignment and batten of H-piles are important for in-the-wet work, templates should be used, or a very accurate lead. For hard driving, H-piles may require driving shoes. Figure A-12 illustrates how a precast concrete stay-in-place template was used to guide underwater battened H-piles for the Richmond-San Rafael Bridge in San Francisco Bay, California. The precast concrete template was subsequently incorporated together with the H-piles and the other precast concrete shells using tremie concrete infill.

(6) *Precast Prestressed Concrete Piles.* Modern precast concrete piles are almost all prestressed and pretensioned (if, for no other reason, to address both handling and driving stresses), and come in a large variety of shapes, sizes, and strengths. Cylindrical concrete pipe piles are addressed in Paragraph A-3*a*(4), while solid precast piles are addressed in the following.

(*a*) Where established precast yards are available, standard sizes of prestressed concrete piles can be more economical than even H-piles, and thus potential in-the-wet use of these standard sizes should be carefully evaluated. Standard solid precast pile shapes include square, octagonal, and round, with sizesand diameters commonly ranging from 10 inches (254 millimeters) to about 54 inches (1370 millimeters). For such standard shapes and sizes, the reinforcing and prestressing can economically be customized to adjust such factors as: i) concrete cover, ii) spiral size and pitch (to provide better concrete core confinement), iii) varying the prestressing level from about 300 psi (2 MPa) to over 1000 psi (7 MPa); iv) varying the longitudinal steel along the length of the pile, v) incorporating or casting items such as jets, driving shoes, lifting eyes, or steel H-piles into the concrete pile, and vi) using epoxy coated steel. It is also economical to customize the concrete to be used in the pile, potentially using i) high-strength, lightweight concrete (not only to reduce weight for transport and handling but also to reduce the foundation weight in weak soils and for seismic cases); ii) very high-strength, standard-weight concrete, where strengths in the range of 12,000 to 14,000 psi (80 to 100 MPa) are practicable; iii) concretes with various admixtures including microsilica fume, anti-corrosion admixtures, and air entrainment (for freeze–thaw protection). Furthermore, in addition to standard load bearing piles, precast concrete sheet piles and fender piles can also be procured from existing precast yards.

Figure A-12. Use of a precast concrete element as a template for battened H-pile on bridge pier for the Richmond–San Rafael Bridge, California.

(*b*) Precast concrete piles typically exhibit superior durability, as compared to either steel or timber piles, for marine applications where the piles extend up through the water. Such issues as corrosion, biological attack, weathering, pollution, or erosion may be a larger concern for other types of piles. Other potential reasons for considering concrete piles include: i) they can stand-up to hard driving and can penetrate debris or rip rap; ii) they can develop good friction with the soil; iii) they have high axial load carrying capacity; iv) they can be easily spliced and connected, and v) they can penetrate into soft and partially decomposed rock. High-capacity prestressed concrete piles are particularly well suited for deep foundations with heavy loads in weak soils, conditions that are frequently encountered with in-the-wet foundations.

(7) *Timber.* Timber piles have been used for thousand of years in-the-wet, and have exhibited excellent durability where they have been buried and thus protected from biological attack, weathering and oxygen, pollution, or erosion.

(8) *Steel Sheet Piles.* Flat sheet piles are frequently used to form cells and other arched structures, and z sheet piles are frequently used to build walls and planar structures, which are very commonly used for in-the-wet marine foundations. Tie-in of the sheet piles into the in-the-wet foundations may require careful consideration.

(9) *Composites.* Polymer composite materials are becoming more commonly used for marine applications, primarily because of their corrosion resistance. Commonly available shapes include: i) sheet piles; ii) wide-flange and H-shapes; and iii) pipe sections. Polymer composite

members typically cost more than steel, concrete, or timber piles, and when used are commonly justified for marine applications based on reduced maintenance and replacement costs.

    c. Drilled Elements.

    (1) *Cast In-situ Piles.* Examples of cast in-situ piles include stone columns (Cemcol), compacted concrete piles (Vibro-Franki, not commonly placed underwater), Atlas screw piles, see Figure A-13 (not yet commonly used underwater according to Reese and Isenhower 2000), and micropiles (which are grouted minipiles or pinpiles installed without drilling).
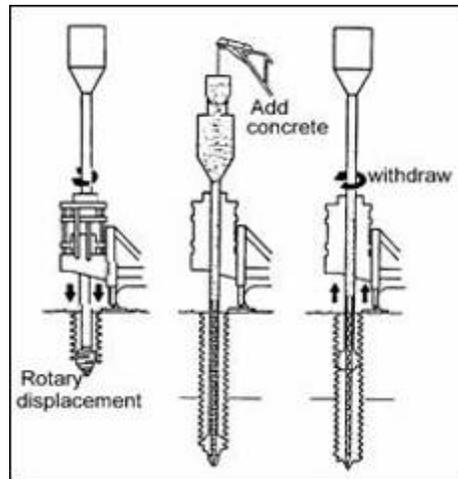
Figure A-13. Cast in-situ pinned foundation (Atlas screw pile method) (after Hollingsworth and Imbo-Burg 1992).

    (*a*) The advantages of cast in-situ piles are:

- No need for permanent casing.
- May be more economical for specific sites.
- No cuttings and, hence, no disposal of cuttings required.

    (*b*) The disadvantages of cast in-situ piles include:

- Limited experience in underwater environments and (for certain proprietary systems, such as Cemcol, Atlas, Vibro, Franki, Vibrex, and Fundex) extremely limited specialized contractor experience.
- Cannot be extended above riverbed without casing and special procedures.

    (*c*) A special case of the cast in-situ pile is represented by the soil and grout cast in-situ foundation, which is generally designed as an improved-site foundation instead of pinned. This type of foundation improvement is used primarily for onshore applications, although deep cement mixing has been used offshore (although primarily outside the United States).

(*d*) Two examples of the soil and grout cast in-situ foundation improvement are jet-grouted columns (grout injected into soil from high-pressure jets) and mixed soil columns (grout mixed with soil, also called deep cement mixing). The advantages and disadvantages mirror those for cast in-situ, as previously noted.

(2) *Drilled Shafts.* The construction of drilled shafts through open water involves drilling holes through underwater soil or rock, and maintaining the hole open long enough to install a reinforcing cage and tremie concrete. Large-diameter drilled shafts (6 feet and larger) are particularly well suited for in-the-wet structures. The primary factors limiting the diameter and capacity of drilled shafts are availability of larger drilling equipment and the stability of larger diameter drill holes during the drilling process.

(*a*) Installation techniques and equipment have been developed to efficiently install drilled shafts in the 6- to 10-foot range, and drilled shafts have been successfully installed up to 16 feet in diameter. These large diameters are a cost efficient replacement for a large number of smaller driven piles, and can be a determining factor in whether a pile cap is required, and if required, where that cap can be positioned. Because of the high stiffness of large diameter drilled shafts, the pile cap can either be eliminated or be positioned off the bottom of the riverbed or seabed. Positioning pile caps off the bottom offers significant cost advantages, because it eliminates the need for costly conventional cofferdams typically used for dewatering the construction site. This technique was used on the piers for the Bath-Woolwich Bridge. See Figures A-14 through A-16.
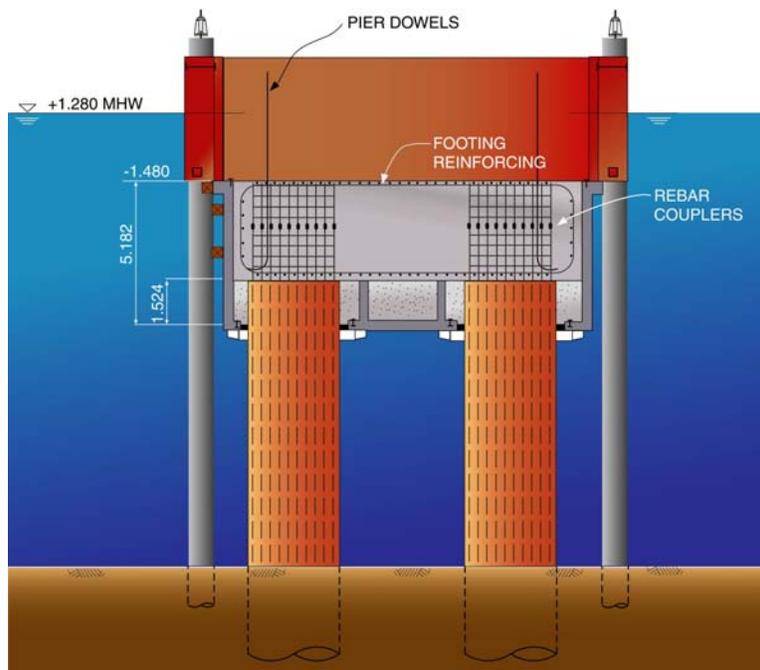
Figure A-14. Pile cap and temporary follower cofferdam used on Bath-Woolwich Bridge.

Figure A-15. Bath-Woolwich Bridge cofferdam being dewatered.

(*b*) Applying this foundation type to in-the-wet construction requires use of an external casing through the water column. The casing extends from the high water elevation into stable soil or into the top of rock. The external casing is installed in a manner to produce a positive seal at the bottom of the casing so that no piping of water or other material enters or exits the shaft excavation (O'Neill and Reese 1999). See Figure A-17, showing temporary casings used on the Bath-Woolwich Bridge Project.

Figure A-16. Bath-Woolwich Bridge dewatered cofferdam.

**Figure A-17. Pile template and temporary casings used
on the Bath-Woolwich Bridge.**

(*c*) The primary purpose of the casing is to protect the shaft concrete from water action during placement and curing of the drilled shaft concrete. For drilling in unstable rock, where drilling fluids such as bentonite or polymer slurries are used, the casing also provides the primary containment system for keeping the drilling fluid from the surrounding water. The casing can be either temporary or permanent.

(*d*) Drilled shafts cause less soil disturbance during installation in comparison to driven piles. Therefore, there is less danger of disturbing adjacent structures or underwater embankments.

- *Small-Diameter Drilled Shafts (less than 6 feet).* The stiffness of the structure being supported will dictate the maximum spacing of the drilled shafts. This spacing will determine the load carrying requirements of each shaft and drilled shaft diameter. Small-diameter drilled shafts are therefore better suited to relatively thin structures such as tailraces on low-head dams.

- *Large-Diameter Drilled Shafts (6 feet or greater).* As a general rule for a given in-the-wet foundations, fewer large-diameter drilled shafts are more economical than a larger number of smaller diameter shafts. This is attributable to the relatively high cost and complexity associated with the underwater connection of the drilled shafts to the structure. However, drill-hole stability tends to decrease with increasing drill-hole diameter. In some cases, the decrease is so great that only by casing the drilled hole can the pile be constructed.

(3) *Augered Cast-In-Place Piles.* The augered cast-in-place pile system has been used onshore since the 1940s. Two methods are available. One consists of drilling a hollow-stem auger to depth and then pumping grout or concrete into the hollow stem as the auger is withdrawn.

Reinforcing rods may be inserted into the concrete through the hollow-stem auger (Neate 1989). The excavated soil is brought to the surface as cuttings. For use of this system under water, external casing would be required if the concrete extends above the mudline. The second method uses a continuous-flight auger to excavate soil inside a driven casing. Numerous equipment manufacturers and techniques are available for this popular (onshore) method.

(4) *Pin Piles (Micropiles) (5–12 inch diameter).* The use of pin piles (or micropiles) has increased in recent years, especially in situations with difficult access or restricted vertical clearance. For example, bridge foundation rehabilitation has become a major market for these piles (Pearlman et al. 1997). Pin piles are small-diameter drilled and grouted piles. Their diameters range from 5 to 12 inches (127 to 305 milimeters), and their axial compression and tension capacities range from 50 to 200 tons (445 to 1780 kN). The most common installation technique is to rotary-drill an open-ended steel pipe into the subsurface. A reinforcing bar cage is then installed inside the casing, and pressurized grout (or tremied grout in rock formations) is pumped into the casing to extend the "bond zone" below the open end of the pile.

(5) *Stone Columns.*

(*a*) Underwater stone columns have been placed to increase a soil's shear strength and bulk modulus. Pennine, Ltd. (1998) reported the installation of 2.6-foot- (0.8-meter-) diameter by 66-foot- (20-meter) long stone columns in water depths up to 108 feet (33 meters) from a floating barge using vibro-compaction. The productivity rate was greater than one column per hour, including barge positioning.

(*b*) Hayward Baker (1999b) reported installing 270 stone columns (3-foot- [1-meter-] diameter) using a vibro-replacement method through water depths up to 40 feet (12 meters) to prevent liquefaction damage to existing underwater piles at a Vancouver port facility.

(6) Multiple closely spaced driven steel piles were driven at water depths of up to 70 meters as a means of enhancing the bracing and shear resistance for the foundations of the Rion-Anterion Bridge in Greece and concrete piles are now being installed under the Venice (Italy) Storm Surge Bassin. They are conceptually planned to stabilize an underwater clay slope against seismic-induced failure.

d. Gravity Base.

(1) *Introduction.*

(*a*) Gravity-based foundations are broadly classified because their load-carrying capacity is primarily determined by the size and weight. These foundations are similar in function and design to the land-based foundations that are classified as "shallow," as their embedment depths are generally less than their lateral dimensions. Typical gravity foundations are constructed of precast concrete elements that are either floated or lifted in and seated on the seafloor or an improved-site foundation. Once installed, the underbases may be filled with cast-in-place concrete or grout via underwater tremie tubes or other type of ballasting materials. For example, the main pier bases supporting the Confederation Bridge in Canada are lift-in precast concrete ele-

ments resting on a tremied concrete bed over mudstone and siltstone layers. Each of the 44 pier bases weighing 6000 tons rests on a ring footing of tremied concrete having an approximate diameter of 67 feet (20 meters) and an approximate thickness of 6 feet (1.8 meters).

(*b*) Smaller gravity-based foundations have been constructed of steel instead of concrete. For example, offshore foundations for wind turbines have been constructed of steel gravity foundations that are basically steel boxes sunk to the seabed and filled with olivine (DWTMA 1998). The steel box size was designed to be 42 by 42 feet (14 by 14 meters) with a weight of about 90 tons (100 metric tonnes) for a water depth to 30 feet (9 meters). Very large gravity-base caissons, with diameters of up to 140 meters and weighing several hundred thousands of tons have been installed offshore of Norway, Scotland, and Netherlands.

(2) *Lift-in Steel Shell.* Lift-in steel gravity base shells have been used for in-the-wet construction, such as the plate steel cylindrical cans that were lifted in by the St. Louis District to use as mooring cells. To minimize the cost of preparing the rock foundation for this project, after cleaning, the exposed rock surfaces beneath the footprint of the cans were carefully surveyed, and then the bottom of the cans were cut to match the bottom rock contours. Then, cans were accurately positioned using staggered vertical pintle guides, and the irregular outer bottom perimeters of the cans were sealed to the rock surface using diver placed sand bags. Then, the in-the-wet foundation was completed using tremie concrete placed on the inside of the cans.

(3) *Rock Foundation.* Rock foundations cannot usually be excavated under water to the tolerance required for direct bearing on the underside of a lift-in or float-in gravity base structure. It is, therefore, necessary to pre-install a thin gravel base that is screeded to a specified level tolerance. This technique was used successfully on the tunnel elements for the Oresund Immersed Tube Tunnel connecting Denmark and Sweden. Alternatively, hard landing points can be pre-installed to a precise elevation, and the gap between the bottom of the lift-in or float-in structure and the rock foundation in-filled with grout. This method was used for supporting the lift-in foundations for the Confederation Bridge connecting Prince Edward Island with the mainland of Canada. See Figures A-18 through A-20

Figure A-18. Guide towers used to establish landing points for precast bridge piers on rock at Confederation Bridge.

Figure A-19. Base of the guide tower leg before inflation of grout bags, Confederation Bridge.
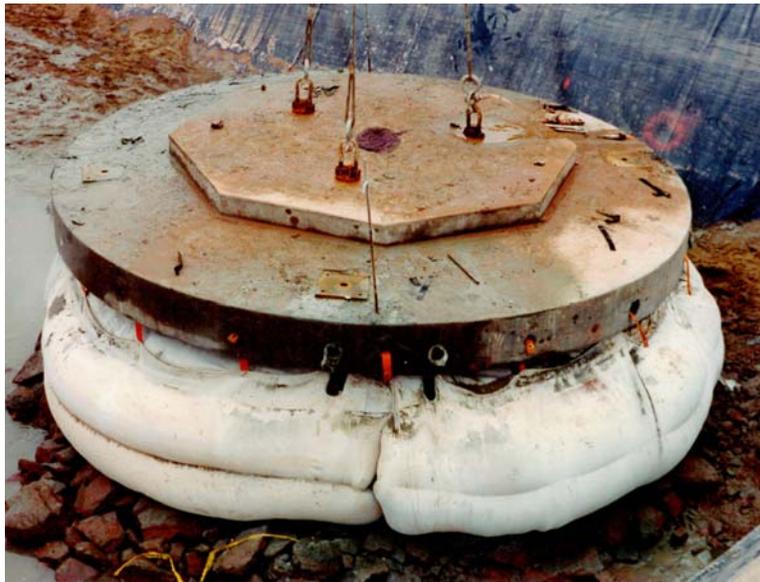
Figure A-20. Inflated grout bags at test landing point, Confederation Bridge.

(4) *Soil Foundation.* API (1989) presents clear guidelines for the design of in-the-wet foundations for gravity structures founded on a variety of different soil types, and surface inclinations. However, it is valuable to consider that, for gravity base structures' foundation on very weak soils,: i) buoyancy can be used to reduce the structural loads on the soil foundation, or ii) skirts can used to transfer the bearing load down to deeper, higher capacity soil layer, such as was done for the Troll GBS platform in Norway.

(5) *Stabilized Soil*. An improved-site foundation modifies the existing soil (or rock) in the seabed or riverbed. Methods include preloading, dredging, backfill, leveling, consolidation, or other engineered modifications (Figure A-21). In the case of immersed tunnels, pipelines, breakwalls, dry docks, and some float-in and lift-in structures, an improved site may be the only foundation system needed.

(*a*) Dowse (1979) described a potentially innovative improved-site methodology employing self-supporting (hydrostatic pressure) sand islands for foundations for oil platforms in water up to 200 feet (61 meters) deep. Basically, an impervious membrane is filled with dredged sand, and the water is then pumped out. Such hydrostatically supported, confined-pressure sand islands are capable of supporting surface loads that are proportional to the drained shear strength of the sand. Each island can be built within 2 weeks, depending on the specific design. Disadvantages are the amount of dredging required to supply the sand fill, the limitations of a soft soil under-base, and the construction handling and placement of the impervious membrane.

(*b*) Dredging, fill placement, consolidation, soil improvement, and mat reinforcing are some of the methods used during construction of an improved-site foundation.

(*c*) An improved-site may be the least complex of all foundations because the foundation itself is not "connected" to the superstructure. An improved-site foundation is usually composed of a leveled surface (excavated or backfilled) overlying either modified or natural soil and rock material. Modification of the soil or rock implies that those materials have been physically or chemically altered to be able to transmit the loading imposed by the superstructure into deeper material without failure from bearing capacity, settlement, or differential settlement.
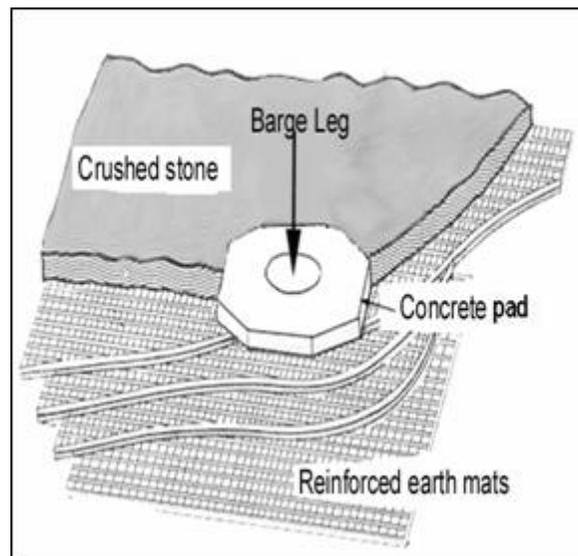
Figure A-21. Example of an improved site foundation (after Abbott and George 1989).

(*d*) Alternatively, the prepared surface may underlie man-made materials. such as geotextiles or bridging mats. The superstructure itself is designed to transmit the imposed loading to the underlying foundation material through its footprint (foundation), which has been designed (sized) to minimize the settlement and movement of the structure. A mat of crushed stone may be placed and screeded to grade.

(*e*) An underwater embankment foundation must be designed for stability and minimal settlement in a way that is similar to an onshore design. The onshore methods for computing slope stability may be applied, with the caveat that spatial variation will be analyzed. As the underwater variability will likely be greater than for onshore slopes, solutions yielding the single critical shear surface and minimum factor of safety should be expanded to include spatially variable factors of safety. Liedtke and Wright (1999) analyzed this approach using the UTEXAS3 software package.

(*f*) Dredging, excavation, and (back)fill placement design principles are similar to those used onshore. Namely, unsuitable materials are removed and replaced with more suitable material. Disadvantages of dredging and backfilling include environmental concerns in sensitive marine or riverine environments, contaminated dredged material issues, and possible scarcity of nearby suitable fill materials.

(*g*) In addition to excavation and replacement, in-situ modifications may also be required to increase the soil or rock strength, reduce settlement, or to improve stability. In sands, the primary goal is to increase the relative density. In clays and weak rocks, the primary goal is to increase the undrained shear strength. Three techniques available to achieve those goals are mechanical, hydraulic, and chemical modifications. Mechanical modifications include physical rearrangements of the soil matrix and include modifications by inclusions or confinement, or both. Hydraulic modifications include dewatering schemes. Chemical modifications include grouting and soil mixing schemes (Hausmann 1990).

(*h*) Mechanical modifications using reinforcing mats, steel mats, prefab rock blankets, or geotextile fabrics have the effect of creating dense layers with improved strength characteristics, as well as serving as filters to minimize erosion or scour. The foundation bearing capacity and settlement design then becomes a multiple-layer subgrade analysis, with stiffer and more competent layers overlying softer strata. For example, Abbot and George (1989) determined that steel reinforcement meshes overlaid with granular layers would reduce potential settlement by 66 percent and increase shear strength by a factor of 800 as compared to the unimproved site.

(*i*) Mechanical modification using vibro-compaction is sometimes called vibro-flotation if water jetting accompanies the vibration. Vibro-compaction mechanically rearranges soil particles into a denser configuration by the use of vibrating shafts and is used exclusively for cohesionless soils. Vibrating shaft equipment may be specially designed or may simply be a vibratory hammer attached to a pipe pile. The effectiveness of vibro-compaction in silty sands is marginal to good but in silts and clays is poor (Hayward Baker 1999a). The design process for vibro-compaction consists of first calculating the predicted settlement (knowing soil gradation and relative density) and then designing sufficient densification using vibro-compaction to reduce the expected settlement to a desired value. The final density is determined by acceptance testing us-

ing standard penetration, cone penetrometer, pressuremeter, or actual load tests. Several case histories document the use of vibro-compaction techniques to densify cohesionless soils. Tate (1961) reported that vibro-flotation performed on drained sands achieved a relative density increase up to 90 percent from the original 50 percent; Millard and Hassani (1971) disclosed that use of a vibratory hammer and pipe pile to densify a cohesionless soil increased the bearing capacity by 100 percent and reduced design settlement by 50 percent; and Jonker (1987) cited similar results using underwater vibro-compaction of the cohesionless materials in the Eastern Scheldt project.

(*j*) Closely spaced piles may be driven underwater to strengthen and stiffen the soil. Hydraulic modification to consolidate cohesive soils involves drainage techniques. Both vertical and horizontal drains have been successfully used underwater. The first underwater installation of vertical drainage wicks in North America was documented during site improvement prior to construction of the Pascagoula (Mississippi) Naval Base. More than 4000 wicks were placed underwater to consolidate the underlying soft bottom silts at the site of a new berthing pier. A 20-foot (6-meter) sand surcharge was used to cause the consolidation (Munn 1989). Disadvantages of wick drains include the need for specialized equipment, the requirement to provide consolidation surcharge, and the time required for consolidation to achieve significant gains in soil strength.

(*k*) Horizontal drainage to dewater silt layers at underwater depths of 64 feet (20 meters) has been done using a vacuum pumping system (Figure A-22). The purpose of dewatering was to allow larger volumes of dredged material to be placed in an underwater disposal area. Collector pipes were installed as successive silt layers were added to the disposal area.
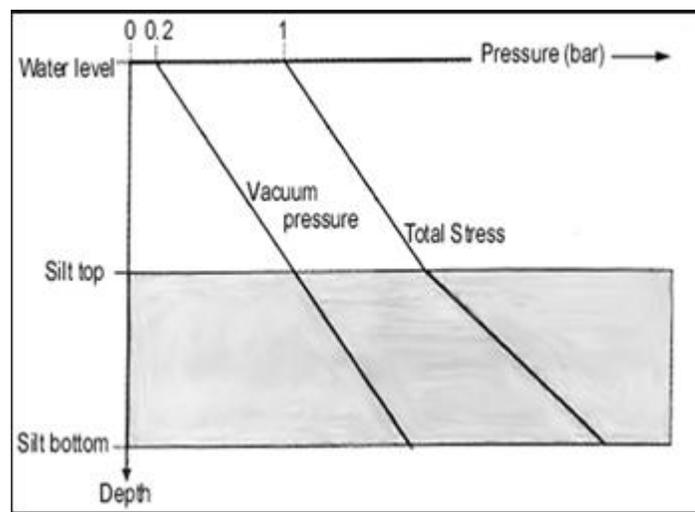
Figure A-22. Suction dewatering in an underwater silt layer (after Aerts et al. 1999).

(*l*) Storage capacity of the 986- by 1300- by 16-feet (300- by 400- by 5-meters) underwater disposal area was increased about 20 percent by dewatering the silt. Density after dewatering was increased by about 5 percent (Aerts et al. 1999).

(*m*) Chemical modification techniques have been used for relatively shallow soft soil sea-beds for bottom-founded offshore oil structures in the Arctic and for harbor areas of Japan. In the Alaskan Beaufort Sea, man-made islands have been constructed in the relatively shallow water depths of 10 feet (3 meters) to support drilling platforms. Halebsky and Wetmore (1986) discussed several methods of soil modification and concluded that, of the methods proposed (dredging and backfilling, soil wick drains, soil freezing, and deep cement mixing), deep cement mixing offered an economical advantage. The dredging and backfilling and soil wick drain methods were assumed to increase the original soil strength by a factor of 2, whereas the deep cement mixing method (in-situ soil strengthening using cement, or DCM) was found to increase the original soil strength by a factor of 70. In many locations, especially in Arctic regions, soils with shear strengths as low as 500 psf (24 kPa) are found. As exploratory oil-drilling structures require soil shear strengths of 1000 to 2500 psf (50 to 20 kPa) and permanent structures require greater shear strengths, increasing the soil shear strengths by a factor of 10 to 20 would make most weak soils acceptable.

(*n*) The major disadvantages of the DCM technology include the specialized equipment that is required and the required time for curing. This technology for underwater applications is more commonly used in Japan than in the United States. Japanese DCM barges operate in water depths to 230 feet (70 meters) and modify the soil to depths of 165 feet (51 meters) beneath the seafloor. Typical curing times to achieve maximum design strengths are 2 to 3 months.

(6) *Caisson.*

(*a*) *Suction.* The placement of suction piles (also called buckets, skirts, suction caissons, anchor piles, or suction anchors, see Figure A-23) operates on the principle of foundation under-pressure . The suction pile is placed on the sea bottom, and the entrapped water is pumped out to create an under-pressure relative to the surrounding water pressure, which causes the pile or caisson foundation to penetrate into the seabed (Senpere and Auvergne 1982). These are commonly called "skirts" in the offshore industry literature if they are attached to gravity-based foundations, especially tension leg platforms. (It should be noted, however, that the offshore literature also refers to "skirt" piles on jacket platforms, which are driven piles instead of suction piles.)

(*b*) *Advantages.* The advantages of self-penetrating (with suction) piles may include one or more of the following:

- Less equipment required for installation.
- Shorter installation time.
- May be fairly easily removed or relocated.
- Very effective in resisting vertical, and nearly vertical, uplift forces.

(*c*) *Disadvantages.* The disadvantages of self-penetrating (with suction) pile include:

- Highly dependent on soil conditions (technology somewhat limited on predicting penetration depth).
- Only suitable for water depths over approximately 100 feet (30 meters).

- Excessive differential pressure (under-pressure) may cause internal soil heave.
- Relatively limited and unproven technology outside the offshore industry.
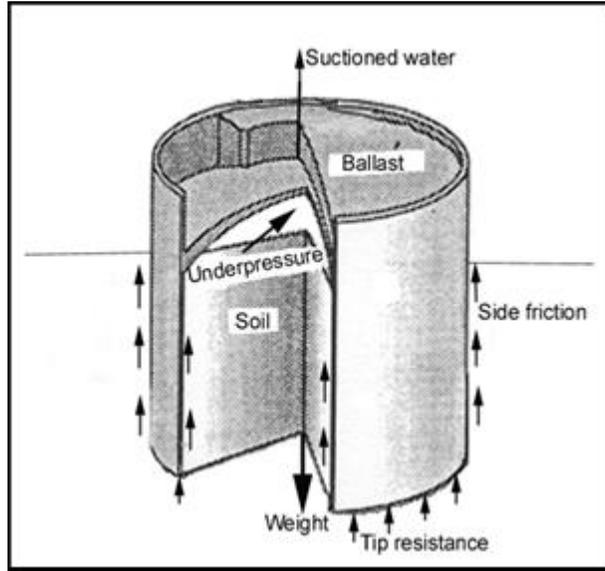
Figure A-23. Suction pile foundation (after Anderson and Jostad 1999).
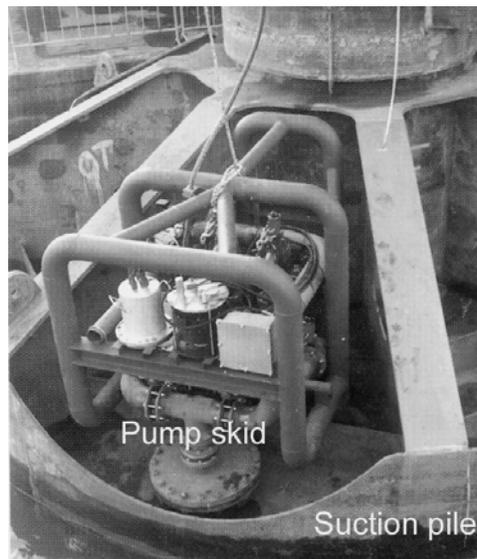
Figure A-24. Suction cassion (pile) installation (after Senpere and Auvergne 1972).

(*d*) *Installation.* As mentioned above, suction caissons are installed by self-penetration into the seabed. The initial phase is self-weight penetration as the caisson is lowered onto the seabed and allowed to "sink" under its own weight into the mud line (see Figure A-24). Further penetration requires an under-pressure in the cavity of the suction caisson, which creates an ad-

ditional driving force. As under-pressure is applied (usually by a skid-mounted pump on top of the caisson, see Figure A-25), the caisson penetrates the soil until the pressure inside the skirt walls tends to equalize with the external pressures. The rate at which the pore pressures tend to equalize and mechanisms by which the underlying soil tends to fail are ascribable to the material and strength properties of the soil. Set-up effects (time-dependent properties) are similar to those found in pile driving (Andersen and Jostad 1999).
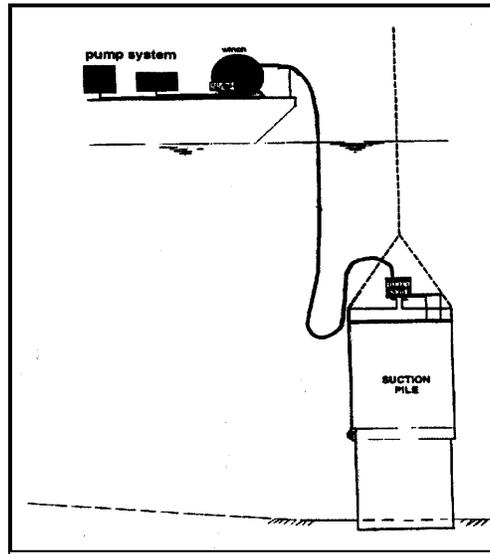
Figure A-15. Pile-mounted suction pump skid (after Suction Pile Technology, Inc. 1999).

(*e*) *Positioning*. Proper 3-D positioning of the foundation is required during the installation process. The installation procedures must be consistent with the design criteria, and advance planning for alternate or supplemental procedures should be made prior to mobilization of the marine construction equipment (Sullivan and Ehlers 1972). Tanaka et al. (1987) studied the installation procedures for placing a gravity-based foundation on an offshore Japan seabed. Model testing and numerical simulation confirmed the technical feasibility and provided recommendations prior to actual installation.

(*f*) *Design Criteria*. The design criteria for a suction pile is similar to the gravity-based and pinned foundation design methodologies. For the suction pile, the penetration resistance is a function of the soil's resistance to bearing and friction and is calculated as the sum of the shear along the walls and the bearing capacity at the tip. The penetration analysis includes calculation of the under-pressure (the difference between the hydrostatic water pressure outside the pile and the water pressure inside the pile cavity). The safety margin against soil heave inside the cavity must be determined to calculate the allowable under-pressure (Andersen and Jostad 1999). The calculation of self-weight penetration of the suction pile is a critical design aspect that must be addressed owing to its effect on the pile cavity volume.

(*g*) *Bearing and Holding Capacity*. The bearing capacity and holding (tension) capacity of suction piles are calculated by limit equilibrium methods. The critical failure surface depends on the combination of vertical, horizontal, and moment loads acting on the pile. Andersen and Jostad (1999) provide comments on design concepts and procedures for suction pile (skirted foundations and anchor) design. The following design aspects need to be analyzed for suction piles:

- *Penetration*.
    - Resistance.
    - Necessary and allowable under-pressure.
    - Soil heave inside cavity.
    - Maximum penetration depth.
    - Factor of safety.
    - Removal analysis if needed.

- *Capacity*.
    - Limiting equilibrium modeling.
    - Shear strengths.
    - Soil crack effects.

- *Displacements*.
    - Consolidation.
    - Cyclic and permanent displacements due to cyclic loads.

- *Soil Spring Stiffnesses* (dynamic analysis).

- *Soil–Structure Interaction*.

- *Effect of Cyclic and Repeated Loads*.

(*h*) *Excavated*. These methods include installing the pile, caisson, or drilled shaft during or after excavation of the soil or rock (see Figure A-26). The advantages of the excavated pile include the following:

- Lengths are easily adjusted to match the bearing stratum.
- Cuttings supplement or verify site investigation borings.
- Very large diameters are possible for extraordinary loads, compared with pile groups.
- Suitable for sites that preclude driven foundations, such as cases in which large rocks and boulders are encountered.
- The material used to construct the pile is not subject to handling or driving stress.
- Very long piles are possible.
- Construction noise and vibration are reduced.
- There is less influence on adjacent structures due to ground heave or disturbance.

Typical disadvantages of excavated piles include:

- Concrete and grout require special equipment and inspection considerations.
- Placement and inspection of reinforcing bar cage may require special techniques or considerations.
- Cannot be extended above riverbed without casing and special procedures.
- Possible end-bearing capacity reduction in sandy stratum.
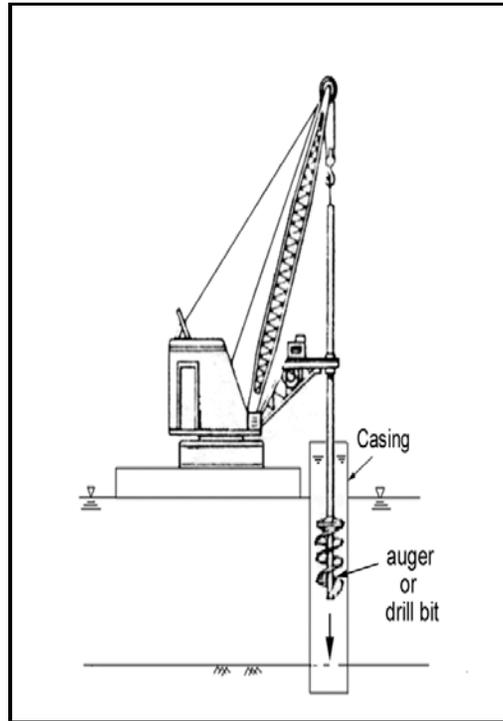
Figure A-26. Excavating a cased borehole.

(7) *Rock Anchors*. Rock anchors have been used for the rehabilitation of in-the-wet foundations, particularly for seismic retro-fitting. The use of rock anchors (installed in-the-wet) have also been proposed for retro-fit work on stilling basins, to control uplift, when dewatering the stilling basins for repair (including for J.T Myers lock, Cannelton Lock, and Newburgh Lock). Rock anchors are not commonly used for new Civil Works in-the-wet foundations be-cause rock anchors: i) have limited ductility, ii) are susceptible to accelerated corrosion unless adequately protected, and iii) are typically more expensive than other design solutions. Further-more, to be effective, rock anchors typically need to be post-tensioned, which for in-the-wet con-struction, normally means that a reaction pipe must extend from the foundations up through the water for the jack to react against.

(8) *Cut-off Walls*. Cut-off walls can be installed through the water and be tied into the structure underwater by tremie concrete closure pours. This was successfully done on Braddock Dam (See case history in Paragraph A-11) where both upstream and downstream sheet pile walls were installed through 40 feet of water. In addition to sheet piles, cut-offs underwater can be constructed using slurry, jet grouting, and injection grouting.

(*a*) *Sheet Pile*. Installing sheet piles in water is a standard procedure. However, constructing an underwater sheet pile wall with a top elevation 30 to 50 feet under water is not a standard procedure. Installing an underwater sheet pile cut-off requires different equipment and procedures. The sheet pile cut-off can of course be driven with long sheets that extend above water and the sheets can be cut off to grade underwater, but this requires handling excessively long sheet piles and the use of divers to cut underwater. An alternate procedure is to install king-piles at a spacing of 15 to 20 feet using a bottom founded or floating template for accurate spacing and alignment, and then to install sheet piles of the exact length between the king-piles. After installation of the king-piles, the sheets are threaded up between the king-piles with the tops of the sheet piles above water. Once a set of sheets between a pair of king-piles is threaded up, the set of sheets is then driven to grade with an underwater hammer or follower attached to the hammer. This procedure was used successfully on the upstream and downstream cut-off walls installed at Braddock Dam (Figures A-27 and A-28).

Figure A-27. Installing king-piles for underwater cutoff wall at Braddock Dam.

(*b*) *Slurry*. Slurry walls have been used to construct underwater cut-off walls for dams and large piers and shafts in Japan. To install slurry walls through the water, it is necessary to contain the slurry to prevent it from mixing with the surrounding water. This can be done by installing sheet piles that extend from the mud line to the water surface and filling with a cohesive soil, then constructing the slurry wall.

(*c*) *Jet Grouting*. Jet grouting through the water can be done efficiently from floating equipment; however, the discharge exiting the drill hole is much more difficult to control and can cause serious contamination of the surrounding water. Silt curtains can be used; however, they become more difficult to install and maintain as the water depth or currents, or both, increase. Use of an anti-washout admixture will help reduce segregation.

(*d*) *Injection Grouting*. Injection grouting can be done efficiently from floating equipment and can be as effective a cut-off technique below water as above. However, because the grouting is done below water, special equipment is required to detect loss of grout at the ground surface or heaving of the ground surface.

(*e*) *Underdrains and Relief Wells.* Vertical drains and horizontal drainage layers can be efficiently installed from floating equipment before floating-in or lifting-in precast elements. Alternatively, relief wells can be installed through sleeves cast in the in-the-wet structure after it has been landed and stabilized.

Figure A-28. King-piles floating template.

A-4. Site Exploration.

a. Increased Need Because of Inability to Visually Inspect Foundation. Foundations frequently contribute on the order of 40 to 50 percent of the cost of a structure, and can be a major source of delays and claims owing to the uncertain nature of the founding material, and thus merit extensive exploration. Furthermore, for in-the-wet work, the designer cannot rely upon visual inspection of the foundation subgrade, and must plan a more rigorous inspection by other means.

b. Evaluation of Aquifers—Artesian Aquifers.

(1) It is important to evaluate both aquifers and artesian aquifers prior to developing an in-the-wet foundation. Aquifers and artesian aquifers can occur in both porous granular soils and fractured rock. Failure to clearly identify, delineate, and characterize the nature of the aquifer, and the groundwater in the aquifer, can potentially lead to a variety of problems, including: i) inadvertent contamination of the groundwater in the aquifer if penetrated by the foundations; ii) boiling or piping from artesian groundwater pressures; and iii misinterpretation of soil design parameters.

(2) The most conventional means to characterize such aquifers is to carefully drill, and then plug, boreholes. Samples should be collected from the boreholes to characterize both the porous media in the aquifer, and the groundwater. As a minimum groundwater pressure measurements should be taken, but, depending on nature of the site and foundations, additional information that could be measured includes: i) hydraulic gradients; ii) permeability–transmissivity; and c) pump and draw-down rates. The following subparagraph addresses geophysical methods, some of which can be used to evaluate aquifers.

   c. Geophysical Methods for Broad Evaluation of Discontinuities. Geophysical methods have evolved as major investigative tools for underwater site assessment and geotechnical exploration. They are useful for detecting and delineating geological deformational features, including bottom and sub-bottom faults, scarps, rock and mud slides; for obstacle detection; landscape mapping; and for mapping other features critical to underwater foundation location and design. Geophysical methods are discussed in EM 1110-1-1802.

   (1) Near-surface geophysics do different functions at much shallower depths, but the principles are the same. Sound waves (acoustic and seismic), electrical resistance, electromagnetic signals, gravimetry, and radar principles and procedures are used in near-surface geophysical investigations, and each method is chosen for suitability depending on the intended function of the investigation (Beasley et.al. 1997a,b). Acoustic and seismic methods are typically used for underwater site investigations; the required depth of investigation usually dictates which method is employed. Acoustic methods (sub-bottom profiling, side-scan sonar, and echo-sounding) exploit compression wave propagation techniques and provide excellent results for determining stratigraphy just below the mud line. Seismic methods, exploiting reflected or refracted shear wave propagation, are useful for deeper investigations. In general, the denser the sub-bottom material is, the deeper the penetrability is for these two methods. The geophysical acoustic methods and hydrographic surveying methods often overlap owing to the available equipment and technology. Multibeam echo-sounding and side-scan sonar equipment is capable of providing detailed landscape mapping of the seabed (or river bottom), which allows interpretation of geologic deformational features as well as providing physical positioning data for engineering surveys (Reson 1999, ORE 1999, Ultra Electronics 1999). Side-scan sonar methods are also used for quality control and monitoring of dredging and ocean disposal of dredged material. Mosher et al. (1997) investigated using side-scan sonar mosaics to monitor ocean disposal at depths to 822 feet (250 meters).

   (2) Geophysical seismic investigations for offshore exploration use 3-D and 4-D (three- and four-dimensional time-lapsed) interpretation methods to map stratigraphy and reservoir deposits thousands of feet below the seabed. Advancing technology, such as ocean bottom cable systems, allows higher resolution and more accurate repeatability because of the fixed receiver (geophone and hydrophone) locations on and inside the seabed (Beasley et.al. 1997a,b).

   (3) Other geophysical technologies commercialized in the offshore industry that may have use for characterizing foundation sites are the seismic-while-drilling (SWD) method, in which the advancing drill bit serves as the seismic source, and the wireline vertical seismic profiling (VSP) method (McMillin 1999). The major advantage of the SWD method is reduced costs during drilling as the VSP method is not conducted in real time during the drilling operation.

(4) High-resolution seismic data are also collected in shallow-water environments such as rivers and tidal areas. Typically, only 2-D interpretation methods are used, because of increased costs in conducting and interpreting 3-D high-resolution surveys. River bottom soil and rock stratification may be detected to depths greater than 100 feet (30 meters) with 1-foot (0.3-meter) vertical resolution when interpreting 2-D data. Advanced processing and interpretation techniques allow a look at greater depths with higher resolutions. Instead of ocean bottom cables, the receivers are located on surface-towed platforms. Typically, a single channel system with a boomer source and a receiver is used, but multichannel systems are also used at more geologically complex sites.

(5) Rivers and canals are special challenges to geophysical investigations owing to navigation restrictions and changing currents during the course of an investigation, as well as interpreting multiple mode reflection geometry data affected by changes in river bottom and bank slopes (Toth et al.1997). As the available computer equipment and processing software become more adapted to shallow-water surveys, transition to high-resolution 3-D methods will likely become more commonplace in shallow-waterway project investigations.

  d. Specialized In-situ Investigation Tools. The topic of the use of specialized in situ investigation tools is not unique to in-the-wet foundation work, it is an ever changing field, and it requires the use of specially trained personnel and special equipment. Well over 50 different in situ field testing devices have been developed, and many of these are variations of more common in situ tests. It is recommended that the foundation designer consult with experts, or geotechnical testing firms, before specifying the use of any specialized in situ investigation tools. Nevertheless, their use can offer advantages for some in-the-wet foundation applications and thus selected in situ tools are briefly discussed in the following.

(1) The common Standard Penetration Test, SPT (ASTM D1586), has a variation SPTT that includes measurement of torque.

(2) The common Vane Shear Test, VST (ASTM D2573), has a variation VST/r that includes measurement of rotation angle.

(3) The relatively common Cone Penetration Test, CPT (ASTM D3441/D5778) includes numerous variations including: i) SCPT with downhole seismic data measurement; ii) PCPT with porewater pressure measurement; ii) PCPT/d is a PCPT test including time effects dissipation tests; iv) SPCPT measures combined bearing, skin friction, seismic data, and porewater pressure; and v) RCPT with resistivity measurements.

(4) The relatively common Pressuremeter Test, PMT (ASTM D4679) also includes numerous variations including: i) SBPMT, which is a self-boring pressuremeter (see discussion in Paragraph A-11*a*), ii) PIPMT, which is a push-in type; iii) FDPMT, which is a full-displacement type; iv) CPMT, which is a FDPMT plus a CPT cone; and v) PCPMT, which is a FDPMT with a PCPT piezocone.

(5) The relatively common Flat Dilatometer Test, DMT, comes in several variations including: i) DMT/t, which includes blade thrust; ii) DMT/c, which includes deflation readings, and iii) DMT/d, which includes dissipation readings with time decay. Also the SDMT combines a Downhole Test, DHT, with a dilatometer, DMT.

(6) The Downhole Test, DHT, has a variation DHT/d, which includes damping with depth (i.e., attenuation).

(7) The relatively common Cross-hole Test, CHT, has a variation CHT/d, which in addition to P- and S-waves, includes damping ratio to measure attenuation.

(8) Additionally, there are numerous in situ tools for measuring thermal properties of the soil, as well as frozen soil, and soil with gas hydrates, which are not discussed here.

A-5.   Subgrade Preparation and Improvement. In-the-wet design and construction infers that the foundation preparation and improvement will be through the water. Not all subgrade improvement methods are applicable for underwater subgrades. When evaluating potential ground preparation and improvement methods for underwater subgrades, it is important to consider the constructability or feasibility of the technique under the specific conditions of the site. These include: water depths, current speed and direction, tidal variation, wave action and interaction with ice. All of these factors can play a significant influence on the cost, risk and feasibility of the ground improvement method at a specific marine site. For an overview and description of available ground improvement techniques see ETL 1110-1-185. This ETL lists and describes 20 subgrade improvement methods and the applicability of each to different ground conditions. The applicability of each of these methods to underwater subgrade preparation and improvement is discussed below.

a.   Soil and Rock Foundation Preparation. The following sub-paragraphs address various aspects of in-the-wet foundation preparation for soil and rock, including: dredging and excavation, rock blasting, subgrade clean-up, subgrade stabilization, backfill, screeding and grading, compaction, scour protection, and maintenance dredging. Some methods for underwater rock excavation that are not discussed in the following sub- paragraphs include use of: i) surface supported chisels, ii) underwater hammers with a chisel attachment (e.g., hydro-hammer), iii) barge mounted backhoes, and backhoes with chisel attachment (e.g., Ho-Ram), iv) weighted heavy-duty clamshell buckets; v) heavy-duty clamshell buckets with underwater hydraulic closing mechanisms; vi) cluster drilling, and vii) cutterhead dredger. In the past, good results have been experienced with all of these tools in various marine foundation projects.

(1) *Excavation and Dredging.* Both terms relate to the removal of undesirable soil or rock materials and are often used interchangeably. Excavation generally implies a smaller operation, such as removing soil inside a cofferdam. Dredging implies a larger area operation and is usually required during installation of improved-site foundations and gravity-based foundations. Reasons for excavation and dredging include removing unsuitable materials such as loose or weathered rock or mud overburden, cutting specified depths and slopes in competent materials, retrieving bulk samples, and cleaning (removing) soft bottom (sediment) materials after dredging or backfilling operations have been completed (Johnson et al. 1972).

(*a*) Dredging to remove overlying soft materials may be done using airlifts, suction (hydraulic) dredges, or specialized trenching equipment. Dredging firmer materials may be done using clamshell (orange-peel) draglines (Figure A-29) or bucket (dipper) dredges in shallower water. Deepwater dredging is done with continuous dragline, cable with attached cutters, ocean dredging vessels (suction hydraulic or hopper with cutterheads, or long reach hydraulic excavators), or ROVs. Explosives are used to remove pinnacles and boulders, and drilling or blasting are used in rocky areas (Gerwick 1974). Harris (1942), Tate (1961), Millard and Hassani (1971), Johnson et al. (1972), and Loeken (1980) describe dredging operations on numerous projects, including the Navy's drydock projects, New York's Yonkers sewage treatment plant, San Francisco's marine terminal, Hong Kong's Plover Cove Dam, Ghana's Akosombo Dam, and offshore borrow areas. Also, the Tennessee-Tombigbee canal project used specialty "ripper-type" cutter heads for rock excavation.
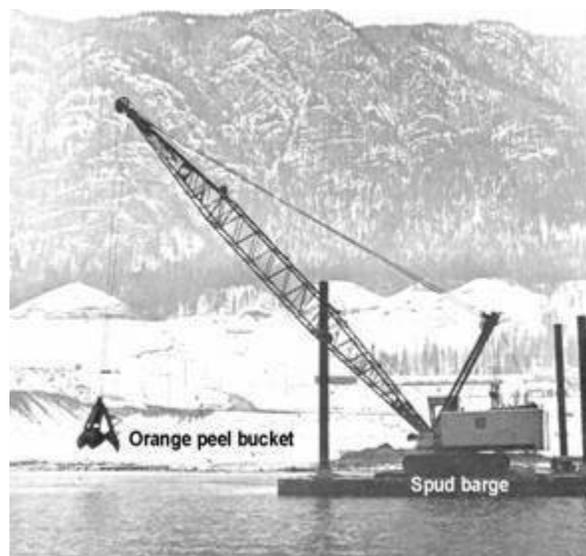
Figure A-29. Clamshell excavation and soil sampling in the Columbia River (after Bazett and Foxall 1972).

(*b*) Large-scale dredging and backfilling operations may use oceangoing vessels. Large, oceangoing, trailing suction hopper dredges have been specially designed for removing and backfilling bottom materials at depths to 430 feet (130 meters). These dredges pump slurried bottom material into vessel hoppers via suction pipes connecting the bottom-trailing dragheads to the vessel's main dredge pumps. The dredge then transports the material to the disposal location, where the hopper load is discharged for placement as underwater fill or is wasted. As of 2000, the world's largest trailing suction hopper dredger is Belgian *Jan De Nul's* with 116,450-cubic foot (33,000-cubic meter) hopper capacity. Smaller dredges are available for dredging near-offshore and inland rivers to depths necessary for maintaining navigation, usually less than 50 feet (15 meters). Submerged dredging platforms allow access to deeper locations, and are typically used offshore. One Netherlands company has developed an ROV capable of dredging to depths of 3300 feet (1000 meters) (Van Oord 1999). Specialized sub-sea ROV trenchers for pipeline burial are also available (Rushfeldt 1981; Dansette and Robertson 1994).

(2) *Blasting*. Drilled-in explosive fracturing has a long history in underwater rock dredging. The hole must be cased down into firm material so it is either driven in or drilled into the rock. The "OD" method is employed, by which the waterproofed charges of powder are lowered down, preferably using primacord. Sand is placed on top of the powder ("Stemming") and the leads of the primacord are brought out at the top of the casing and tied to a small float. The casing is pulled and the primacord is tied to the barge. After a series of holes have been so charged, the barge pulls back 60 to 100 meters and the round is fired. Acoustic blasting caps may also be used but are sometimes unreliable, especially if covered by silt.

(*a*) The effectiveness of the charge is increased by leaving the overburden in place and by blasting a full face at a time.

(*b*) Holes are typically drilled on a 2- to 3-meter spacing, using a template, to a depth of "half the spacing," a little more if this is a ship channel where no pinnacle is to extend above grade. Powder factors of 1.2 to 1.8 kg/m^3 and 60% powder will give best fracturing for later excavation. Note that blasted rock has a substantially higher volume than solid rock.

(*c*) Existing structures can be protected by controlling size and spacing of blasts and by air bubbling (Gerwick 2000).

(*d*) Notable underwater blasting projects included the Third Boston Harbor Tunnel and the Honshu-Shikoku Bridges in Japan. The Corps of Engineers has had a substantial number of channel deepening projects involving underwater blasting, which have been carried out by contractors.

(*e*) Underwater blasting can achieve any effect that can be achieved using in-the-dry blasting; however, underwater blasting has several considerations that are different from in-the-dry blasting, including:

- Cost of drilling charge holes.
- Influence of shock waves on marine life.
- Logistics.
- OSHA safety considerations for underwater blasting include:
- All non-approved vessels shall be more than 1500 feet (457 meters) away from the blast.
- No swimmers, or divers, shall be allowed in the water during blasting.
- Only water-resistant blasting caps should be used.
- Stray electrical currents, including those from galvanic cells, shall be avoided.
- Only approved blasters shall conduct all blasting operations.
- Blast flags shall be used during blast operations.

(*f*) In some cases where the underwater rock is layered and especially if it has multiple fractures, it can be removed mechanically without requiring blasting. Where appropriate, removal of underwater rock by mechanical means, such as by a chisel, or closely spaced drill holes, may be more economical and appropriate than blasting.

(3) *Airlift, Vacuum.* Airlifts, eductors, and underwater pumps are frequently used to remove loose material from a submerged foundation, as described in the following.

(*a*) Airlifts work by injecting a stream of air, underwater, into the bottom of a vertical pipe, or hose. The presence of the air within the vertical tube, or hose, reduces the bulk density of liquid, or slurry, thus reducing the pressure within the tube or hose, causing the liquid, and any loose material, at the bottom of the tube or hose to flow up into it, and to be discharged out of the top.

(*b*) Eductors introduce a high velocity water jet directed upward into the bottom of a submerged tube or hose. This causes the liquid, and any loose material, at the bottom of the tube or hose to be sucked up into the pipe or hose, and to be discharged out of the pipe. Typically, the top of the eductor pipe or hose remains underwater and points to the side of the excavation, where the loose material is discharged.

(*c*) Underwater pumps (such as Toyomenka pumps) can be very effective in sucking up or vacuuming loose material from an underwater foundation site.

(4) *Stabilization.* Frequently, excavated or dredged foundations must be stabilized before the subgrade can be prepared further. This is particularly true for sites with: i) fast moving water, ii) movable beds, including moving submerged debris, and iii) excessive sedimentation. Techniques for stabilizing the exposed subgrade include the following.

(*a*) Use of temporary flow deflection barriers, erected upstream of the site.

(*b*) Use of temporary mattresses, including fascine mats, articulated mats, and grout mats.

(*c*) Use of clay, or temporary stone, blankets dumped on the bottom, by means including: i) bottom dump barges, ii) material pushed off the side of a deck barge by a bulldozer; iii) clam shell, and d) tremie pipe.

(5) *Grading.* Typically, grading of underwater subgrades requires placement of an engineered backfill layer or layers, followed by screeding or compaction, as discussed in the following sub-paragraphs. Occasionally, the native material can be leveled, followed by other foundation preparation methods such as: i) underbase grouting or tremie concrete placement; ii) compaction, or iii) other foundation improvements.

b. Placement of Select Backfill. Select or engineered backfill may be required for a variety of purposes underwater, including: i) infill, ii) drainage, iii) scour protection, iv) filtration, and v) to facilitate subsequent screeding or compaction. As discussed briefly earlier, backfill can be placed by a variety of means, including:

- Tremie pipe.
- Bucket/clamshell.
- Side casting.
- Bottom dump barge.

- Hydraulic means (including use of long reach hydraulic excavators).

Unless special provisions are made to achieve, verify, and maintain the backfill quantities and elevations, then the designers are required to specify 50 percent additional backfill to account for material losses during underwater placement. Traditionally, payment for backfill material is based on the volumes of measure measured in the dry prior to placement; however, with adequate underwater survey information, it is possible to base payment on the volume of material actually in the specified locations. Hydraulically placed sand backfill will have low density and is therefore subject to liquefaction unless densified.

(1) *Leveling and Screeding*. If the superstructure is founded on an improved-site or gravity-based foundation, the bottom must be leveled to specified tolerances after dredging or backfilling. Typical out-of-level tolerances were described by Gerwick (1974) as being about 2 inches (5 centimeters) on hard bottoms to 12 inches (30 centimeters) on softer bottoms (sand and silt). Gerwick also stated that most leveling sequences include the following steps.

(*a*) A well-graded, crushed (not natural gravel) rock base course can be placed by barge bottom-dumping (effective only to about 100-foot [30-meter] depths), through a fall-pipe, or lowered in buckets. Natural gravel is not desirable owing to its tendency to displace laterally (slide) under load. Typical rock base courses use a maximum aggregate size of about 6 inches (15 centimeters).

(*b*) If desired, a smaller layer (screed course) up to 40 inches (1 meter) thick with maximum aggregate size of 3 inches (7.5 centimeters) can be placed on top of the rock base to enable more accurate screeding of the smaller particle sizes. Typically, the screed course is placed using a tremie pipe (fall pipe).

(*c*) Densification of the screed course by vibration is possible.

(*d*) Many types of screeding devices have been used on various underwater projects. All are designed to provide stable methods for leveling within the specified tolerances. The historical method for shallow-water depths used a heavy steel beam that was dragged across the bottom from a surface platform. Other devices are manually operated underwater and require divers. Self-leveling frames with remotely controlled positioning devices are used in deeper water. An electronically controlled hydraulic dredge head was used to screed the foundation layer for the Oresund tunnel.

(2) *Grading—Measuring and Monitoring*. Although it is sometimes possible to measure backfill volumes in-the-dry prior to placement, underwater grading requires in-the-wet measurement and monitoring of the graded surface. Means for such measurement and monitoring (both for technical and payment purposes) include the following.

- Tell-tale poles with targets, or survey towers, extending above the water surface from either the screed, or the screeded surface itself, working together with an above water survey system.
- Bathymetry (ranging from high to low resolution).
- Acoustic imagery.
- Underwater survey robots.

- Side scanning sonar.
- Pre-positioned bottom transponders.
- Sounding rods/poles and lines.
- Underwater reference frames/templates/molds.

(3) *Compaction Requirements.* Some underwater foundations require compaction of the subgrade material (either native, or backfill). That the specified degree of compaction has been achieved underwater can be verified by various means, including:

- An underwater test area that can be dewatered after compaction for examination and testing.
- Underwater cores combined with Standard Penetration Tests, SPT.
- Cone Penetrometer Tests, CPT.
- Gamma radiation density meters.
- Underwater survey (see grading measuring and monitoring above) before and after compaction to check for elevation or volume changes in the surface.

Common means for underwater compaction include:

- Dynamic compaction (e.g. the Menard method), has reduced efficiency underwater.
- Explosive compaction, has reduced efficiency underwater.
- Vibro-probe or vibro-floation compaction.
- Vibro-replacement.
- Vibrated compaction plates/rollers.
- Compaction piles.
- At depth, compaction grouting can be used.

c. Scour Protection. The following sub-paragraphs discuss various means of scour protection including: riprap, geotexile mats, and articulated concrete mats. Scour stone is another commonly used means of scour protection. Scour stone can be placed by any of the methods used for placing backfill, discussed previously. Tremie pipes can be used to place relatively thin layers of scour stone underwater with minimal disturbance to the riverine environment. Also, it is possible underwater to mechanically level (via compaction or vibration) scour stone with nominal sizes up to approximately 9-inches (230 millimeters). Such leveled layers of scour stone exhibit enhanced scour protection as compared with layers with rough un-compacted surfaces.

(1) *Riprap, Armor Stone.* Riprap and armor stone are the most common means of scour protection for Civil Works projects subjected to high currents, waves, and swells. The design of standard riprap and armor stone scour protection is not addressed here, and guidance on this topic can be found in numerous USACE documents, including EM 1110-2-1601 and EM 1110-2-1100.

(2) *Geotextile Mats.* Geotextile mats are frequently used for both temporary scour protection, and for permanent scour protection for projects with appropriate service lives. Geotextile fabric, sometimes attached to wood frames to form fascine mattresses, is often used as a filter layer or construction aid between the native soil and the scour stone or riprap.

(*a*) On a larger scale, the Oosterschelde (Eastern Scheldt) storm surge barrier in The Netherlands used fabricated scour protection mats consisting of layers of sand, fine gravel, and coarse gravel separated by layers of synthetic geotechnical fabric. As this project has a 200-year design life, the designers assumed that the geotechnical fabric would deteriorate leaving the layers of sand, fine gravel, and coarse gravel to provide long-term scour protection (Figure A-30). These mats were rolled onto large floating drums and were deployed using the giant mat-laying pontoon Cardium shown in Figure A-31.

(*b*) Geotechnical fabric can also be used to form grout mattress bags, which are filled with cement grout (or bentonite/cement grout) to provide scour protection. Olmsted Dam is using such a system as a construction aid to protect the footprint of the dam against scour until the piles can be driven, and the precast concrete shells installed and connected to the piles with tremie concrete.

(3) *Articulated Concrete Mats.* Articulated concrete mats are frequently used for both permanent, and temporary, scour protection of in-the-wet foundations. The Oosterschelde (Eastern Scheldt) storm surge barrier in The Netherlands used articulated concrete mats together with the geotextile/sand/gravel mats, both for permanent scour protection (see Figures A-31 and A-32), and used different articulated concrete block mattresses for permanently founding the precast concrete piers. It should be noted that it is common to attach a geotechnical filter fabric as a backing to articulated mats to conveniently meet both filter and scour criteria in an easily installed system.

d. Deposition, Maintenance Dredging During Construction.

(1) In appropriate cases, provisions must be made to maintain dredging deposits during construction. If riverbed migration causes most of the deposition, then control of this can be aided by excavating an upstream trench that is periodically dredged during construction to remove recently deposited material, as was the case for Olmsted Dam.

(2) In some cases, relatively thin layers of deposition build-up on top of structural surfaces, which must be cleaned prior to further construction of the structure. This was the case for the Oosterschelde (Eastern Scheldt) storm surge barrier in The Netherlands, where the Dutch made the custom built vessel *Macoma*, which vacuum-cleaned the founding mats immediately before the 18,000 tonne precast concrete piers were placed, before more sand could be tidally deposited on the foundation mats. Note that the *Macoma* also served as a pre-positioned, and pre-anchored, mooring pontoon for the Ostrea catamaran, which was carrying the pier modules. It is also possible to remove such thin layers of deposition using: i) jetting, ii) submerged pumps, and iii) removing protective sheets or mats with deposition on top of them.

Figure A-30. Sand, pea-gravel and coarse-gravel mattress for the Oosterschelde Barrier.
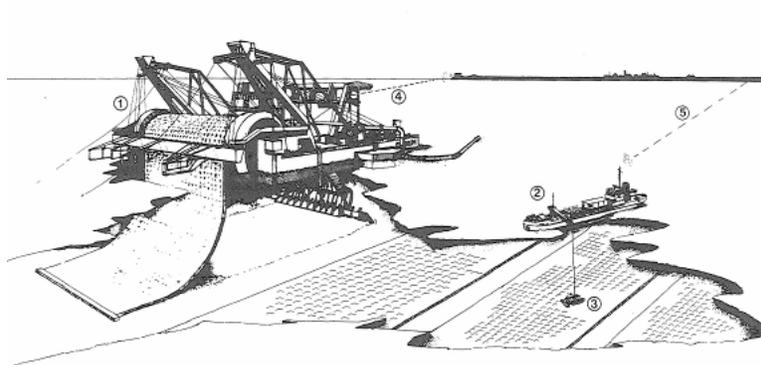
Figure A-31. Placing and cleaning geotextile/sand/gravel mats for the Oosterschelde Barrier: 1. Cardium; 2. Wijker Rib; 3. Portunus; 4. infra-red position location; 5. radio position location.
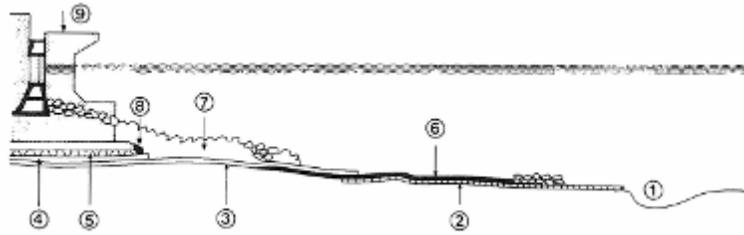
Figure A-32. Scour protection for Oosterschelde Barrier: 1. scour trench; 2. concrete articulated mat; 3. bottom geotextile/sand/gravel mat; 4. upper geotextile/sand/gravel mat; 5. articulated concrete block mat; 6. mastic asphalt slabs; 7. rip rap; 8. gravel bags; 9. barrier pier.

e. Soil Replacement. This is technically a viable alternative but gets very expensive with increasing water depths and current conditions. This method also raises concerns with environmental contamination of the water column during both removal and replacement of the bottom material. Silt curtains can be used to contain the spread of sediments in the water; however, they are expensive to install and maintain especially in deep water or currents exceeding 1 fps.

f. Admixture Stabilization. For underwater use, the admixtures are typically introduced through deep soil mixing. See Paragraph A-5*p, Deep Soil Mixing,* below.

g. Roller Compacted Concrete. This method is not applicable for underwater use owing to the loss of cement during the mixing and compaction process. However, graded stone has been roller compacted successfully in 20 meters of water for the Great Belt Bridge in Denmark.

h. Deep Dynamic Compaction.

(1) This method has been successfully used on underwater sand and rock embankments. However, it is not considered to be efficient for underwater ground improvement. The method uses a large weight dropped from a given height, and its effectiveness is a function of the weight and speed at impact of the falling mass. In water, both of these are significantly reduced; the speed is affected by drag and the weight is affected by the buoyancy of the mass.

(2) If dynamic compaction is used in the vicinity of underwater slopes, piezometers should be installed in the slopes to monitor the build up of pore water pressure and the potential increased risk of slope failure.

i. Vibro-Compaction. This technique is well suited for underwater ground densification of loose free draining sands. The vibrating probe is suspended from the tip of a crane boom and is relatively insensitive to barge motion. However, it may not be applicable for densifying loose material around existing structures. There is a risk that the existing structure will either rise up or settle, depending on whether the net specific gravity of the structure exceeds the specific gravity of the liquefied ground, typically in the range of 1.5.

(1) If layers of silt or clay cap the ground that is to be densified, they will prevent drainage and seriously limit the efficiency of the vibro-compaction. Therefore, vertical drains should be installed prior to starting vibro-compaction. These drains can be wick drains, gravel drains, or stone column (see below).

(2) Internal vibration of a zone of underwater loose sand or gravel does not compact the near surface layer. Vibratory plate compactors have been used successfully underwater to compact this upper layer.

j.   Stone Columns (Vibro-Replacement). The technique for installing stone columns is very similar to vibro-compaction, and, therefore, well suited to being done from floating equipment. One advantage of stone columns and sand compaction piles is that these systems both densify and provide vertical drainage at the same time.

k.   Gravel Drains. Gravel drains are a type of stone column, and, therefore, also well suited to being built from floating equipment. However, the primary function of the gravel drain is to induce consolidation through the dissipation of excess pore water pressures generated during earthquakes. In an underwater environment subject to siltation over time, these drains may become clogged with silt, and, thereby, be prevented from working during an earthquake. This may be overcome by placement of a graded stone layer on top of the gravel drain to prevent silt infiltration.

l.   Sand and Gravel Compaction Piles. The construction equipment and techniques for sand and gravel compaction piles are the same as vibro-compaction, and therefore, well suited to being done from floating equipment. Like stone columns, these two methods both densify and provide vertical drainage at the same time.

m. Explosive Compaction. This technique is typically less effective underwater than on dry land. With explosives, compaction is induced by both the blast pressure and by gravity. While the blast pressure is more effectively transmitted underwater than above, the effects of gravity are reduced by about 50 percent owing to the buoyancy of the submerged ground.

n.   Permeation Grouting and Compaction Grouting. These techniques work as effectively below water as they do above. However, because the grouting is performed below water, it requires special instrumentation to detect loss of grout at the ground surface or heaving of the ground surface.

o.   Jet Grouting. Jet grouting works as effectively below water as above. However, the discharge exiting the drill hole is much more difficult to control and can cause serious environmental problems by contaminating the surrounding water. Silt curtains can be used, but they become more difficult to install and maintain as the water depth or currents increase. Use of anti-washout admixture will help reduce contamination.

p.   Deep Soil Mixing. Deep soil mixing has been successfully done underwater. This process has been used extensively in Japan for underwater ground improvement. The technique was also used successfully on the underwater ground improvement for the Fort Point Channel

Immersed Tube Tunnel in Boston. The cost for deep soil mixing underwater is typically twice the cost of underwater sand compaction piles.

q. Mini-Piles. Mini- or micro-piles can be installed from floating equipment and work effectively underwater. For float-in or lift-in structures, the mini- or micro-piles can also be installed after landing of the elements. With this method, the mini-piles are installed through sleeves in the precast elements and then grouted.

r. Prefabricated Vertical Drains. Prefabricated vertical drains (wick drains) are effective in consolidating silts and clays. They can be efficiently installed through the water from floating equipment. Their effectiveness, however, can be reduced by heavy siltation unless free drainage at the top is maintained. A drainage layer of gravel at the top of the drains is typically provided for long term drainage. Fill or surcharge backfill can be placed to accelerate the consolidation if time is critical.

s. Electro-Osmosis. This technique will work both above and below water. Electro-osmosis is based on decreasing pore water pressure and causing settlement by gravity. The effectiveness of this technique for underwater use is therefore reduced by the buoyancy of the submerged ground.

t. Buttress Fills (Surcharge). This technique works underwater; however, it achieves compaction by gravity, and the net surcharge load for a given depth of fill is reduced by about 50 percent by the buoyancy effect of the ground. This loss in surcharge weight, in combination with the very high relative cost of placing and removing soil underwater, significantly reduces the cost effectiveness of this technique underwater. This method also raises concerns with environmental contamination of the water column during both placement and removal of the surcharge material. Silt curtains can be used to contain the spread of sediments in the water; however, they are expensive to install and maintain, especially in deep water or in currents exceeding even 1 fps.

(1) When placing underwater fills, the air content of the fill at time of placement has a very significant effect its control. Air bubbles attached to the fine particles give them added buoyancy. This has the effect of both segregating the fines and making the fill act as a heavy liquid on the bottom. One technique for minimizing this effect is to thoroughly saturate the fill prior to underwater placement (Gerwick 1999).

(2) Tremie pipes can be effectively used to place underwater fills. This technique minimizes segregation, is environmentally sound, and helps to minimize spreading of the fill on the bottom. If rock is being placed with a tremie pipe, the pipe diameter should be 3 to 5 times the diameter of the largest rock to prevent plugging. The smaller diameter pipes should be limited to placing naturally rounded aggregate and the larger used for crushed rock.

(3) When underwater fills are placed, they tend to act like a heavy fluid with a specific gravity in the range of 1.5. It is, therefore, essential to be careful when placing underwater fills adjacent to existing structures, such as pipelines, to ensure that the heavy liquid does not flow under the structure, lift it, and shift it horizontally. Rock berms placed around the perimeter of

the fill area are an effective means of limiting the spread of hydraulically placed sand for under-water fills.

(4) In silts and clays the effectiveness of the surcharge can be greatly increased by combining this technique with vertical drains. These can be prefabricated wicks, stone columns, or sand compaction piles.

u. Biotechnical Stabilization and Soil Bioengineering. This technique has been used successfully on the Mississippi River to control scour under caissons where willow mattresses are sunk on the river bottom before landing and sinking of the caissons.

A-6. Location and Positioning Methods.

a. Introduction.

(1) Control of positioning is fundamental for in-the-wet foundation construction, and re-quires more advance planning than in-the-dry foundation construction. The fact that construction of the underwater foundations cannot be observed directly, and that construction equipment may be floating and moveable, means that surveying and positional control requires the use of more advanced systems and procedures such as GPS, templates, guides, survey towers or poles, sonic transponders, and adjustable positioning devices such as flat jacks, hydraulic rams, jack screws, spuds, mooring lines, winching systems, dynamic positioning and adjustable shims. The use of floating construction equipment may require control of the six degrees of vessel motion (heave, pitch, roll, sway, surge, and yaw) in response to environmental forces that may have three com-ponents (steady/quasi-static, low-frequency, and high-frequency).

(2) Guidance for positioning large prefabricated float-in and lift-in units can also be found in the Innovations for Navigation Projects Research Program report "Positioning Systems for Float-in and Lift-in Construction in Inland Waterways" (Yao and Gerwick 2002), which contains information that is also useful for positioning systems for in-the-wet foundation con-struction.

b. Global Positioning System (GPS). Global positioning system (GPS) surveying is ad-dressed in EM 1110-1-1003 and EM 1110-1-1004, and uses the NAVSTAR Global Positioning System established by the U.S. Defense Department. GPS is relatively mature (USACE has used static GPS since 1983, and dynamic GPS for real-time control of marine floating plants since the early 1990's), but it is still a changing industry with a wide range of available private industry vendors and products with a wide range of applications and accuracies. Up to date information on GPS systems, vendors, procedures, equipment and applications may be obtained through the US Army Topographic Engineering Center's web site: http://www.tec.army.mil/info_links.html.

(1) For in-the-wet civil works foundation applications, it is common to use GPS survey-ing methods together with conventional survey control systems. In particular, conventional lev-eling methods are generally used to determine height elevations of benchmarks for establishing vertical control. When the in-the-wet foundation construction is nearshore, then GPS-derived po-sitions may be used to provide primary horizontal control of monument locations, from which

construction alignment work may be done using conventional surveying by instruments moni-toring targets on templates or guides extending above water from the underwater foundations. Alternately, GPS surveying equipment may be positioned on marine floating plants (such as dredgers, pile drivers, screeders, cranes, hydrographic survey vessels, etc.) for direct positional control.

(2) NAVSTAR consists of a system of satellites that circle the globe in controlled orbits, while continuously broadcasting their positions to locations worldwide. A GPS device receives this broadcasted positional data and calculates its distance from each visible satellite. With at least three visible satellites, the GPS device uses triangulation to calculate latitude and longitude location coordinates. With at least four visible satellites a GPS device can also calculate its alti-tude.

(3) Each satellite broadcasts both a precise signal and a standard signal. The precise sig-nal is only available to the U.S. military, while the standard signal is available for civil applica-tions. Standard level (code phase) GPS systems have accuracies from 1 to 30 meters (as of 1 May 2000 Federal regulations have discontinued "Selectivity Availability," which was use to limit standard signal accuracy to within 100 meters and the U.S. Defense Department reserves the right to reintroduce the practice in certain geographical areas), which can be useful not only for positioning the foundation elements but also for tracking equipment and personnel on the construction site. Differential GPS (DGPS) can provide meter-level control (the generic term "meter-level GPS" refers to 2-D accuracies ranging between 0.5 to 1 meter), which should be sufficient for monitoring the real-time operations of most floating plants. WAAS (Wide Area Augmentation System) is only available in North America and is a system of satellites and ground stations that provide GPS signal corrections, without the need to establish an independent ground station for DGPS. Differential Carrier-Phase GPS can provide centimeter-level control (the generic term "centimeter-level GPS" refers to 1-D, 2-D, or 3-D GPS accuracies ranging between 1 and 10 centimeters), which may be sufficient for monitoring many structural in-the-wet foundation applications (frequently used in conjunction with conventional survey systems and targets on templates or guides). The designer should be aware that systems with increasing levels of positional accuracy have higher expenses than less accurate systems.

c. Templates. The use of templates is standard practice for positional control of in-the-wet foundation construction (see Paragraph A-11 for numerous examples, including those shown in Figures: A-63, A-78, A-79, A-87, A-88, and A-115). Templates can be used for such activities as: bottom preparation (e.g., dredging, screeding, and ground improvement), pile driving, cut-off wall installation, and controlled demolition. Templates are fabricated in-the-dry, transported to the wet site, installed to the surveyed tolerances, and used to control the positioning of the final structure. Templates can be either temporary, such as steel frames, or stay-in-place, such as a precast concrete template tied into the final structure by means of tremie concrete. Templates can be either fixed, or adjustable, and typically can exert some force on the production element being installed to keep it in place.

d. Guides. Marine foundation construction uses static guide systems, such as mas-ter/king piles, dolphins, horn guides, taut lines, and stabling guides, to complement templates (and are frequently placed on templates) for positional control of production elements (see Para-

graphs A-2 and A-11 for numerous examples including those shown in Figures A-3, A-4, A-5, A-104 and A-110). If stringent positional control is required, then active guide systems such as flat jacks, hydraulic rams, wedge shims, and jack screws can be used. Such guide systems can be pre-installed and surveyed (say by a template), or they be attached to a previously installed element or an in-coming element. Grout bags can also be pre-installed, and inflated, using a survey tower, in order to act as a guide. Casting and molds can also be made of underwater interfaces, then transported to the surface, where customized guides can be made using the castings and molds to match the underwater interface. Guides can either extend to the surface, as may be the case for dolphins or master/king piles, or they may remain fully submerged, as may be the case for pintles or horn guides.

e. Optical Surveys. Optical, and laser, survey systems can readily be employed to survey templates, or survey towers and poles that extend from the foundations to above the water.

f. Sonic Transponders. Sonic transponders are frequently used for positional control in the offshore industry, or anywhere else, where water depths are too great to allow templates or other positional guides to extend above the surface.

A-7. Installation and Construction Methods.

a. Staging—Temporary Works.

(1) *Pile Templates.* An essential factor in the feasibility of implementing in-the-wet construction techniques is the ability to pre-install foundations underwater to an accuracy that allows mating of the precast float-in or lift-in element to the pre-installed foundation. Templates are the primary tool used to position and maintain piles and drill casings to a specified tolerance. Templates can be divided into two categories: floating and bottom founded.

(*a*) Floating templates can be frames attached to the crane barge or they can be supported on their own pontoons and moored with anchors or spud piles. The effectiveness of a floating template to meet a given set of tolerances depends on the current, wind, and wave conditions at the particular site. Water depth can also be a significant factor in the verticality of the installed pile and the horizontal position at cut-off in deep water. Floating templates can be effective in positioning piles or drill casing at the surface of relatively calm water to within ±6 to 12 inches.

(*b*) Bottom founded templates are typically supported by spud piles driven through sleeves in the template frame. See Figure A-33 of the bottom founded template used to install the drill shaft casings for the Bath-Woolwich Bridge. The frames are floated into position on temporary floats that are removed after the spud piles have been driven, and the frame is pinned-off above water to the spud piles. This lower frame is typically positioned to within ±12 inches at the surface. A sliding frame with a pocket or sleeve for the piles is then positioned on top of the lower frame. See Figure A-34 of the upper sliding frame used at Bath-Woolwich. This upper or sliding template is slid to a tolerance within fractions of an inch and locked into position. The pile or drill casing is then stabbed through the upper template. Bottom founded two-stage templates of this type can effectively position piles or drill casings to within ±2 inches at the surface and to within a vertical tolerance of 1 percent.

Figure A-33. Bottom founded template used to install drilled shaft casings at the Bath-Woolwich Bridge.

Figure A-34. Sliding upper guide used at Bath-Woolwich Bridge.

(2) *Pile Top Cofferdams for Underwater Cut-off.* In-the-wet construction often requires the cut-off of piles or drill casings underwater. This can be done using underwater robotic cutting tools from either the inside of the pile or from the outside; however, divers have achieved precision (±2 to 3 inches) cut-off using guidance from the surface. Alternatively, temporary circular cofferdams with a diameter about 2 feet larger than the pile or casing can be lowered over the outside and sealed to the exterior by inflatable seals (Figure A-35). These cofferdams extend from above high water to a few feet below the cut-off point. After sealing, the annulus between the pile and the cofferdam is dewatered and cut-off from the inside of the pile.

**Figure A-35. Local section of pile top cofferdam for un-
derwater cutoff of piles or casings.**

(3) *Guide Frames*. The mating of the lift-in or float-in elements to the pre-installed foundations is typically done underwater and out of site. Guides are therefore used to force the lift-in or float-in elements to mate during the final stages of threading the foundation elements onto the pre-installed foundations. Where existing structures are available, such as an adjacent lock for a float-in dam segment, the guide frame can be mounted on top of the lock. This technique was successfully used to position the first dam segment at Braddock Dam. See Figures A-36 through A-38. After landing the first segment, the guide was transferred to the leading end of the first segment and used to guide the second segment. If there are no adjacent structures to use as a support for the guide frame, an alternate method is to attach the guide frame to the foundation element. This method was used successfully to position the float-in pile cap for the New Carquinez Strait Bridge at the north end of San Francisco Bay. Each tower foundation for this suspension bridge is supported by twelve 10-foot-diameter drilled shafts in 100 feet of water. The drilled shafts were pre-installed and cut-off 12 feet below high tide prior to floating in the precast pile cap. Four L-shaped guide frames were welded to the top of the drilled shaft casing, just below the cut-off point. These four frames acted as a corral to box in the floating pile caps in the 3 knot tidal currents as it was ballasted down onto the drill shaft casings.

Figure A-36. Positioning Braddock Dam with guide attached to existing lock wall.
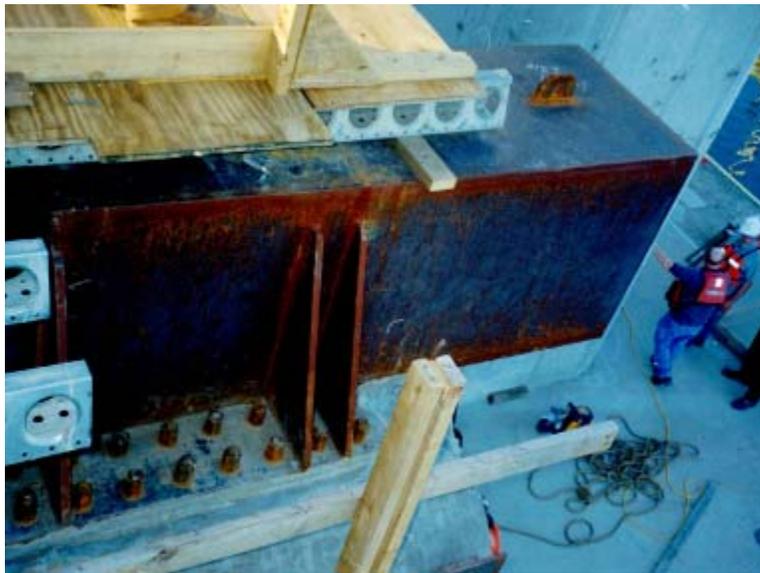
Figure A-37. Side view of male guide arm positioned above the horn-guide at Braddock Dam.

**Figure A-38. Plan view of the horn-guide at Braddock
Dam as the dam segment is ballasted to grade.**

(4) *Staging Interface with Final Structure.* Installation templates for the piles or drilled casing, and guide frames for mating of the float-in elements, are typically removed once they have completed their function. This eliminates any interference with the completion of remaining structure.

b. Drilling Methods. The type of drilling method and equipment are usually determined by the contractor. Selection of the method is based on ground or rock characteristics, local practice within the area where the work is done, and available equipment. For in-the-wet construction, drilling can be done from an access trestle, a jack-up barge or, more typically, from floating equipment. When drilling is done from floating equipment, it is necessary to isolate the drill string from the barge motion to give a consistent downward force on the drill face. This can be done by allowing the drill string to rest on the bottom of the drill hole or by mounting the drill unit on top of the pile.

(1) *Rotary Methods.* Rotary-drilling, as the name implies, is the circular motion of a drill bit on the face of the drill hole. The three main elements of a rotary-drilling are the drill rig, the drill shaft (or kelly bar), and the drill tool. The drill rig consists of the drive motor for turning the drill shaft, rotary table for transmitting torque from the drive motor to the drill shaft, and the transport system, usually a truck crane, crawler crane, track-mounted boom. The drill shaft is suspended from the rig and passes through the rotary table. Figure A-39 shows a drill rig in operation. The drill tool is attached to the bottom of the drill shaft and cuts the soil or rock at the face of the drill hole. A variety of bits are used for the cutting, including drilling buckets, flight augers, rock augers, core barrels, shot barrels, full-face excavators, and under-reamers or belling buckets. All of these tools have been used successfully for drilled shaft construction both on land and over open water (Figures A-40 through A-43).

Figure A-39. Typical rotary drill rig Bath-Woolwich bridge.

Figure A-40. Double wall shot barrel with roller cone bits.

Figure A-41. Rotary barrel working over water.

Figure A-42. Double wall shot barrel drill tools at Bath-Woolwich Bridge.

(2) *Percussion Methods*. Percussion drilling involves, first, breaking up the rock or hard soil with an impact tool, and then removing the broken material with a clamshell bucket or some other form of vertical lift. The three main elements of percussion drilling are the crane used to do the heavy lifting, the impact tool that breaks up of the rock, and the lift mechanism that removes the broken material from the drill hole. The crane can be either a truck crane, crawler crane mounted on a flat-deck barge, or a marine crane. The breaking tool is typically a heavy mass with a wedged-shaped face at the bottom end designed to concentrate the impact force at the drill face. This tool can be either lifted and dropped by the crane ("churn drill") or it can be operated by a hydraulic motor in the drill string. The broken material can be cleaned out with a clamshell bucket, by airlift, or by circulating drill fluid. Down-the-hole drills are very fast and efficient in

hard rock but may not develop sufficient reaction in plastic clays; they cause some fractures in the sides of the drilled hole.

Figure A-42. Barrel with 72-inch diameter and cutters welded to bottom of casing to seat it into top of weak rock—Bath-Woolwich Bridge.

(3) *Other Methods*. Other drilling methods include rodless drills and pile-top drill rigs(Figures A-44 through A-47). The rodless drilling consists of down-the-hole motors that drive excavating cutters that rotate in a column of bentonite drilling slurry (O'Neill and Reese 1999). The cutters breakup the rock and the cuttings are removed from the drill hole by reverse circulation of the drill slurry. Pile top drill rigs are similar, but the drive motor sits on top of the pile and turns the drill string with the drill bit at the bottom. The drill cuttings are brought to the surface with the circulating drill slurry through the drill string. From the pile top, the slurry is pumped through a flexible line to settling tanks where drill cuttings are removed and the drill slurry is reconditioned before being pumped back into the drill hole.

Figure A-44. Pile-top drill rig used for 10-foot-diameter drilled shafts on the New Carquinez Bridge.
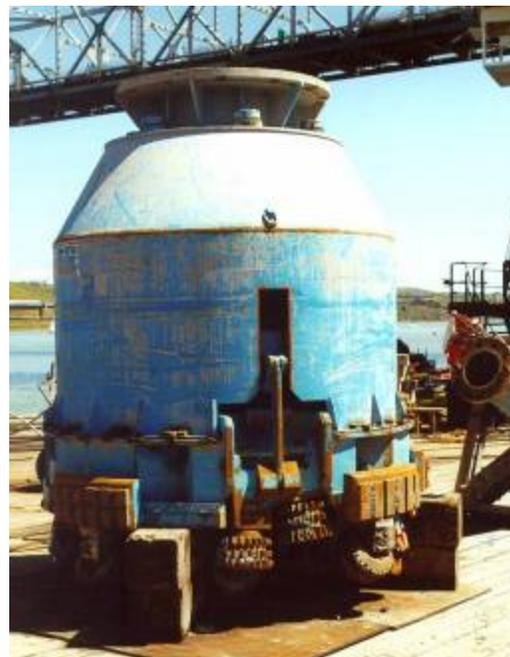
Figure A-45. Under-reamer bit with 10-foot diameter used on the drilled shafts on the New Carquinez Bridge.

Figure A-46. Rotary cone drill bits used on the New Carquinez Bridge.

Figure A-47. Centralizers used on the drill string at the New Carquinez Bridge.

    c. Driving Methods.

      (1) *Introduction.* Pile foundations are installed using impact-driven, vibration-driven, jacking, jetting, suction, or drilling techniques, or combinations of these. The most common method of installing onshore piles is dynamic impact from a variety of hammer types. Pile-driving hammers include the simple drop hammer, single-acting hammer, double-acting hammer, differential hammer, hydraulic hammer (which can be used underwater), and the diesel hammer

(ASCE 1984). Impact hammers require a D/t-ratio of about 50 to withstand buckling stresses under sustained hard driving. See API (1989) RP2A for guidance.

(*a*) Hammer energy is supplied by air, steam, hydraulic pressure, diesel, or electricity. Vibratory hammers produce dynamic forces by internal counter-rotating masses using air, steam, electric, or hydraulic energy sources. Jetting, jacking, and screwing methods are specialized, and their use for underwater piles has not been widely found in the literature. The majority of underwater piles have been installed using impact-driven methods, although vibratory-driven methods are gaining in popularity for offshore operations. The principal reason that offshore vibratory hammers are not as popular as impact hammers is because of the larger offshore pile weights, which require greater energy for penetration (Jonker 1987). Vibratory hammers have been assembled in groups of two and four (and more) for installing large diameter steel cylinder piles and (in China) for installing large concrete cylinder piles.

(*b*) Offshore piles driven with above-water steam-impact hammers dominated through the 1970s. Pipe piles ranging from 30 to 48 inches (75 to 120 centimeters) in diameter with non-spliced lengths from 200 to 350 feet (60 to 110 meters) were routinely driven (total spliced pile lengths) up to 800 feet (240 meters) below the mud line in the Gulf of Mexico. Offshore pile hammers (typically single-acting steam-driven) with rated energies from 60,000 to 300,000 ft-lb (80 to 400 kNm) were used to drive these piles to their design penetrations. The resulting ultimate pile load capacities were up to 3500 tons in compression and 2000 tons tension (Sullivan and Ehlers 1972). Pile-driving experience in the hard clays of the North Sea caused overestimation of pile hammer capabilities and underestimation of the pile bearing capacities (Fox et al. 1970). Offshore (above-water) pile hammer manufacturers included Vulcan, MKT, Delmag, and Menck. Steam hammer energy capacities increased to 600,000 ft-lb (810 kNm) by the mid-1970s.

(*c*) Today, hydraulic hammers up to 1700kNm (1,300,000 ft-lb) of energy per blow are increasingly being used to drive high capacity steel cylinder piles. These hammers have been used to drive 8-foot diameter steel piles to over 300 feet depth for the East Bay Replacement Bridge across San Francisco Bay.

(2) *Driven Caissons*. Driven caissons are essentially large steel, and prestressed concrete, driven pipe piles. Hammers for such piles include: air-steam hammers, impact hammer, hydraulic hammers, and vibratory hammers, which are discussed in the following subparagraphs. See also the discussion on large diameter pipe piles in the sub- paragraph on pile types.

(3) *Underwater Impact Hammers*. Development of the submersible hammer in the mid-1970s opened up new frontiers in pile-driving history. Pile followers, steam lines, and costly anchored derrick ships were no longer needed. The Hydroblok hydraulic hammer (by HBM, Hollandsche Beton Groep N.V.) operated an internal piston using nitrogen gas supplied through an umbilical cord to the underwater cage attached to the pile top. The rated energy of the HBM 3000 was 800,000 ft-lb (1,080 kNm). It could be used to drive an 84-inch- (2.1-meter-) diameter pipe pile (Jansz et al. 1976). Jansz (1977) reported that this underwater hydraulic hammer successfully drove piles in the Gulf of Mexico and the North Sea. In hard clay locations, a method to vertically align piles topped with the HBM hammer was devised.

(*a*) Alignment and lateral support problems of submerged pile hammers were addressed (Jansz and Brockhoff 1979). For the Eastern Scheldt project, an HBM 1500 hammer was used for driving mooring anchor piles. Sitter (1980) reported the development of an underwater hammer (HBM 4000) with rated capacity of over 1 million ft-lb (1 350 kN-m) of energy. The Menck company, which previously had made steam hammers, began making underwater hammers (such as the model shown in Figure A-48).

(*b*) Another submersible hammer was manufactured by the Raymond Company (RU-300). Its rated energy was 300,000 ft-lb (404 kNm). Its first offshore assignment was off the coast of New Zealand on the Maui jacket platforms that have 48-inch- (1.2-meter-) diameter open-ended pipe piles driven 230 feet (70 meters) below the mud line (Gendron et al. 1978; Rennie and Fried 1979).

(*c*) Several other innovative pile-driving methods were introduced during the late 1970s. These developments include the submerged water hammer pile driver, which used seawater pressure to create a cyclic water-driven impact mechanism to drive the pile (Wisotsky 1978), and electro-osmosis, which used direct current and achieved considerable pile-driving resistance reduction in clay deposits through the soil chemistry bond force reduction (Rose and Grubbs 1979).

Figure A-48. Menck 500T underwater hammer (after Menck 1999).

(*d*) Slender hydraulic underwater hammers were introduced in 1980 by Menck, Inc. By 1983, the hammers were instrumented with an adequate internal monitoring system, comparable to the above-water hammers (Van Zandwijk 1986). Slender hammers proved to be much more versatile for offshore jacket pinning than were the larger diameter first generation of underwater

hammers. Currently, underwater hammers manufactured by Menck GmbH (Germany) and IHC Hydrohammer BV (Netherlands) are used for offshore applications.
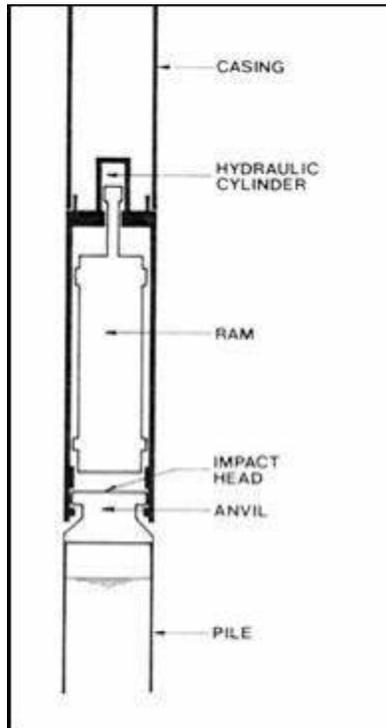
Figure A-49. Underwater hydraulic slender hammer (after VanZandwijk 1986).

(*e*) Both manufacturers build hydraulic hammers for underwater applications. Menck's largest hydraulic hammer (MHU 3000T) has a rated energy of 2.4 million ft-lb (3300 kNm) and weighs 400 tons. IHC's largest hydraulic hammer (the S-2300, shown in Figure A-49) has a rated energy of 1.7 million ft-lb (2300 kNm) and weighs 260 tons. Piles up to 102 inches (2.5 meters) outside diameter may be driven with these hammers (Menck 1999, IHC 1999).

(4) *Underwater Vibratory Hammers.* An alternate method of installing piles is pile driving using vibratory methods. Vibratory hammer technology was developed by the Russians for onshore applications in the 1960s. The German and Japanese foundation industry quickly embraced this technology and developed an extensive array of vibratory hammers. The Americans became interested in vibratory hammers in the early 1970s.

(*a*) Vibratory hammer power is supplied by electricity or hydraulics. The vibratory hammers that use hydraulic power normally use a diesel engine to drive the hydraulic pump, which in turn drives the excitation motor on the vibratory hammer. In the United States, hydraulic vibratory hammer systems have become dominant. The major manufacturers, including Vulcan, ICE (see Figure A-50), and MKT, offer hydraulic systems exclusively (Warrington 1989).
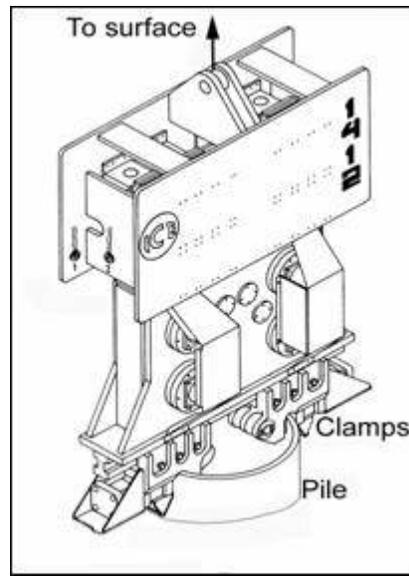
**Figure A-50. ICE 1412 hy-
draulic vibratory pile ham-
mer (after Jonker 1987).**

(*b*) The vibratory hammer does not incrementally impart energy to the pile–soil system as does an impact hammer, but instead provides a rapidly alternating dynamic force to the pile. The hammers are designed to impart this dynamic force to the pile at a range of frequencies, i.e., from approximately 300 to about 7200 rpm (5 to 120 Hz), depending on the type of pile to be driven. Frequencies above 30 Hz are considered to be "high frequency." These hammers gener- ally provide for lowered transmission of ground excitation to neighboring structures. Hammers weighing greater than 12 tons with the capability of transferring dynamic force greater than 200 tons to the pile are available. Pile Buck, Inc. (1999) maintains a comprehensive list of manufac- turers and equipment capabilities.

(*c*) The vibratory hammers are especially well-suited to installing casings for drilled- shaft piles.

(*d*) Case histories of offshore vibratory-driven piles are not as numerous as those for im- pact-driven piles, especially before the late 1980s. The evolution from above-water to underwa- ter vibratory hammer use occurred much faster than did the impact hammer evolution. The bene- fits of underwater vibratory hammers outweighed the above-water benefits. Use of the underwater hammers began at offshore locations in the North Sea and Adriatic (Jonker 1987, 1988; Ligterink et al. 1990; Ligterink and Martin 1992).

(*e*) Pile drivability using a vibratory hammer is analyzed using the same 1-D wave propagation concept as for impact hammers. The model is somewhat different for vibratory hammers as compared to impact hammers, including modifications to wave equation computer programs. Chua et al. (1987) showed the applicability of the wave equation approach to vibratory hammer driving by modifying the TTI program (VIBEWAVE). Jonker (1988) adapted the TNOWAVE program to vibratory driving. Ligterink et al. (1990) used the adapted TNOWAVE

program to analyze the soil's resistance to vibratory driving during installation of 42-inch (1.07-meter) piles. The vibratory pile-driving analyzer (VPDA) finite difference model was developed by Moulai-Khatir et al.(1994). The Wisconsin Vibratory Pile Driving Analyzer (WiscVPDA) was developed by Bosscher et al. (1998).

(*f*) A list of considerations for selecting impact versus vibratory underwater hammers follows:

- No pile followers (chasers) are needed for underwater driving with either hammer.
- The vibratory hammer and pile are clamped together and are handled as one unit.
- The vibratory hammer can be activated prior to the pile tip touching the seabed. The methodology avoids the need for a temporary guide template (illustrated in Figure A-51).
- The use of impact hammers is more common for offshore applications. Consequently, they have been used for a larger variety of soil types and pile sizes.
- Soil frictional resistance may be overestimated when using vibratory hammers.
- Vibratory hammers are used with piles requiring a minimum vertical tolerance, which allows easier realignment during penetration.
- A combination of hammers allows faster installation of a pile. The vibratory hammer can be used for initial positioning followed by limited driving to achieve lateral support, whereas the impact hammer can be used for the remainder of penetration.
- Above-water impact hammers generally have higher vibration and noise levels than vibratory hammers. Hydraulic impact hammers generally have lower vibration levels than diesel hammers.
- Vibratory hammers cannot develop as much capacity nor reach as great a penetration as hydraulic hammers.
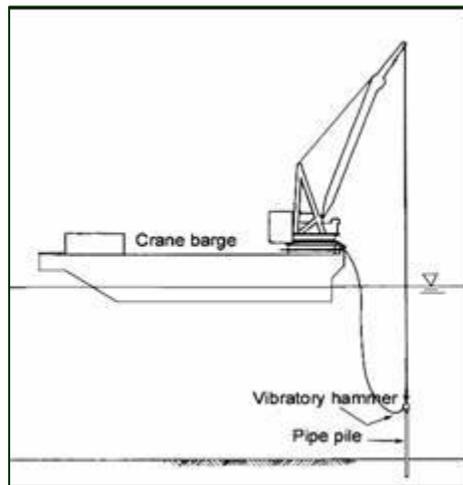
Figure A-51. Driven pile without underwater template (after Jonker 1988).

(*g*) Technology is now available to reduce noise levels during above-water impact driving. Numerous manufacturers offer protective shields or encapsulation modules as noise muf-

flers. For example, one onshore contractor designed and built a hydraulic impact pile-driver that was about 12 decibels quieter than an equivalent system for driving concrete precast piles near an urban development. Totally enclosing all moving parts and eliminating all metal-to-metal contact reduced the noise level to the point at which the ropes flapping against the rig's mast was the only appreciable noise (*Ground Engineering* 1999). For driving tubular and standard steel sheet piles, a Japanese company has recently demonstrated in the United States a hydraulic pile-driving system that operates at less than 60 decibels of noise and is virtually vibration free (Phair 2000).

(5) *Followers.* Followers have two technical definitions, they are: i) a structural member between the pile hammer and the pile to transmit blows when the pile head is below the reach of the hammer; and ii) an attachment used to extend the upper portion of the helmet. Frequently, the follower must serve the same function as a helmet in that they frequently need to accommodate cushions to protect the pile, and they must provide a structural transition between the shape of the hammer and the shape of the pile. Followers are also frequently used to allow for driving either through a template, or through a water column when an above water hammer is being used. Other potential functions for followers include: i) accommodating an extension, or device, up from the top of the pile; ii) providing a port for water to escape from the pile during driving, and iii) serving as a small cofferdam to allow access in-the-dry to the top of the pile. Typically, the pile head must be cut square in order to accept a follower.

(6) *Spin-in.* Spin-in piles essentially consist of the "spin-fin" piles developed by PND Engineers (Peratovich Nottingham & Drage, Inc.) throughout the 1980's. Spin-fin piles were developed for numerous applications where large uplift and tension forces are expected. A spin-fin pile is a driven pipe with steel fins welded at a batter near the tip. The fins can be either shop, or field, fabricated. Because of their unique geotechnical deformation characteristics, these piles exhibit redundancy without catastrophic failure, and are thus more reliable tension piles. End bearing, or pile compression, capacity is also greatly increased. Such piles are frequently used in marine applications such as mooring dolphins where large tensile forces are commonly encountered.

(7) *Jetting.* In-the-wet jetting considerations include: i) internal jets (either attached to the inside of a steel pile, or within the wall thickness of a concrete pile) can be built into large pipe pipes to break-up the pile plug, or alternately, the hammer can be periodically removed and an independent jet can be inserted to break-up the plug—it should also be noted that air-lifting can be used together with jetting to remove a plug; ii) the soil can be pre-jetted to loosen and disrupt dense soils; and iii) independent external jets can be used provided that either the friction with the soil can be restored, or is not critical to the pile capacity. Note that external jetting, and pre-jetting, are discouraged in situations where the capacity of the foundation element relies on skin friction.

d. *Self-Excavating Caissons (Open Caissons).*

(1) Self-excavating caissons typically consist of large, prefabricated, concrete or steel caissons (that need not be cylindrical) that are sunken into the soil in-the-wet, down to a more suitable bearing layer. In this document, the term self-excavating should be taken to mean re-

mote-excavation, as contrasted to caissons sunk in-the-dry (and without the use of underbase compressed air for miners, which can lead to caissons disease), where workmen and equipment can be positioned at the bottom of the excavation to continue under-mining the caisson as it sinks. In-the-wet excavation systems that have been used for sinking caissons include: i) clam-shell buckets; ii) water jet eductor systems that first liquefy and then eject the liquefied soil; and iii) air-lifts used to remove loose material from the excavation. Typically, a detailed structural or FEM analyses, or both, must be performed for each controlling stage of the sinking operation to account for the potential unbalanced support conditions for the caisson being sunk. Examples of self-excavating foundations include the bridges piers for the Bencia-Martinez, and the West Bay Bridge, in San Francisco Bay, California.

(2) In addition to the in-the-wet excavation system, some kind of control needs to be established to keep the caisson aligned vertically as it sinks. Methods that can practicably be used to control the alignment of the caisson, include: i) for open bottom, sub-divided caissons, compressed air can be used in the different cells; ii) supplemental piles (or other reaction points) can be driven/positioned adjacent to the caisson to help control its alignment; and iii) computer-ized systems can be used to control the excavation process itself to control the alignment.

e. Underbase Grouting, Tremie Concrete, Grout Bags, Seal, Load Transfer. Underbase grout and tremie concrete placements have many similarities and many fundamental differences. Both procedures are used to provide structural continuity between the base of a prefabricated element and the foundations. Underbase grout placements are typically thinner than tremie con-crete placements, for reasons including: i) heat of hydration, ii) bleeding, iii) shrinkage, and v) expense. Both procedures require seals to contain the grout or tremie concrete; however, the seals for tremie concrete tend to be larger and more robust than the seals for grouting operations.

(1) Although underbase grout and underbase tremie concrete placements are commonly used for offshore, nearshore, and inland waterways "in-the-wet" foundations, it is not practicable to provide thorough coverage of this somewhat complex topic in this guidance document. It is recommended that either engineers with prior experience, or consulting experts, be used to de-sign the technical aspects of this portion of the foundations work. Potential problems in the un-derbase that can arise in poorly planned grout and tremie concrete placements include: i) forma-tion of segregated zones, laitance layers, and lenses of excessive bleed water; ii) wash-out of the cement particles by the surrounding water; iii) exposure of the marine life to excessive alkalinity; iv) leaking, or blow-out, of the seals; v) accidental up-lift of the structure; vi) accidental lateral displacement of the structure; vii) hydrostatic overloading of sidewalls and side forms; viii) acci-dental trapping of water within the placement; ix) excessive heat of hydration; x) excessive shrinkage and creep; xi) unacceptable delay in setting time of the mix owing to over-dosing with admixtures; xii) formation of unplanned cold joints; xiii) incomplete encapsulation of reinforcing steel, if used; xiv) unintended interaction with landing piles, or tension piles; xv) "green", or re-cently hydrated, grout and remie concrete can be degraded by excessive vibrations and move-ments, such as those from pile driving, blasting, settlements, etc.; and xvi) contamination from silt, sediment, and other foreign matter. It is also common to require an underbase grout or tre-mie concrete placement test, or tests, under controlled conditions, to work out construction diffi-culties in advance, and to verify the contractor's ways and means.

(2) Potential uses for underbase grout, or tremie concrete, placements include:

(*a*) To create a uniform bearing condition for a gravity foundation.

(*b*) To close-off and seal the base of a structure to the soil, rock, concrete, or cut-off system, to inhibit piping or water transmission beneath the structure.

(*c*) To structurally tie-in piles and drilled shafts to their foundations.

(*d*) To connect structural elements (precast panels, etc.) to their foundations.

(*e*) To construct structural reinforced concrete elements under water.

(3) For both grout, and tremie concrete, it is generally important to reduce washout (mixing with the water) of the cement by one (or more) of various means including:

(*a*) Use of anti-washout admixtures, as was used on the grout beneath precast concrete panels for the rehabilitation of Chicago Lock, and as will be used for the tremie concrete on Olmsted Dam.

(*b*) Use of flexible grout hoses (and less commonly flexible tremie concrete lines) beneath the structure, as is common practice beneath offshore gravity base platforms.

(*c*) Use of sub-compartmentalization together with uniform flow front, as was done using grout for Braddock Dam, and will be done using tremie concrete for Olmsted Dam.

(*d*) When possible, sloping the underside of the structure being grouted or tremied, can help to facilitate flow, and to help minimize mixing with the water.

(*e*) For both grout and tremie concrete, it is important to initiate the flow properly, and to introduce grout and tremie concrete into subsequent ports only after the grout or tremie concrete has reached that port.

(4) With regards to grouting, providing at least a 3-foot (1-meter) head can assist in assuring proper grout flow. Furthermore: i) the consistence and workability of the grout should be periodically checked using a flow cone; ii) the grout should be designed to have low bleed properties; iii) the grout should be introduced sequentially in a series of injection ports, and when possible the quality of the grout exiting each subsequent injection port should be checked before grout injection begins from that port; and iv) the up-lift pressure on the bottom of the structure being grouted should be checked to ensure that the structure does not unintentionally lift-up.

(5) Heat of hydration may present a problem for thicker underbase grout placements, which could potentially lead to thermally induced cracking and other distress of the grout layer. For grout layers thicker than approximately 1 foot (0.3 meter), designers should carefully evaluate the heat of hydration produced together with the heat flow from the grout layer. Means to reduce the heat of hydration include:

- Replacing some of the portland cement with pozzolans.
- Using a low heat portland cement.
- Minimizing the cement content while still meeting the given performance requirements.
- Adding sand or rock flour to the grout.

For placements thicker than approximately 3 feet (1 meter), it is common to replace grout with tremie concrete, as discussed in the following.

(6) For underbase placements thicker than approximately 3 feet (1 meter) it is common to use tremie concrete rather than grout for reasons, including: i) tremie concrete exhibits less heat of hydration and less shrinkage than grout; ii) tremie concrete can be less expensive than grout for larger placements; and iii) it can be difficult to engineer tremie concrete to flow properly in layers less than approximately 3 feet (1 meter) (note that it is currently planned to use a nominally 3-foot- [1-meter-] thick tremie concrete underbase for the Inner Harbor Navigation Canal replacement lock). Indeed, to get tremie concrete to flow properly in layers from approximately 3 feet (1 meter) to approximately 6 feet (2 meters) may require the addition of both high range water reducers, and anti-washout admixtures, to obtain a mix with both the proper workability properties. Furthermore, for underbase tremie concrete placements, it is common to limit the distance between tremie pipe locations to approximately 20 to 25 feet (6 to 7.5 meters), or less, unless the thickness of the tremie concrete layer is appreciably greater than 6 feet (2 meters).

(7) The underbase grout or tremie concrete can be contained on the sides by several means, including: i) compression seals, j-seals, wedge seals, or inflatable seals; ii) grout bags acting in direct bearing; iii) grout bags designed and positioned to resist lateral load without direct bearing; and iv) containment by sheet piles, sand bags, rock berms, or fabric containment barriers. The containment system is typically not only positioned around the periphery of the structure, but also frequently used to subdivide the underbase space into zones that can be more readily controlled.

(8) Grout bags merit particular mention as they have proven to be a very versatile way to provide a containment seal in that: i) they can be pre-installed and can expand into a wide variety of geometries of spaces; ii) after the grout has set they are strong and durable; and iii) they can be designed with both redundancy and with checking systems. However, grout bags are a sophisticated sealing system and they should only be fabricated, installed, and filled by qualified companies and personnel with prior experience.

(9) The designer also needs to carefully evaluate load transfer through the underbase placement to ensure that the foundation functions as intended. Factors regarding the final load transferred through the underbase placement that the designer needs to consider include: i) the influences of buoyancy and placement head; ii) load carried by any landing piles or seats, including the influence of using any adjustable supports, such as a flat jack, during, or after the underbase placement; iii) construction sequence; iv) whether the interface with the structure is intended to act compositely with regard to shear and tension as well as compression; and v) whether any secondary grouting of the interface with the structure is planned.

f. Cut-offs and Splicing.

(1) *Divers.* For safety reasons it is always desirable to minimize the use of divers; however, for some cases the limited use of divers may be the most practicable means for cutting or splicing piles underwater. Divers typically cannot work in water velocities exceeding approximately 3 fps (1 m/s) without supplemental safety systems, such as taut lines, cages, attachment systems, and reference systems, use of which may limit the diver's efficiency. Furthermore, for the diver to determine where he/she needs to make the cut, or splice, may require supplemental systems such as: i) jigs, ii) clear water bags, and iii) reference markings or lines on the pile. Common systems cutting systems used by divers include: i) "Zero thrust" high-pressure water jet/abrasive cutting; ii) hydraulic power tools; iii) heat cutting systems such as thermal lance, or plasma arc; iv) controlled underwater explosive charges; and v) mechanical breakers, and diamond wire, or circular, saws for concrete piles.

(2) *Casing Cutters.* The offshore industry has development a number of proprietary underwater casing cutters that can be acquired. Such cutting machines require rigging for handling and may, or may not, require diver support for installation and operation. Typically, such machines eliminate the need for torch cutting and the need for secondary fine cuts and surface dressing, as many of these machines leave a beveled surface suitable for welding. Casings can be cut by either internally or externally mounted systems. E.H. Wachs makes one such externally mounted cutter for up to 36-inch- (0.91-meter-) diameter casings. However, the use of casing cutters can be expensive, as it entails mobilization and demobilization of specialized equipment, crew, and supplies; as well as the expense associated with standby time.

(3) *Remote Systems.* The offshore industry has developed a number of remotely operated vehicles, ROV's, and robots, suitable for cutting steel pipe piles underwater. As many of these systems have been custom developed for deepwater uses, and thus have limited application for relatively shallow inland waterways projects, they will not be discussed further here.

(4) *Dewatering Casing.* It is also practicable to attach a local cylindrical cofferdam around the exterior of a pile to allow access for cutting the pile in-the-dry. Such a system was used for the drilled shaft casings for Braddock Dam.

A-8. Test Programs.

a. Load Test.

(1) *Static—Compression, Tension, Lateral.* Static load tests are conducted to determine the shaft (or pile) performance and the soil response in resisting applied axial and lateral loads. Static load tests can either be conducted in-the-wet or in-the-dry adjacent to the site. A load test is usually made for one or more of the following reasons (Whitaker 1976): i) to obtain a load-settlement or load-displacement relationship, ii) to obtain or validate the design safety factor against failure, and iii) to determine the true ultimate bearing capacity as a check on the design value.

(*a*) Pile load test configurations are axial (compression, tension, and quick load), mono-tonic lateral, and cyclic lateral. These test configurations are detailed in ASTM Standard D1143 , ASTM Standard D3689 , and ASTM Standard D3966 . In-the-Wet Pile Load Tests (for axial loads) are usually conducted in the following manner.

(*b*) Piles are driven full length or extended above water by splices during driving to the same tip elevation and spacing as the permanent piles. Thus, these test piles are tested above water. The reaction is typically obtained by jacking against a platform whose supports, e.g., re-action piles, are spaced away from the test piles so that they don't influence the test pile.

(*c*) A jacking frame applies the load to the test pile. It reacts against the support piles and a gauge records the movement under each increment of jacking load. A swivel bearing at the jack ensures against accidental eccentricity.

(*d*) As the test pile is free-standing in the water column, L/r must be checked. Where the column effect is too great, the test pile can be encased through the water column in a large di-ameter pipe or cage and guided.

(*e*) Axial uplift can be tested by a T-headed beam, reacting against the platform. A jack on each end of the T-headed beam furnishes the force and measures the load. The straps tying this beam to the pile must be conservatively designed for shear in the pile head as failure can re-sult in serious injury to personnel.

(*f*) Lateral load tests (static) are usually made for underwater piles by laterally jacking two test piles apart at the surface (which may not be practicable for deep water depths). This is not a very rigorous test as deflections at the surface are large and minor variations in soils and jacking details bias the results. The use of underwater jacks at the soil line can reduce this bias, but complicate the test logistics.

(*g*) It is generally better to perform a lateral load test by using a braced rigid platform (it can be the same platform as used for the axial compression test) and pulling the test pile towards it, using a center hole jack. The minor deflection of the platform needs to be incorporated by sur-vey. The lateral test should ideally be made with the normal (unfactored) dead load applied in compression but the details are complicated so it is usually neglected or incorporated by calcula-tion.

(*h*) The Osterberg Cell is excellent for testing large, heavily loaded pipe piles in both compression and tension. As before, the pile is extended to the surface. As described, it gives both an upward and downward reaction. The downward reaction must be adequate to resist the jacking force. The upward reaction must also not fail by the concrete core slipping within the pipe. The tremie concrete must use a very well-designed mix to ensure that it flows by the flat jack and bears on both the underside as well as on the top of the Osterberg Cell. Currently, for large diameter heavily-loaded piles, the Osterberg Axial Test (see Paragraph A-8*a*(3)) is pre-ferred. For smaller piles and other shapes and types, the platform tests are still state-of-the-art.

(*i*) The quick load test is permitted as a faster alternative. Its two loading configurations are the constant-rate-of-penetration (CRP) and the quick maintained-load test (ASCE 1993). The CRP test was developed in the United Kingdom by Whitaker (1976) and is not as popular in the United States as is the maintained-load test (ASCE 1993). In the maintained-load test, the load is applied in increments of 10 percent of the proposed design load and is maintained for a constant time interval of a few minutes. In the CRP test, the load is applied to cause pile head settlement at a predetermined constant rate, usually 0.01 to 0.1 inch/minute. Several static bearing capacity determination methods (including the Corps of Engineers method) are listed in ASCE (1993).

(*j*) Axial pile load tests conducted in the past used superimposed dead weights. Current practice includes the use of a hydraulic ram that is jacked against a loaded platform or against a test frame anchored by reaction piles (ASCE 1993), as discussed previously.

(*k*) Load tests conducted onshore are much more numerous and are relatively less costly than those conducted offshore. The offshore industry literature contains several references to offshore load tests, especially for research or in riskier environments such as calcareous soils. For example, load tests conducted from both a fixed offshore platform and a floating vessel are described by Angemeer et. al. (1973) and Angemeer et al. (1975) for offshore Australian sites containing calcareous soils. Dutt and Cheng (1984) conducted pullout load tests on 2-foot- (0.66-meter-) diam pipe piles in offshore carbonate sands. A research program described by Matlock et al.(1998) conducted load tests on pipe piles from a fixed platform in the Gulf of Mexico. J The Pittsburgh District conducted an extensive full-scale in-the-wet pile load tests for Braddock Dam, which can serve as a guide for future inland waterways in-the-wet pile load tests. Load-test piles were tested for axial and lateral loads (Schaffer et al. 2000).

(*l*) The majority of referenced offshore load tests are conducted under axial tension loading (monotonic pullout tests) to determine skin friction capacity. An offshore load test for bending, tension, and compression of pipe piles driven into carbonate bedrock, was detailed by Settgast (1980). Vines and Hong (1984) reported tests on large-diameter laterally loaded pipe piles in coral at an offshore port site in Saudi Arabia.

(*m*) Load tests done onshore with the results applied to offshore structures have also been conducted. Tension and compression loading tests were conducted on 2-foot- (0.66-meter-) diameter pipe piles in stiff clay in Belgium (Heerema 1979). Grosch and Reese (1980) performed cyclic axial loading tests on small-scale (1-inch-diameter) piles pushed beyond the end of shallow boreholes in soft clay. Lu (1986) evaluated small-scale pile models in carbonate sands, while Karlsrud and Nadim (1990) reported tests on small-scale (6-inch-diameter) piles subjected to tension and compression loadings in clay. Zuidberg and Vergobbi (1996) discussed load tests on 30-inch- (0.76-meter-) diameter pipe piles in dense silica sand for the European Initiative on Piles in Dense Sands (EURIPIDES) Project. Onshore testing of pipe piles driven and grouted in calcareous soils was done by Randolph et al. (1996). Onshore axial load tests on 4-ft (1.2-m)-diameter pipe piles in clay were conducted for the Corp's future in-the-wet IHNC project in New Orleans (DACW29-99-B-0085).

(*n*) Test programs can also include exploratory pile and sheet pile driving, and programs to better define rock profiles and drivability of piles and sheets. Although such exploratory pro-

grams are not for determining load capacities of the foundation elements, they can provide very beneficial information that can be incorporated into the tender documents to allow bidders to have a better understanding of the subsurface conditions and foundation requirements.

(2) *Indicator Piles.* It is generally advisable to drive indicator piles as early as practicable on an in-the-wet foundation project. Indicator piles can also be used for some re-strike tests to determine pile set-up. The indicator pile can either be positioned just out-side of the foundation area, or can be incorporated into the final foundation, provided provisions are made of supplementing the capacity of the indicator piles, if they are found to have insufficient capacity. A good distribution of indicator piles throughout the site is recommended to preview the driving characteristics of the foundation soils, chosen pile element, and driving system prior to ordering production piles.

(3) *Osterberg.* Load tests conducted on drilled shafts sometimes use the Osterberg load cell (see Figure A-52). As compared to conventional load tests conducted with a reaction load or hold-down shafts with a heavy reaction beam, the load test employing the Osterberg load cell is relatively easy and inexpensive (Greer and Gardner 1986).

(*a*) The Osterberg method, which was developed in 1984, allows separate measurements of end-bearing and friction loads versus deflection for each load increment.

(*b*) Typically, a flat pressure cell that covers the bottom of the shaft hole is hydraulically pressed simultaneously against both the bottom of the shaft hole and the bottom of the shaft. Depending upon the cell placement within the drilled shaft, gauges can be used to measure the cell's downward movement against the soil and its upward movement against the shaft. From these data, load-deflection curves are generated. The most common loading sequence is the ASTM D1143 (Osterberg 1998). After the testing is completed, the cell can be grouted in place, and the test shaft can be used as a production shaft.

(*c*) Over 300 load tests using Osterberg cells have been conducted in over 10 countries. Drilled shafts (bored piles or caissons), driven pipe piles, and driven precast concrete piles up to 10 feet (3 meters) in diameter, driven to depths up to 300 feet (90 meters), have been tested. Loads up to 15,000 tons (135 MN) have been applied using Osterberg cells. Osterberg (1998) lists several installation methods and cell locations for determining side shear resistance and end-bearing values.

(4) *Pile Driving Analyzer.* Dynamic pile testing using the Pile Driving Analyzer (PDA), can frequently eliminate the need for costly and time-consuming static pile load tests. ASTM D4945 (ASTM 1993) sets a standard for conducting such dynamic pile tests. The PDA displays (in real time) and records dynamic measurements from strain transducers and accelerometers attached near the top of the pile being driven. If a follower is used, the instrumentation may be attached near the top of the follower. For each hammer blow, the PDA processes signals from the gauges to calculate the hammer energy, the pile stresses, and the pile bearing capacity, based on the physical pile properties input by the PDA operator. The PDA can also detect pile damage during driving.
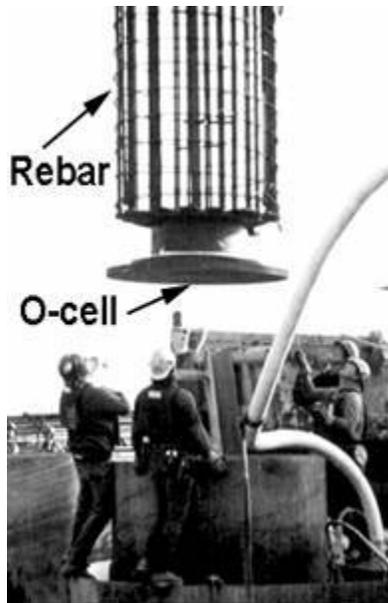
**Figure A-52. Osterberg cell
at the rebar cage bottom (af-
ter DiMillio 1999).**

(*a*) The test pile is normally driven to a predetermined distance above the specified tip before the monitoring begins. Typically, the pile is then monitored while being driven to within 1 foot of the specified tip elevation, and then the pile is allowed to "set-up," usually overnight. Then the pile is usually driven the final specified tip elevation. The data recorded by the PDA can later be analyzed using the CAPWAP analytical software program.

(*b*) Dynamic Pile Monitoring is believed to be very reliable for piles driven in granular soils; however, for silts and clays, this method is less reliable because these soils offer larger damping resistances to pile driving that are not yet accurately modeled.

(*c*) It can be more difficult to conduct pile driving analyses, PDA, and re-strike tests in-the-wet than in-the-dry. Typically, the instrumentation for the PDA must be positioned above water, which may require either that a follower be used, or that the piles being monitored extend above water, with subsequent underwater pile cut-off, if required. In either case, results may not be representative of the actual production piles. The need for re-strikes on underwater piles requires that it be practicable to relocate the pile top under water and to reposition either the pile driving hammer, or the follower, back on top of the pile, which normally does not present a problem if this activity is pre-planned.

(5) *Statnamic*. The Statnamic load test was developed jointly by Berminghammer Foundation Equipment of Canada and TNO Building & Construction Research of the Netherlands, and can be used for either axial, or lateral, pile load testing. The Statnamic test burns a solid fuel/propellant within a gas pressure chamber with piston (with a silencer) to exert a steadily increasing outward force on reaction masses, and an equal inward force on the pile. The loading increases to a maximum before unloading by controlled venting of the pressure. Built-in

instrumentation (load cell and laser sensor) record the dynamic load and displacement during the test, which can then be used to determine the static equivalent load-displacement behavior, and the ultimate capacity, of the pile.

(*a*) Statnamic testing overcomes many of the practical and logistical difficulties associated with conducting axial and lateral pile load tests over water (Figure A-53 illustrates how water can be used as a counter weight for over water tests.) Traditional static pile load tests over water can be costly, dangerous, time consuming, and uncertain. Dynamic pile load test methods induce high accelerations, and pile load-displacement behavior is controlled by the action of the stress wave. Lateral tests over water are conducted by laying the Statnamic apparatus on its side on a barge that bears laterally against the foundations.

(*b*) Advantages of the Statnamic test include:

- Applied loads over 30 MN (3400 tons).
- Lower cost compared to a static load test.
- No reaction piles are required.
- Production piles can be tested without prior planning.
- Simple logistics.
- Reduced testing time.

(*c*) Limitations of the Statnamic test include:

- The load can only be applied above water, which is a limitation for foundations with underwater production pile-heads.
- Axial tension tests cannot be performed.
- Very large piles cannot be tested to ultimate capacity.

b. Model Test/Numerical Test. Physical model of load tests can be used to both verify and to refine numerical model predictions and hind-casts. It is important that numerical prediction be made before the physical load tests are conducted, both to avoid bias and to help determine what type, and what range, of data need to be collected from the physical test.

c. Verification (Continuity) Tests. As part of QA verification (or continuity), tests should be conducted as required by the project specifications.

(1) *Coring.* Underwater concrete coring is an essential technology to have available for most in-the-wet foundation projects using concrete.

(2) *Geophysical Methods for Defects.* Sub-Paragraph A-9*l* addresses geophysical methods for detecting defects/discontinuities.

Figure A-53. How water can be used as a counterweight for an over-water statnamics test.

A-9.   Quality Control/Quality Assurance.

a. Introduction.

(1) Quality control and quality assurance (QA/QC) are essential in engineering design and construction. Quality control and assurance elements are especially important for in-the-wet construction projects where the foundations are prepared, installed, and tested in an underwater, remote location, often from a floating platform on the river or marine surface. Visual inspection of the foundation before, during, and after construction is generally not possible, as compared with the onshore process where the aboveground portion of the foundation is easily viewed and accessible. Specialized techniques and equipment are needed to install quality foundations underwater, and adequate QA/QC procedures must be specified and followed to achieve and ensure successful installation.

(2) Quality control and quality assurance are not the same functions, and are more than an inspection process. Quality control refers to setting up and maintaining a system that provides a means to *control and measure* specified characteristics (of the underwater foundations). Quality assurance *verifies* that the quality control system is performing adequately. Quality control and assurance operate hand-in-hand to enable the underwater foundation to be constructed in strict accordance with the engineering design and specifications by monitoring, recording, and certifying the entire process. Adequate QA/QC is the collective responsibility of all parties con-

cerned with designing, specifying, manufacturing, purchasing, and installing the various project components (Batten 1987). Although the QA/QC system applies to the planning, conceptual design, and engineering design phases of a project, itss most commonly, and better, defined application is in project construction. Constructing the underwater foundation according to the design plans and specifications requires a high degree of QA/QC because of the complexities and variables involved in contractual construction projects. Preventing the introduction of defective materials or poor workmanship in a construction project before these problems occur constitutes an effective quality control and inspection scheme (API 1989).

  b. Procedures—More Unknowns Require More Diligence. One aspect of QA/QC for in-the-wet construction is due diligence in planning and developing clear procedures to be followed for the execution of the work. By means of such careful planning, it is frequently the case that higher quality foundation construction is actually achieved for in-the-wet work than for in-the-dry work. Frequently, the development of acceptable procedures may also entail the performance of mock-up, trial, and indicator tests (and possibly prototype tests) to both verify and to refine the planned procedures. Also, the typically higher construction costs per day associated with in-the-wet construction justifies the more extensive planning to avoid construction delays. Such planning should also include contingency measures for addressing unexpected occurrences. Also, in-the-wet foundations may require a greater use of redundant systems, back-up plans, and possibly more conservative designs than conventional methods.

  c. CSL (Cross-Hole Sonic Logging). The most common integrity testing technique is the cross-hole sonic logging, CSL, which is conducted within pre-installed tubes in cast-in-place piles or drilled shafts (illustrated in Figure A-54). A transducer is lowered into one water-filled tube and emits an acoustic signal that is received by a hydrophone (geophone) transducer in a separate tube. The signal's compression (P-) wave is detected and compared to the transit time of a signal transmitted through intact material with a given modulus of elasticity. If material defects are present, the wave speed will be reduced relative to that of intact material. Accurate transducer positioning helps to pinpoint the location of a defect. For accurate P-wave transmission, the concrete material must be fully cured; however, less accurate quality control checks can be made shortly after the concrete has reached initial set. Chernauskas and Paikowsky (1999) describe the first use of a portable personal computer-based CSL test on concrete drilled shafts in the United States with the Pile Integrity Sonic Analyzer software package. It is advisable to substantiate any anomalies identified by CSL, by means of a coring program.

  (1) Single-hole sonic logging (SSL) uses only one tube in the drilled shaft. This method is normally used after construction is completed and it has been determined that there is a need to check the shaft integrity by coring a single hole through the shaft.

  (2) Parallel seismic logging is similar to geophysical cross-hole seismic testing in that holes are drilled into the surrounding soil (Figure A-55). A casing is inserted into each hole, the annulus is grouted, and the casing is filled with water. Both SSL and parallel logging use equipment and interpretation procedures similar to those used in the CSL test.
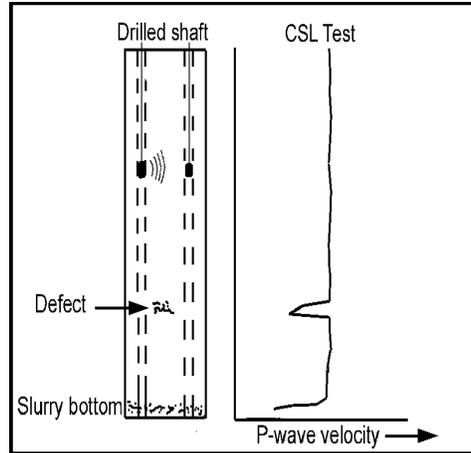
Figure A-54. Crosshole sonic logging (CSL) integrity test.
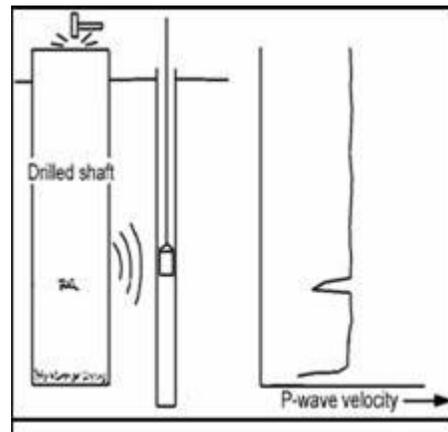
Figure A-55. Parallel seismic integrity test.

d. Gamma-Gamma Tests. Gamma-Gamma logging is normally used to measure the concrete quality placed within drilled shafts. In this regard, it is an alternative to the CSL test discussed in Paragraph A-9*b* above. Although Gamma-Gamma tests can give accurate measurements of concrete density up to distances of approximately 3 inches away from the inspection tube, it does have the following limitations: i) expense, ii) inability to identify concrete that is slow to set, as that concrete is similarly dense, and iii) risk to personnel from exposure to radioactivity.

e. ROV/AUV (Remotely Operated Vehicle/Autonomous Underwater Vehicles). Remotely operated vehicles, ROVs, and autonomous underwater vehicles, AUVs, have been extensively developed for the offshore industry and their use can minimize the need for divers on a job. ROVs are tethered and controlled from the surface, while AUVs are preprogrammed and operate autonomously underwater until they AUV resurface. ROVs are typically much more common, are less expensive, and can do more practical work than can AUVs. The efficient use of ROVs and AUVs normally requires planning, for example, to ensure that attachment points for the ROV/AUV are pre-positioned and pre-planned on the prefabricated structure before it 1s

installed underwater. AUVs are frequently limited to data collection activities, while ROVs have been designed for a broad range of activities, from surveys, to construction, to repairs, operations, and maintenance. The need for ROVs and AU's in the relatively shallow inland waterways has been limited to date, as frequently other means can more easily used to achieve the same result. Figure A-56 presents a few examples of different ROVs. ROVs can either be purchased from organizations such as Oceaneering, or Seascape, or a ROV's services can be hired from companies such as The Bluestream Group.

Figure A-56. Examples of ROVs.

f. Acoustic Cameras. Within the past decade, extensive progress has been made on the quality and resolution of acoustic cameras that use sound instead of light to produce real-time images of objects underwater. Frequently, it is not practicable to use light to image objects underwater because of turbidity, and, frequently, acoustic cameras can provide resolutions on the order of millimeters. Nevertheless acoustic cameras have limitations as compared to normal optical cameras including: i) they have a limited sight range; ii) they have a low signal-to-noise ratio. thus the presence of background noise can be a challenge; and iii) they have limited resolution compared to optical camera. High-resolution acoustic cameras are becoming available for an increasing range of uses. This field is rapidly changing, so it may be advisable to contact the Engineer Research and Development Center, ERDC at: James.A.Evans@erdc.usace.army.mil who used such a system at the Olmsted Locks and Dam project. Figure A-57 presents an image of a Dual-Frequency Identification (DIDSON) System.

Figure A-57. DIDSON Acoustic Imaging System.

g. Optical Surveys. Optical surveys, using conventional in-the-dry equipment and personnel, of equipment, vessels, frames, templates, shells, and hulls extending above the water

are an essential aspect of most in-the-wet foundation construction projects. As optical surveys are not unique to in-the-wet construction they are not addressed in detail here.

h. *Divers.* Although it is desirable to minimize the use of divers, it is advisable that, on complex in-the-wet foundation projects, some degree of diver QA activity be planned for. Divers can only do a limited amount of work underwater, largely because of their nearly neutral buoyant condition. Thus, divers generally need something to react against, such as a hand or foot hold, or a taut line, to exert leverage and to resist water currents. Divers also generally work in conditions with both limited communications and limited visibility, which can present both safety hazards and can limit inspection and work. Divers need to work in teams and can generally only work underwater for limited periods. All diver operations require a well prepared work plan, as well an emergency evacuation plan in the event of diver injury.

i. *Above Water Extensions/Tell-Tales/Survey Towers*. Above water extensions/tell-tales/survey towers are commonly used for in-the-wet foundation construction together with optical surveys to determine the location of underwater elements. It is an important part of QA to plan for survey targets for such above water extensions.

j. *Coring/Verification/Cameras/Pressures Sensors*. The ability to take underwater cores, optical pictures, and pressure readings are fundamental for many in-the-wet projects. Frequently, the use of optical cameras underwater can be facilitated by locally displacing dirty water with clear water by such methods are placing the camera in a clear plastic bag filled with clear water and placing the bag against the object to be imaged, or to use a viewing box that flushes and contains clear water around the object being imaged.

k. *Soundings/Hydrographic Surveys*. Soundings and hydrographic surveys are a fundamental aspect of most in-the-wet foundation projects. Generally such surveys should be conducted early enough in the design process to: i) determine whether the underwater bed conditions are static, or dynamic, and, if they are changing, how do they change over time; and ii) determine site conditions to be used in the design, including the potential presence of debris, old submerged structures/formations, scour-holes, and other pertinent features. EM 1110-2-1003 presents a comprehensive and current manual for planning and conducting hydrographic surveys.

l. *Geophysical Methods*. Seismic survey techniques may be used to assist exploratory borings and to identify any unusual features, including boulders, discontinuities, and cavities of any structurally significant size prior to final design. High-resolution geophysical surveys may make use of boreholes. One technique uses a group of boreholes, where seismic (acoustic) transponders and transducers are deployed in the boreholes to image (via tomography) the subsurface features. Another technique is similar to the first but it employs an impactor (dropped weight) on the surface, together with a seismic transponder. Examples of locating subterranean discontinuities and cavities with the geophysical methods include extensive surveys performed by the USGS, particularly in Florida at Orange Lake, Kingsley Lake, Lowry and Magnolia Lakes, and Drayton Island, or surveys performed by private firms such as IntegrityTesting.

m. *Test Procedures/Mockups*. Conducting advance tests of construction procedures, and mock-up tests conducted onshore are an important aspect of in-the-wet foundation construction.

Such tests not only serve to verify the contractor's means and methods, but also give the contractor the opportunity to: i) train his/her workmen; ii) troubleshoot and improve his/her techniques; and iii) provide QA documentation of procedures that are hard to document during the actual construction.

n. Education/Training of Team—Knowledge Resources. Design quality is the responsibility of everyone involved in the development of a project. Several QA steps are involved in the design of Corps projects before plans and specifications are inked, as outlined in Engineer Regulation 1110-2-1150. Appendix F of the ER is an internal management control checklist to ensure that all engineering functions are conducted in a cost-efficient manner. Adequate coordination, communication, and lessons-learned applications are stressed to achieve a high standard of technical quality during the design phase.

(1) Appendix E of ER 1110-2-1150 includes instructions for writing a "report on engineering considerations and instructions for field personnel" who will be responsible for the oversight of the contractors' quality control program. It is stressed that this report should be distributed to appropriate construction field personnel for their suggestions and input prior to completion of the design phase. Field personnel may have a more intimate knowledge of site conditions and potential site-specific design problems than do the engineering design personnel. Hence, the field comments and suggestions should be incorporated early on, especially during the design phase.

(2) The Corps of Engineers has published engineer manuals, technical manuals, and other guidance to ensure consistent criteria and requirements for developing design and engineering documents for civil and military projects.

(3) The Corps of Engineers has a partnering agreement with the Association of Deep Foundations and Drilled Shaft Contractors, and free ITR/PEER reviews can be setup with expert panels of high ranking officials from drilling companies to review foundation projects. Also, a comprehensive listing of Federal standards, design guides, engineer manuals, national standards, building codes, and trade organization standards can be found at *http://www.hnd.usace.army.mil*.

A-10. Instrumentation.

a. Considerations.

(1) *Construction*. Numerous considerations must be carefully evaluated regarding the use and installation of temporary and permanent instruments during in-the-wet foundation construction, including: i) durability of the instrumentation to withstand incidental construction loading; ii) access underwater for installing the instrumentation; iii) access for lead wires carrying data from the instrumentation, and iv) access for repair or replacement of damaged instrumentation. Useand installation of temporary and permanent instrumentation during construction requires thorough planning. Temporary instruments used only during construction can be used both to ensure that the structure and foundations are built correctly and to avoid hazardous conditions.

(2) *Permanent or Long Term.* Permanent or long-term instrumentation is frequently used to verify that design assumptions have been met both immediately after construction, and after any long-term changes to the foundations. Long-term instrumentation can be used to monitor a wide variety of foundation responses including: temperature, groundwater pressures, accelerations, strains, stresses, pressures, displacements or deformations, rotations, forces, and elevations, all of which can be processed to verify or invalidate the initial design assumptions.

(3) *Difficult Installations.* Difficult, in-the-wet installation conditions, either during, or after construction, merit careful consideration and planning. Potential failure of instrumentation under such difficult installation conditions requires both: i) planning for means to repair or replace the damaged instrumentation, or ii) the provision of redundant instrumentation so that adequate data can be gathered even if some gages fail.

b. *Instruments.*

(1) *Strain Gages.* Strain gages can be mounted onto structural foundation elements, such as: i) piles; ii) drilled shafts; iii) reinforcing steel; or iv) concrete. Strain gages can be used uniaxially, or in rosettes to resolve multi-axial and shear strains. The gages must be adequately bonded (e.g., welded, or glued) to a prepared surface, or cast into the concrete, and frequently must be coated and sleeved and the lead wire routed in rigid piping for protection in field conditions.

(2) *Tiltmeters.* Tiltmeters can be used for: i) structural response tests (for design verification); ii) automated inspection and long-term surveillance; iii) detection of hazardous conditions (such as angle changes from excessive settlement, or excessive loading); iv) construction monitoring; and v) machine and gate positioning and control. Tiltmeters provide a precise and dependable record of angular movements, typically with respect to gravity. Measured rotations can readily be converted to structural displacements, moduli, moments, and shears using standard engineering formulae. Tiltmeters are typically discrete units that are designed for prolonged field exposure conditions.

(3) *Inclinometers.* In some disciplines inclinometers serve much the same function as tiltmeters (see previous sub-paragraph); however, for geotechnical purposes, inclinometers use torpedo shaped probes that can be run down casings to determine ground movements, settlements, and heave. Typically, a local microcontroller in the probe manages data collection from the sensor (typical sensor types include servo-accelerometers, force-balance and uniaxial), applies calibration factors to determine angle changes, and transmits the data for recording and processing. Similar to tiltmeters, inclinometers can be used for: i) structural response tests; ii) construction monitoring; iii) long-term surveillance; and iv) detection of hazardous conditions.

(4) *Pressure Transducers.* Pressure transducers, and piezometers, measure liquid and gas pressures. Pressure transducers have a variety of pressure/temperature service ranges, and can be deployed in a variety of manners including: i) mounted on the face of a structure; ii) suspended in a water or air column; or iii) buried to monitor groundwater pressures. The area of the pressure diaphragm on a pressure transducer is typically much smaller than that for an earth pressure cell

(see the following sub-paragraph) as there is no need to average variations in pressure of the medium being measured.

(5) *Earth Pressure Cells.* Earth pressure cells consist of two thin steel plates welded together around their periphery and spaced apart by a narrow cavity filled with de-aired oil. Changing earth pressure squeezes the two plates together causing a corresponding increase of fluid pressure inside the cell, which is typically measure electronically to determine the earth pressure. The plates are normally large enough to average small soil variations, in order to provide average design values. Earth pressure cells can be installed: i) in engineered backfill or embankments; or ii) at the surface of concrete, or steel structures.

(6) *Load Cells.* Load cells can be used for monitoring the performance of rock and soil anchors if these features are part of the design.

A-11. Select Case Histories and Lessons Learned.

*a. Olmsted Approach Walls Foundations (Louisville District).*

(1) *Summary.* The Olmsted Locks and Dam project is located on the Ohio River approximately 16.5 miles upstream of its confluence with the Mississippi River. It consists of twin 1200-foot locks with floating and fixed approach walls, five 110-foot-wide tainter gate bays, a 1400-foot-wide, navigable, pass boat-operated wicket gate section, and a 400-foot section of fixed weir (Figure A-58). The project will have a total cost over 1 billion dollars, and as such is one of the largest civil works project underway in the Corps of Engineers.

(*a*) The approach walls for the project consist of four floating concrete pontoons ranging from 160 to 1700 feet in length and a 565-foot-long fixed precast land wall. Each of the floating walls will be anchored to a pylon structure and have an isolated nose pier for protection (Figure A-59). The pylon structures, nose piers, and the fixed land wall all have large diameter drilled shaft foundations. The foundations are designed to resist very large lateral loads from barge impacts and seismic forces. Axial loading is minimal. The drilled shafts are 10 feet in diameter and 140 to 145 feet in length. The permanent casings range in thickness from 0.75 to 2.12 inches and extend to the bottom of the shaft. The shafts are driven and drilled approximately 98 feet into the ground. The foundation materials consists of 2 to 20 feet of alluvial sands, 50 feet of very dense fine sand and interbedded clay of the McNairy formation, and 30 feet of hydrothermally altered rock of the Fort Payne formation. The casings were driven with an HPSI 21,445 in.-lb vibratory hammer and drilled with a Hain casing-mounted drill. A total of 37 shafts were required for this project, all installed from a floating plant in the river.

Figure A-58. Artist rendering of completed Olmsted Locks and Dam.
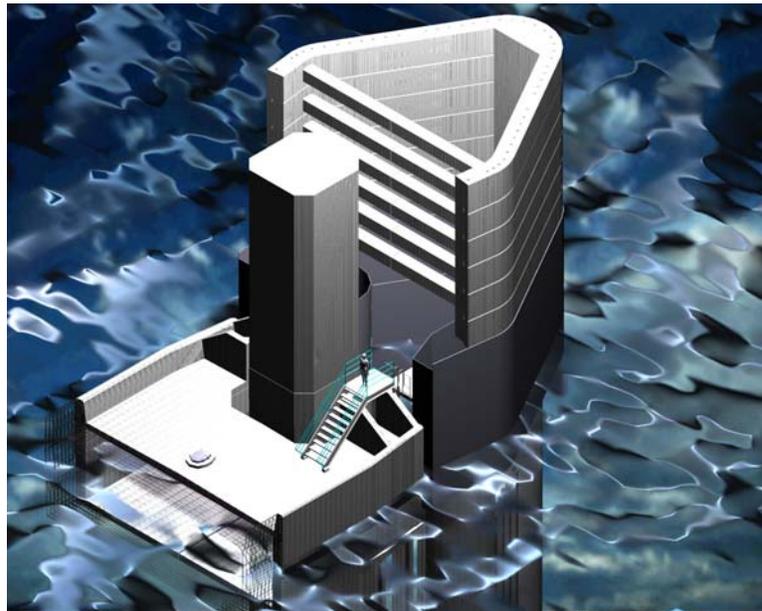
Figure A-59. Rendering of completed nose pier and pylon.

(*b*) After the casings are driven to the final tip elevation, they are cleaned out, the rebar cage is installed, and the casings are filled with tremie concrete. After sufficient curing, the integrity of the concrete is evaluated by Cross-Hole Sonic Logging tests performed in 8 steel tubes that are installed as part of the rebar cage.

(*c*) The hard work and cooperation displayed by the U.S. Army Corps of Engineers, INCA Engineers, Inc., and Massman Construction Company put this unique project on the path to successful completion.

(2) *Subsurface Conditions.* Within the riverbed and along the banks, the McNairy is overlain by recent alluvium. The alluvial deposits near the Illinois bank consist mainly of re-worked silts, clays, and occasional fine sand lenses, some of which are likely a result of colluvial materials from landslides. These materials are classified according to the United Soils Classification System as SC, CL, and CH, and have SPT N-values that range from 0 to 22 blows-per-foot. These materials are very soft to stiff, and are normally consolidated. Covering most of the riverbed and the Kentucky bank, the alluvium is a poorly graded sand (SP), with N-values ranging from 0 to 15 bpf, with an average of 6 bpf.

(*a*) Below the alluvium, a soil stratum known as the McNairy Formation is present throughout the site. The McNairy (a Cretaceous-aged soil deposit) consists of approximately 80 percent very fine to fine dense sands with the remainder being interbedded layers of stiff clays and silts. It was likely deposited in a fluvial deltaic environment. This formation ranges from 40 feet in the thalwag of the river, to 120-feet thick on the Illinois bank. The sands were classified as SP. The clays and silts are classified as CL and ML, respectively. The sands are very uniform throughout the formation, with grain sizes predominately ranging from 0.002 in. to 0.01 inch with a mean of 0.006 inch. Self-Boring Pressuremeter tests conducted in the McNairy indicate that the magnitude of effective in-situ horizontal stress is very high. K_0 (the ratio of horizontal to vertical effective stress) averages about 4.5 in this stratum. Even though the in-situ effective horizontal stress is very high, the Over-consolidation Ratios (OCR) determined from consolidation tests on clay samples taken from various depths in the McNairy average 2.0 with little variation with depth. The void ratio determined from undisturbed sand samples ranges from 0.74 to 0.91. The specific gravity of the sand is between 2.64 and 2.68. The sands have an average drained friction angle (φ') of 31 degrees. In the clay layers, peak values of φ' range from 19 to 22 degrees with residual values as low as 10 degrees. The average saturated unit weight of the soil is 120 pcf. The N-values in the McNairy range from 40 to over 100. The groundwater within the McNairy sands is pressurized with heads up to 10 feet above the river water surface.

(*b*) The Illinois Geological Survey evaluated core samples of the material found below the McNairy Formation, determining that the Paleozoic rock formation directly underlying McNairy is a leached and silicified Mississippian formation, known as the Fort Payne Formation (John Nelson, personal communication). The Fort Payne is normally a dark-colored, siliceous limestone that contains bands of dark chert. In the Olmsted area, nearly all of the carbonate minerals have been leached or dissolved away or replaced with silica, leaving a dark brown, silt-like rock that contains occasional bands of hard chert. Several samples of unaltered Fort Payne limestone were obtained. Early in the Olmsted project site investigations, the altered Fort Payne materials were incorrectly identified as a zone II member of the McNairy formation.

(3) *Drilled Shaft Design.* The large lateral loads from possible barge impact and earthquakes governed the design of the drilled shaft foundations. Site-specific ground motions were developed for the site by Geomatrix Consultants. The Operating Basis Earthquake (OBE) with a return period of 144 years has a peak ground acceleration at the base rock of 0.065 gravities. The Maximum Design Earthquake (MDE) with at return period of 1000 years has a peak baserock acceleration of 0.37 gravities. Site response analysis was then performed to develop the freefield ground surface motions for seismic design. For barge impact, forces up to 4000 kips were applied to the nose piers. Smaller forces ranging from 300 to 1000 kips were used for impacts on

the floating pontoons. As the performance of full-scale lateral load tests are difficult and very expensive for larger diameter shafts, self boring pressuremeter (SBPM) tests were done at each nose pier location and used to develop *P-Y* curves for design. The tests were performed by Dr. Jean Benoit and the drilling by FMSM Engineers. The SBPM is shown in Figure A-60.
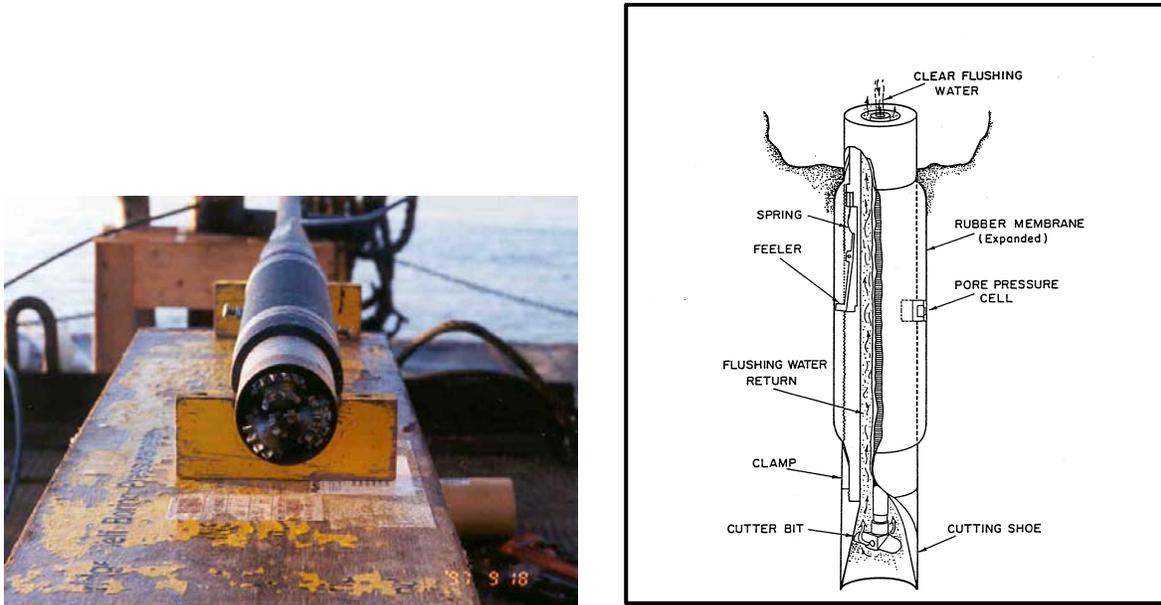
Figure A-60. Self boring pressure meter.

(*a*) The non-linear load-displacement (*P-Y*) curves for the soils adjacent to the embedded portion of the shafts were developed using the recommendations by the American Petroleum Institute (API 1993). The results of the SBPM tests and lateral load test on H-piles performed for the Locks' foundation were then used to refine the *P-Y* curves. As the zones of influence of the closely spaced drilled shafts overlap, the load transfer characteristics along a shaft are affected by adjacent shafts. The interaction among drilled shafts was accounted for by using *P*-multipliers to scale the *P-Y* curves. These multipliers were developed based on an empirical correlation developed by Dunnavant and O'Neill (1986).

(*b*) An initial dynamic analysis of the entire system was based on a unified model in which the *P-Y* curves were converted to equivalent linear spring supports. This was done entirely with GTSTRUDL; however, this method required many iterations and converged very slowly. The final design was completed by separating the structure into a linear superstructure model and a nonlinear substructure model. The superstructure was modeled with GTSTRUDL and the substructure stiffness determined with LPILE. To verify the validity of this approach for the seismic design, a three dimensional soil–structure interaction analysis was conducted using SASSI (Lysmer et. al. 1981) on one nose pier and pylon group.

(*c*) The shaft embedment depths were determined to ensure long pile behavior for all load conditions. Flexible long pile behavior was required to minimize permanent set in the pile after removal of loading. Two approaches were used to establish the required embedment depth. First, a series of analyses were made starting with a very long shaft. The shaft length was then

incrementally shortened until the shaft head deflection was no longer constant. The second method used was to increase the depth of the shaft incrementally until there were two crossings of the zero deflection axis on a plot of depth vs. deflection.

(*d*) The structural moment capacity of the shafts was determined by considering the composite strength of the concrete, reinforcing steel, and the permanent casings. The drilled shafts are 10.0 feet in diameter and 140–145 feet in length. The permanent casings range in thickness from 0.75 to 2.12 inches and extend to the bottom of the shafts. In the maximum moment areas the reinforcing steel consists of 72 #18 bars. Figure A-61 shows the drilled shaft layout for the Nose Piers and Pylons.
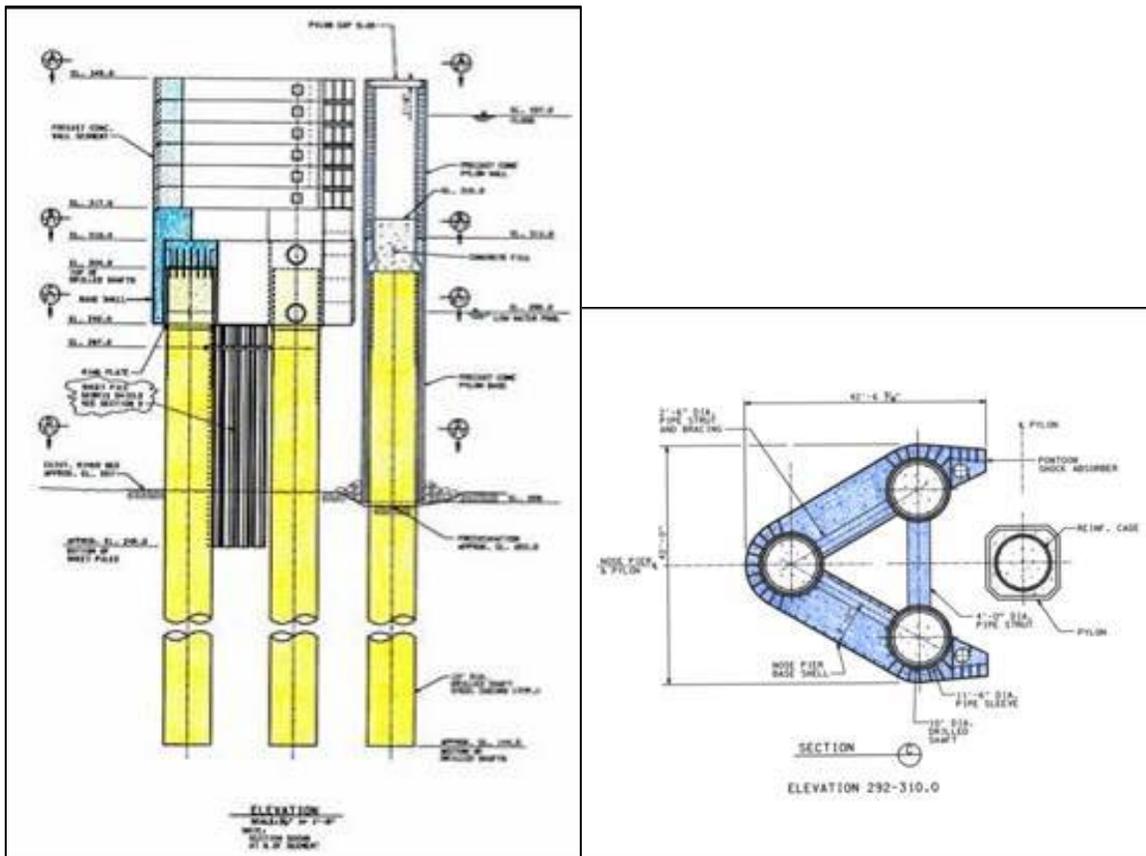
Figure A-61. Nose pier and pylon shaft layout.

(4) *Shaft Construction*. Construction of the 10-foot-diameter drilled shafts (Figure A-62) required a relatively complex set of procedures to obtain the desired design parameters for the project. The drilled shafts were designed to resist large lateral loads from barge impact as well as seismic loading. Based on these requirements, specific construction constraints were required to that ensure lateral resistance was not lost during driving of the casings.

(*a*) Prior to driving, a fabricated heavy steel template was installed at each nose pier location. The template was designed to hold the casings within the specified tolerances against significant lower Ohio River currents. Figure A-63 shows the template used for the drilled shaft in-

stallations. The template was spudded with eight 6-foot-diameter piles to remain stable once it was positioned and leveled from the barge.

Figure A-62. Shaft casings with 10-foot diameter.

Figure A-63. Nose pier template.

(*b*) A sequence of construction steps was originally developed, based on experience and design constraints, to minimize the possibility of lateral stress relief. The procedures were as follows.

- Drive the casing to practical refusal using the vibratory hammer.
- Loosen material within the casing to within 10 feet of the casing tip.
- Remove material within the casing to within 10 feet of the tip.
- Loosen material to the casing tip.
- Remove material within the casing to the casing tip.
- Loosen material to a depth 6 feet below the casing tip.
- Remove material to a depth 6 feet below the casing tip.

(*c*) The casings were driven into the substrata using a HPSI 2000 vibratory hammer with a rated eccentric moment of 21,445 in.-lb (see Figure A-64). To obtain the optimal driving amplitude, the operating frequency and vibrating mass were initially varied.

Figure A-64. HPSI Model 2000 vibratory hammer.

(*d*) During the course of driving, periodic clean out of the inside of the casing was required. This was done with various cleanout buckets and brushes. Figure A-65 shows the drilling tools. Water levels inside the casings needed to be maintained above or near the river level at all times. When material was to be loosened or removed within 10 feet of the tip, slurry was required to replace the water in the casing. The slurry consisted of AQUAGEL bentonite with a density maintained between 66 and 75 pcf. The contractor was eventually allowed to use the cuttings in the slurry mixture and was able to maintain a fluid density near 80 pcf without adding bentonite.

(5) *Driving Issues*. Driving began at the upstream riverside nose pier (NP1) in late September of 2000. At the beginning, the contractor encountered several mechanical difficulties with the hammer and power pack. In general, the hammer bearings and power pack were put under tremendous strain in attempting to achieve the final tip elevation of 159.

(*a*) Based on repeated attempts, the contractor could drive the casings to the top of the Fort Payne formation, clean out the casing and add slurry, then attempt to redrive. After several subsequent attempts, drilling out 6 feet in front of the tip was required to advance the shaft to the final tip elevation.

Figure A-65. Pile top drill with 9-foot cleanout bucket with extendable wings to 9 ft 8 inches.

(*b*) The eight upstream drilled shafts were advanced to their final tip elevation using the revised procedure. However, casing refusal above the final tip elevation (near the top of the Fort Payne formation) was encountered during driving the casings for the downstream riverside nose pier. Several attempts were made to advance the shafts, including using the more powerful (26,000 in.-lb) Tandem King Kong APE Vibratory Hammer (Figure A-66). The skin friction on

the casing had "set-up" from the time initial driving halted until cleanout. This made it impossible to advance the casings even after 6 feet of material was drilled out below the tips.

**Figure A-66. King Kong APE vibratory hammer.**

(6) *Design Revisions*.

(*a*) The design tip elevation was 159. The four casings for the downstream riverside nose pier and pylon refused between eleveations 179 and 188. The design was modified for these shafts. The tops of the casings were cut off at the design top elevation. An uncased socket was then drilled to the original tip elevation of 159. Additional reinforcing steel was added to compensate for the absence of the casing. Number 18 reinforcing bars placed in 4 bar bundles were required to achieve the required moment capacity. To reduce the risk of early refusal of the remaining casings and having the thickened high moment capacity section of the casings in the wrong place, the remaining shafts were modified by reducing the casing embedment depths and increasing the length of the reinforcement (Figure A-67).
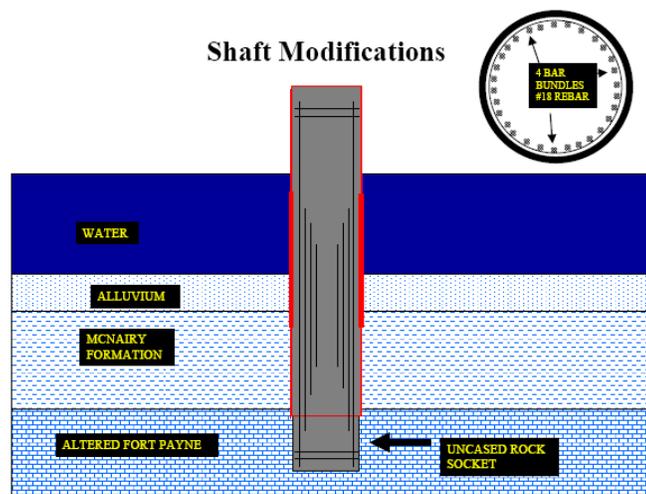
Figure A-67. Drilled shaft design modification.

(*b*) During the installation of the lower land wall shaft 3, the casing refused in the soil well above the top of the Fort Payne formation. Before the contractor could employ alternative methods for advancing the casing, the river rose above the elevation of the work platform and templates. Even though the excavation was full of a drilling mud, the walls of the excavation collapsed, undermining the casing, which then fell under its own weight (Figure A-68). To remedy this situation, the casing was raised, then driven to proper alignment, and a compaction grouting program was implemented to restore the strength and confinement of the soil around the casing.
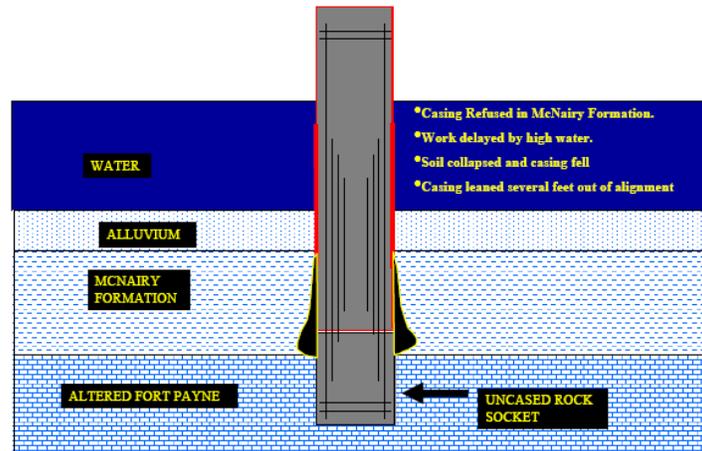
Figure A-68. Lower land wall shaft 3 excavation collapse.

(7) *Reinforcement and Concrete Placement.*

(*a*) Concrete was placed in the drilled shafts in two sequences. An unreinforced 8-foot thick "plug" of tremie concrete was placed in the bottom of the shaft. Next, #6 hoop bars were positioned adjacent to the shaft and vertical reinforcements consisting of #18 bars were fixed to a circular lifting ring and spliced by treaded couplers. The vertical reinforcement was then lowered entirely into the shaft and slowly raised to add the hoop bars. As the completed cage was lowered back into the shaft casing, eight steel access tubes for integrity testing were installed at equal spacing within the cage. See Figure A-69 for typical reinforcement installation. The tubes extended above the shaft and were fitted with removable caps.

(*b*) Concrete was placed via of a floating batch plant. The floating batch plant was located on a 35- by 200-foot barge and had a capacity of 75 cubic yards per hour. Concrete from the batch plant was delivered to a tremie hopper via a 5-inch diameter pump (Figure A-70). Slump for the concrete placed was 6 to 9 inches. Logs were kept of slump loss, concrete temperature, and volume discharged. Concreting operations varied but it generally took about 10 hours to fill one shaft.

Assembly Ring

Figure A-69. Typical reinforcement installation using lifting ring and crane.

(8) *Integrity Testing*. Once the concrete was successfully placed, testing was required to verify the integrity of the shaft. Cross-hole Sonic Logging (CSL) was used to determine the shaft integrity.

Figure A-70. Tremie concrete placement.

(*a*) CSL tests newly placed drilled shafts for voids, soil intrusions, slightly cementitious defect seams, water filled zones, poor concrete, and suitable bottom placement of concrete. This test method is based upon sending a signal through the material being tested and recording the arrival time of the signal and the energy associated with that signal. CSL tests the "cross-hole" material and provides a profile of the drilled shaft as the test apparatus is raised through access tubes installed prior to concrete placement.

(*b*) To begin testing, a signal transmitter and receiver are lowered on a calibrated line, such that the two components are always at the same elevation during testing. The components are lowered to the bottom and then raised during testing at about 1 fps. The transmitter sends a signal impulse, which is received as the components are raised. The information is recorded for processing.

(*c*) Because the distance between the two tubes is known, the wave speed can be evalu-ated. A log of depth vs. arrival time can then be plotted. In addition, the energy of the signal can be plotted vs. depth on the same chart. Defects will cause a reduction in velocity and perhaps en-

ergy due to increased travel path or signal dissipation. Sample test results are shown in Figure A-71. The plot on the left shows high quality concrete with no defects while the plot on the right shows soft material at the bottom of the shaft. Defects were repaired by grouting.
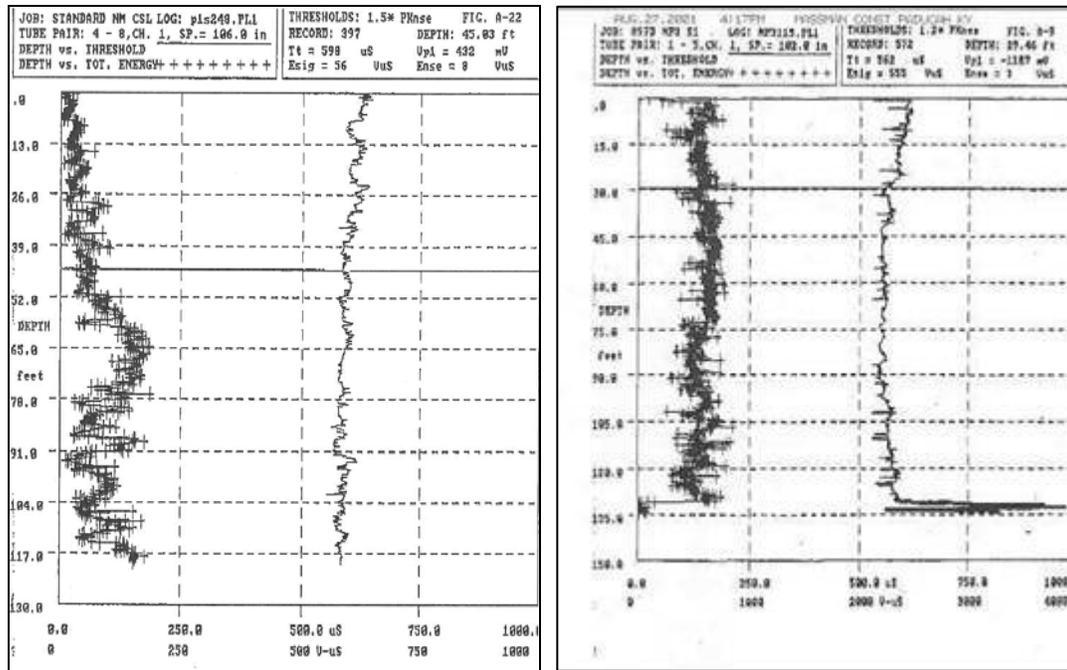
Figure A-71. Cross-hole sonic logging result.

(9) *Conclusions*. Construction of an innovative design for large lateral loads in difficult subsurface conditions has proven to be challenging.

(*a*) Soil behavior during driving of large diameter shafts by vibratory methods is not well understood in the engineering community, and therefore warrants special considerations. The skin friction developed by some of the casings could not be overcome by the largest vibratory hammers available.

(*b*) Designers and contractors must often be flexible to achieve project success for these types of foundations. Careful field observations are often the key to making these proper revisions.

(*c*) Integrity testing on large diameter shafts in water is prudent and should be used for critical structures. The data gathered helps verify proper construction procedures and promotes designer confidence if revisions are later required.

(*d*) To date, the teamwork exhibited by the Corps of Engineers, INCA Engineers, and Massman Construction has proven successful in overcoming all project challenges.

b.   Inner Harbor Navigation Canal (New Orleans District). The Inner Harbor Naviga-tion Canal (IHNC) Lock Replacement project will consist of float-in precast concrete elements resting on underwater pinned foundations. Prior to design and construction, a pile load test and installation study contract was awarded. The contract study included installation of nine 48-inch- (1.2-meter-) diameter, open-ended steel pipe piles driven to a maximum depth of 190 feet (58 meters) below ground surface. Several innovative features for underwater foundation installation were addressed in this contract.

(1) The technical specifications (DACW29-99-B-0085) included these sections:

02355   Pile Load Test
02451   Steel Pipe Piles
02497   Dynamic Pile Testing

(2) Contractor QC plans were required to address material delivery, storage, handling, installation, testing, and removal procedures. In addition to material specifications meeting American Society for Testing and Materials requirements, the API (1989) RP-2A-LRFD re-quirements were specified, and only one splice per pile was allowed. Provisions to monitor hori-zontal and vertical alignments and plumbness during underwater installation were included, with underwater alignment tolerances of 6 inches (15 centimeters) vertical, 6 inches (15 centimeters) horizontal, and within 1.5 percent of length (plumbness).

(3) Only two of the nine pipe piles were specified to be driven underwater in the naviga-tion channel, which is approximately 30 feet (9 meters) deep. The remaining seven (onshore) piles were driven inside 6-foot- (1.8-meter-) diameter steel casings in which the soil had been excavated and then backfilled with water to simulate underwater pile driving. All piles and cas-ing were driven with submersible impact hydraulic hammers or submersible vibratory (electric or hydraulic) hammers (Figure A-72). The piles were instrumented and analyzed using the Pile Driving Analyzer and CAPWAP dynamic load capacity packages.

(4) Restriking and compression load tests were conducted on five of the onshore piles to determine ultimate pile capacity. Noise and vibration measurements (decibels and inches/second) were specified and taken at two locations during driving of the test piles onshore and underwater for comparison with background readings. Upon completion of the driving and testing, it was specified that the piles and casings were to be removed and the voids filled with a tremied ce-ment–bentonite–sand mixture.

Figure A-72. Pile-mounted submersible hammer.

c. New Braddock Dam Foundations (Pittsburgh District).

(1) *Introduction.* The foundation system for the new Braddock Dam was constructed completely in-the-wet, and included sheet pile cut-off walls, step-tapered drilled shafts, and driven H-piles. While two float-in segments were being fabricated off-site, work continued concurrently at the Braddock project site to complete the dam foundation system. The basic dam foundation system is composed of upstream and downstream cut-off walls, a graded gravel base, and a grid of reinforced concrete drill shafts that extend from the riverbed into bedrock. A series of 150 steel bearing piles form the support for the dam's tailrace area. In addition to these foundation features, the left closure weir utilized an in-the-wet approach to stabilize the in-situ material encapsulated within the sheet pile closure cells to make an impermeable cut-off.

(2) *Pre-existing Foundation Conditions.* The alluvial deposits above the top of rock along the proposed dam axis were a maximum thickness of 50 feet near the landwall and less than 30 feet under the existing fixed crest dam. Top of rock was nearly horizontal, averaging at elevation 670 and varying by 2± feet with no discernible trend. In several initial borings, a thin sandstone layer was encountered at the top of rock. Because this layer was thin and discontinuous, the sandstone was not distinguished from the clayshale in generalized representations of the subsurface stratigraphy. The clayshale varied in thickness from 10 feet along the existing river wall to 14 feet along the left abutment. Below the clayshale was siltstone. Approximately the upper 2 feet consisted of a highly fractured, silty claystone. Below this 2-foot zone lay the top of upper firm rock (UFR). This rock was slightly to moderately fractured clayey siltstone, which was encountered at approximately elevation 658 along the existing river wall to elevation 654

along the left abutment. The predominant discontinuities in rock strata at the dam site consisted of horizontal bedding planes, which were most frequent in the clayshale. The greatest concentration of discontinuities occurred in the upper 3 to 5 feet of clayshale, with joint spacing on the order of 1 inch being common. Although the quality of the clayshale tended to improve with depth, the entire unit readily slaked upon exposure to air, resulting in additional separations along bedding planes (some occurring rapidly, within minutes). Vertical joints were also encountered at the top of the clayshale, with some inferred to extend 3 to 5 feet deep. The depth of chemical weathering was also confined to the upper 3 to 5 feet of the clayshale. Although no other prominent joint sets were identified, joints of various orientations appear throughout both the clayshale and siltstone, and were presumed attributable to valley stress relief. Soil samples obtained from the alluvial deposits within the area of the proposed dam were tested in the laboratory for classification, gradation, and moisture content. Relative density of the soil samples retrieved from the alluvium were correlated using Standard Penetration Test (SPT) results. Soil dry density was estimated based on a relationship with relative density. Strength tests consisting of R-Bar triaxial compression and direct shear tests were also done on Lasky samples taken within the alluvium. The alluvial soils are predominantly cohesionless, loose to dense in consistency (with the majority of the alluvial samples classified as medium dense), and classify as sandy gravel, silty gravelly sand, gravelly sand, and clayey gravelly sand.

(3) *Prerequisite Testing for Design.* Besides typical exploratory sampling and testing, a drilled shaft load test program was conducted in the summer 1998. The test program had two primary goals. First, the resulting axial and lateral loading characteristics of the test shafts were evaluated and used by designers to complete the final drilled shaft design. Secondly, the construction procedures used for the test shafts were evaluated for developing criteria and procedures for construction of the production drilled shafts.

(4) *Prerequisite Testing for Construction.* The first order of business for construction of the new dam's foundation system was to complete an exploratory drilling and driving program.

(5) *Dredging.* Pre-excavation for the dam foundation occurred first and consisted of excavating the riverbed from the existing lock river wall to the left bank abutment toe. This excavation was approximately 140-feet wide and ranged in depth between 14 and 32 feet across the approximate 650-foot reach along the dam axis. This phase of work provided the footprint for the dam foundations, as well as the necessary draft for the delivery, positioning, and set-down of the float-in segments. Figure A-73 shows the basic profile of this dredging effort.

(*a*) Precise depths of cuts and side slopes were achieved by using a Liebherr R-994 hydraulic excavator with a reach approaching 50 feet. The excavator worked from atop a 210- × 54-foot spud barge. The bucket of the excavator was 7.2 cubic yards. A transducer affixed to the bucket transmitted real-time information to an onboard microprocessor in the operator's cab. The real-time information allowed the operator to see the depth and angle of cuts in comparison to the design template that was overlaid on a monitor in the cab. With this feature, the operator had positive control to make the depth cuts and side slopes to the exact design template. Figures A-74 through A-76 show the dredging equipment in use.
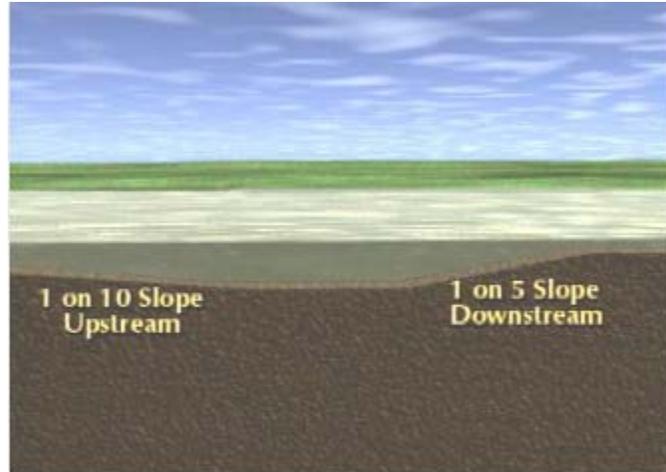
Figure A-73. Basic dredging profile.

Figure A-74. Liebherr R-994.

Figure A-75. Pre-excavation at dam's footprint.

Figure A-76. Hydraulic backhoe dredging dam footprint.

(*b*) Later work involved maintenance dredging of the river bottom inside the area bounded by the seepage cut-off walls after installation of the foundation's drilled shafts and prior to the set-down of the floating dam segments. This dredging was done immediately before set-down of the float-in dam segments with air lift equipment.

(6) *Seepage Cut-Off Walls.* After completing pre-excavation of the riverbed to elevation 690, steel sheet piling was installed to provide both upstream and downstream seepage cut-off walls. These walls restricted flow paths beneath the new dam and acted as retaining walls during other stages of the work. Figure A-77 shows the area about which the cut-off walls were installed.

Figure A-77. Alignment of seepage cut-off walls.

(*a*) The cut-off walls extended from the existing right abutment and wrapped around the upstream, left side and downstream sides of the dam and tailrace. The upstream cut-off walls

were located 3 feet from the upstream face of the new dam. Downstream and on the left side cut-off walls were located only 1.5 feet from dam and tailrace.

(*b*) In their final configuration, the sheet pile cut-off walls provided the primary seepage barrier below the base of the dam. During construction, however, the cut-off walls served as retaining walls for various stages of work on the dam. The sheet pile cut-off walls were installed prior to any other foundation work for the new dam. The sheet-pile walls were driven to top of rock at approximately elevation 669. Once installed, the area within the cut-off walls was dredged to approximately elevation 682.7 for installation of drilled shafts. At this point in the construction, the cut-off walls served as retaining walls to hold the alluvial river bottom material away from the dam foundation. In later phases of construction, the downstream cut-off provided temporary lateral support for the dam during installation of downstream scour protection, which involved replacement of a considerable depth of riverbed soil with large rock fill.

(7) *Upstream Cut-off Wall*. The upstream cut-off was installed using a barge-mounted pile driver over 35 feet of water. An International Construction Equipment (ICE) Model 44-50 vibratory hammer was used for all driving operations. Specially fabricated steel master piles (HP12x59) were first driven at approximately 19-foot centers to provide lateral support and maintain the alignment of the wall as they were driven. Each master pile had a pair of sheet pile jaws welded along the flanges for its full length. A floating two-piece guide frame was used to install the master piles (Figures A-78 and A-79).

(*a*) The guide frame was long enough to install four master piles from one setting of the frame, and was secured with two spuds. The master piles were stabbed through "driving holes" within the guide frame and driven to top of rock. The floating frame was repositioned as the wall extended across the river. Previously driven master piles were ordered sufficiently long and permitted to extend above the waterline.

(*b*) The visible tops of these piles were used as reference points by surveyors to project the driving line and aided the repositioning of the guide frame (Figure A-80).

Figure A-78. Floating guide frame template.

**Figure A-79. Assembly of floating guide frame.**

Figure A-80. Using master piles to monitor alignment of cut-off wall.

(*c*) Once a sufficient number of master piles had been driven and checked for alignment and tolerance, steel (AZ13 section) sheet piles, approximately 45 feet in length, were driven through the riverbed materials and seated into the underlying bedrock. Sheet piles had been ordered to exact lengths from the elevation data gathered during the exploratory drilling and driving program that was conducted prior to any foundation work. Sheet piles were driven in pairs with the interlock between each pair seal welded for its entire length. Having every other interlock sealed improved the seepage cut-off characteristics of the wall. Four sheet pile pairs were driven between each pair of master piles, interlocked at the special jaw sections welded to the master piles. Because sheet piles were of exact length, the top elevation would be beneath the waters surface. To permit driving below the waterline, the vibratory hammer was extended with a follower. Sheet piles were driven to within ±1 inch of a predefined top of pile elevation (eleva-

tion 694.0). Surveyors monitored precisely measured indicators marks on the follower to determine the elevation of the sheet piles.

(*d*) Master piles were cut-off at the sheet pile elevations after all sheets had been checked for alignment, elevation, and tolerances. Divers using magnetic templates, which fit the section of the master pile, cut-off the master piles with exothermic burning rods.

(8) *Downstream and Left Side Cut-off Walls.* The downstream and left side cut-off walls were installed in a similar manner, with the exception that 24-inch-diameter steel master piles were used in lieu of the HP section piles. In addition to seepage control, the downstream and left side cut-off walls acted as retaining structures. These walls provided support of the alluvium materials under the dam and tailrace as the new dam was to be founded on drilled shafts. Both wall designs allowed all of the alluvial material downstream of their alignment to be excavated to rock for placement of the new dam's scour protection. Figure A-81 shows the configuration of the downstream cut-off.
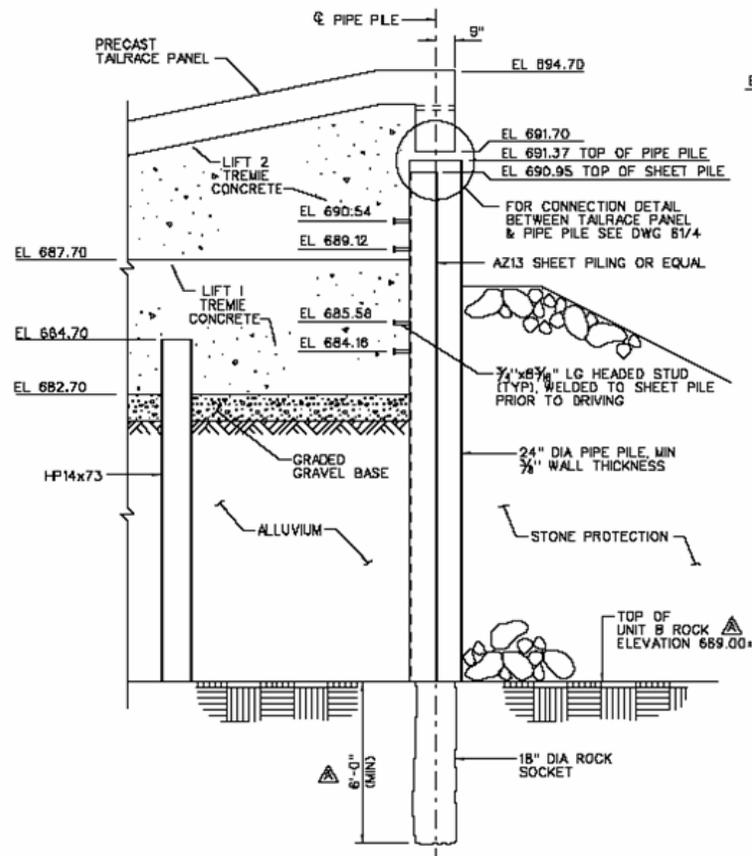
Figure A-81. Downstream cut-off wall configuration.

(*a*) The downstream cut-off wall consisted of an AZ13 section sheet pile wall with 24-inch-diameter master piles spaced at approximately 20-ft centers. Because the (AZ) sheet piles were only seated into rock, the design required that the sheets be anchored near the top of the pile. To accomplish this, the design called for the sheet piles to be driven with headed shear studs

pre-attached to the upstream flange of every sheet. These studs anchored the sheets to the concrete infill placed beneath the tailrace slabs. With anchorage now provided at the top, the downstream sheet piling could retain the alluvial material below the tailrace during the excavation for placement of the scour protection. There was not enough infill concrete to adequately anchor the sheet piles along the left side cut-off. For this portion of wall, an Arbed Tube Wall© was used. It was then possible to horizontally transfer lateral earth pressures to the master-piles by the sheet piles. The master piles then transferred the load vertically to into the rock via the rock socket.

(*b*) Sheet piles were interlocked with the steel pipe (master) piles by continuous jaw sections that were pre-welded to the pipe. The steel pipe piles were provided with a reinforced concrete rock-socket and were filled with (tremie) concrete. In addition to providing stiffness for the wall and assisting in keeping the sheet piles in line during driving, the pipe piles provided the support for the downstream edge of the dam's tailrace slabs.

(c) Based on the results of the exploratory drilling program, the top of rock profile along the cut-off wall alignment was known accurately. This allowed sheet piles to be ordered to exact lengths and the headed studs to be welded in the proper location prior to driving.

(*d*) The downstream and left side cut-off walls were also installed using a barge-mounted pile driver. A vibratory hammer was again used for all driving operations. Specially fabricated steel master piles (24-inch-diameter pipe) were first driven at approximately 19-foot centers to provide lateral support and maintain the alignment of the walls as they were driven. The same floating two-piece guide frame was used to install the master piles (Figures A-82 and A-83). The master (pipe) piles were stabbed through "driving holes" within the guide frame and driven to top of rock. Alluvial material inside the pipe pile was then removed. The rock socket was drilled and cleaned out. Then a reinforcement cage was placed within the socket, and the pipe filled with concrete to about 3 feet below the design cut-off elevation. Figure A-84 shows the completed downstream master (pipe) piles before sheets were driven.

(*e*) The floating frame was repositioned as the line of master piles extended across the river. The visible tops of these piles were used as reference points by surveyors to project the driving line and aided the repositioning of the guide frame.

(*f*) Once all master piles had been filled with concrete, steel (AZ13 section) sheet piles were driven through the riverbed materials and seated into the underlying bedrock. Sheet piles had been ordered to exact lengths and were driven with a follower in pairs (see Figure A-85). Four sheet pile pairs were driven between each pair of master piles, interlocked at the special jaw sections welded to the master piles. The top of each sheet was driven to within ±3 inches of a predefined top of pile elevation (elevation 691.7). Master (pipe) piles were then cut-off at elevation 691.7 with a tolerance of ±3 inches.

Figure A-82.   Driving downstream cut-off wall master piles using floating guide template.

Figure A-83. Driving downstream cut-off wall master piles.

Figure A-84. Downstream cut-off wall master piles—24-inch-diameter pipe.

Figure A-85. Driving sheets for left side cut-off wall.

(*g*) After the cut-off walls were completed, the area between the walls was once again dredged and then covered with a layer of crushed gravel base. Construction of the drilled shafts and tailrace H-pile foundation systems began after the cut-off walls and gavel base were finished.

(9) *Drilled Shafts.* The foundation for the new dam used 89 reinforced concrete drilled shafts. Two types of shafts were installed, 77 foundation drilled shafts and 12 set-down drilled shafts. The set-down shafts supported the weight of the floating segments until they were grouted to their permanent foundation shafts. Foundation and set-down drilled shafts measured 78 inches in diameter, with 72-inch-diameter rock sockets. The average depth of each shaft was about 40 feet, with another 15 to 20 feet of rock socket. Figure A-86 shows the arrangement of the drilled shafts.
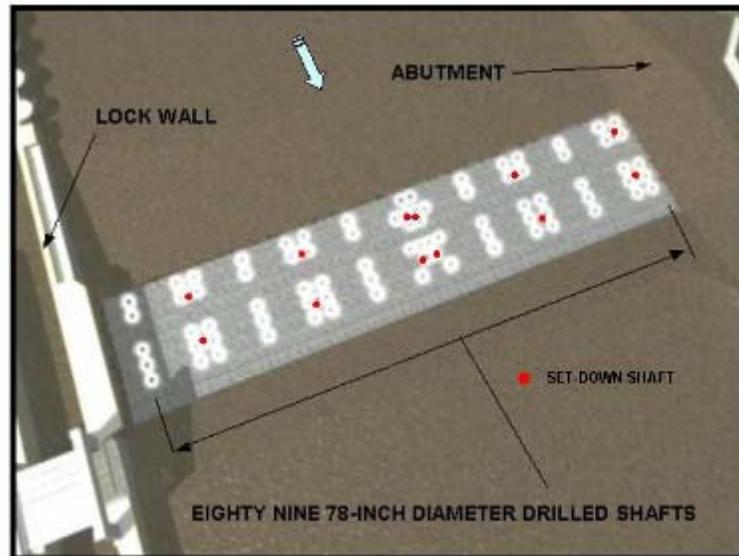
Figure A-86. Drilled shaft layout.

(*a*) One-inch thick permanent steel casings (ASTM A252, Grade 3, f_y=45 ksi) were used through the soil portion of each drilled shaft.

(*b*) All drilled shafts were positioned and installed with the use of a two-piece bottom-founded template positioned ahead of the actual casing installation. The bottom half of the template consisted of a four-legged structure that was set within ±12 inches of the required pile location. Spuds held the frame in place. The top half of the template was an independent guide frame (or jig), which slid horizontally on top of the fixed lower frame. The top frame was then pre-positioned to an accuracy of ±1 inch relative to the theoretical centerlines of the drilled shafts before stabbing the casing through it. A 15-foot long "guide tube" aided in keeping the casing plumb as it was lowered into position and driven. Figures A-87 and A-88 show the template being used.

(*c*) Casings were installed with vibratory pile driving equipment positioned on spud barges. An International Construction Equipment (ICE) Model 44-50 vibratory hammer was used for all driving operations (see Figure A-89). Each casing was checked for horizontal alignment and vertical plumb every 5 feet of advancement as it was being vibrated through the overburden and seated into the upper rock layer. Drilling and shaft clean-out were done using drill buckets and augers. The drill was equipped with a Kelly-bar of sufficient weight to maintain suitable downward force on the cleanout tools.

(*d*) The 72-inch-diameter rock socket was then drilled using a combination of shale drilling bucket, shale auger, and a cleanout bucket. The rock socket was then cleaned with an air-lift and cleanout bucket (see Figure A-90). A thorough inspection of the socket was made immediately before the reinforcing cage was placed.

Figure A-87. Positioning drilled shaft templates.

Figure A-88. Installation of template "guide tube."

Figure A-89. Vibratory driving drilled shaft casing.

Figure A-90. Clean out of drilled shaft casing.

(*e*) The reinforcing cage was then set in the casing and concrete was placed inside the casing by tremie method (see Figures A-91 through A-93). The concrete was specially formulated with pozzolans and admixtures to promote free flowing characteristics and anti-washout

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properties. The highly flowable mix consolidated without vibration and possessed the ability to flow readily through tremie pipes and to flow laterally through steel reinforcement cages. The ascending level of the concrete was continuously monitored (from the work template) with weighted measuring tapes.

Figure A-91. Drilled shaft reinforcing cage ready for installation.

Figure A-92. Preparations for drilled shaft concrete placement.

(*f*) For the main foundation drilled shafts, the concrete placement was stopped about 10 feet below the final cut-off elevation (683.86 ± ½ inch) to allow installation of a structural shear pin. This placed the first lift of concrete about 3 feet above the top of rock, thus providing a suit-able seal to dewater the casing. Figure A-94 shows the arrangement of a typical foundation drilled shaft.

(*g*) Each shear pin assembly was lowered into position within the dewatered casing and rigidly secured in place. Precise horizontal and vertical location of the pin was achieved by translating established land-based control down the casing with a plumb line. Figure A-95 shows

a workman guiding shear pin into position within a dewatered casing with reference to the plumb line.


Figure A-93. Drilled shaft concrete placement by tremie method.

(*h*) For the 12 set-down shafts, the (tremie) concrete was placed just short of the final cut-off elevation (683.33 ± 1 inch). The casing was then dewatered and a steel bearing plate was set and cast in-the-dry to a vertical tolerance of ±½ inch of the theoretical design elevation. A "cone form" was used to form the tapered edges surrounding the bearing plate. Flat surfaces surrounding the plate were hand trawled. Figure A-96 shows the arrangement of a typical set-down drilled shaft

(*i*) Removal of the work template and dewatering for in-the-dry shaft work could not be done until the initial lift of (tremie) concrete had reached a minimum compressive strength of 2000 psi. As the template was critical to the overall drilled shaft production, a non-destructive means to monitor the concrete strength had to be used. To monitor the strength and integrity of concrete, each shaft was tested with a nondestructive testing (NDT) method called Cross-hole Sonic Logging (CSL). CSL testing was conducted after at least threedays of curing time, but before seven calendar days of curing time. Only the concrete portions placed by tremie method required NDT. The CSL testing established the integrity of each drilled shaft and located any voids or defective areas in the concrete. Six CSL access tubes were installed within each drilled shaft. Access tubes had round, uniform internal walls that were free of defects and obstructions, including at pipe joints, to permit free unobstructed passage of the CSL source and receiver probes. All joints within access tubes were watertight. Each access tubes was fitted with a watertight shoe on the bottom and a removable cap on the top. CSL test equipment included a microproces-

sor based CSL system for display of individual CSL records, analog–digital conversion and re-cording of CSL data, analysis of receiver responses, and printing of CSL logs; an ultrasonic source and receiver probes for testing tubes; an ultrasonic voltage pulsar to excite the source with a synchronized triggering system to start the recording system; a depth measurement device to determine and record depths, and appropriate filter, amplification, and cable systems for CSL testing.
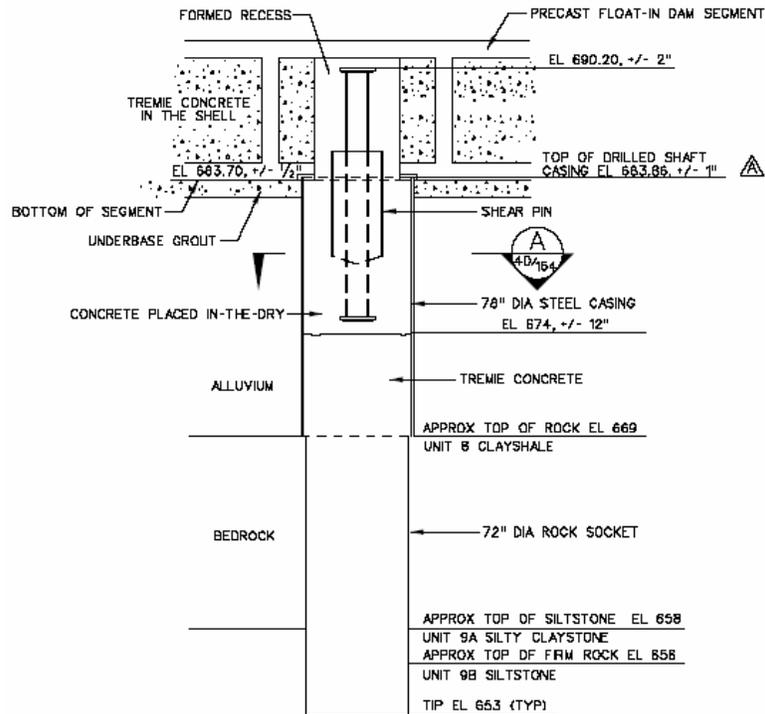

Figure A-94. Foundation drilled shaft configuration.
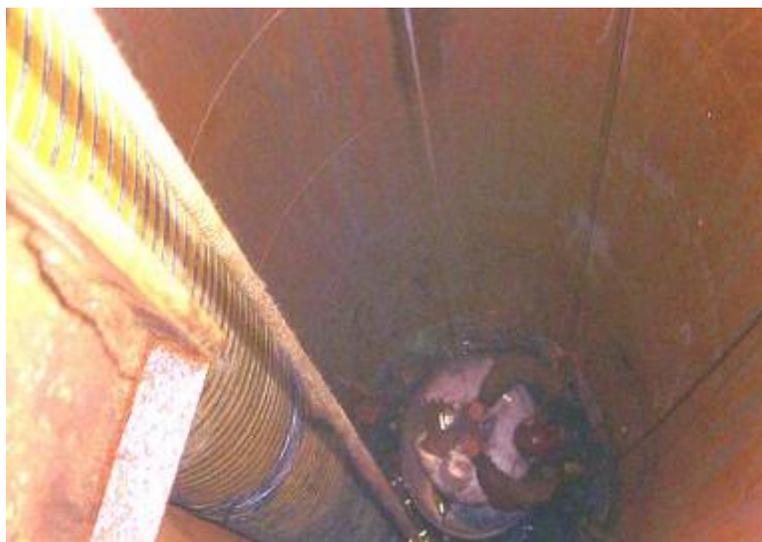
Figure A-95. Installation of shear pin inside dewatered casing.

Figure A-96. Set down drilled shaft configuration.

Figure A-97. Installation of coffer pipe for casing cut-off.

(*j*) The drill shaft designs called for extremely precise cut-off tolerances (± 1 inch) for the casings. Divers and robotic cutters could have been used, but the general contractor completed this task with the use of a unique coffering system (see Figure A-97). A pipe coffer was used to locally dewater the area about each drilled shaft, thus allowing cut-off to be done precisely from within a dewatered work area inside the permanent casings. The coffer pipe was approximately 1½-inches larger in diameter than the 78-inch-diameter permanent casings, providing a ¾-inch annulus. A 4-inch diameter o-ring seal, installed about the I.D. of the coffer pipe, was pushed outwards under

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low-pressure air (~100 psi) to engage the outer surface of the permanent casing just below the final cut-off line. Once the seal was engaged, the coffer pipe (and permanent casing) was pumped out with a pump integral to coffer pipe. All final cut-offs could then be done with great precision in-the-dry. The final casing cut-off elevation was translated down the permanent pipe from land-based control points. The cut-off line was scribed along the I.D. of the pipe and curved magnetic templates were affixed to provide positive guides during the burning operations. Once cut-off, the coffer pipe was refilled, the seal disengaged, and the coffer pipe lifted away.

(10) *Left Closure Weir.* The final task of completing the new Braddock Dam was the installation of a section of closure weir on the south (left) portion of the dam. The closure weir was positioned in a space between Pier 5 of concrete dam section and the left abutment wall. The closure weir was composed of two sheet-pile cells, 52-feet in diameter, three connecting arcs located between each cell, and two closure sections between the cells. Figure A-98 shows the general plan of this closure. The circular sheet pile cells were driven into the riverbed down to bedrock, using conventional pile-driving equipment. To maintain the stability of the left abutment wall, it was not possible to remove all in-situ materials from within the cells and arcs. Some material had to remain to provide a passive wedge to support the toe of the left abutment wall. Because this weir served as a cut-off, it was required that the remaining material be stabilized to provide a more impermeable barrier. The remaining material was leveled off and jet-grouted to make the soil impermeable. Figure A-99 and A-100 shows the areas within the cells that were stabilized. The remaining materials within the arcs and closure sections were stabilized to approximately the same levels as their companion cells. A series of concrete placements underwater filled the remainder of the cells and arcs, forming cylinders and arcs of concrete, banded with steel. The weir was finally capped with a reinforced concrete slab.

(*a*) In-situ materials remaining in the cells, arcs, and closures was stabilized by a jet grouting process. The jet grouting process created a "soilcrete" by using a stabilizing grout mix that was delivered at high velocity through the end of a monitor inserted in a bore hole. The monitor used was a double-phase fluid drill designed to simultaneously convey air and grout at a high velocity. Surrounding it within a concentric collar of compressed air enhanced the excavating action of the high velocity grout jets. A series of overlapping soilcrete columns were placed by lifting (and rotating if necessary) the monitor at a smooth and constant speed. Doing so cut the soil with fluid and air to achieve a thorough mixing and consistent, continuous geometry and quality of the grouted column. Figure A-101 shows sheet piles for a cell of the Left Closure Weir being driven.

(*b*) The three basic parameters for the resulting stabilized material were:

- The resulting stabilized material had a saturated unit weight of 115 pcf (specific gravity = 1.84).
- The stabilized material's permeability was 5×10^{-5} cm/s, or less.
- The unconfined compressive strength was 200 psi, or greater.
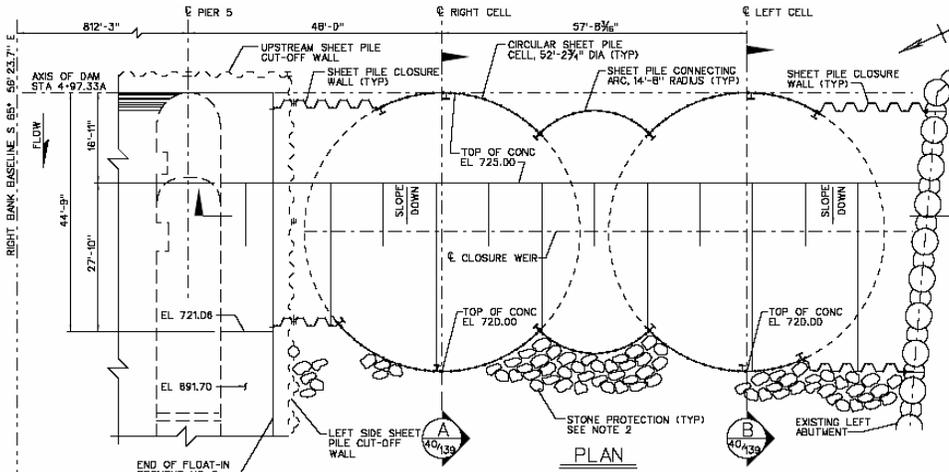
Figure A-98. Plan view of left closure weir.
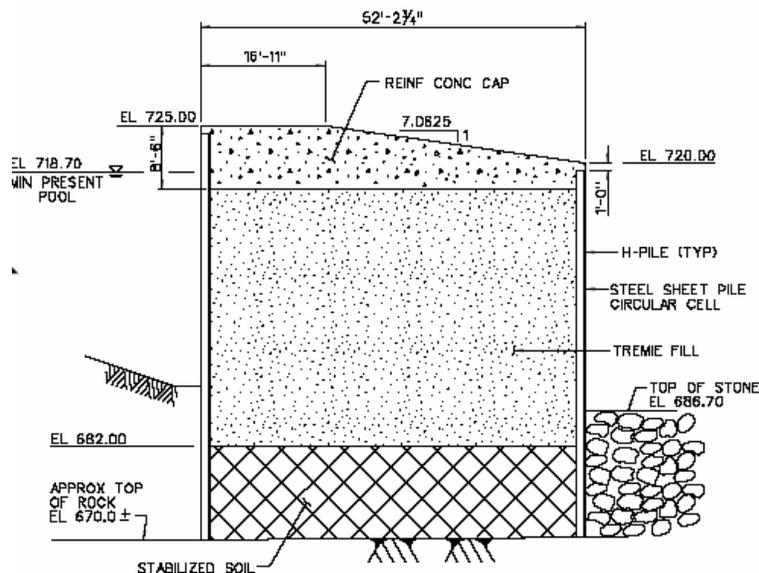
Figure A-99. Section of cell showing area of stabilized soil

(*c*) Hayward Baker (HB), who proposed the use of their proprietary Superjet Grouting System, stabilized the soil. HB's Superjet Grouting process was similar to a double rod jet grouting system in that it uses high velocity grout jets to erode and mix pre-engineered cement grouts with in-situ soil. The variation between the two techniques relies primarily upon different tooling that is designed to minimize jet flow resistance in the grout monitor and provide maximum energy to the high velocity grout jets. The revised tooling and slower (monitor) pulling rates allowed for greater 'cutting' distances and higher unit weights of the in-situ materials than would be otherwise provided by a conventional double rod jet grouting system.

(*d*) Drilling and control of the monitor was accomplished with a hydraulic rotary drill (Bauer BG15H). The drill rod was a double stem-type rod with an O.D. of 4½ inches. Borehole diameter was between 5½ to 6-inches using a standard drag-type bit.

(*e*) All soil stabilization activities were conducted from a temporary trestle that was erected above the sheet pile cells, arcs, and closures. Grout was mixed and pumped from an ad-jacent barge directly through hosing connected to the monitor.
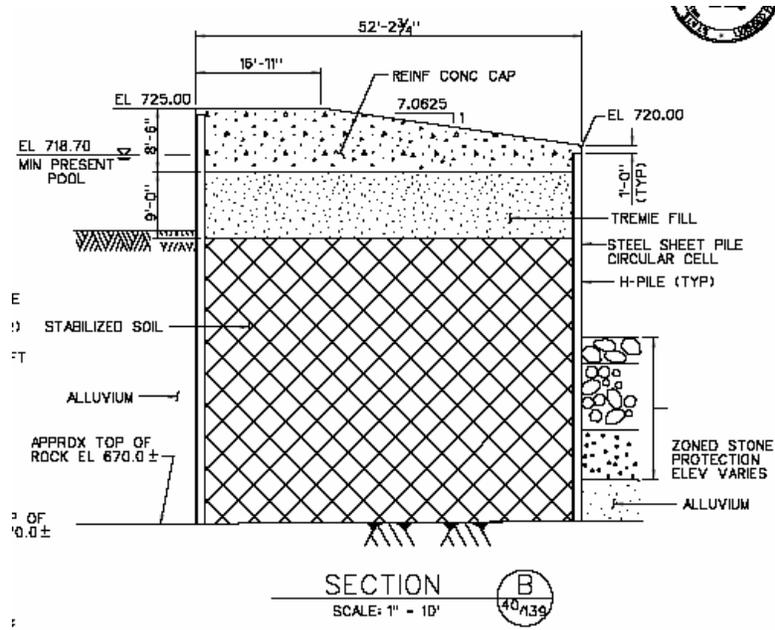

Figure A-100. Section of cell showing stabilized soil area.

Figure A-101. Driving sheet pile cell for left closure weir.

(*f*) Jet grouting followed an established grid of soilcrete columns as shown in Figure A-102. A test section of three soilcrete columns was completed initially. The test area was used to determine if permeability requirements were being met and to establish the final design mix for the grout. Each of the three columns forming the test area were grouted using different mixes to

test the unit weight variations. Figure A-103 shows a rotary drill rig operation with monitor with temporary work trestle.
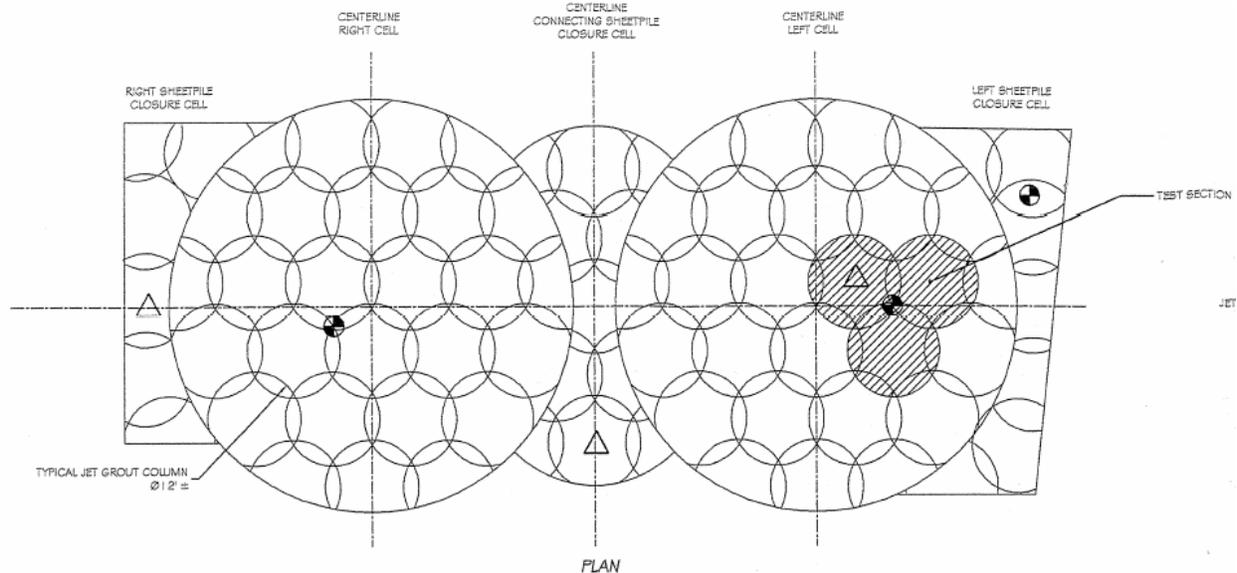
Figure A-102. Jet grouting pattern for left closure weir.

(*g*) To satisfy the required permeability, a water/cement ratio between 1:1 and 4:1 was used. The exact w/c ratio was established from the test section. The quality of the grout mix was monitored consistently as the material was injected into the monitor with measurements being made with a micro-motion t-series density meter to assure the design density would fall within ± 0.05.

(*h*) During jet grouting operations, some of the grout/soil mixture generated would exit up the drilling annulus as spoil. This spoil remained within the cells until all jet grouting operations were completed. The spoil layer was then removed with air-lift equipment prior to placing the tremie concrete lift.

(*i*) Confirmation testing was done following grouting of the test columns and production columns. Testing included core drilling of a column or interstitial point of several columns, testing of core samples (saturated unit weight and strength), and in-situ permeability testing of the core location. In place permeability was tested by installing standpipe piezometers in freshly jet grouted columns. Once the soilcrete had cured for 7 days, the piezometer was used to measure the column permeability using falling head techniques. The permeability at each test location could then be calculated from the results of the falling head tests.

Figure A-103. Rotary drill rig with monitor with temporary work trestle.

 d. Mississippi River Lock and Dam No. 24.

 (1) *Downstream Protection Cell (St. Louis District).*[*] This project was the construction of a lock guide wall protection cell utilizing a steel cylindrical can, 35 feet in diameter, and 45 feet tall. The can was constructed by individually fabricating steel rings and then stacking and welding them together. The steel was 1-inch thick plate on rubbing side and ½-inch plate elsewhere. Note that in 2003 the plate split. Additional rubbings strips were required and installed to distribute impacts to concrete. The can was internally stiffened with five frames for handling, concreting, and pile sockets. Mooring posts and supports were pre-attached to the shell. The can was fabricated approximately 80 miles downstream of Lock 24. The cost of the project was $850,000 while the Government estimate was $1,300,000.

 (*a*) Construction tolerances were:

- ± 12 inches from end of guidewall.
- ± 0.5 degree plumb ±1.0 degree rotation.
- Flush to –6.0 inches from guidewall face.
- 6.0-inch maximum gap between steel and rock foundation.

 (*b*) The can was installed on three drilled pipe (guide) piles in approximately 25 feet of water. A "Stinger" used to precisely locate the hole. A 30–inch-diameter outer casing was socketed 6 inches into the rock. The drilling within casing was concurrent with navigation through the lock. The holes were grouted and piles placed during a 48 hour navigation closure.

 (*c*) The 20-inch-diameter piles were installed with centralizers welded to pile. The piles were placed through wet grout. The plumb was checked by level and conventional survey. Two

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[*] Project Contact—Jeff Stamper (MVS)—314-331-8226.

piles ended up obviously out of plumb, but this did not hinder achieving the tolerances of the cell. The piles act as guides, resist current forces, and help support the weight of the cell (see Figure A-104).

Figure A-104. Installation of a guide pile.

(*d*) In conjunction with a demonstration project by the Engineer Research and Development Center, Global Positioning Systems were tested for real time monitoring of the casing and cell positions (see Figure A-105 through A-107). Sensors were placed on the crane boom and cell for monitoring horizontal position during installation.
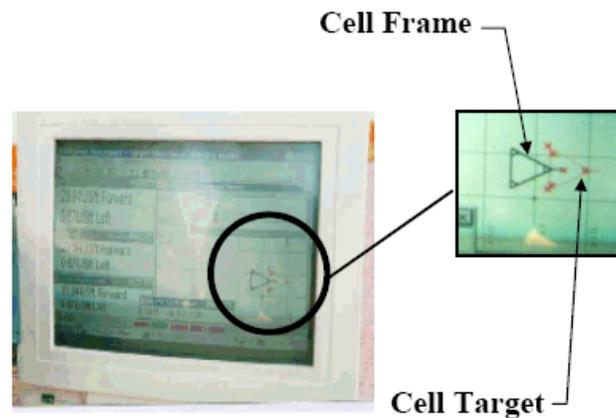
Figure A-105. GPS monitoring display.

Figure A-106. GPS target sensor on crane boom.
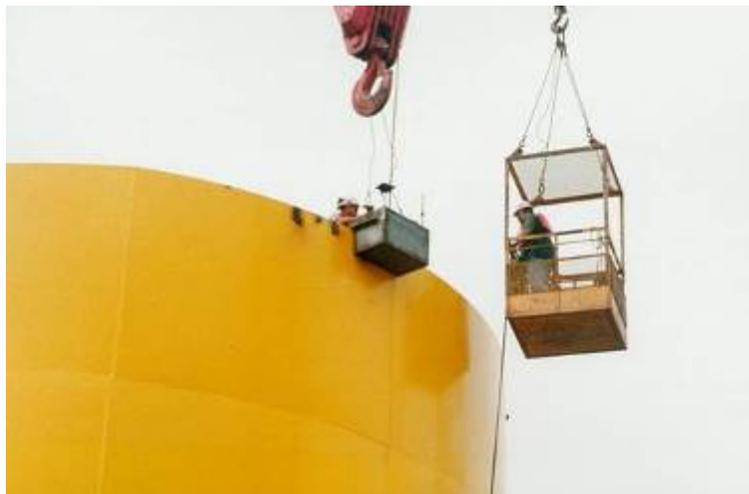
Figure A-107. GPS sensor on steel cell.

(*e*) The bottom of cell was scribed and cut to the contour of the bedrock (see Figure A-108). This prevented large gaps between the steel and rock and helped contain the concrete fill. The cell (80 tons) was placed by two cranes on a floating plant (see Figure A-109). The cell was positioned over the guide piles by the push boat and some booming of the cranes. The draft of

the floating plant helped block river currents. The use of staggered stabbing guides helped make threading of the frame over the guide piles easier (see Figure A-110). A 72-hour lock closure was required for cell placement and the first lift of concrete.

**Figure A-108. Bottom of cell scribed and cut to match rock surface.**

(*f*) Once the cell was set, a stone seal was place around exterior perimeter (see Figure A-111). Divers then inspected the stone seal and added sandbags where required. The foundation of the cell was cleaned by airlift and a tremie concrete seal layer placed (see Figure A-112). Figure A-113 shows the complete cell.
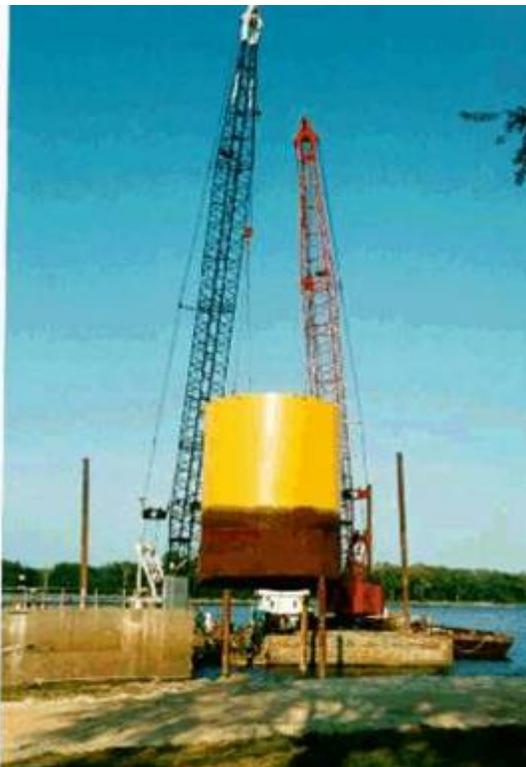
Figure A-109. Cell installation.   **Figure A-110. Stabbing guide.**
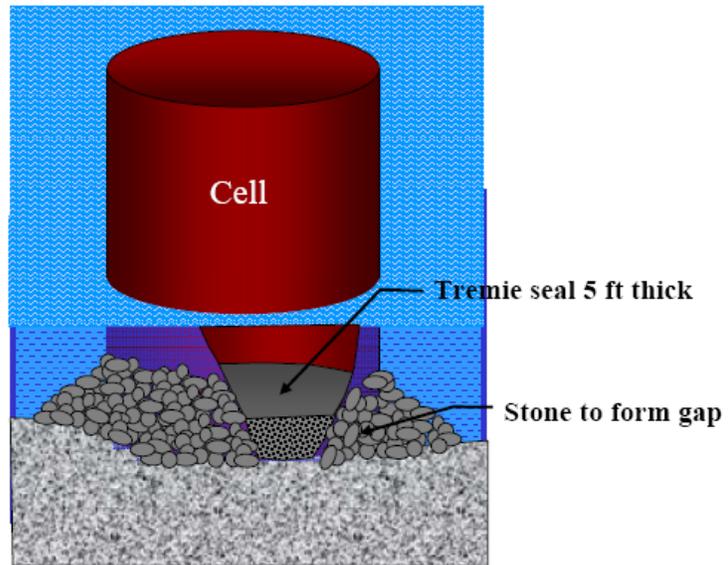
Figure A-111. Stone seal and tremie concrete.

Figure A-112. Placing tremie concrete.

Figure A-113. Downstream view of completed cell.

  e. *KY Lock Addition—Highway and Railroad Bridge Foundations (Nashville District).*[*]
To make room for a lock chamber addition, both a railroad line and state highway must be relocated off of the Kentucky Dam crest. The Nashville District is constructing in-the-wet, large diameter, drilled shaft foundations for new bridge structures just downstream of the dam (see Figure A-114).

  (1) The foundation system consists of 88 Shafts (82 constructed in-the-wet)—6 foot diameter for the highway and 7 foot diameter for the railroad bridge (see Figure A-115). Permanent casings were used above rock. The design intent was to screw (seat) the casings into the rock ,1–2 feet. Because of the hardness of the rock, most were actually embedded only a few inches. Some sealing problems occurred. To overcome this problem, 5 feet of concrete was placed on top-of-rock, inside the temporary casing, before the permanent casing was set.

Figure A-114. Typical marine pier.

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[*] Contacts: Paul Ross (geologist) Tony Ellis (project engineer) 270-362-2163 Nashville District.

Figure A-115. Floating template.

(2) The specifications required four exploratory borings at each pier location, one per shaft. After award, the contractor drilled NQ holes at each shaft location and findings were used to determine final tip elevations. The foundation material was a karst Fort Payne limestone with large solution features. The rock was very hard, exceeding 40,000 psi on some tests, and contained numerous beds, lenses, and nodules of chert.

(3) Shaft construction was started with the kerf cutting and core recovery method (see Figures A-116 through A-119). This was required so inspection of the rock core could indicate any problems or required remediation of the top of the rock socket. It was later determined that this method was inefficient.

Figure A-116. Driving temporary casing.

Figure A-117. Soil clean out with auger bucket.

Figure A-118. Kerf cutting bit.   **Figure A-119. Kerf core retriever.**

(4) The Wirth Co. in Germany built a casing top drill to complete the work that utilized a full face bit with reverse circulation (see Figures A-120 through A-122). This method was much faster, cutting approximately 2 feet per hour.

Figure A-120. Wirth pile top reverse circulation drill.

Figure A-121. Wirth full face bit.

Figure A-122. Permanent casing set inside temporary casing.

(5) QC included video using the Shaft Inspection Device (SID) and the sonar caliper to determine the shape of the rock socket, verify shaft verticality, and detect solution features in the walls (see Figures A-123 through A-126). Templates were used with GPS and conventional surveying for positioning. Cross-hole Sonic Log (CSL) testing was used to verify concrete quality (see Figures A-127 through A-128). One land based highway abutment shaft had two 3-foot zones of poor concrete and required grouting. Figure A-129 shows setting the pier base form.

Figure A-123. Shaft Inspection Device (SID).


Figure A-124. Sonar caliper.
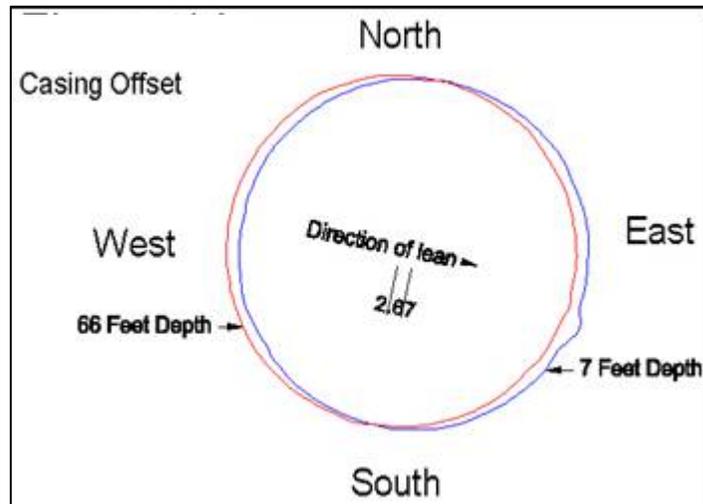
Figure A-125. Sonar caliper plot showing casing lean.
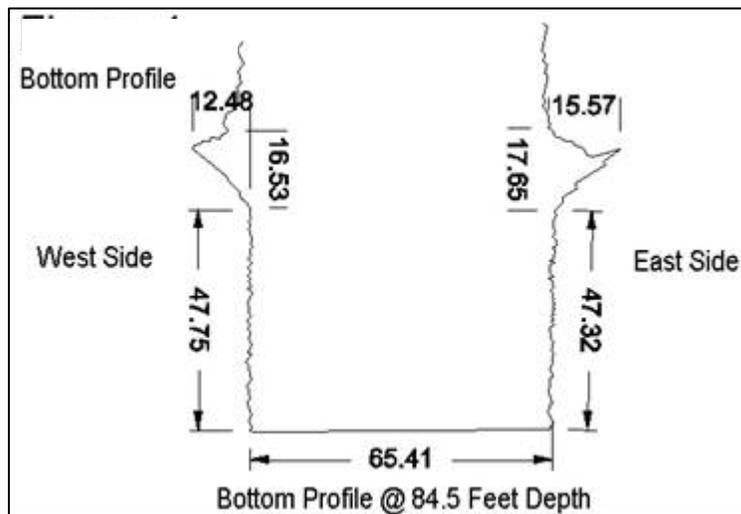
Figure A-126. Sonar caliper plot showing void in rock socket excavation walls.

Figure A-127. Rebar cage with CSL tubes.   **Figure A-128. Placing tremie concrete.**

Figure A-129. Setting pier base form.

(6) Pool fluctuations and currents from dam discharge or Ohio River backwater limited construction time. They had to deal with large variations in tailwater elevations, up to approximately 36 feet, which affected spud lengths, template elevations, drilling tools, and boom lengths.

(7) Other factors included requirements to relocate all mussels at every shaft location by hand with divers and restrictions on drilling during fish spawning periods. Construction was re-

stricted during fish spawning periods in early spring, times that drilling activities were already suspended because of high tailwater levels. Figure A-130 shows the completed piers.

Figure A-130. Bridge piers.

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