

Gerwick, Jr. B.C. "Substructures of Major Overwater Bridges."  
*Bridge Engineering Handbook.*  
Ed. Wai-Fah Chen and Lian Duan  
Boca Raton: CRC Press, 2000

# 47

## Substructures of Major Overwater Bridges

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## 47.1 Introduction

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The design and construction of the piers for overwater bridges present a series of demanding criteria. In service, the pier must be able to support the dead and live loads successfully, while resisting environmental forces such as current, wind, wave, sea ice, and unbalanced soil loads, sometimes even including downslope rock fall. Earthquake loadings present a major challenge to design, with cyclic reversing motions propagated up through the soil and the pier to excite the superstructure. Accidental forces must also be resisted. Collision by barges and ships is becoming an increasingly serious hazard for bridge piers in waterways, both those piers flanking the channel and those of approaches wherever the water depth is sufficient.

Soil–structure foundation interaction controls the design for dynamic and impact forces. The interaction with the superstructure is determined by the flexibility of the entire structural system and its surrounding soil.

Rigid systems attract very high forces: under earthquake, the design forces may reach 1.0 g, whereas flexible structures, developing much less force at longer periods, are subject to greater deflection drift. The design must endeavor to obtain an optimal balance between these two responses. The potential for scour due to currents, amplified by vortices, must be considered and preventive measures instituted.

Constructibility is of great importance, in many cases determining the feasibility. During construction, the temporary and permanent structures are subject to the same environmental and accidental loadings as the permanent pier, although for a shorter period of exposure and, in most cases, limited to a favorable time of the year, the so-called weather window. The construction processes employed must therefore be practicable of attainment and completion. Tolerances must be a suitable compromise between practicability and future performance. Methods adopted must not diminish the future interactive behavior of the soil–structure system.

The design loadings for overwater piers are generally divided into two limit states, one being the limit state for those loadings of high probability of occurrence, for which the response should be essentially elastic. Durability needs to be considered in this limit state, primarily with respect to corrosion of exposed and embedded steel. Fatigue is not normally a factor for the pier concepts usually considered, although it does enter into the considerations for supplementary elements such as fender systems and temporary structures such as dolphins if they will be utilized under conditions of cyclic loading such as waves. In seismic areas, moderate-level earthquakes, e.g., those with a return period of 300 to 500 years, also need to be considered.

The second limit state is that of low-probability events, often termed the “safety” or “extreme” limit state. This should include the earthquake of long return period (1000 to 3000 years) and ship collision by a major vessel. For these, a ductile response is generally acceptable, extending the behavior of the structural elements into the plastic range. Deformability is essential to absorb these high-energy loads, so some damage may be suffered, with the provision that collapse and loss of life are prevented and, usually, that the bridge can be restored to service within a reasonable time.

Plastic hinging has been adopted as a principle for this limit state on many modern structures, designed so that the plastic hinging will occur at a known location where it can be most easily inspected and repaired. Redundant load paths are desirable: these are usually only practicable by the use of multiple piles.

Bridge piers for overwater bridges typically represent 30 to 40% of the overall cost of the bridge. In cases of deep water, they may even reach above 50%. Therefore, they deserve a thorough design effort to attain the optimum concept and details.

Construction of overwater bridge piers has an unfortunate history of delays, accidents, and even catastrophes. Many construction claims and overruns in cost and time relate to the construction of the piers. Constructibility is thus a primary consideration.

The most common types of piers and their construction are described in the following sections.

## **47.2 Large-Diameter Tubular Piles**

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### **47.2.1 Description**

Construction of steel platforms for offshore petroleum production as well as deep-water terminals for very large vessels carrying crude oil, iron, and coal, required the development of piling with high axial and lateral capacities, which could be installed in a wide variety of soils, from soft sediments to rock. Lateral forces from waves, currents, floating ice, and earthquake as well as from berthing dominated the design. Only large-diameter steel tubular piles have proved able to meet these criteria (Figures 47.1 and 47.2).



FIGURE 47.1 Large-diameter steel tubular pile, Jamuna River Bridge, Bangladesh



FIGURE 47.2 Driving large-diameter steel tubular pile.



**FIGURE 47.3** Steel tubular pile being installed from jack-up barge. Socket will be drilled into rock and entire pile filled with tremie concrete.

Such large piling, ranging from 1 to 3 m in diameter and up to over 100 m in length required the concurrent development of very high energy pile-driving hammers, an order of magnitude higher than those previously available. Drilling equipment, powerful enough to drill large-diameter sockets in bedrock, was also developed (Figure 47.3).

Thus when bridge piers were required in deeper water, with deep sediments of varying degrees or, alternatively, bare rock, and where ductile response to the lateral forces associated with earthquake, ice, and ship impact became of equal or greater importance than support of axial loads, it was only natural that technology from the offshore platform industry moved to the bridge field.

The results of this “lateral” transfer exceeded expectations in that it made it practicable and economical to build piers in deep waters and deep sediments, where previously only highly expensive and time-consuming solutions were available.

### 47.2.2 Offshore Structure Practice

The design and construction practices generally follow the Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms published by the American Petroleum Institute, API-RP2A [1]. This recommended practice is revised frequently, so the latest edition should always be used. Reference [2] presents the design and construction from the construction contractor’s point of view.

There are many variables that affect the designs of steel tubular piles: diameter, wall thickness (which may vary over the length), penetration, tip details, pile head details, spacing, number of piles, geometry, and steel properties. There must be consideration of the installation method and its effect on the soil–pile interaction. In special cases, the tubular piles may be inclined, i.e., “raked” on an angle from vertical.

In offshore practice, the piles are almost never filled with concrete, whereas for bridge piers, the designer's unwillingness to rely solely on skin friction for support over a 100-year life as well as concern for corrosion has led to the practice of cleaning out and filling with reinforced concrete. A recent advance has been to utilize the steel shell along with the concrete infill in composite action to increase strength and stiffness. The concrete infill is also utilized to resist local buckling under overload and extreme conditions. Recent practice is to fill concrete in zones of high moment.

Tubular piles are used to transfer the superimposed axial and lateral loads and moments to the soil. Under earthquake, the soil imparts dynamic motions to the pile and hence to the structure. These interactions are highly nonlinear. To make matters even more complex, the soils are typically nonuniform throughout their depth and have different values of strength and modulus.

In design, axial loads control the penetration while lateral load transfer to the soil determines the pile diameter. Combined pile stresses and installation stresses determine the wall thickness. The interaction of the pile with the soil is determined by the pile stiffness and diameter. These latter lead to the development of a  $P$ - $y$  curve,  $P$  being the lateral shear at the head of the pile and  $y$  being the deflection along the pile. Although the actual behavior is very complex and can only be adequately solved by a computerized final design, an initial approximation of three diameters can give an assumed "point of fixity" about which the top of the pile bends.

Experience and laboratory tests show that the deflection profile of a typical pile in soft sediments has a first point of zero deflection about three diameters below the mudline, followed by deflection in reverse bending and finally a second point of zero displacement. Piles driven to a tip elevation at or below this second point have been generally found to develop a stable behavior in lateral displacement even under multiple cycles of high loading.

If the deflection under extreme load is significant,  $P$ - $\Delta$  effects must also be considered. Bridge piers must not only have adequate ultimate strength to resist extreme lateral loads but must limit the displacement to acceptable values. If the displacement is too great, the  $P$ - $\Delta$  effect will cause large additional bending moments in the pile and consequently additional deflection.

The axial compressive behavior of piles in bridge piers is of dominant importance. Settlement of the pile under service and extreme loads must be limited. The compressive axial load is resisted by skin friction along the periphery of the pile, by end bearing under the steel pile tip, and by the end bearing of the soil plug in the pile tip. This latter must not exceed the skin friction of the soil on the inside of the pile, since otherwise the plug will slide upward. The actual characteristics of the soil plug are greatly affected by the installation procedures, and will be discussed in detail later.

Axial tension due to uplift under extreme loads such as earthquakes is resisted by skin friction on the periphery and the deadweight of the pile and footing block.

Pile group action usually differs from the summation of individual piles and is influenced by the stiffness of the footing block as well as by the applied bending moments and shears. This group action and its interaction with the soil are important in the final design, especially for dynamic loading such as earthquakes.

API-RP2A Section G gives a design procedure for driven steel tubular piles as well as for drilled and grouted piles.

Corrosion and abrasion must be considered in determining the pile wall thickness. Corrosion typically is most severe from just below the waterline to just above the wave splash level at high tide, although another vulnerable location is at the mudline due to the oxygen gradient. Abrasion typically is most severe at the mudline because of moving sands, although suspended silt may cause abrasion throughout the water column.

Considering a design lifetime of a major bridge of 100 years or more, coatings are appropriate in the splash zone and above, while sacrificial anodes may be used in the water column and at the mudline. Additional pile wall thickness may serve as sacrificial steel: for seawater environment, 10 to 12 mm is often added.

### 47.2.3 Steel Pile Design and Fabrication

Tubular steel piles are typically fabricated from steel plate, rolled into “cans” with the longitudinal seam being automatically welded. These cans are then joined by circumferential welds. Obviously, these welds are critical to the successful performance of the piles. During installation by pile hammer, the welds are often stressed very highly under repeated blows: defective welds may crack in the weld or the heat-affected zone (HAZ). Welds should achieve as full joint penetration as practicable, and the external weld profile should merge smoothly with the base metal on either side.

API-RP2A, section L, gives guidance on fabrication and welding. The fabricated piles should meet the specified tolerances for both pile straightness and for cross section dimensions at the ends. These latter control average diameter and out-of-roundness. Out-of-roundness is of especial concern as it affects the ability to match adjacent sections for welding.

Inspection recommendations are given in API-RP2A, section N. Table N.4-1, with reference to structural tubulars, calls for 10% of the longitudinal seams to be verified by either ultrasonic (UT) or radiography (RT). For the circumferential weld seams and the critical intersection of the longitudinal and circumferential seams, 100% UT or RT is required.

Because of the typically high stresses to which piles supporting bridge piers are subjected, both under extreme loads and during installation, as well as the need for weldability of relatively thick plates, it is common to use a fine-grained steel of 290 to 350 MPa yield strength for the tubular piles.

Pile wall thickness is determined by a number of factors. The thickness may be varied along the length, being controlled at any specific location by the loading conditions during service and during installation.

The typical pile used for a bridge pier is fixed at the head. Hence, the maxima combined bending and axial loads will occur within the  $1\frac{1}{2}$  diameters immediately below the bottom of the footing. Local buckling may occur. Repeated reversals of bending under earthquake may even lead to fracture. This area is therefore generally made of thicker steel plate. Filling with concrete will prevent local buckling. General column buckling also needs to be checked and will usually be a maximum at a short distance below the mudline.

Installation may control the minimum wall thickness. The hammer blows develop high compressive waves which travel down the pile, reflecting from the tip in amplified compression when high tip resistance is encountered. When sustained hard driving with large hammers is anticipated, the minimum pile wall thickness should be  $t = 6.35 + D/100$  where  $t$  and  $D$  are in millimeters. The drivability of a tubular pile is enhanced by increasing the wall thickness. This reduces the time of driving and enables greater penetration to be achieved.

During installation, the weight of the hammer and appurtenances may cause excessive bending if the pile is being installed on a batter. Hydraulic hammers usually are fully supported on the pile, whereas steam hammers and diesel hammers are partially supported by the crane.

If the pile is cleaned out during driving in order to enable the desired penetration to be achieved, external soil pressures may develop high circumferential compression stresses. These interact with the axial driving stresses and may lead to local buckling.

The tip of the pile is subject to very high stresses, especially if the pile encounters boulders or must be seated in rock. This may lead to distortion of the tip, which is then amplified during successive blows. In extreme cases, the tip may “tear” or may “accordion” in a series of short local axial buckles. Cast steel driving shoes may be employed in such cases; they are usually made of steels of high toughness as well as high yield strength. The pile head also must be thick enough to withstand both the local buckling and the bursting stresses due to Poisson’s effect.

The transition between sections of different pile wall thickness must be carefully detailed. In general, the change in thickness should not be more than 12 mm at a splice and the thicker section should be beveled on a 1:4 slope.



FIGURE 47.4 Large-diameter tubular steel pile being positioned.

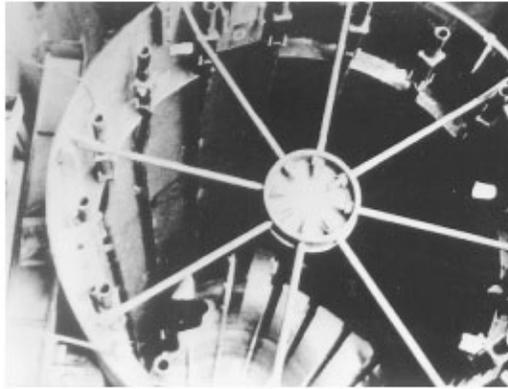
#### 47.2.4 Transportation and Upending of Piles

Tubular piles may be transported by barge. For loading, they are often simply rolled onto the barge, then blocked and chained down. They may also be transported by self-flotation. The ends are bulkheaded during deployment. The removal of the bulkheads can impose serious risks if not carefully planned. One end should be lifted above water for removal of that bulkhead, then the other. If one bulkhead is to be removed underwater by a diver, the water inside must first be equalized with the outside water; otherwise the rush of water will suck the diver into the pipe. Upending will produce high bending moments which limit the length of the sections of a long pile (Figure 47.4). Otherwise the pile may be buckled.

#### 47.2.5 Driving of Piles

The driving of large-diameter tubular piles [2] is usually done by a very large pile hammer. The required size can be determined by both experience and the use of a drivability analysis, which incorporates the soil parameters.

Frequently, the tubular pile for a bridge pier is too long or too heavy to install as a single section. Hence, piles must be spliced during driving. To assist in splicing, stabbing guides may be preattached to the tip of the upper segment, along with a backup plate. The tip of the upper segment should be prebeveled for welding.



**FIGURE 47.5** Arrangement of internal jet piping and “spider” struts in large-diameter tubular pile.

Splicing is time-consuming. Fortunately, on a large-diameter pile of 2 to 4 m diameter, there is usually space to work two to three crews concurrently. Weld times of 4 to 8 h may be required. Then the pile must cool down (typically 2 h) and NDT performed. Following this, the hammer must be repositioned on top of the pile. Thus a total elapsed time may be 9 to 12 h, during which the skin friction on the pile sides “sets up,” increasing the driving resistance and typically requiring a number of blows to break the pile loose and resume penetration.

When very high resistance is encountered, various methods may be employed to reduce the resistance so that the design pile tip may be reached. Care must be taken that these aids do not lessen the capacity of the pile to resist its design loads.

High resistance of the tubular pile is primarily due to plugging of the tip; the soil in the tip becomes compacted and the pile behaves as a displacement pile instead of cutting through the soil. The following steps may be employed.

1. *Jetting internally to break up the plug, but not below the tip.* The water level inside must be controlled, i.e., not allowed to build up much above the outside water level, in order to prevent piping underneath. Although a free jet or arrangement of jets may be employed, a very effective method is to manifold a series of jets around the circumference and weld the down-going pipes to the shell (Figure 47.5). Note that these pipes will pick up parasitic stresses under the pile hammer blows.
2. *Clean out by airlift.* This is common practice when using large-diameter tubular piles for bridge piers but has serious risks associated with it. The danger arises from the fact that an airlift can remove water very rapidly from the pile, creating an unbalanced head at the tip, and allowing run-in of soil. Such a run-in can result in major loss of resistance, not only under the tip in end bearing but also along the sides in skin friction.

Unfortunately, this problem has occurred on a number of projects! The prevention is to have a pump operating to refill the pile at the same rate as the airlift empties it — a very difficult matter to control. If structural considerations allow, a hole can be cut in the pile wall so that the water always automatically balances. This, of course, will only be effective when the hole is below water. The stress concentrations around such a hole need to be carefully evaluated. Because of the risks and the service consequences of errors in field control, the use of an airlift is often prohibited. The alternative method, one that is much safer, is the use of a grab bucket (orange peel bucket) to remove the soil mechanically. Then, the water level can be controlled with relative ease.

3. *Drilling ahead a pilot hole, using slurry.* If the pile is kept full of slurry to the same level as the external water surface, then a pilot hole, not to exceed 75% of the diameter, may be drilled ahead from one to two diameters. Centralizers should be used to keep the drilled hole properly

aligned. Either bentonite or a polymer synthetic slurry may be used. In soils such as stiff clay or where a binder prevents sloughing, seawater may be used. Reverse circulation is important to prevent erosion of the soils due to high-velocity flow. Drilling ahead is typically alternated with driving. The final seating should be by driving beyond the tip of the drilled hole to remobilize the plug resistance.

4. *External jetting.* External jetting relieves the skin friction during driving but sometimes permanently reduces both the lateral and axial capacity. Further, it is of only secondary benefit as compared with internal jetting to break up the plug. In special cases, it may still be employed. The only practicable method to use with long and large tubular piles is to weld the piping on the outside or inside with holes through the pile wall. Thus, the external jetting resembles that used on the much larger open caissons. As with them, low-pressure, high-volume water flow is most effective in reducing the skin friction. After penetration to the tip, grout may be injected to partially restore the lateral and axial capacity.

### 47.2.6 Utilization of Piles in Bridge Piers

There are several possible arrangement for tubular piles when used for bridge piers. These differ in some cases from those used in offshore platforms.

1. The pile may be driven to the required penetration and left with the natural soil inside. The upper portion may then be left with water fill or, in some cases, be purposely left empty in order to reduce mass and weight; in this case it must be sealed by a tremie concrete plug. To ensure full bond with the inside wall, that zone must be thoroughly cleaned by wire brush on a drill stem or by jet.

For piles fixed at their head, at least 2 diameters below the footing are filled with concrete to resist local buckling. Studs are installed in this zone to ensure shear transfer.

2. The pile, after driving to final penetration, is cleaned out to within one diameter of the tip. The inside walls are cleaned by wire brush or jet. A cage of reinforcing steel may be placed to augment the bending strength of the tubular shell. Centralizers should be used to ensure accurate positioning. The pile is then filled with tremie concrete. Alternatively, an insert steel tubular with plugged tip may be installed with centralizers, and the annular space filled with tremie grout. The insert tubular may need to be temporarily weighted and/or held down to prevent flotation in the grout.

Complete filling of a tubular pile with concrete is not always warranted. The heat of hydration is a potential problem, requiring special concrete mix design and perhaps precooling.

The reasons for carrying out this practice, so often adopted for bridge piers although seldom used in offshore structures, are

- a. Concern over corrosion loss of the steel shell over the 100-year lifetime;
- b. A need to ensure positively the ability of the permanent plug to sustain end bearing;
- c. Prevention of local buckling near the mudline and at the pile head;
- d. To obtain the benefits of composite behavior in stiffness and bending capacity.

If no internal supplemental reinforcement is required, then the benefits of (b), (c), and (d) may be achieved by simple filling with tremie concrete. To offset the heat of hydration, the core may be placed as precast concrete blocks, subsequently grouted into monolithic behavior. Alternatively, an insert pile may be full length. In this case, only the annulus is completely filled. The insert pile is left empty except at the head and tip.

The act of cleaning out the pile close to the tip inevitably causes stress relaxation in the soil plug below the clean-out. This will mean that under extreme axial compression, the pile will undergo a small settlement before it restores its full resistance. To prevent this, after the concrete plug has hardened, grout may be injected just beneath the plug, at a pressure that will restore the compactness of the soil but not so great as to pipe under the tip or fracture the foundation, or the pile may be re-seated by driving.

3. The tubular pile, after being installed to design penetration, may be filled with sand up to two diameters below the head, then with tremie concrete to the head. Reinforcing steel may be placed in the concrete to transfer part of the moment and tension into the footing block. Studs may be pre-installed on that zone of the pile to ensure full shear transfer. The soil and sand plug will act to limit local buckling at the mudline under extreme loads.

4. A socket may be drilled into rock or hard material beyond the tip of the driven pile, and then filled with concrete. Slurry is used to prevent degradation of the surface of the hole and sloughing. Seawater may be used in some rocks but may cause slaking in others such as shales and siltstone. Bentonite slurry coats the surface of the hole; the hole should be flushed with seawater just before concreting. Synthetic slurries are best, since they react in the presence of the calcium ion from the concrete to improve the bond. Synthetic polymer slurries biodegrade and thus may be environmentally acceptable for discharge into the water.

When a tubular pile is seated on rock and the socket is then drilled below the tip of the pile, it often is difficult to prevent run-in of sands from around the tip and to maintain proper circulation. Therefore, after landing, a hole may be drilled a short distance, for example, with a churn drill or down-the-hole drill and then the pile resealed by the pile hammer.

Either insert tubulars or reinforcing steel cages are placed in the socket, extending well up into the pile. Tremie concrete is then placed to transfer the load in shear. In the case where a tubular insert pile is used, its tip may be plugged. Then grout may be injected into the annular space to transfer the shear.

Grout should not be used to fill sockets of large-diameter tubulars. The heat of hydration will damage the grout, reducing its strength. Tremie concrete should be used instead, employing small-size coarse aggregate, e.g., 15 mm, to ensure workability and flowability.

Although most sockets for offshore bridge piers have been cylindrical extensions of the tubular pile, in some offshore oil platforms belled footings have been constructed to transfer the load in end bearing. Hydraulically operated belling tools are attached to the drill string. Whenever transfer in end bearing is the primary mechanism, the bottom of the hole must be cleaned of silt just prior to the placement of concrete.

### **47.2.7 Prestressed Concrete Cylinder Piles**

As an alternative to steel tubular piling, prestressed concrete cylinder piles have been used for a number of major overwater bridges, from the San Diego–Coronado and Dunbarton Bridges in California to bridges across Chesapeake Bay and the Yokohama cable-stayed bridge (Figures 47.6 and 47.7). Diameters have ranged from 1.5 to 6 m and more. They offer the advantage of durability and high axial compressive capacity. To counter several factors producing circumferential strains, especially thermal strains, spiral reinforcement of adequate cross-sectional area is required. This spiral reinforcement should be closely spaced in the 2-m zone just below the pile cap, where sharp reverse bending occurs under lateral loading.

Pile installation methods vary from driving and jetting of the smaller-diameter piles to drilling in the large-diameter piling (Figure 47.8).

### **47.2.8 Footing Blocks**

The footing block constructed at the top of large-diameter tubular piles serves the purpose of transmitting the forces from the pier shaft to the piles. Hence, it is subjected to large shears and significant moments. The shears require extensive vertical reinforcement, for both global shear (from the pier shaft) and local shear (punching shear from the piles). Large concentrations of reinforcement are required to distribute the moments. Post-tensioned tendons may be effectively utilized.



**FIGURE 47.6** Large-diameter prestressed concrete pile, Napa River Bridge, California.



**FIGURE 47.7** Prestressed concrete cylinder pile for Oosterschelde Bridge, the Netherlands.

Although the primary forces typically produce compression in the upper surface of the footing block, secondary forces and particularly high temporary stresses caused by the heat of hydration produce tension in the top surface. Thus, adequate horizontal steel must be provided in the top and bottom in both directions.



**FIGURE 47.8** Installing concrete cylinder pile by internal excavation, jetting, and pull-down force from barge.

The heat of hydration of the cementitious materials in a large footing block develops over a period of several days. Due to the mass of the block, the heat in the core may not dissipate and return to ambient for several weeks.

The outside surface meantime has cooled and contracted, producing tension which often leads to cracking. Where inadequate reinforcement is provided, the steel may stretch beyond yield, so that the cracks become permanent. If proper amounts of reinforcement are provided, then the cracking that develops will be well distributed, individual cracks will remain small, and the elastic stress in the reinforcement will tend to close the cracks as the core cools.

Internal laminar cracking may also occur, so vertical reinforcement and middepth reinforcement should also be considered.

Footing blocks may be constructed in place, just above water, with precast concrete skirts extending down below low water in order to prevent small boats and debris from being trapped below. In this case, the top of the piles may be exposed at low water, requiring special attention to the prevention of corrosion.

Footing blocks may be constructed below water. Although cofferdams may be employed, the most efficient and economical way is usually to prefabricate the shell of the footing block. This is then floated into place. Corner piles are then inserted through the structure and driven to grade. The prefabricated box is then lowered down by ballasting, supported and guided by the corner piles. Then the remaining piles are threaded through holes in the box and driven. Final connections are made by tremie concrete.

Obviously, there are variations of the above procedure. In some cases, portions of the box have been kept permanently empty, utilizing their buoyancy to offset part of the deadweight.

Transfer of forces into the footing block requires careful detailing. It is usually quite difficult to transfer full moment by means of reinforcing inside the pile shell. If the pile head can be dewatered,



**FIGURE 47.9** Large steel sheet pile cofferdam for Second Delaware Memorial Bridge, showing bracing frames.

reinforcing steel bars can be welded to the inside of the shell. Cages set in the concrete plug at the head may employ bundled bars with mechanical heads at their top. Alternatively the pile may be extended up through the footing block. Shear keys can be used to transfer shear. Post-tensioning tendons may run through and around the pile head.

## 47.3 Cofferdams for Bridge Piers

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### 47.3.1 Description

The word *cofferdam* is a very broad term to describe a construction that enables an underwater site to be dewatered. As such, cofferdams can be large or small. Medium-sized cofferdams of horizontal dimensions from 10 to 50 m have been widely used to construct the foundations of bridge piers in water and soft sediments up to 20 m in depth; a few have been larger and deeper (Figure 47.9). Typical bridge pier cofferdams are constructed of steel sheet piles supported against the external pressures by internal bracing.

A few very large bridge piers, such as anchorages for suspension bridges, have utilized a ring of self-supporting sheet pile cells. The interior is then dewatered and excavated to the required depth. A recent such development has been the building of a circular ring wall of concrete constructed by the slurry trench method (Figures 47.10 and 47.11). Concrete cofferdams have also used a ring wall of precast concrete sheet piles or even cribs.

### 47.3.2 Design Requirements

Cofferdams must be designed to resist the external pressures of water and soil [3]. If, as is usual, a portion of the external pressures is designed to be resisted by the internal passive pressure of the soil, the depth of penetration must be selected conservatively, taking into account a potential sudden reduction in passive pressure due to water flow beneath the tip as a result of unbalanced water pressures or jetting of piles. The cofferdam structure itself must have adequate vertical support for self-load and equipment under all conditions.

In addition to the primary design loads, other loading conditions and scenarios include current and waves, debris and ice, overtopping by high tides, flood, or storm surge. While earthquake-induced loads, acting on the hydrodynamic mass, have generally been neglected in the past, they



**FIGURE 47.10** Slurry wall cofferdam for Kawasaki Island ventilation shaft, Trans-Tokyo Bay Tunnels and Bridge.



**FIGURE 47.11** Concrete ring wall cofferdam constructed by slurry trench methods.

are now often being considered on major cofferdams, taking into account the lower input accelerations appropriate for the reduced time of exposure and, where appropriate, the reduced consequences.

Operating loads due to the mooring of barges and other floating equipment alongside need to be considered. The potential for scour must be evaluated, along with appropriate measures to reduce the scour. When the cofferdam is located on a sloping bank, the unbalanced soil loads need to be properly resisted. Accidental loads include impact from boats and barges, especially those working around the site.



**FIGURE 47.12** Dewatering the cofferdam for the main tower pier, Second Delaware Memorial Bridge.

The cofferdam as a whole must be adequately supported against the lateral forces of current waves, ice, and moored equipment, as well as unbalanced soil loads. While a large deep-water cofferdam appears to be a rugged structure, when fully excavated and prior to placement of the tremie concrete seal, it may be too weak to resist global lateral forces. Large tubular piles, acting as spuds in conjunction with the space-frame or batter piles may be needed to provide stability.

The cofferdam design must be such as to integrate the piling and footing block properly. For example, sheet piles may prevent the installation of batter piles around the periphery. To achieve adequate penetration of the sheet piles and to accommodate the batter piles, the cofferdam may need to be enlarged. The arrangement of the bracing should facilitate any subsequent pile installation.

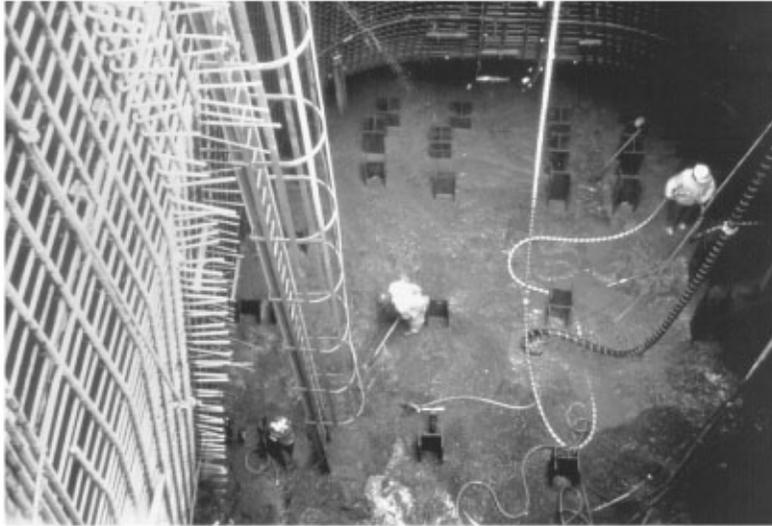
To enable dewatering of the cofferdam (Figure 47.12), a concrete seal is constructed, usually by the tremie method. This seal is designed to resist the hydrostatic pressure by its own buoyant weight and by uplift resistance provided by the piling, this latter being transferred to the concrete seal course by shear (Figure 47.13).

In shallow cofferdams, a filter layer of coarse sand and rock may permit pumping without a seal. However, in most cases, a concrete seal is required. In some recent construction, a reinforced concrete footing block is designed to be constructed underwater, to eliminate the need for a separate concrete seal. In a few cases, a drainage course of stone is placed below the concrete seal; it is then kept dewatered to reduce the uplift pressure. Emergency relief pipes through the seal course will prevent structural failure of the seal in case the dewatering system fails.

The underwater lateral pressure of the fresh concrete in the seal course and footing block must be resisted by external backfill against the sheet piles or by internal ties.

### **47.3.3 Internally Braced Cofferdams**

These are the predominant type of cofferdams. They are usually rectangular in shape, to accommodate a regular pattern of cross-lot bracing.



**FIGURE 47.13** Pumped-out cofferdam showing tremie concrete seal and predriven steel H-piles.

The external wall is composed of steel sheet piles of appropriate section modulus to develop bending resistance. The loading is then distributed by horizontal wales to cross-lot struts. These struts should be laid out on a plan which will permit excavation between them, to facilitate the driving of piling and to eliminate, as far as practicable, penetration of bracing through the permanent structure.

Wales are continuous beams, loaded by the uniform bearing of sheet piles against them. They are also loaded axially in compression when they serve as a strut to resist the lateral loads acting on them end-wise. Wales in turn deliver their normal loads to the struts, developing concentrated local bearing loads superimposed upon the high bending moments, tending to produce local buckling. Stiffeners are generally required.

While stiffeners are readily installed on the upperside, they are difficult to install on the underside and difficult to inspect. Hence, these stiffeners should be pre-installed during fabrication of the members.

The wales are restrained from global buckling in the horizontal plane by the struts. In the vertical plane they are restrained by the friction of the sheet piles, which may need to be supplemented by direct fixation. Blocking of timber or steel shims is installed between the wales and sheet piles to fit the irregularities in sheet pile installation and to fill in the needed physical clearances.

Struts are horizontal columns, subject to high axial loading, as well as vertical loads from self-weight and any equipment that is supported by them. Their critical concern is stability against buckling. This is countered in the horizontal plane by intersecting struts but usually needs additional support in the vertical plane, either by piling or by trussing two or more levels of bracing.

The orthogonal horizontal bracing may be all at one elevation, in which case the intersections of the struts have to be accommodated, or they may be vertically offset, one level resting on top of the other. This last is normally easier since, otherwise, the intersections must be detailed to transmit the full loads across the joint. This is particularly difficult if struts are made of tubular pipe sections. If struts are made of wide-flanged or H-section members, then it will usually be found preferable to construct them with the weak axis in the vertical plane, facilitating the detailing of strut-to-strut intersections as well as strut-to-wale intersections. In any event, stiffeners are required to prevent buckling of the flanges.

For deep-water piers, the cofferdam bracing is best constructed as a space-frame, with two or more levels joined together by posts and diagonals in the vertical plane. This space-frame may be completely prefabricated and set as a unit, supported by vertical piles. These supporting piles are

typically of large-diameter tubular members, driven through sleeves in the bracing frame and connected to it by blocking and welding.

The setting of such a space-frame requires a very large crane barge or equivalent, with both adequate hoisting capacity and reach. Sometimes, therefore, the bracing frame is made buoyant, to be partially or wholly self-floating. Tubular struts can be kept empty and supplemental buoyancy can be provided by pontoons.

Another way to construct the bracing frame is to erect one level at a time, supported by large tubular piles in sleeves. The lower level is first erected, then the posts and diagonal bracing in the vertical plane. The lower level is then lowered by hoists or jacks so that the second level can be constructed just above water and connections made in the dry.

A third way is to float in the prefabricated bracing frame on a barge, drive spud piles through sleeves at the four corners, and hang the bracing frame from the piles. Then the barge is floated out at low tide and the bracing frame lowered to position.

#### **47.3.4 Circular Cofferdams**

Circular cofferdams are also employed, with ring wales to resist the lateral forces in compression. The dimensions are large, and the ring compression is high. Unequal loading is frequently due to differential soil pressures. Bending moments are very critical, since they add to the compression on one side. Thus the ring bracing must have substantial strength against buckling in the horizontal plane.

#### **47.3.5 Excavation**

Excavation should be carried out in advance of setting the bracing frame or sheet piles, whenever practicable. Although due to side slopes the total volume of excavation will be substantially increased, the work can be carried out more efficiently and rapidly than excavation within a bracing system.

When open-cut excavation is not practicable, then it must be carried out by working through the bracing with a clamshell bucket. Struts should be spaced as widely as possible so as to permit use of a large bucket. Care must be taken to prevent impact with the bracing while the bucket is being lowered and from snagging the bracing from underneath while the bucket is being hoisted. These accidental loads may be largely prevented by temporarily standing up sheet piles against the bracing in the well being excavated, to act as guides for the bucket.

Except when the footing course will be constructed directly on a hard stratum or rock, overexcavation by 1 m or so will usually be found beneficial. Then the overexcavation can be backfilled to grade by crushed rock.

#### **47.3.6 Driving of Piles in Cofferdams**

Pilings can be driven before the bracing frame and sheet piles are set. They can be driven by underwater hammers or followers. To ensure proper location, the pile driver be equipped with telescopic leads, or a template be set on the excavated river bottom or seafloor.

Piling may alternatively be driven after the cofferdam has been installed, using the bracing frame as a template. In this case, an underwater hammer presents problems of clearance due to its large size, especially for batter piles. Followers may be used, or, often, more efficiently, the piles may be lengthened by splicing to temporarily extend all the way to above water. They are then cut off to grade after the cofferdam has been dewatered. This procedure obviates the problems occasioned if a pile fails to develop proper bearing since underwater splices are not needed. It also eliminates cutoff waste. The long sections of piling cutoff after dewatering can be taken back to the fabrication yard and re-spliced for use on a subsequent pier.

All the above assumes driven steel piling, which is the prevalent type. However, on several recent projects, drilled shafts have been constructed after the cofferdam has been excavated. In the latter case, a casing must be provided, seated sufficiently deep into the bottom soil to prevent run-in or blowout [see Section 47.2.6, item (4)].

Driven timber or concrete piles may also be employed, typically using a follower to drive them below water.

### 47.3.7 Tremie Concrete Seal

The tremie concrete seal course functions to resist the hydrostatic uplift forces to permit dewatering. As described earlier, it usually is locked to the foundation piling to anchor the slab. It may be reinforced in order to enable it to distribute the pile loads and to resist cracking due to heat of hydration.

*Tremie concrete* is a term derived from the French to designate concrete placed through a pipe. The term has subsequently evolved to incorporate both a concrete mix and a placement procedure. Underwater concreting has had both significant successes and significant failures. Yet the system is inherently reliable and concrete equal or better than concrete placed in the dry has been produced at depths up to 250 m. The failures have led to large cost overruns due to required corrective action. They have largely been due to inadvertently allowing the concrete to flow through or be mixed with the water, which has caused washout of the cement and segregation of the aggregate.

Partial washout of cement leads to the formation of a surface layer of laitance which is a weak paste. This may harden after a period of time into a brittle chalklike substance.

The tremie concrete mix must have an adequate quantity of cementitious materials. These can be a mixture of portland cement with either fly ash or blast furnace slag (BFS). These are typically proportioned so as to reduce the heat of hydration and to promote cohesiveness. A total content of cementitious materials of 400 kg/m<sup>3</sup> (~700 lb/cy) is appropriate for most cases.

Aggregates are preferably rounded gravel so they flow more readily. However, crushed coarse aggregates may be used if an adequate content of sand is provided. The gradation of the combined aggregates should be heavy toward the sand portion — a 45% sand content appears optimum for proper flow. The maximum size of coarse aggregate should be kept small enough to flow smoothly through the tremie pipe and any restrictions such as those caused by reinforcement. Use of 20 mm maximum size of coarse aggregate appears optimum for most bridge piers.

A conventional water-reducing agent should be employed to keep the water/cementitious material ratio below 0.45. Superplasticizers should not normally be employed for the typical cofferdam, since the workability and flowability may be lost prematurely due to the heat generated in the mass concrete. Retarders are essential to prolong the workable life of the fresh mix if superplasticizers are used.

Other admixtures are often employed. Air entrainment improves flowability at shallow water depths but the beneficial effects are reduced at greater depths due to the increased external pressure. Weight to reduce uplift is also lost.

Microsilica may be included in amounts up to 6% of the cement to increase the cohesiveness of the mix, thus minimizing segregation. It also reduces bleed. Antiwashout admixtures (AWA) are also employed to minimize washout of cementitious materials and segregation. They tend to promote self-leveling and flowability. Both microsilica and AWA may require the use of superplasticizers in which case retarders are essential. However, a combination of silica fume and AWA should be avoided as it typically is too sticky and does not flow well.

Heat of hydration is a significant problem with the concrete seal course, as well as with the footing block, due to the mass of concrete. Therefore, the concrete mix is often precooled, e.g., by chilling of the water or the use of ice. Liquid nitrogen is sometimes employed to reduce the temperature of the concrete mix to as low as 5°C. Heat of hydration may be reduced by incorporating substantial amounts of fly ash to replace an equal portion of cement. BFS–cement can also be used to reduce



**FIGURE 47.14** Placing underwater concrete through hopper and tremie pipe, Verrazano Narrows Bridge, New York.

heat, provided the BFS is not ground too fine, i.e., not finer than  $2500 \text{ cm}^2/\text{g}$  and the proportion of slag is at least 70% of the total.

The tremie concrete mix may be delivered to the placement pipe by any of several means. Pumping and conveyor belts are best because of their relatively continuous flow. The pipe for pumping should be precooled and insulated or shielded from the sun; conveyor belts should be shielded. Another means of delivery is by bucket. This should be air-operated to feed the concrete gradually to the hopper at the upper end of the tremie pipe. *Placement down the tremie pipe should be by gravity feed only* (Figure 47.14).

Although many placements of tremie concrete have been carried out by pumping, there have been serious problems in large placements such as cofferdam seals. The reasons include:

1. Segregation in the long down-leading pipe, partly due to formation of a partial vacuum and partly due to the high velocity;
2. The high pressures at discharge;
3. The surges of pumping.

Since the discharge is into fresh concrete, these phenomena lead to turbulence and promote intermixing with water at the surface, forming excessive laitance.

These discharge effects can be contrasted with the smooth flow from a gravity-fed pipe in which the height of the concrete inside the tremie pipe automatically adjusts to match the external pressure of water vs. the previously placed concrete. For piers at considerable depths, this balance point will be about half-way down. The pipe should have an adequate diameter in relation to the maximum

size of coarse aggregate to permit remixing: a ratio of 8 to 1 is the minimum. A slight inclination of the tremie pipe from the vertical will slow the feed of new concrete and facilitate the escape of entrapped air.

For starting the tremie concrete placement, the pipe must first be filled slightly above middepth. This is most easily done by plugging the end and placing the empty pipe on the bottom. The empty pipe must be negatively buoyant. It also must be able to withstand the external hydrostatic pressure as well as the internal pressure of the underwater concrete. Joints in the tremie pipe should be gasketed and bolted to prevent water being sucked into the mix by venturi action. To commence placement, with the tremie pipe slightly more than half full, it is raised 150 mm off the bottom. The temporary plug then comes off and the concrete flows out. The above procedure can be used both for starting and for resuming a placement, as, for example, when the tremie is relocated, or after a seal has been inadvertently lost.

The tremie pipe should be kept embedded in the fresh concrete mix a sufficient distance to provide backpressure on the flow (typically 1 m minimum), but not so deep as to become stuck in the concrete due to its initial set. This requires adjustment of the retarding admixture to match the rate of concrete placement and the area of the cofferdam against the time of set, keeping in mind the acceleration of set due to heat as the concrete hydrates.

Another means for initial start of a tremie concrete placement is to use a pig which is forced down the pipe by the weight of the concrete, expelling the water below. This pig should be round or cylindrical, preferably the latter, equipped with wipers to prevent leakage of grout and jamming by a piece of aggregate. An inflated ball, such as an athletic ball (volleyball or basketball) must never be used; these collapse at about 8 m water depth! A pig should not be used to restart a placement, since it would force a column of water into the fresh concrete previously placed.

Mixes of the tremie concrete described will flow outward on a slope of about 1 on 8 to 1 on 10. With AWAs, an even flatter surface can be obtained.

A trial batch with underwater placement in a shallow pit or tank should always be done before the actual placement of the concrete seal. This is to verify the cohesiveness and flowability of the mix. Laboratory tests are often inadequate and misleading, so a large-scale test is important. A trial batch of 2 to 3 m<sup>3</sup> has often been used.

The tremie concrete placement will exert outward pressure on the sheet piles, causing them to deflect. This may in turn allow new grout to run down past the already set concrete, increasing the external pressure. To offset this, the cofferdam can be partially backfilled before starting the tremie concreting and tied across the top. Alternatively, dowels can be welded on the sheets to tie into the concrete as it sets; the sheet piles then have to be left in place.

Due to the heat of hydration, the concrete seal will expand. Maximum temperature may not be achieved for several days. Cooling of the mass is gradual, starting from the outside, and ambient temperature may not be achieved for several weeks. Thus an external shell is cooling and shrinking while the interior is still hot. This can produce severe cracking, which, if not constrained, will create permanent fractures in the seal or footing. Therefore, in the best practice, reinforcing steel is placed in the seal to both provide a restraint against cracking and to help pull the cracks closed as the mass cools.

After a relatively few days, the concrete seal will usually have developed sufficient strength to permit dewatering. Once exposed to the air, especially in winter, the surface concrete will cool too fast and may crack. Placing insulation blankets will keep the temperature more uniform. They will, of course, have to be temporarily moved to permit the subsequent work to be performed.

### **47.3.8 Pier Footing Block**

The pier footing block is next constructed. Reinforcement on all faces is required, not only for structural response but also to counteract thermal strains. Reference is made to Section 31.2.8 in which reinforcement for the footing block is discussed in more detail.

The concrete expands as it is placed in the footing block due to the heat of hydration. At this stage it is either still fresh or, if set, has a very low modulus. Then it hardens and bonds to the tremie concrete below. The lock between the two concrete masses is made even more rigid if piling protrudes through the top of the tremie seal, which is common practice. Now the footing block cools and tries to shrink but is restrained by the previously placed concrete seal. Vertical cracks typically form. Only if there is sufficient bottom reinforcement in both directions can this shrinkage and cracking be adequately controlled. Note that these tensile stresses are permanently locked into the bottom of the footing block and the cracks will not close with time, although creep will be advantageous in reducing the residual stresses.

After the footing block has hardened, blocking may be placed between it and the sheet piles. This, in turn, may permit removal of the lower level of bracing. As an alternative to bracing, the footing block may be extended all the way to the sheet piles, using a sheet of plywood to prevent adhesion.

### 47.3.9 Pier Shaft

The pier shaft is then constructed. Block-outs may be required to allow the bracing to pass through. The internal bracing is removed in stages, taking care to ensure that this does not result in overloading a brace above. Each stage of removal should be evaluated.

Backfill is then placed outside the cofferdam to bring it up to the original seabed. The sheet piles can then be removed. The first sheets are typically difficult to break loose and may require driving or jacking in addition to vibration. Keeping in mind the advantage of steel sheet piles in preventing undermining of the pier due to scour, as well as the fact that removal of the sheets always loosens the surrounding soil, hence reducing the passive lateral resistance, it is often desirable to leave the sheet piles in place below the top of the footing. They may be cut off underwater by divers; then the tops are pulled by vibratory hammers.

Antiscour stone protection is now placed, with an adequate filter course or fabric sheet in the case of fine sediments.

## 47.4 Open Caissons

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### 47.4.1 Description

Open caissons have been employed for some of the largest and deepest bridge piers [4]. These are an extension of the “wells” which have been used for some 2000 years in India. The caisson may be constructed above its final site, supported on a temporary sand island, and then sunk by dredging out within the open wells of the caisson, the deadweight acting to force the caisson down through the overlying soils (Figure 47.15). Alternatively, especially in sites overlain by deep water, the caisson may be prefabricated in a construction basin, floated to the site by self-buoyancy, augmented as necessary by temporary floats or lifts, and then progressively lowered into the soils while building up the top.

Open caissons are effective but costly, due to the large quantity of material required and the labor for working at the overwater site. Historically, they have been the only means of penetrating thorough deep overlying soils onto a hard stratum or bedrock. However, their greatest problem is maintaining stability during the early phases of sinking, when they are neither afloat nor firmly embedded and supported. Long and narrow rectangular caissons are especially susceptible to tipping, whereas square and circular caissons of substantial dimensions relative to the water depth are inherently more stable. Once the caisson tips, it tends to drift off position. It is very difficult to bring it back to the vertical without overcorrecting.

When the caisson finally reaches its founding elevation, the surface of rock or hard stratum is cleaned and a thick tremie concrete base is placed. Then the top of the caisson is completed by casting a large capping block on which to build the pier shaft.



FIGURE 47.15 Open-caisson positioned within steel jackets on “pens.”

#### 47.4.2 Installation

The sinking of the cofferdam through the soil is resisted by skin friction along the outside and by bearing on the cutting edges. Approximate values of resistance may be obtained by multiplying the friction factor of sand on concrete or steel by the at-rest lateral force at that particular stage,  $f = \emptyset K_0 wh^2$  where  $f$  is the unit frictional resistance,  $\emptyset$  the coefficient of friction,  $w$  the underwater unit weight of sand,  $K_0$  the at-rest coefficient of lateral pressure, and  $h$  the depth of sand at that level.  $f$  is then summed up over the embedded depth. In clay, the cohesive shear controls the “skin friction.” The bearing value of the cutting edges is generally the “shallow bearing value,” i.e., five times the shear strength at that elevation.

These resistances must be overcome by deadweight of the caisson structure, reduced by the buoyancy acting on the submerged portions. This deadweight may be augmented by jacking forces on ground anchors.

The skin friction is usually reduced by lubricating jets causing upward flow of water along the sides. Compressed air may be alternated with water through the jets; bentonite slurry may be used to provide additional lubrication. The bearing on the cutting edges may be reduced by cutting jets built into the walls of the caisson or by free jets operating through holes formed in the walls. Finally, vibration of the soils near and around the caisson may help to reduce the frictional resistance.

When a prefabricated caisson is floated to the site, it must be moored and held in position while it is sunk to and into the seafloor. The moorings must resist current and wave forces and must assist in maintaining the caisson stable and in a vertical attitude. This latter is complicated by the need to build up the caisson walls progressively to give adequate freeboard, which, of course, raises the center of gravity.

Current force can be approximately determined by the formula

$$F = CA\rho \frac{V^2}{2g}$$

where  $C$  varies from 0.8 for smooth circular caissons to 1.3 for rectangular caissons,  $A$  is the area,  $\rho$  is the density of water, and  $V$  is the average current over the depth of flotation. Steel sheet piles develop high drag, raising the value of  $C$  by 20–30%.

As with all prismatic floating structures, stability requires that a positive metacentric height be maintained. The formula for metacentric height,  $\overline{GM}$  is

$$\overline{GM} = \overline{KB} - \overline{KG} + \overline{BM}$$

where  $\overline{KB}$  is the distance from the base to the center of buoyancy,  $\overline{KG}$  the distance to the center of gravity, and  $\overline{BM} = I/V$ .

$I$  is the moment of inertia on the narrowest (most sensitive) axis, while  $V$  is the displaced volume of water. For typical caissons, a  $\overline{GM}$  of +1 m or more should be maintained.

The forces from mooring lines and the friction forces from any dolphins affect the actual attitude that the structure assumes, often tending to tip it from vertical. When using mooring lines, the lines should be led through fairleads attached near the center of rotation of the structure. However, this location is constantly changing, so the fairlead attachment points may have to be shifted upward from time to time.

Dolphins and “pens” are used on many river caissons, since navigation considerations often preclude mooring lines. These are clusters of piles or small jackets with pin piles and are fitted with vertical rubbing strips on which the caisson slides.

Once the caisson has been properly moored on location, it is ballasted down. As it nears the existing river or harbor bottom, the current flow underneath increases dramatically. When the bottom consists of soft sediments, these may rapidly scour away in the current. To prevent this, a mattress should be first installed.

Fascine mattresses of willow, bamboo, or wood with filter fabric attached are ballasted down with rock. Alternatively, a layer of graded sand and gravel, similar to the combined mix for concrete aggregate, can be placed. The sand on top will scour away, but the final result will be a reverse filter.

In order to float a prefabricated caisson to the site initially, false bottoms are fitted over the bottom of the dredging wells. These false bottoms are today made of steel, although timber was used on many of the famous open caissons from the 19th and the early part of the 20th centuries. They are designed to resist the hydrostatic pressure plus the additional force of the soils during the early phases of penetration. Once the caisson is embedded sufficiently to ensure stability, the false bottoms are progressively removed so that excavation can be carried out through the open wells. This removal is a very critical and dangerous stage, hazardous both to the caisson and to personnel. The water level inside at this stage should be slightly higher than that outside. Even then, when the false bottom under a particular well is loosened, the soil may suddenly surge up, trapping a diver. The caisson, experiencing a sudden release of bearing under one well, may plunge or tip.

Despite many innovative schemes for remote removal of false bottoms, accidents have occurred. Today's caissons employ a method for gradually reducing the pressure underneath and excavating some of the soil before the false bottom is released and removed. For such constructions, the false bottom is of heavily braced steel, with a tube through it, typically extending to the water surface. The tube is kept full of water and capped, with a relief valve in the cap. After the caisson has penetrated under its own weight and come to a stop, the relief valve is opened, reducing the pressure to the hydrostatic head only. Then the cap is removed. This is done for several (typically, four) wells in a balanced pattern. Then jets and airlifts may be operated through the tube to remove the soil under those wells. When the caisson has penetrated sufficiently far for safety against tipping, the wells are filled with water; the false bottoms are removed and dredging can be commenced.

### 47.4.3 Penetration of Soils

The penetration is primarily accomplished by the net deadweight, that is, the total weight of concrete steel and ballast less the buoyancy. Excavation within the wells is carried down in a balanced pattern until the bearing stratum is reached. Then tremie concrete is placed, of sufficient depth to transfer the design bearing pressures to the walls.

The term *cutting edge* is applied to the tips of the caisson walls. The external cutting edges are shaped as a wedge while the interior ones may be either double-wedge or square. In the past, concern over concentrated local bearing forces led to the practice of making the cutting edges of heavy and expensive fabricated steel. Today, high-strength reinforced concrete is employed, although if obstructions such as boulders, cobbles, or buried logs are anticipated or if the caisson must penetrate rock, steel armor should be attached to prevent local spalling.

The upper part of the caisson may be replaced by a temporary cofferdam, allowing the pier shaft dimensions to be reduced through the water column. This reduces the effective driving force on the caisson but maintains and increases its inherent stability.

The penetration requires the progressive failure of the soil in bearing under the cutting edges and in shear along the sides. Frictional shear on the inside walls is reduced by dredging while that on the outside walls is reduced by lubrication, using jets as previously described.

Controlling the penetration is an essentially delicate balancing of these forces, attempting to obtain a slight preponderance of sinking force. Too great an excess may result in plunging of the caisson and tipping or sliding sidewise out of position. That is why pumping down the water within the caisson, thus reducing buoyancy, is dangerous; it often leads to sudden inflow of water and soil under one edge, with potentially catastrophic consequences.

Lubricating jets may be operated in groups to limit the total volume of water required at any one time to a practicable pump capacity. In addition to water, bentonite may be injected through the lubricating jets, reducing the skin friction. Compressed air may be alternated with water jetting.

Other methods of aiding sinking are employed. Vibration may be useful in sinking the caisson through sands, especially when it is accompanied by jetting. This vibration may be imparted by intense vibration of a steel pile located inside the caisson or even by driving on it with an impact hammer to liquefy the sands locally.

Ground anchors inserted through preformed holes in the caisson walls may be jacked against the caisson to increase the downward force. They have the advantage that the actual penetration may be readily controlled, both regarding force exerted and displacement.

Since all the parameters of resistance and of driving force vary as the caisson penetrates the soil, and because the imbalance is very sensitive to relatively minor changes in these parameters, it is essential to plan the sinking process in closely spaced stages, typically each 2 to 3 m. Values can be precalculated for each such stage, using the values of the soil parameters, the changes in contact areas between soil and structure, the weights of concrete and steel and the displaced volume. These need not be exact calculations; the soil parameters are estimates only since they are being constantly modified by the jetting. However, they are valuable guides to engineering control of the operations.

There are many warnings from the writings of engineers in the past, often based on near-failures or actual catastrophes.

1. Verify structural strength during the stages of floating and initial penetration, with consideration for potentially high resistance under one corner or edge.
2. In removing false bottoms, be sure the excess pressure underneath has first been relieved.
3. Do not excavate below cutting edges.
4. Check outside soundings continually for evidence of scour and take corrective steps promptly.
5. Blasting underneath the cutting edges may blow out the caisson walls. Blasting may also cause liquefaction of the soils leading to loss of frictional resistance and sudden plunging. If blasting is needed, do it before starting penetration or, at least, well before the cutting edge reaches the hard strata so that a deep cushion of soil remains over the charges.
6. If the caisson tips, avoid drastic corrections. Instead, plan the correction to ensure a gradual return to vertical and to prevent the possibility of tipping over more seriously on the other side. Thus steps such as digging deeper on the high side and overballasting on the high side should be last resorts and, then, some means mobilized to arrest the rotation as the caisson nears vertical. Jacking against an external dolphin is a safer and more efficient method for correcting the tipping ([Figure 47.16](#)).



**FIGURE 47.16** Open caisson for Sunshine Bridge across the Mississippi River. This caisson tipped during removal of false bottoms and is shown being righted by jacking against dolphins.

Sinking the caisson should be a continuous process, since once it stops the soil friction and/or shear increase significantly and it may be difficult to restart the caisson's descent.

#### **47.4.4 Founding on Rock**

Caissons founded on bare rock present special difficulties. The rock may be leveled by drilling, blasting, and excavation, although the blasting introduces the probability of fractures in the underlying rock. Mechanical excavation may therefore be specified for the last meter or two. Rotary drills and underwater road-headers can be used but the process is long and costly. In some cases, hydraulic rock breakers can be employed; in other cases a hammer grab or star chisel may be used. For the Prince Edward Island Bridge, the soft rock was excavated and leveled by a very heavy clamshell bucket. Hydraulic backhoes and dipper dredges have been used elsewhere. A powerful cutter-head dredge has been planned for use at the Strait of Gibraltar.

#### **47.4.5 Contingencies**

The planning should include methods for dealing with contingencies. The resistance of the soil and especially of hard strata may be greater than anticipated. Obstructions include sunken logs and even sunken buried barges and small vessels, as well as cobbles and boulders. The founding rock or stratum may be irregular, requiring special means of excavating underneath the cutting edge at high spots or filling in with concrete in the low spots. One contingency that should always be addressed is what steps to take if the caisson unexpectedly tips.

Several innovative solutions have been used to construct caissons at sites with especially soft sediments. One is a double-walled self-floating concept, without the need for false bottoms. Double-walled caissons of steel were used for the Mackinac Strait Bridge in Michigan. Ballast is progressively filled into the double-wall space while dredging is carried out in the open wells.

In the case of extremely soft bottom sediments, the bottom may be initially stabilized by ground improvement, for example, with surcharge by dumped sand, or by stone columns, so that the caisson may initially land on and penetrate stable soil. Great care must, of course, be exercised to maintain control when the cutting edge breaks through to the native soils below, preventing erratic plunging.



**FIGURE 47.17** Excavating within pressurized working chamber of pneumatic caisson for Rainbow Suspension Bridge, Tokyo. (Photo courtesy of Shiraishi Corporation.)

This same principle holds true for construction on sand islands, where the cutting edge and initial lifts of the caisson may be constructed on a stratum of gravel or other stable material, then the caisson sunk through to softer strata below. Guides or ground anchors will be of benefit in controlling the sinking operation.

## 47.5 Pneumatic Caissons

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### 47.5.1 Description

These caissons differ from the open caisson in that excavation is carried out beneath the base in a chamber under air pressure. The air pressure is sufficient to offset some portion of the ambient hydrostatic head at that depth, thus restricting the inflow of water and soil.

Access through the deck for workers and equipment and for the removal of the excavated soil is through an airlock. Personnel working under air pressure have to follow rigid regimes regarding duration and must undergo decompression upon exit. The maximum pressures and time of exposure under which personnel can work is limited by regulations. Many of the piers for the historic bridges in the United States, e.g., the Brooklyn Bridge, were constructed by this method.

### 47.5.2 Robotic Excavation

To overcome the problems associated with working under air pressure, the health hazards of “caisson disease,” and the high costs involved, robotic cutters [5] have been developed to excavate and remove the soil within the chamber without human intervention. These were recently implemented on the piers of the Rainbow Suspension Bridge in Tokyo (Figures 47.17 and 47.18).

The advantage of the pneumatic caisson is that it makes it possible to excavate beneath the cutting edges, which is of special value if obstructions are encountered. The great risk is of a “blowout” in which the air escapes under one edge, causing a rapid reduction of pressure, followed by an inflow of water and soil, endangering personnel and leading to sudden tilting of the caisson. Thus, the use of pneumatic caissons is limited to very special circumstances.

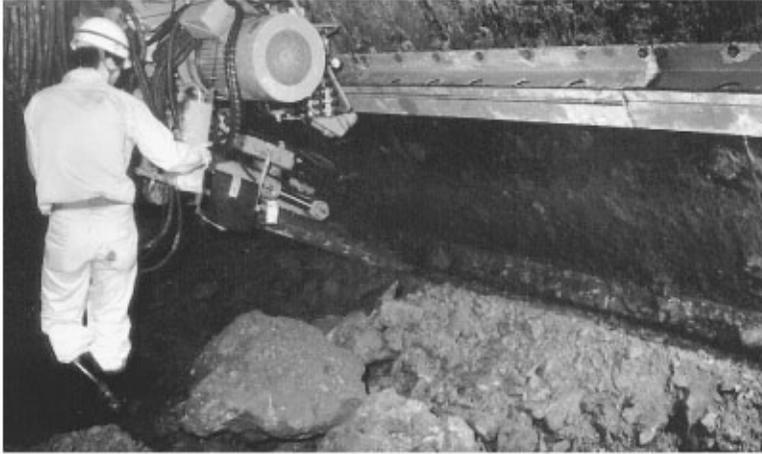


FIGURE 47.18 Excavating beneath cutting edge of pneumatic caisson. (Photo courtesy of Shiraishi Corporation.)

## 47.6 Box Caissons

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### 47.6.1 Description

One of the most important developments of recent years has been the use of box caissons, either floated in or set in place by heavy-lift crane barges [4]. These box caissons, ranging in size from a few hundred tons to many thousands of tons, enable prefabrication at a shore site, followed by transport and installation during favorable weather “windows” and with minimum requirements for overwater labor. The development of these has been largely responsible for the rapid completion of many long overwater bridges, cutting the overall time by a factor of as much as three and thus making many of these large projects economically viable.

The box caisson is essentially a structural shell that is placed on a prepared underwater foundation. It is then filled with concrete, placed by the tremie method previously described for cofferdam seals. Alternatively, sand fill or just ballast water may be used.

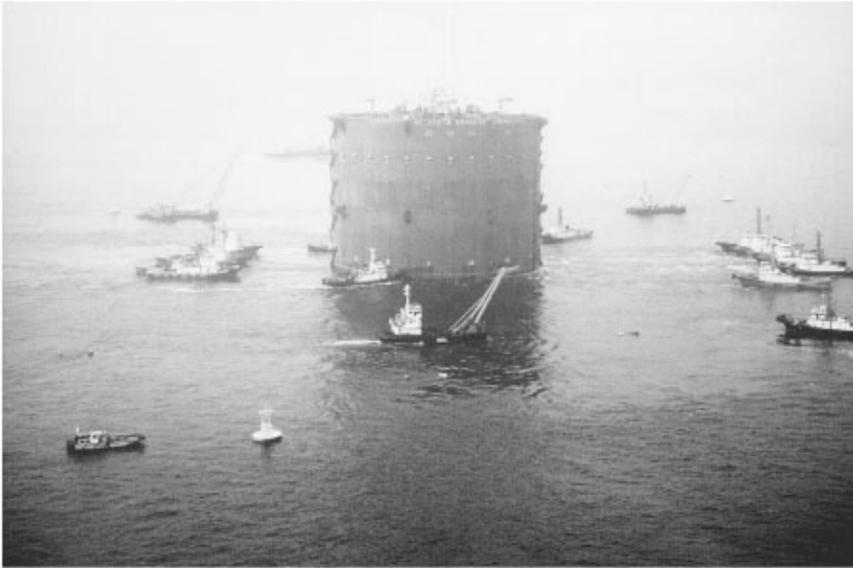
Although many box caissons are prismatic in shape, i.e., a large rectangular base supporting a smaller rectangular column, others are complex shells such as cones and bells. When the box caisson is seated on a firm foundation, it may be underlain by a meter or two of stone bed, consisting of densified crushed rock or gravel that has been leveled by screeding. After the box has been set, underbase grout is often injected to ensure uniform bearing.

### 47.6.2 Construction

The box caisson shell is usually the principal structural element although it may be supplemented by reinforcing steel cages embedded in tremie concrete. This latter system is often employed when joining a prefabricated pier shaft on top of a previously set box caisson.

Box caissons may be prefabricated of steel; these were extensively used on the Honshu–Shikoku Bridges in Japan (Figures 47.19 and 47.20). After setting, they were filled with underwater concrete, in earlier cases using grout-intruded aggregate, but in more recent cases tremie concrete.

For reasons of economy and durability, most box caissons are made of reinforced concrete. Although they are therefore heavier, the concurrent development of very large capacity crane barges and equipment has made their use fully practicable. The weight is advantageous in providing stability in high currents and waves.



**FIGURE 47.19** Steel box caisson being positioned for Akashi Strait Bridge, Japan, despite strong currents.



**FIGURE 47.20** Akashi Strait Suspension Bridge is founded on steel box caissons filled with tremie concrete.

### 47.6.3 Prefabrication Concepts

Prefabrication of the box caissons may be carried out in a number of interesting ways. The caissons(s) may be constructed on the deck of a large submersible barge. In the case of the two concrete caissons for the Tsing Ma Bridge in Hong Kong, the barge then moved to a site where it could submerge to launch the caissons. They were floated to the site and ballasted down onto the predredged rock base. After sealing the perimeter of the cutting edge, they were filled with tremie concrete.

In the case of the 66 piers for the Great Belt Western Bridge in Denmark, the box caissons were prefabricated on shore in an assembly-line process. (Figures 47.21 and 47.22). They were progressively moved out onto a pier from which they could be lifted off and carried by a very large crane barge to their site. They were then set onto the prepared base. Finally, they were filled with sand and antiscour stone was placed around their base.



**FIGURE 47.21** Prefabrication of box caisson piers for Great Belt Western Bridge, Denmark.



**FIGURE 47.22** Prefabricated concrete box caissons are moved by jacks onto pier for load-out.

A similar procedure has been followed for the approach piers on the Oresund Bridge between Sweden and Denmark and on the Second Severn Bridge in Southwest England. For the Great Belt Eastern Bridge, many of the concrete box caissons were prefabricated in a construction basin (Figure 47.23). Others were fabricated on a quay wall.

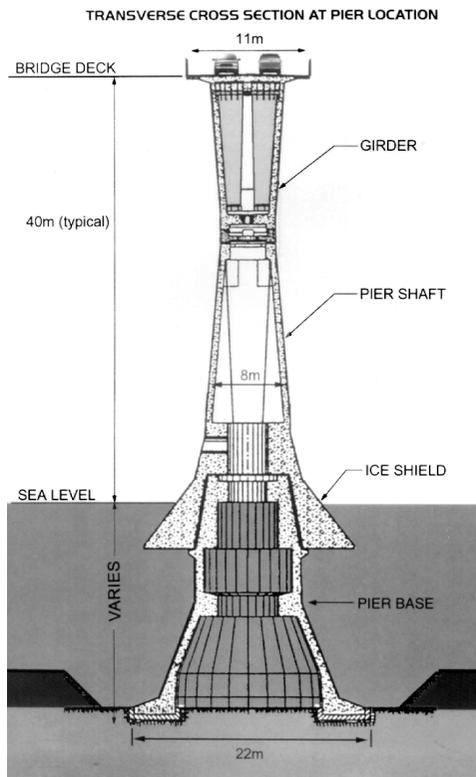
For the Prince Edward Island Bridge, bell-shaped piers, with open bottom, weighing up to 8000 tons, were similarly prefabricated on land and transported to the load-out pier and onto a barge, using transporters running on pile-supported concrete beams (Figures 47.24 through 47.26). Meanwhile, a shallow trench had been excavated in the rock seafloor, in order to receive the lower end of the bell. The bell-shaped shell was then lowered into place by the large crane barge. Tremie concrete was placed to fill the peripheral gap between bell and rock.

#### **47.6.4 Installation of Box Caissons by Flotation**

Large concrete box caissons have been floated into location, moored, and ballasted down onto the prepared base (Figure 47.27). During this submergence, they are, of course, subject to current, wave, and wind forces. The moorings must be sufficient to control the location; “taut moorings” are therefore used for close positioning.



**FIGURE 47.23** Large concrete box caissons fabricated in construction basin for subsequent deployment to site by self-flotation, Great Belt Eastern Bridge, Denmark.



**FIGURE 47.24** Schematic representation of substructure for Prince Edward Island Bridge, Canada. Note ice shield, designed to reduce forces from floating ice in Northumberland Strait.

The taut moorings should be led through fairleads on the sides of the caisson, in order to permit lateral adjustment of position without causing tilt. In some cases where navigation requirements prevent the use of taut moorings, dolphins may be used instead. These can be faced with a vertical rubbing strip or master pile. Tolerances must be provided in order to prevent binding.



FIGURE 47.25 Prefabrication of pier bases, Prince Edward Island Bridge, Canada.



FIGURE 47.26 Prefabricated pier shaft and icebreaker, Prince Edward Island Bridge, Canada.

Stability is of critical importance for box caissons which are configured such that the water plane diminishes as they are submerged. It is necessary to calculate the metacentric height,  $\overline{GM}$ , at every change in horizontal cross section as it crosses the water plane, just as previously described for open caissons.

During landing, as during the similar operation with open caissons, the current under the caisson increases, and scour must be considered. Fortunately, in the case of box caissons, they are being



**FIGURE 47.27** Prefabricated box caisson is floated into position, Great Belt Eastern Bridge, Denmark. Note temporary cofferdam above concrete caisson.

landed either directly on a leveled hard stratum or on a prepared bed of densified stone, for which scour is less likely.

As the base of the caisson approaches contact, the prism of water trapped underneath has to escape. This will typically occur in a random direction. The reaction thrust of the massive water jet will push the caisson to one side. This phenomenon can be minimized by lowering the last meter slowly.

Corrections for the two phenomena of current scour and water-jet thrust are in opposition to one another, since lowering slowly increases the duration of exposure to scour. Thus it is essential to size and compact the stone of the stone bed properly and also to pick a time of low current, e.g., slack tide for installation.

### **47.6.5 Installing Box Caissons by Direct Lift**

In recent years, very heavy lift equipment has become available. Jack-up, floating crane barges, and catamaran barges, have all been utilized (Figures 47.28 through 47.31). Lifts up to 8000 tons have been made by crane barge on the Great Belt and Prince Edward Island Bridges.

The box caissons are then set on the prepared bed. Where it is impracticable to screed a stone bed accurately, landing seats may be preset to exact grade under water and the caisson landed on them, and tremie concrete filled in underneath.

Heavy segments, such as box caissons, are little affected by current — hence can be accurately set to near-exact location in plan. Tolerances of the order of 20 to 30 mm are attainable.

### **47.6.6 Positioning**

Electronic distance finders (EDF), theodolites, lasers, and GPS are among the devices utilized to control the location and grade. Seabed and stone bed surveys may be by narrow-beam high-frequency sonar and side-scan sonar. At greater depths, the sonar devices may be incorporated in an ROV to get the best definition.

### **47.6.7 Grouting**

Grouting or concreting underneath is commonly employed to ensure full bearing. It is desirable to use low-strength, low-modulus grout to avoid hard spots. The edges of the caisson have to be sealed by penetrating skirts or by flexible curtains which can be lowered after the caisson is set in place,



**FIGURE 47.28** Installing precast concrete box caissons for Second Severn Crossing, Bristol, England. Extreme tidal range of 10 m and high tidal current imposed severe demands on installation procedures and equipment.



**FIGURE 47.29** Lifting box caisson from quay wall on which it was prefabricated and transporting it to site while suspended from crane barge.

since otherwise the tremie concrete will escape, especially if there is a current. Heavy canvas or submerged nylon, weighted with anchor chain and tucked into folds, can be secured to the caisson during prefabrication. When the caisson is finally seated, the curtains can be cut loose; they will restrain concrete or grout at low flow pressures. Backfill of stone around the edges can also be used to retain the concrete or grout.

Heat of hydration is also of concern, so the mix should not contain excessive cement. The offshore industry has developed a number of low-heat, low modulus, thixotropic mixes suitable for this use. Some of them employ seawater, along with cement, fly ash, and foaming agents. BFS cement has also been employed.

Box caissons may be constituted of two or more large segments, set one on top of the other and joined by overlapping reinforcement encased in tremie concrete. The segments often are match-cast to ensure perfect fit.



**FIGURE 47.30** Setting prefabricated box caisson on which is mounted a temporary cofferdam, Great Belt Eastern Bridge, Denmark.



**FIGURE 47.31** Setting 7000-ton prefabricated box caisson, Great Belt Western Bridge, Denmark.

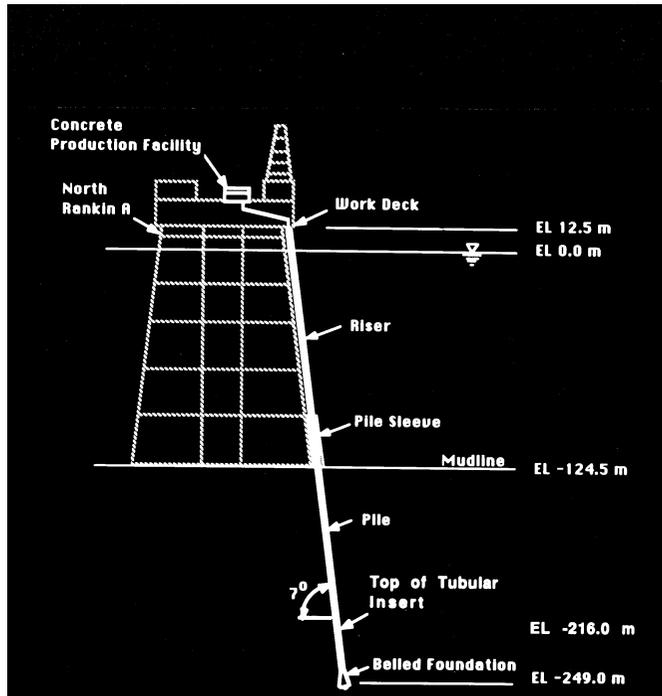
## 47.7 Present and Future Trends

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### 47.7.1 Present Practice

There is a strong incentive today to use large prefabricated units, either steel or concrete, that can be rapidly installed with large equipment, involving minimal on-site labor to complete. On-site operations, where required, should be simple and suitable for continuous operation. Filling prefabricated shells with tremie concrete is one such example.

Two of the concepts previously described satisfy these current needs. The first, a box caisson — a large prefabricated concrete or steel section — can be floated in or lifted into position on a hard seafloor. The second, large-diameter steel tubular piles, can be driven through soft and variable soils to be founded in a competent stratum, either rock or dense soils. These tubular piles are especially suitable for areas of high seismicity, where their flexibility and ductility can be exploited to reduce the acceleration transmitted to the superstructure (Figure 47.32). In very deep water, steel-framed



**FIGURE 47.32** Conceptual design for deep-water bridge pier, utilizing prefabricated steel jacket and steel tubular pin piles.

jackets may be employed to support the piles through the water column (Figure 47.33). The box caisson, conversely, is most suitable to resist the impact forces from ship collision. The expanding use of these two concepts is leading to further incremental improvements and adaptations which will increase their efficiency and economy.

Meanwhile cofferdams and open caissons will continue to play an important but diminishing role. Conventional steel sheet pile cofferdams are well suited to shallow water with weak sediments, but involve substantial overwater construction operations.

### 47.7.2 Deep Water Concepts

The Japanese have had a study group investigating concepts for bridge piers in very deep water, and soft soils. One initial concept that has been pursued is that of the circular cofferdam constructed of concrete by the slurry wall process [3]. This was employed on the Kobe anchorage for the Akashi Strait Bridge and on the Kawasaki Island Ventilation Structure for the Trans-Tokyo Bay tunnels, the latter with a pumped-out head of 80 m, in extremely soft soils in a zone of high seismicity (see Section 47.3.1).

Floating piers have been proposed for very deep water, some employing semisubmersible and tension “leg-platform” concepts from the offshore industry. While technically feasible, the entire range of potential adverse loadings, including accidental flooding, ship impact, and long-period swells, need to be thoroughly considered. Tethered pontoons of prestressed concrete have been successfully used to support a low-level bridge across a fjord in Norway.

Most spectacular of all proposed bridge piers are those designed in preliminary feasibility studies for the crossing of the Strait of Gibraltar. Water depths range from 305 m for a western crossing to 470 m for a shorter eastern crossing. Seafloor soils are highly irregular and consist of relatively weak sandstone locally known as flysch. Currents are strong and variable. Wave and swell exposure is significant. For these depths, only offshore platform technology seems appropriate.

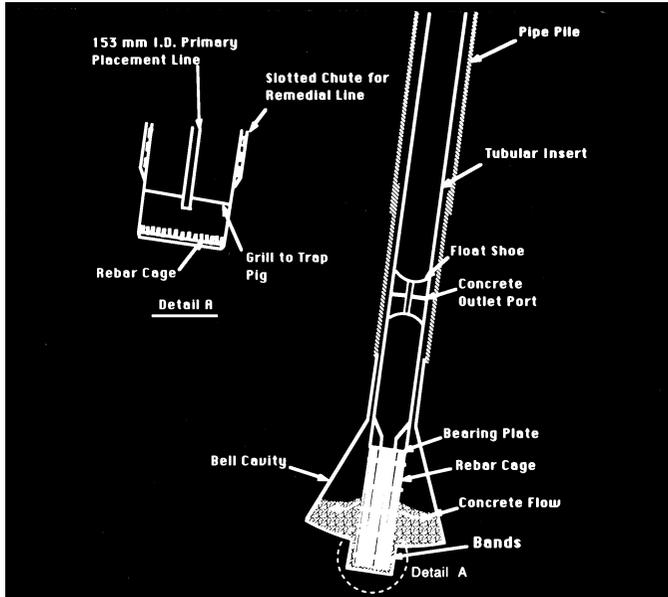


FIGURE 47.33 Belled footing provides greater bearing area for driven-and-drilled steel tubular pile.

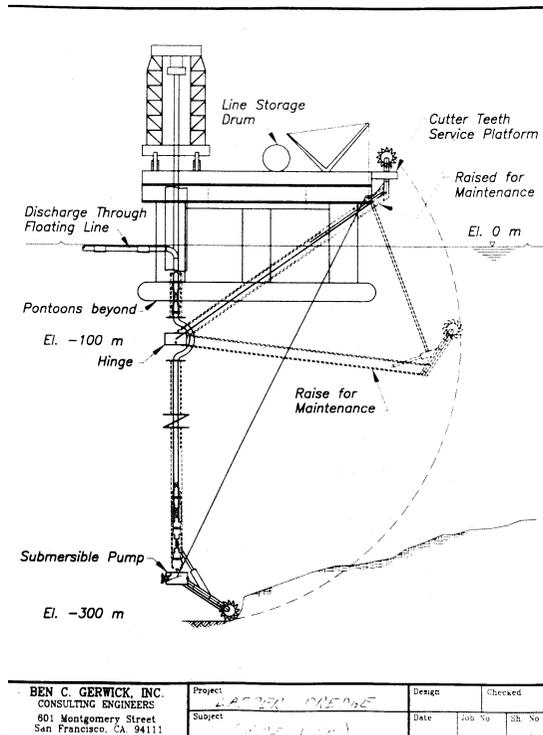
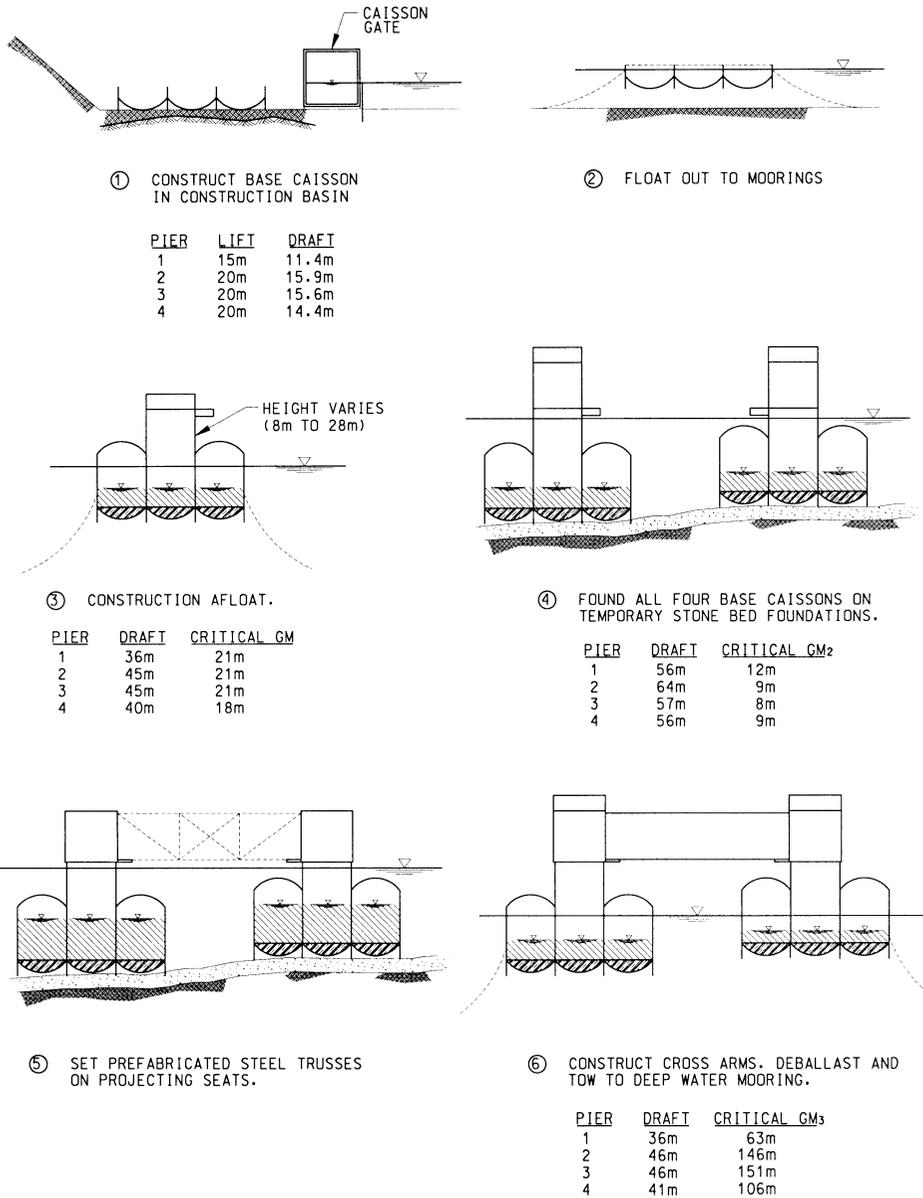


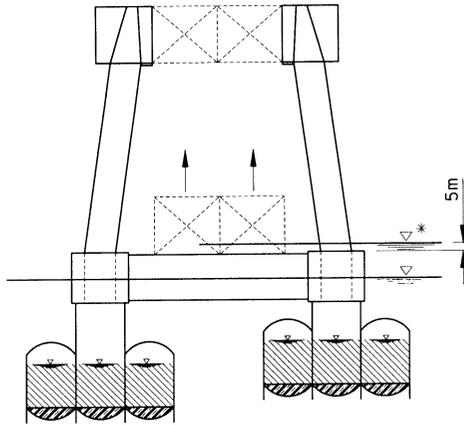
FIGURE 47.34 Concept for preparation of seabed for seating of prefabricated box piers in 300 m water depth, Strait of Gibraltar.



**FIGURE 47.35** (a,b,c) Fabrication and installation concept for piers in 300 m water depth for crossing of Strait of Gibraltar.

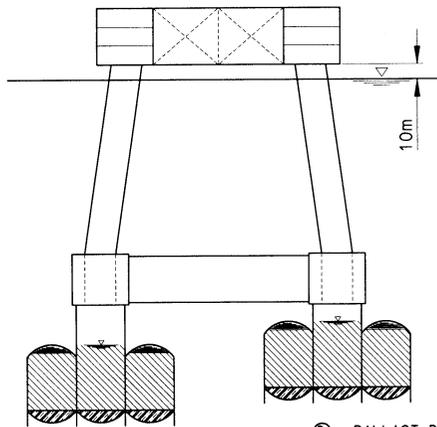
Both steel jackets with pin piles and concrete offshore structures were investigated. Among the other criteria that proved extremely demanding were potential collision by large crude oil tankers and, below water, by nuclear submarines.

These studies concluded that the concrete offshore platform concept was a reasonable and practicable extension of current offshore platform technology. Leveling and preparing a suitable foundation is the greatest challenge and requires the integration and extension of present systems of dredging well beyond the current state of the art (Figure 47.34). Conceptual systems for these structures have been developed which indicate that the planned piers are feasible by employing an extension of the concepts successfully employed for the offshore concrete platforms in the North Sea (Figure 47.35).



- ⑦ BALLAST DOWN TO MID-HEIGHT OF CROSS-ARMS. CONSTRUCT ALL 4 SHAFTS AND CONICAL TOPS. RAISE PLATFORM FABRICATION TRUSSES AND SECURE TO SHAFT TOP CONES AND UPPER FALSEWORK (TRUSS WEIGHTS ~ 2000T)

PIER	DRAFT*	CRITICAL GM <sub>4</sub>
1	96m	22m
2	106m	34m
3	95m	36m
4	97m	27m



- ⑧ BALLAST DOWN TO DEEP DRAFT. COMPLETE UPPER PLATFORM, VERTICAL CROSS WALLS FORMED WITH REBAR TIED BUT NO CONCRETE PLACED.

PIER	DRAFT*	CRITICAL GM <sub>5</sub>
1	103m	5m
2	281m	21m
3	246m	16m
4	196m	6m

FIGURE 47.35 (continued)

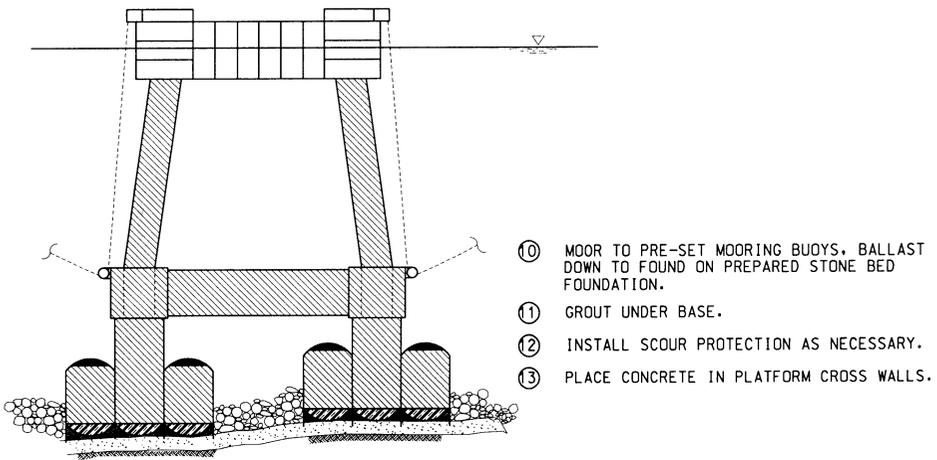
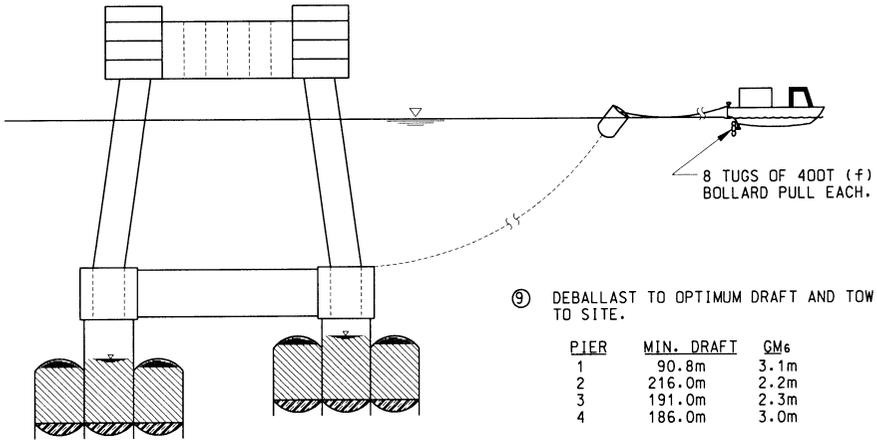


FIGURE 47.35 (continued)

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