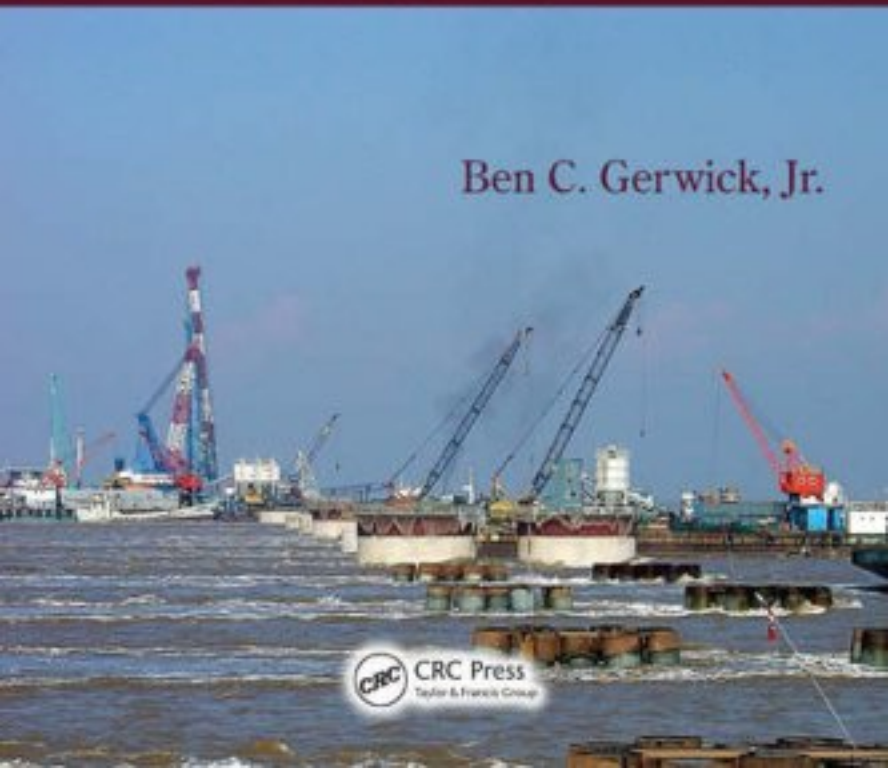
CONSTRUCTION OF MARINE AND OFFSHORE STRUCTURES

Third Edition

Ben C. Gerwick, Jr.



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Third Edition

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Third Edition

Ben C. Gerwick, Jr.

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To the great pioneers in Marine and Offshore Construction who were

undeterred by violent storms and massive ice.

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Preface

This third editon has been intensively augmented and revised to include the latest developments in this rapidly expanding field. The intensified search for oil and gas, the catastrophic flooding of coastal regions and the demands for transportation, bridges, submerged tunnels and waterways have led to the continuing innovation of new technology which is now available for use on more conventional projects as well as those at the frontiers.

This text is intended as a guide and reference for practicing engineers and constructors for use in the marine environment. It is also intended as a text for graduate engineering students interested in this highly challenging endeavour.

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I wish to acknowledge the help of many members of our company, Ben C. Gerwick, Inc. making available information on the current construction of marine and offshore projects, also the willing responses to my queries from other sources in the industry.

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Author

Ben C. Gerwick, Jr. is the author of *Construction of Prestressed Concrete*, first, second, and third editions, and the first and second editions of *Construction of Marine and Offshore Structures*.

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1

Physical Environmental Aspects of Marine and Offshore Construction

1.1 General

The oceans present a unique set of environmental conditions that dominate the methods, equipment, support, and procedures to be employed in construction offshore. Of course, this same unique environment also dominates the design of offshore structures. Many books have addressed the extreme environmental events and adverse exposures as they affect design. Unfortunately, relatively little attention has been given in published texts to the environment's influence on construction. Since the design of offshore structures is based to a substantial degree upon the ability to construct them, there is an obvious need to understand and adapt to environmental aspects as they affect construction. These considerations are even more dominant in many coastal projects where breaking waves and high surf make normal construction practices impossible. To a lesser extent, they have an important role in harbor and river construction.

In this chapter, the principal environmental factors will be examined individually. As will be emphasized in this book, a typical construction project will be subjected to many of these concurrently, and it will be necessary to consider their interaction with each other and with the construction activity.

1.2 Distances and Depths

Most marine and offshore construction takes place at substantial distances from shore, and even from other structures, often being out of sight over the horizon. Thus, construction activities must be essentially self-supporting, able to be manned and operated with a minimum dependency on a shore-based infrastructure.

Distance has a major impact upon the methods used for determining position and the practical accuracies obtainable. The curvature of Earth and the local deviations in sea level should be considered. Distance affects communication. Delivery of fuel and spare parts and transportation of personnel must be arranged. Distance requires that supervisory personnel at the site be capable of interpreting and integrating all the many considerations for making appropriate decisions. Distance also produces psychological effects. People involved in offshore construction must be able to work together in harmony and to endure long hours under often miserable conditions.

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Offshore regions extend from the coast to the deep ocean. Construction operations have been already carried out in 1500-m water depth, exploratory oil drilling operations in 6000 m, and offshore mining tests in similar water depths. The average depth of the ocean is 4000 m, the maximum over 10,000 m, a depth larger than the distance that Everest rises above sea level. The ocean depths, even those in which work is currently carried out, are inhospitable and essentially dark, and thus require special equipment, tools, and procedures for location, control, operations, and communication. Amazing technological developments have arisen to meet these demands: the work submersible, remote-operated vehicles (ROVs), fiber optics, acoustic imaging, and special gases for diver operations. While some of these advances have extended the capabilities of humans in the deep sea, it is important to recognize the limitations that depth still places on construction operations.

1.3 Hydrostatic Pressure and Buoyancy

The external pressure of seawater acting on a structure and all of its elements follows the simple hydraulic law that pressure is proportional to depth, where h = depth, Vw = density of seawater, and P = unit pressure,

$$P = Vwh \tag{1.1}$$

This can be very roughly expressed in the SI system as 10 kN/m^2/m of depth. More accurately, for seawater, the density is 1026 kg/m^3.

Hydrostatic pressure acts uniformly in all directions: downward, sideways, and up. The pressure is, of course, influenced by wave action: directly below the crest, the hydrostatic pressure is determined by the elevation of the crest and is therefore greater than that directly below the trough. This effect diminishes with depth, with differences due to moderate waves becoming negligible at 100 m and those due to storm waves becoming negligible at 200 m.

Hydrostatic pressure is also transmitted through channels within and beneath structures and within channels (pores) in the soil. The difference in pressure causes flow. Flow is impeded by friction. The distribution of hydrostatic pressure in the pores of soils under wave action is thus determined by the water depth, wavelength, wave height, and friction within the pores or channels. The effects from wave action usually disappear at 3–4 m in depth.

Hydrostatic pressure is linked with the concept of buoyancy. Archimedes' principle is that a floating object displaces a weight of water equal to its own weight. From another viewpoint, it can be seen that the body sinks into the fluid (in this case, seawater) until its weight is balanced by the upward hydrostatic pressure. In the case of a submerged object, its net weight in water (preponderance) can also be thought of as the air weight less either the displaced weight of water or the difference in hydrostatic pressures acting upon it.

Hydrostatic pressure not only exerts a collapsing force on structures in total, but also tends to compress the materials themselves. This latter can be significant at great depths, and even at shallower depths for materials of low modulus like, for example, plastic foam. Confined liquids or gases, including air, also are decreased in volume and increased in density when subjected to hydrostatic pressure. This decreases the volume and buoyancy while increasing the density.

Hydrostatic pressure forces water through permeable materials, membranes, cracks, and holes. In the cases of cracks and very small holes, flow is impeded by frictional

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forces. At the same time, capillary forces may augment the hydrostatic force, and raise the water level above the ambient. Hydrostatic pressure acts in all directions. Thus, on a large-diameter jacket leg, which has a temporary closure, it will produce both transverse circumferential compression and longitudinal compression. The combined stresses may lead to buckling.

It is important for the construction engineer to remember that full external hydrostatic pressure can be exerted in even a relatively small hole like, for example, an open prestressing duct or duct left by removal of a slip-form climbing rod. Hydrostatic pressure acting on gases or other fluids will transmit its pressure at the interface to the other substance. Thus, where an air cushion is utilized to provide increased buoyancy to a structure, the pressure at the interface will be the hydrostatic pressure of the seawater.

The density of seawater increases slightly with depth. This can be important in determining net weight of objects at great depths. The density of seawater also varies with temperature, salinity, and the presence of suspended solids such as silts. See Chapter 22, "Construction in the Deep Sea," in which the effects are quantified.

Special care must be taken during inshore or near-shore operations, where buoyancy, freeboard, and underkeel clearance are critical, and where large masses of fresh water may be encountered, with their lowered density and consequent effect on draft. An example of such suddenly occurring reduction of buoyancy is the annual release of the lake behind St. George Glacier in Cook Inlet, or a flood on the Orinoco River, whose effects may extend almost to Trinidad. A more static situation exists north of Bahrain in the Arabian Gulf, where fresh water emerges from seafloor aquifers.

1.4 Temperature

The surface temperature in the seas varies widely from a low of −2°C (28°F) to a high of 32°C (90°F). The higher temperatures decrease rapidly with depth, reaching a steady-state value of about 2°C (35°F) at a depth of 1000 m (3280 ft.). However, water and soil temperatures at 250 m depth on Australia's Northwest Shelf exceed 30°C.

Temperatures of individual masses and strata of seawater are generally distinct, with abrupt changes across the thermal boundaries. This enables ready identification of global currents; for example, a rise in temperature of as much as 2°C may occur when entering the Gulf Stream.

While horizontal differentiation (on the surface) has long been known, vertical differentiation and upwelling have recently been determined as major phenomena in the circulation of the sea. Rather definite boundaries separate zones of slightly different temperature, chemistry, and density. These zones will have recognizably different acoustic and light transmission properties, and the boundaries may give reflections from sonic transmissions.

Temperature affects the growth of marine organisms, both directly and by its effect on the amount of dissolved oxygen in the water. Marine organisms are very sensitive to sudden changes in the temperature: a sudden rise or fall produces a severe shock that either inhibits their growth or kills them. Cold water contains more dissolved oxygen than warm water.

Air temperatures show much greater variation. In the tropics, day air temperatures may reach 40°C. In semi-enclosed areas such as the Arabian–Persian Gulf and the Arabian Sea, air temperatures may even reach 50°C. Humidity is extremely high in such areas, resulting in rapid evaporation, which can produce a "salt fog" in the mornings, causing saline condensation to form on the surfaces of structures.

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The other extreme is the Arctic, where air temperatures over the ice may reach −40°C to −50°C. When the wind blows, air friction usually raises the temperature 10°C–20°C. However, the combination of low temperature and wind produces "wind chill," which severely affects the ability of people to work. Wind may similarly remove heat from materials (weldments or concrete surfaces, for example) far more rapidly than when the air is merely cold but still.

Air temperature in the temperate zones varies between these extremes. The ocean's thermal capacity, however, tends to moderate air temperatures from the extremes that occur over land. The rate of sound transmission varies with temperature. The temperature of the surrounding seawater has an important effect on the behavior of material, since it may be below the transition temperature for many steels, leading to brittle failure under impact. Properties of many other materials, such as concrete, improve slightly at these lower temperatures. Chemical reactions take place more slowly at lower temperatures: this, combined with the decrease in oxygen content with depth, reduces greatly the rate of corrosion for fully submerged structures.

Temperature also has a major effect on the density (pressure) of enclosed fluids and gases that may be used to provide buoyancy and pressurization during construction. The steady temperature of the seawater will tend to bring the enclosed fluid to the same temperature. Where this enclosed fluid, such as oil, is subject to transient phenomena, density and thermal gradients will be set up in it.

The atmosphere immediately above seawater is greatly modified by the water temperature. Nevertheless, it can be substantially below freezing, as for example in the sub-Arctic, or substantially above the water temperature, as in areas off Peru, where cold water contrasts with warm air. This produces a thermal gradient and thermal strains in structures that pierce the water plane. These above-water structures may also be directly heated by the sun. Thus, there may be a significant expansion of the deck of a barge or pontoon, leading to overall bending of the hull, with high shears in the sides and longitudinal bulkheads. Conversely, at night, the radiation cooling may lower the air temperature well below that of day.

Where the structure contains heated products, such as hot oil, or extremely cold products, such as liquefied natural gas (LNG), the thermal strains may be severe and require special attention, particularly at points of rigidity like structural intersections and corners. These thermal strains are discussed more fully in Chapter 4.

1.5 Seawater and Sea–Air Interface Chemistry

1.5.1 Marine Organisms

The dominant chemical characteristic of seawater is, of course, its dissolved salts, which typically constitutes 35 parts per thousand (3.5%) by weight. The principal ions are sodium, magnesium, chloride, and sulfate. These ions are of importance to the construction of structures in the ocean in many ways. Chloride (Cl^-) acts to reduce the protective oxidized coatings that form on steel and thus accelerates corrosion.

Magnesium (Mg_2^+) will gradually replace the calcium in various chemical constituents of hardened concrete. Magnesium salts are soft and tend to high permeability and solubility. Sulfates (SO_4^-) attack concrete, especially in fresh water. They affect both the cement paste and the aggregates, causing expansion and disintegration. Fortunately, the other constituents of seawater tend to inhibit sulfate attack.

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Oxygen is present in the air immediately adjacent to the seawater–air interface and is also present in the water in the form of entrapped air bubbles and dissolved oxygen. Oxygen plays an essential role in the corrosion of steel in the sea environment, whether the steel is exposed, coated, or encased in concrete. Carbon dioxide (CO_2) and hydrogen sulfide (H_2S) are also dissolved in seawater in varying degrees depending on location and temperature. They lower the pH of seawater. In addition, H_2S may cause hydrogen embrittlement of steel.

Entrapped bubbles of water vapor, as in foam, may collapse suddenly, leading to cavitation, which pits and erodes the surface of concrete structures. This phenomenon occurs when the surface of a structure is exposed to high-velocity local flow, as with surf, or over a spillway.

Silt and clay are suspended in water, usually in colloidal form, as the result of river runoff and also as the result of bottom erosion and scour due to current and waves. Colloidal silt in fresh water will drop out of suspension upon encountering seawater: this, as well as reduced velocity, accounts for the formation of deltas. The zone or band where such deposition takes place is often very narrow, resulting in a disproportionate deposition and buildup in this zone. Fine sand, silts, and clays, and even gravel may also be carried along with strong currents or wave action to be deposited as soon as the velocity drops below critical for that particle size and density. This results in horizontal stratification of deposits. The colloidal and suspended silts render vision and optics difficult due to their turbidity, which scatters light rays. Thus in many harbors, rivers, and estuaries, diver and submersible observations are limited when using normal light spectra.

Moving silt, sand, and gravel may erode surfaces, removing coatings and paint as well as the protective film of rust from steel, exposing fresh surfaces to corrosion.

Marine organisms have a number of adverse effects upon sea structures. The first is the increase of drag due to the obstruction of the free flow of water past the surface of the structure. This is caused by the "fouling" of ship bottoms. Mussels may clog intakes to power plants, or eels may enter circulating water systems and then grow and plug the system. Barnacles and algae increase the diameter of steel piles. Fouling increases the size of the member and more important, increases the surface roughness. Because of this latter, the drag coefficient, C_D, used in Morrison's equation, is often increased by 10%–20%.

Fortunately, most marine organisms have a specific gravity only slightly greater than that of the seawater itself; thus, they do not add an appreciable mass. They also tend to be fragile, and are often torn or broken off by storms. Barnacles and sea urchins secrete an acid that pits and erodes steel. Sea urchins are partially active near the sand line and can attack the steel piling and jacket legs.

Mollusks secreting acids bore into rocks and soft concrete. Very aggressive mollusks exist in the Arabian–Persian Gulf. These bore holes into the hard limestone aggregate of high-strength concrete: they also can eat through bitumastic coatings on steel piles. Marine organisms have occurred at depths up to 60 m, but they are concentrated near the surface where sunlight penetrates.

Of particular importance to the constructor is the attack of marine organisms on timbers. Teredo enter into wood through a relatively small hole, eating out the heart, while Limnoria attack the surface of the wood, generally when it is in the tidal range. The action of teredo may be very rapid, especially in fast-flowing clean seawater. Untreated timber piles have been eaten off within a period of three months!

Fish bite, attacking fiber mooring lines. This is of increasing concern for deep-sea operations. Sharks apparently exercise their teeth on the lines, causing them to fray, which then attracts smaller fish. Fish bite is especially severe in the first month or two of exposure, apparently due to the curiosity of the sharks. Fish bite attacks occur in depths up to 1000 m in sub-Arctic waters and probably twice that depth in tropical waters.

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Marine organisms play a major role in the soil formation on the seafloor and in disturbing and reworking the surficial soils. Walruses apparently plow up large areas of sub-Arctic seafloors in search of mollusks, leading to turbidity and erosion. Algae and slime can form very rapidly on the surfaces of stones and riprap, preventing the subsequent bond with grout and concrete. Mussels, especially zebra mussels, can rapidly build up clusters on substrates of stone and steel. In the case of the anchorage caisson for the Great Belt suspension bridge, a cluster of mussels built up in the short interval between final screeding and the placement of the caisson, preventing it from proper seating.

Marine growth is influenced by temperature, oxygen content, pH, salinity, current, turbidity, and light. While the majority of growth takes place in the upper 20 m, significant growth has occasionally been found at three times that depth. Enclosed areas are protected temporarily during construction by algae inhibitors such as copper sulfate and by covering them to cut off sunlight.

Anaerobic sulfur-based bacteria are often trapped in the ancient sediments of the oil reservoir. Upon release to the saltwater, they convert to sulfates, and upon subsequent contact with air they produce sulfides (H_2S). These bacteria and the sulfides they produce, with the dramatic scientific name of *Theobacillus concretivorous*, attack weak and permeable concrete as well as causing pitting corrosion in steel. Even more serious, the hydrogen sulfide that is formed is deadly poisonous and may be odorless. Hence, entry to compartments previously filled with stored oil must be preceded by thorough purging not only of hydrocarbons, but also of any hydrogen sulfide. These anaerobic bacteria may also react with each other to produce methane and hydrogen. *T. concretivorous* bacteria in a seawater canal in the Arabian Gulf have attacked polysulfide sealants, turning them into a spongy mass.

1.6 Currents

Currents, even when small in magnitude, have a significant effect on construction operations. They obviously have an influence on the movement of vessels and floating structures and on their moorings. They change the characteristics of waves. They exert horizontal pressures against structural surfaces and, due to the Bernoulli effect, develop uplift or downdrag forces on horizontal surfaces. Currents create eddy patterns around structures, which may lead to scour and erosion of the soils. Currents may cause vortex shedding on piles, tethers, mooring lines, and piping.

Even before the start of construction, currents may have created scour channels and areas of deposition, thus creating surficial discontinuities at the construction site. The vertical profile of currents is conventionally shown as decreasing with depth as a parabolic function. Recent studies in the ocean and on actual deepwater projects indicate, however, that in many cases, the steady-state current velocities just above the seafloor are almost as high as those nearer the surface. There are substantial currents in the deep sea, just above the seafloor.

There are several different types of currents: oceanic circulation, geostrophic, tidal, wind-driven, and density currents, as well as currents due to river discharge. Currents in a river vary laterally and with depth. The highest river currents usually occur near the outer edge of a bend. River currents are also augmented locally around the head of jetties and groins. Some of these may be superimposed upon each other, often in different directions (Figure 1.1).

The worldwide ocean circulatory system produces such currents as the Gulf Stream, with a relatively well-defined "channel" and direction and velocity of flow. Other major

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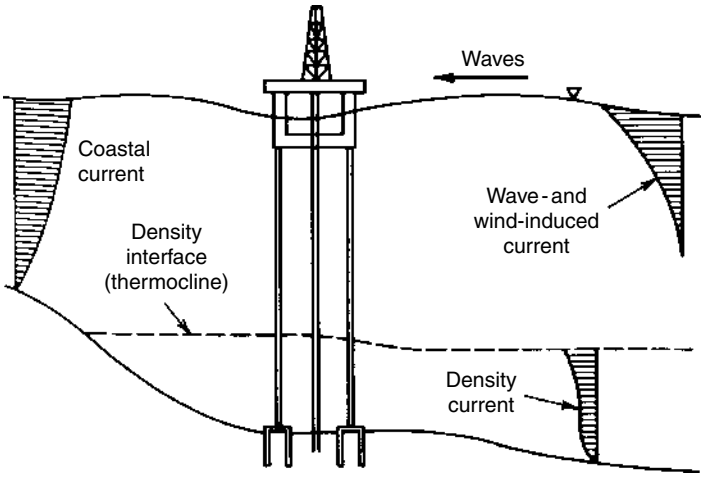

FIGURE 1.1
Wave-current flow field. (Adapted from N. Ismail, *J. Waterway, Port Coast. Ocean Eng., Am. Soc. Civil Engineers,* 1983.)

current systems exist but are often more diffuse, having a general trend but without the characteristics of a river. Thus the prevailing southeasterly trending current along the California and Oregon coasts gives an overall southward movement to sedimentary materials from river outflows. These major currents may occasionally, often periodically, spin off eddies and branches; the lateral boundaries of the current are thus quite variable. Strong currents may thus occur many miles from the normal path of a current such as the Gulf Stream. Within local coastal configurations, a branch of the main current may sweep in toward shore or even eddy back close to shore.

Recent research has indicated that many of these current "streams" are fed by upwelling or downward movements of the waters and that there are substantial vertical components. These will become important as structures are planned and built in deeper waters and will require that accurate measurements be taken at all depths, for both vertical and horizontal components of the current.

Another major source of currents is tidal changes. The stronger tidal currents are usually in proximity to shore but may extend a considerable distance offshore where they are channeled by subsurface reefs or bathymetry. While they generally follow the tidal cycle, they frequently lag it by up to 1 h; thus, a tidal current may continue flooding on the surface for a short period after the tide has started to fall.

Actually tidal currents are often stratified vertically, so that the lower waters may be flowing in while the upper waters are flowing out. This is particularly noticeable where tidal currents are combined with river currents or where relatively fresh water of lower density overlies heavier saltwater. This stratification and directional opposition also occurs at the entrance to major bodies of water like the Strait of Gibraltar, where evaporation from the Mediterranean produces a net inflow.

Since tidal currents are generally changing four times a day, it follows that their velocity and direction are constantly changing. Since the velocity head or pressure acting on a structure varies as the square of this current velocity, it can have a major effect on the mooring and control of structures during critical phases of installation. The current velocities are also superimposed on the orbital particle velocities of the waves, with the pressure and hence forces being proportional to the square of the vectorial addition.

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While in regular harbor channels, the tidal currents may move in and out along a single path; at most offshore sites, the shoreline and subsurface configurations cause the directions to alter significantly, perhaps even rotate, during the tidal cycle. Ebb currents may be directed not only 180° but often 150°, 120°, or even 90° from flood currents, and this variance itself may change periodically. Tidal currents may reach speeds of 7 knots and more.

River currents, especially those of great rivers with large discharges, such as the Orinoco, extend far out to sea. Because the density of the water is less, and perhaps because of silt content, the masses of water tend to persist without mixing for a long period; thus substantial surface currents may reach to considerable distances from shore. River currents may, as indicated earlier, combine with tidal currents to produce much higher velocities on ebb and reduced velocities on flood.

Wind persisting for a long period of time causes a movement of the surface water that is particularly pronounced adjacent to shallow coasts. This may augment, modify, or reverse coastal currents due to other causes.

Deep-water waves create oscillatory currents on the seafloor, so that there is little net translation of soil particles due to waves alone. When, however, a wave current is superimposed upon a steady-state current, the sediment transport is noticeably increased, since its magnitude varies as the cube of the instantaneous current velocity. The vertical pressure differentials from the waves lift the soil particles, which are then transported by the current.

Adjacent to the shore, the translational movement of the waves produces definite currents, with water flowing in on top and out either underneath or in channels. Thus, a typical pattern of the sea will be to build up an offshore bar, over which the waves move shoreward and break on the beach. This piles excess water on the beach, which may move laterally, then run out to the sea. The outflowing current cuts channels in the offshore bar. The seaward-flowing current becomes the infamous "undertow." These lateral and seaward-flowing currents may be a hazard or may be taken advantage of to keep a dredged channel clear through the surf zone.

In the deeper ocean, currents are generated by internal waves, by geostrophic forces, and by deeply promulgated eddies from major ocean streams such as the Gulf Stream. It appears that currents of magnitudes up to 0.5 knots exist on the continental shelf and slope and that currents up to 2.6 knots (1.3 m/s) can be found in the deep ocean.

Strong currents can cause vortex shedding on risers and piles, and vibration of wire lines and pipelines. Vortex shedding can result in scour in shallow water, and it can result in cyclic dynamic oscillations of cables, tethers, moorings, and vertical tubulars, such as piling, which can lead to fatigue. Vortices occur above a critical velocity, typically 2–3 knots. These vortices spin off in a regular pattern, creating alternating zones of low pressure. Vortices and whirlpools can form at the edge of obstructions to river flow, such as around the end of groins or at the edge of an underwater sand wave, leading to severe local scour. Vibration due to vortices on tensioned mooring lines has led to fatigue failure of connecting links and shackles.

Currents develop forces due to drag and to inertia, the latter on the total mass, including that of the structure itself plus any contained material and that of the displaced water.

As mentioned earlier, water moving over a submerged surface or under the base of a structure produces a vertical pressure (uplift or downdrag) in accordance with Bernoulli's theorem. This can cause significant constructional problems, of which the following examples may be given:

1. A large concrete tank being submerged in the Bay of Biscay had its compartments accurately sized for filling to create a known preponderance for controlled sinking, without free surface. When it had been filled and submerged a few meters, the waves moving over the top had their oscillatory motion changed

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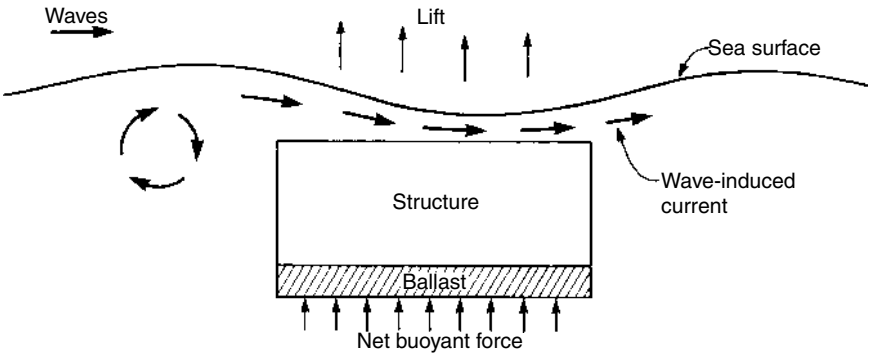

FIGURE 1.2
Hydrodynamic uplift on shallow immersed structure.

to a translatory current, thus creating an uplift force. This has been called the "beach effect" (see Figure 1.2). The tank would sink no further. When, as an emergency measure at 30 m, additional ballast was added to cause sinking to continue, the current effect was reduced and the uplift force was diminished. The tank was now too heavy and plunged rapidly (Figure 1.2).

2. A caisson being submerged to the seafloor behaves normally until close to the bottom, when the current is trapped beneath the base and its velocity increases. This "pulls" the caisson down, while at the same time tending to scour a previously prepared gravel base. In loose sediments, such as the Mississippi River, the loose sand mudline may drop almost as fast as the caisson is submerged, unless antiscour mattresses are placed beforehand.
3. A pipeline set on the seafloor is subjected to a strong current that erodes the sand backfill from around it. The pipeline is now subject to uplift (from the increased current flowing over it) and rises off the bottom. The current now can flow underneath; this pulls the pipeline back to the seafloor, where the process can be repeated. Eventually, the pipeline may fail in fatigue (Figure 1.3).
4. The placement of a structure such as a cofferdam in a river leads to accelerated currents around the leading corners, and the formation of a deep scour hole either at the corners or some distance downstream where a vortex has formed. These have reached depths of 10–20 m below the adjoining bottom and have resulted in general instability.

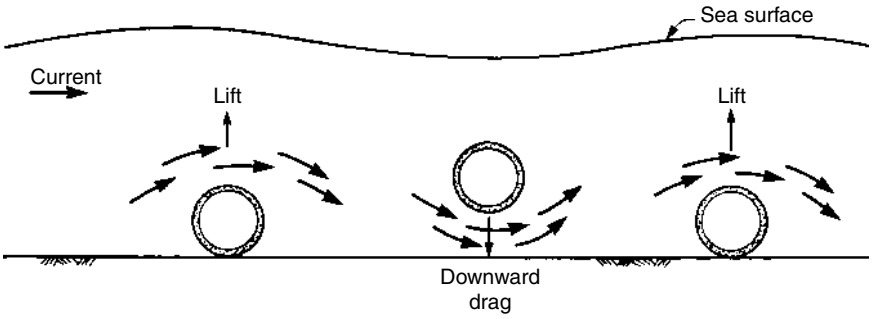

FIGURE 1.3
Oscillating movement of seafloor pipeline due to current.

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5. Velocity caps are flat concrete slabs supported on short posts just above the inlet to a seawater intake. Their purpose is to lower the velocity so that fish won't be sucked into the intake. In shallow water, breaking waves sweep over the top, leading to high cyclic uplift forces. Unless adequately held down, by added weight or drilled in ties, for example, the velocity cap is soon broken loose and destroyed.

Installation of a box caisson pier in the Øresund crossing between Denmark and Sweden led to significant erosion along and under one corner of the base due to currents induced by a storm. Similar erosion occurred under the base of one pylon pier of the Akashi Strait Bridge across Japan's Inland Sea. Currents produce both scour and deposition. It is important to note that eddies formed at the upstream and downstream corners of structures, such as those of a rectangular caisson, produce deep holes, whereas deposition may occur at the frontal and rear faces.

Scour is extremely difficult to predict. Model studies indicate tendencies and critical locations but are usually not quantitatively accurate because of the inability to model the viscosity of water, the grain size and density, and the effect of pore pressures. However, models can be effectively utilized to predict how the currents will be modified around a particular structure.

Currents have a significant effect on the wave profile. A following current will lengthen the apparent wavelength and flatten the wave out, so that its slopes are much less steep. Conversely, an opposing current will shorten the wavelength, increasing the height and steepness. Thus, at an ocean site affected by strong tidal currents, the same incident waves will have quite different effects on the construction operations, depending on the phases of the tidal cycle (Figure 1.4).

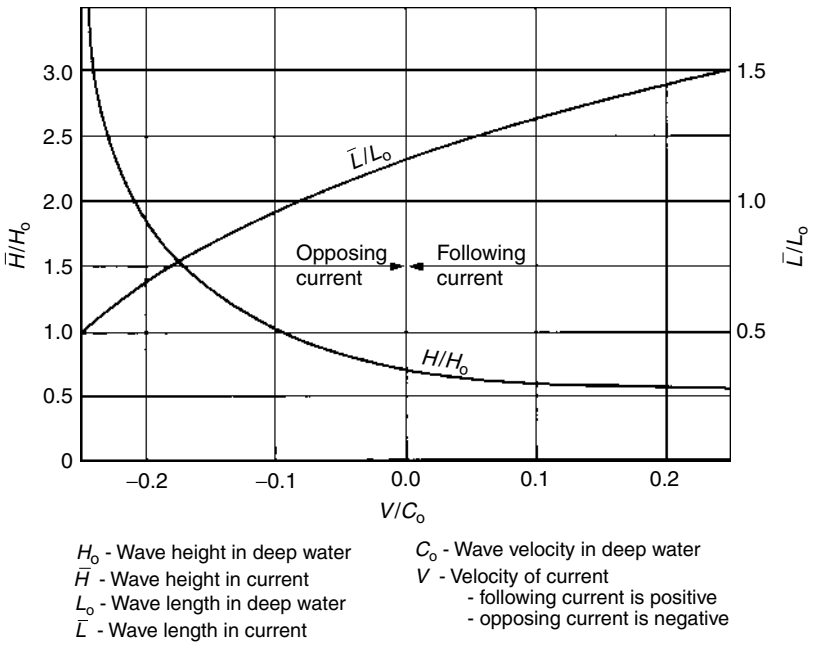

FIGURE 1.4
Changes in wave height and length in an opposing or following current.

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Currents have a serious effect on towing speed and time, a following current increasing the effective speed, an opposing current decreasing it. Translated into time, the decrease in time for a tow of a given distance is only marginally improved by a following current, whereas an opposing current may significantly increase the time required.

By way of example, assume that a towboat can tow a barge 120 miles at 6 knots in still water, thus requiring 20 h. With a following current of 2 knots, the trip will take only 120/(6+2) or 15 h, a saving of 5 h or 25%. With an opposing current of 2 knots, the trip will require 120/(6−2) or 30 h, an increase of 10 h or 50%.

1.7 Waves and Swells

Waves are perhaps the most obvious environmental concern for operations offshore. They cause a floating structure or vessel to respond in six degrees of freedom: heave, pitch, roll, sway, surge, and yaw. They constitute the primary cause of downtime and reduced operating efficiency. The forces exerted by waves are usually the dominant design criterion affecting fixed structures (Figure 1.5).

Waves are primarily caused by the action of wind on water, which through friction transmits energy from the wind into wave energy. Waves that are still under the action of the wind are called "waves," whereas when these same waves have been transmitted beyond the wind-affected zone by distance or time, they are called "swells."

Water waves can also be generated by other phenomena, such as high currents, landslides, explosions, and earthquakes. Those associated with earthquakes (e.g., tsunamis) will be dealt with in Section 1.12. A wave is a traveling disturbance of the sea surface. The disturbance travels, but the water particles within the wave move in a nearly closed elliptical orbit, with little net forward motion.

Wave and swell conditions can be predicted from knowledge of the over-ocean winds. Routine forecasts are now available for a number of offshore operating areas. They are

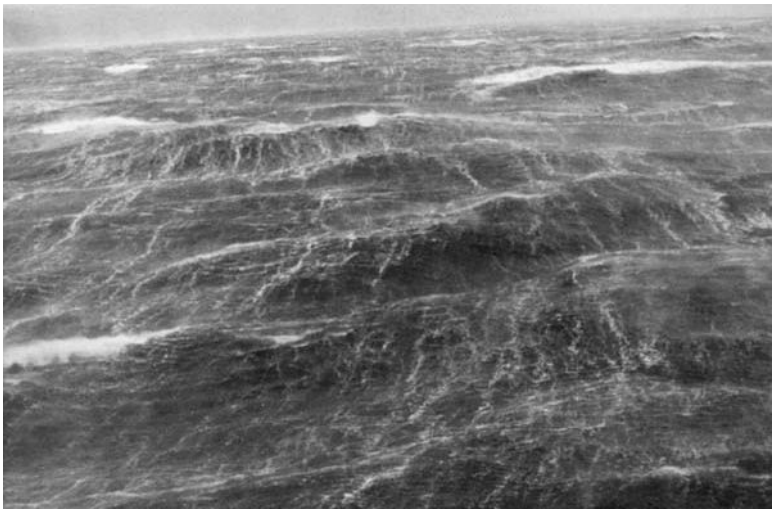

FIGURE 1.5
Long period swells from a distant storm, on which wind waves from a local storm are superimposed.

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provided by governmental services such as the U.S. Naval Fleet Numerical Weather Control at Monterey, California. Many private companies now offer similar services. These forecasts are generally based on a very coarse grid, which unfortunately may miss local storms such as extratropical cyclones.

The height of a wave is governed by the wind speed, duration, and fetch (the distance that the wind blows over open water).

Deep-water wave forecasting curves can be prepared as a guide (see Figure 1.6). These values are modified slightly by temperature; for example, if the air is 10°C colder than the

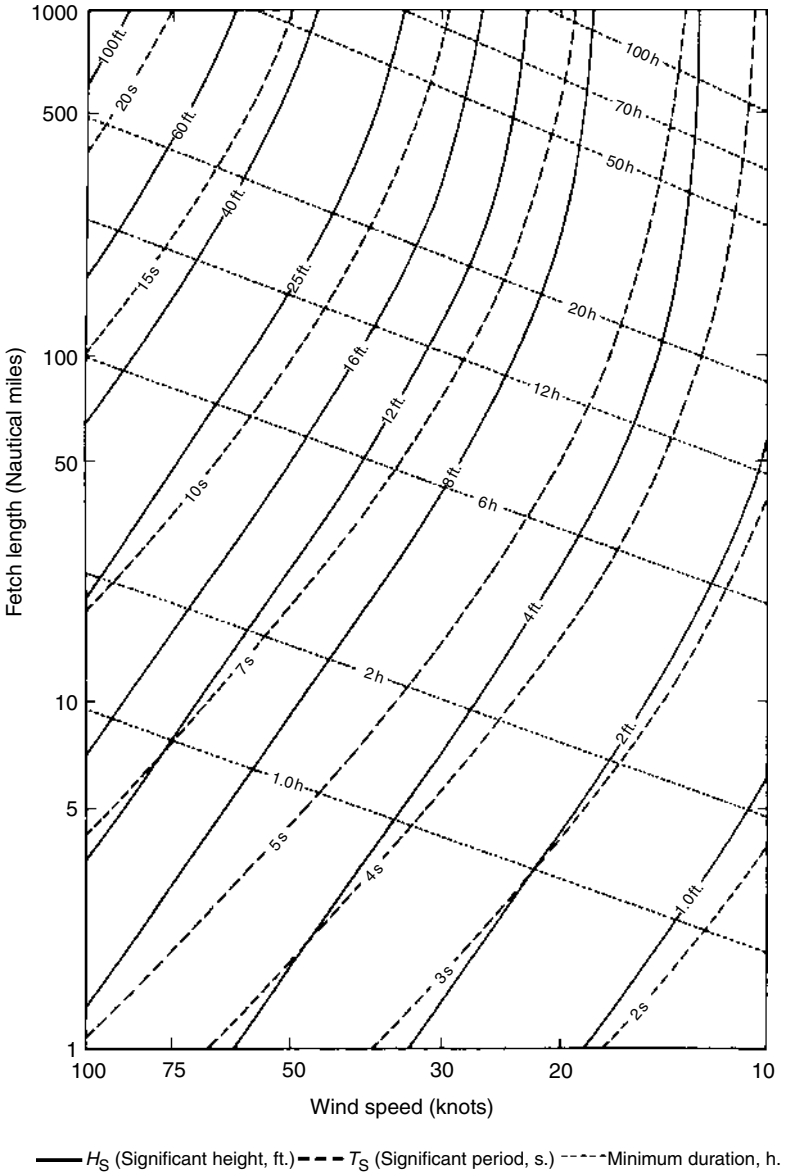

FIGURE 1.6
Deep-water wave forecasting curves. (Adapted from U.S. Army Engineers, *Shore Protection Manual, U.S. Army Engineers*, Coastal Engineering Research Center.)

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sea, the waves will be 20% higher, due to the greater density and hence energy in the wind. This can be significant in the sub-Arctic and Arctic.

Some interesting ratios can be deduced from Figure 1.6:

1. A tenfold increase in fetch increases the wave height 2.5 times.
2. A fivefold increase in wind velocity increases the wave height 13 times.
3. The minimum-duration curves indicate the duration which the wind must blow in order for the waves to reach their maximum height. The stronger the wind, the less time required to reach full development of the waves.

The total energy in a wave is proportional to the square of the wave height. While wave height is obviously an important parameter, wave period may be of equal concern to the constructor. Figure 1.6 gives the typical period associated with a fully developed wave in deep water. Long-period waves have great energy. When the length of a moored vessel is less than one half the wavelength, it will see greatly increased dynamic surge forces.

Waves vary markedly within a site, even at the same time. Therefore, they are generally characterized by their significant height and significant period. The significant height of a wave is the average of the highest one-third of the waves. It has been found from experience that this is what an experienced mariner will report as being the height of the waves in a storm. If the duration of the strong wind is limited to less than the minimum duration, then the wave height will be proportional to the square root of the duration. A sudden squall will not be able to kick up much of a sea.

The majority of waves are generated by cyclonic storms, which rotate counterclockwise in the northern hemisphere and clockwise in the southern hemisphere. The storm itself moves rather slowly, as compared with the waves themselves. The waves travel out ahead of the generating area. Waves within the generating area are termed seas; those which move out ahead are termed swells. Swells can reach for hundreds and even thousands of miles. The area embraced by the cyclone can be divided into four quadrants. The "dangerous quadrant" is the one in which the storm's forward movement adds to the orbital wind velocity.

The Antarctic continent is completely surrounded by open water. It is an area of intense cyclonic activity. Storms travel all the way around the continent, sending out swells that reach to the equator and beyond. The west coast of Africa, from southwest Africa to Nigeria and the Ivory Coast, and the west coast of Tasmania are notorious for the long period swells that arrive from Antarctica. The long, high-energy swells that arrive at the coast of southern California in May are generated by tropical hurricanes in the South Pacific.

The swells eventually decay. Energy is lost due to internal friction and friction with the air. The shorter-period (high-frequency) waves are filtered out first, so that it is the longest of the long-period swells that reach farthest. Swells tend to be more regular, each similar to the other, than waves. Whereas waves typically have significant periods of 5–15 s, swells may develop periods as great as 20–30 s or more. The energy in swells is proportional to their length; thus even relatively low swells can cause severe forces on moored vessels and structures.

Deep-sea waves tend to travel in groups, with a series of higher waves followed by a series of lower waves. The velocity of the group of waves is about half the velocity of the individual waves. This, of course, gives observant constructors the opportunity to wait until a period of successive low waves arrives before carrying out certain critical construction operations of short duration, such as the setting of a load on a platform deck or the stabbing of a pile. Such periods of low waves may last for several minutes.

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The average wave height is about 0.63Hs. Only 10% of the waves are higher than Hs. One wave in 1000 is 1.86 times higher than Hs; this is often considered to be the "maximum" wave, but more recent studies show that the value may be closer to 2. Wave height, H, is the vertical distance from trough to crest, and T is the period, the elapsed time between the passage of two crests past a point. Wavelength, L, is the horizontal distance between two crests. Velocity, V, often termed celerity, C, is the speed of propagation of the wave. Rough rule-of-thumb relationships exist between several of these factors.

Where L is in m, T in s, and V in m/s,

$$L = \frac{3}{2}T^2 \tag{1.2}$$

$$V = \frac{L}{T} = \frac{3}{2}T \tag{1.3}$$

In the English System, where L is in ft., T in s, and V in ft./s,

$$L = 5T^2 \tag{1.4}$$

$$V = \frac{L}{T} = 5T \tag{1.5}$$

As noted in Section 1.6, currents have a significant effect upon wavelength, steepness, and height. A following current increases the length and decreases the height, whereas an opposing current decreases the wavelength and increases the height, thus significantly increasing the wave steepness. Note that the influence of an opposing current is much more pronounced than that of a following current. Note also that the wave period remains constant. When seas or swells meet a strong current at an angle, very confused seas result, with the wave crests becoming shorter, steeper, and sharper and thus hazardous for offshore operations.

Seas are often a combination of local wind waves from one direction and swells from another. Waves from a storm at the site may be superimposed upon the swells running out ahead of a second storm that is still hundreds of miles away. The result will be confused seas with occasional pyramidal waves and troughs.

Waves are not "long-crested"; rather, the length of the crest is limited. The crest length of wind waves averages 1.5–2.0 times the wavelength. The crest length of swells averages three to four times the wavelength. These crests are not all oriented parallel to one another but have a directional spread. Wind waves have more spread than swells. From a practical, operational point of view, the majority of swells tend to be oriented within $\pm 15°$, whereas wind waves may have a $\pm 25°$ spread.

When waves in deep water reach a steepness greater than 1 in 13, they break. When these breaking waves impact against the side of a vessel or structure, they exert a very high local force, which in extreme cases may reach 30 tn./m^2 (0.3 MPa), or 40 psi. The areas subjected to such intense forces are limited, and the impact itself is of very short duration; however, these wave impact forces are similar to the slamming forces on the bow of a ship and thus may control the local design.

Data on wave climates for the various oceans are published by a number of governmental organizations. The U.S. National Oceanic and Atmospheric Administration (NOAA) publishes very complete sets of weather condition tables entitled "Summaries of Synoptic Meteorological Observations" (SSMOs), based on data compiled from ship observations and ocean data buoys. The published tables tend to underestimate the wave

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heights and periods in the Pacific; recent data for the Pacific indicates that there is significant wave energy in longer periods (e.g., 20–22 s) during severe storms. The swells from such storms may affect operations even at a distance of several thousand miles.

The "persistence" of wave environmental conditions is of great importance to construction operations. Persistence is an indication of the number of successive days of low sea states one may expect to experience at a given site and season. To the offshore constructor, persistence is quite a different thing from percentage exceedance of sea states exceeding various heights.

For example, assume that the limiting sea state for a particular piece of construction equipment is 2 m. The percentage exceedance chart may show that seas greater than 2 m occur 20% of the month in question. This could consist of two storms of three-day duration each, interspersed between two twelve-day periods of calm. Such a wave climate would allow efficient construction operations. Alternatively, this 20% exceedance could consist of 10 h of high waves every other day, as typically occurs in the Bass Straits between Australia and Tasmania. Such a wave climate is essentially unworkable with conventional marine equipment.

Typical persistence charts are shown in Figure 1.7 and Figure 1.8. Further discussion on persistence is found in Section 1.8. Wave height–wave period relationships are shown in Figure 1.9.

As swells and waves approach the land or shoal areas, the bottom friction causes them to slow down; the wave front will refract around toward normal with the shore. This is why waves almost always break onshore even though the winds may be blowing parallel to it. Multiple refractions can create confused seas and make it difficult to orient a construction barge or vessel for optimum operational efficiency. At some locations, two refraction patterns will superimpose, increasing the wave height and steepness.

Submerged natural shoals and artificial berms increase the wave height and focus the wave energy toward the center. Waves running around a small island, natural or artificial, not only refract to converge their energy on the central portion but run around the island to

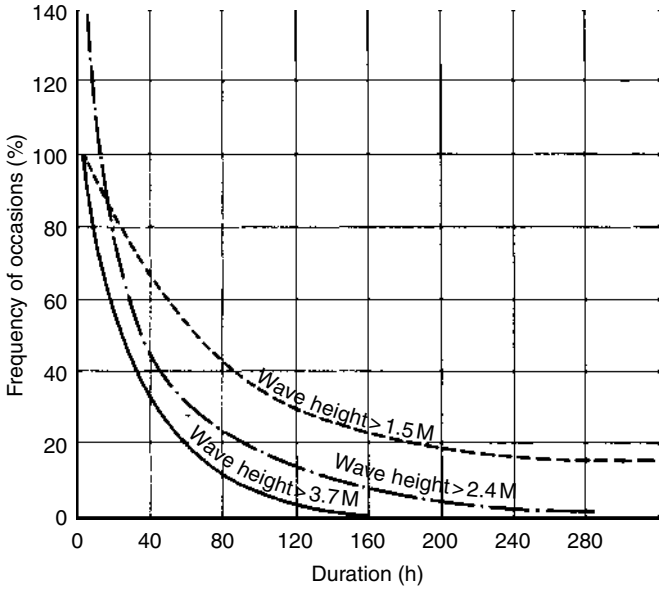

FIGURE 1.7
Persistence of unfavorable seas.

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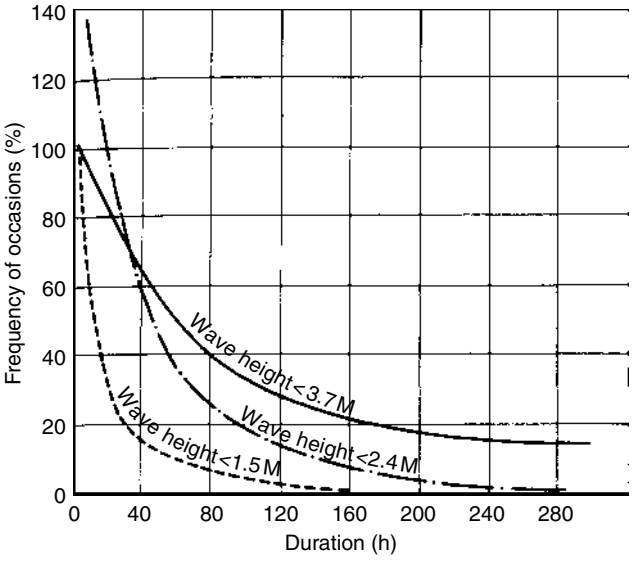

FIGURE 1.8
Persistence of favorable seas.

meet in the rear in a series of pyramidal peaks and troughs. Such amplification of waves and the resultant confusion of the sea surface may make normal construction operations almost impossible. Running along or around the vertical face of a caisson, waves will progressively build up in an effect known as "mach stem" and spill over onto the island, but without radial impact. Both phenomena combined to cause overtopping and difficulty in operations at the Tarsiut Offshore Drilling Island in the Canadian Beaufort Sea.

Waves approaching a shore having a deep inlet or trench through the surf zone will refract away from the inlet, leaving it relatively calm, while increasing the wave energy breaking in the shallow water on either side. As waves and swells move from deep water into shallow water, their characteristics change dramatically. Only their period remains essentially the same. The wavelength shortens and the height increases. This, of course, leads to steepening of the wave, until it eventually breaks.

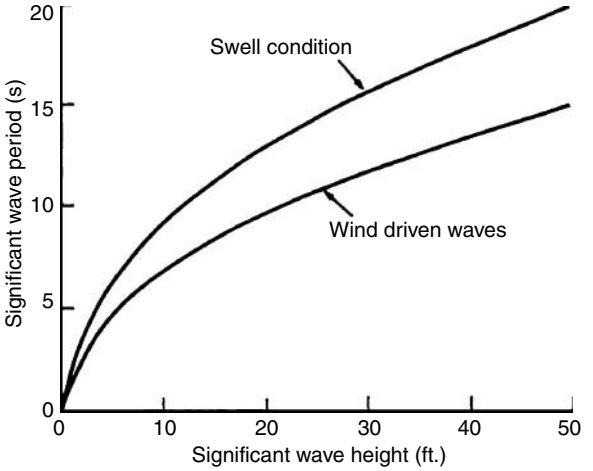

FIGURE 1.9
Wave height–wave period relationship.

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While the above may be thought of as a typical description of coastal phenomena, the same can occur in the open ocean whenever shoaling is encountered. In the Bering Sea, for example, major storms generate high waves that spread into Norton Sound, with its relatively shallow depth over large areas. This results in an extremely difficult construction environment, with high, steep waves breaking and reforming over the shoal areas.

A deep-water wave is one for which the seafloor (the bottom) has essentially no effect. Since waves generate significant orbital motion of the water particles to a depth of about half a wavelength, while normally waves do not exceed 400 m wavelength even in a maximum storm, this has led to the adoption of a 200-m water depth as being "deep water." Most marine and offshore construction to date has taken place in lesser depths and hence, has been subject to the shallow water effects.

Waves and the associated wave-generated currents may cause major sediment transport, eroding sand embankments under construction and causing rapid scour around newly placed structures, especially if the wave effects are superimposed upon currents. Waves breaking over structures, especially flat-topped structures or decks, cause hydrodynamic uplift.

Internal waves are waves that propagate beneath the surface, typically acting along the boundary line (thermocline) between the warm upper ocean waters and the colder, more saline waters, which have greater density. This is usually located 100–200 m below the surface. Internal waves have been measured at a water depth of 1000 m with an internal height of 60 m. These can generate "density currents" as high as 2.6 knots (1.3 m/s) and thus are important for deep-sea and submerged operations. They may penetrate into deep fjords, acting on the interface between the upper, fresher water and the saltwater below.

Solitary waves are generated by landslides and rock slides into bodies of water. Earthquakes often generate tsunamis, which may be due to massive underwater landslides. In constricted water a ship or tug or high-speed boat may generate bow waves. All of these are solitary waves, also known as solitons. They travel long distances, decaying slowly due to bottom and air friction. They possess the unusual characteristic that they pass right through wind-driven waves and swells, without changing them or being changed by them.

Waves impacting against the vertical wall of a caisson or against the side of a barge are fully reflected, forming a standing wave or clapotis, almost twice the significant wave height, at a distance from the wall of one-half wavelength. Very high forces (wave slam) result. Waves, especially breaking waves in shallow water, may produce pore pressure buildup in the seafloor soils, leading to flow slide. Waves impinging on a structure, such as a platform or a coastal cofferdam, may impart repeated impact to the soils below, causing liquefaction and loss of support.

Waves oblique to a seawall will travel along the wall, building up height so that they overflow the top. This is called the Mach-stem effect.

Waves traveling obliquely between two pontoon hulls or two flexible walls, such as an open-ended steel sheet piled trench, build up a resonant harmonic action called "ringing." This is amplified by the out-of-phase travel of the waves on each exterior side.

1.8 Winds and Storms

The predominant patterns of ocean winds are their circulation around the permanent high-pressure areas that cover the ocean, clockwise in the northern hemisphere, counterclockwise in the southern hemisphere. In the tropical and subtropical zones, the extreme heat and the interface between atmosphere and ocean create deep lows, which spawn the violent storms known as tropical cyclones in the Indian Ocean, Arabian Sea, and offshore

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Australia; as hurricanes in the Atlantic and South Pacific; and as typhoons in the western Pacific. The occurrence of such storms in the subtropical and temperate zones is seasonal, late summer to early fall, and is fortunately somewhat infrequent. However, while they are easily spotted by satellite as well as by observations of opportunity, prediction of their route is still highly inaccurate. Thus, there may be several alerts per year at a construction site, necessitating adoption of storm procedures. Most of these will turn out to be false alarms, but significant delays will nevertheless occur.

Typical of the violent cyclonic storms are the typhoons of the western Pacific (Figure 1.10). These occur from May to December and have their source in the Pacific Ocean in the vicinity of the Caroline Islands. These storms travel westward, eventually dissipating over the Philippines, the China coast, and Japan or decaying into North Pacific storms. The diameters of these typhoons range from a maximum of 1000 miles to 100 miles or less. In most cases, the most severe wind, tide, and wave action occurs in a band about 50–100 miles wide. The storm as a whole can move in any direction, even circling back upon its previous track. It normally moves at a speed of less than 20 mph. While the predominant winds in a cyclonic storm are circular, the velocity of translation adds and subtracts. The "dangerous quarter" for navigators is that where the circular and translatory velocities join to produce a maximum.

In the Indian Ocean, east of the Indian subcontinent, tropical cyclones can occur during two seasons of the year, one the southern hemisphere cyclone season, the other the northern hemisphere cyclone season. There are many seas in which tropical cyclones were previously not thought to be present, primarily because there was so little human activity in the area. An example is the Arafura Sea, between Australia and the island of New Guinea. Not only do cyclones exist in this offshore region, but in 1982, one struck Darwin with devastating fury.

In typical offshore operations off the west coast of Australia, there will be a dozen or more cyclones spotted each season. See Figure 1.10. Several, perhaps four, will be close enough to cause a cyclone alert, with all operations shut down and equipment deployed in accordance with the prearranged procedures, either towed to the safety of a harbor or, in the case of larger vessels, towed clear to ride out the storm at sea. However, usually only one, sometimes none, of these cyclones will actually hit the construction site. Nothing

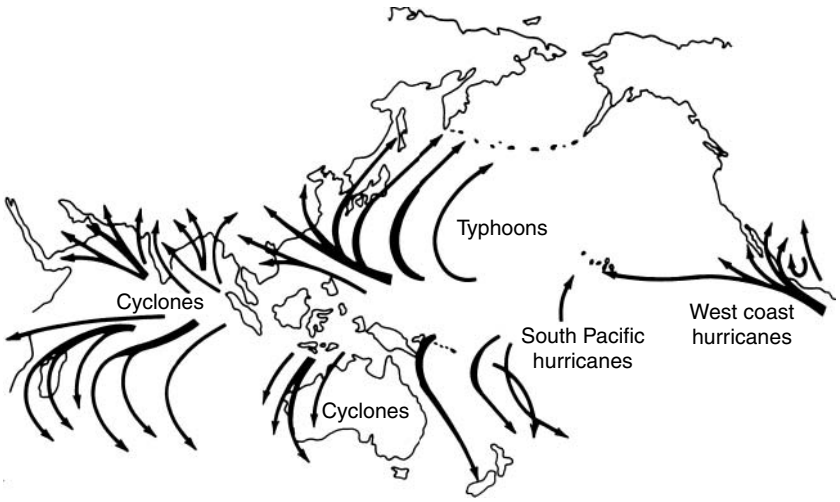

FIGURE 1.10
Major cyclonic storm paths in Pacific and Indian oceans.

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more serious than long swells may arrive. Quite often, it is the interruption to operations rather than actual storm damage that causes additional costs and delays.

Cyclonic storms can also spin off the interface between Arctic (and Antarctic) cold air masses and the warmer air of the temperate zones; these produce the typical winter storms of the North Atlantic, Gulf of Mexico, and North Pacific, and those surrounding the Antarctic continent. These storms tend to act over wide areas (several hundred miles) and persist for two to three days.

As noted in Section 1.7, patterns do exist for specific areas; for example, on the Pacific coast of the United States, six or seven days of storms will usually be followed by six or seven days of good weather. In the North Sea, in February, if there is a "good" day, then the probability of getting a "good" day the next day is 65%. The probability for the next two days is 40%, for the next three days, 5%. Conversely, if today is a "bad" day, the probability of a "good" day tomorrow is only 15%.

Recent activity in the Arctic Ocean has shown that during the brief summer, the interaction between the cold air mass over the ice pack and the warmer air of the adjacent landmasses can similarly spin off a series of intense but very local cyclonic storms, creating strong winds and short steep waves in the shallow open water.

Storm procedures for the constructor involve a sequence of steps: first, to stop operations and secure them against the weather, and second, to move off site, as applicable, to a more protected location in which to moor. Alternatively, the floating construction equipment may be moored on a survival mooring or even towed clear to ride out the storms at sea. These procedures will all be dealt with in later sections of this book.

It will be noted that swells normally arrive prior to the winds. A seasoned mariner will note the sky: cirrus clouds indicate a low-pressure area; they usually radiate from the storm center. The initial event of an approaching storm is often a cessation of the normal high-pressure winds, "the calm before the storm," followed by winds shifting and picking up as a fresh, intermittent breeze. It is often more dangerous to try to enter a harbor during the onset of a storm than to ride it out at sea. The harbor poses the additional problems of tidal currents and their effect on the waves, also of nearby shores and of winds that may be blowing across the channel.

Nautical terminology for wind directions is somewhat contrary. When mariners speak of a north wind, it means that the wind is coming from the north. When they speak of an onshore wind, it is blowing from sea toward land. An offshore wind blows from land toward sea. The term lee side is the sheltered side of a vessel away from that against which the wind is blowing. However, a lee shore is not a sheltered shore but the shore toward which the wind is blowing and hence a dangerous shore. The windward side of a vessel is the side against which the wind is blowing.

Wind speed increases as one goes up from sea level, for example, to the deck of a platform. For instance, the wind at a height of 20 m may be 10% greater than at 10 m, the usual reference height. Near sea level, the friction of the waves decreases the speed significantly. Winds are, of course, not steady but blow in gusts: the 3-s gust, for example, may have one-third to one-half greater speed than the same storm averaged over 1 h.

Wind storms may be of several types. There are the anticyclonic winds sweeping clockwise around the high-pressure areas over the major oceans of the northern hemisphere. Then there are the low-pressure cyclonic storms circulating counterclockwise around deep atmospheric depressions—the winter storms and the tropical cyclones.

In the tropics and subtropics, intense but very localized low-pressure zones result in sudden rain squalls, which occasionally grow into waterspouts. Although high winds of short duration are associated with these phenomena, serious tornado and waterspout activity appears to be extremely rare. However, the squall winds, coming rather suddenly,

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can damage booms or interfere with offshore operations. Since a weather period of several days' duration may have frequent squall activity, this may require extra precautions during that season.

In many areas of the world adjacent to landmasses, high-pressure cold air masses may build up over the land and then suddenly swoop down over the sea. They have many local names, such as "williwaw"; they are associated with clear visibility, usually a cloudless sky, and hence give little advance notice prior to the sight of whitecaps advancing across the sea. Wind velocities may reach 100 km/h (60–70 mph) or more. Because of their short duration, usually only a few hours, the seas are not fully developed; waves will be short and steep. They can catch construction operations unawares and do significant damage. Other offshore winds of longer duration are the Schmall of the Arabian and Persian Gulf and the dense air masses flowing down the slopes of Antarctica and out to sea.

Offshore winds can also blow off the desert; the infamous Santa Ana of Southern California forms as a cyclonic storm over the hot desert interiors, and its intense winds sweep out over the adjacent ocean. Because of lack of moisture, these storms are usually cloudless but full of sand and dust. Similar storms reportedly occur off the west coast of North Africa.

Milder but prevalent types of winds are the onshore–offshore winds typical of all coasts. The land heats up during the day, the air rises, and the wind blows in from the sea during the afternoon. To a lesser extent, the process occurs in the opposite direction in early morning. The onshore afternoon winds can occasionally reach 30 knots or more. These onshore winds may occur almost every afternoon on the U.S. Pacific Coast from June through August, posing a serious impediment to coastal operations.

Winds from two or more sources may be superimposed. The two most common are the high-pressure cyclonic winds plus the onshore afternoon winds. In extreme cases, these combined winds have reached 60 knots. Winds in harbors and estuaries may be considerably altered by local topography. Hills may provide relative calm in their lee, while gaps may intensify the wind speed. Similarly, winds tend to blow up- and downriver with greater intensity than over land and forests.

Storm forecasting services are available for all principal offshore operating areas. New areas, however, may suffer from lack of observation stations, especially in the southern hemisphere. Forecasting services also are not able to forecast local storms very well; all they can do is to warn when barometric pressures and air mass temperatures are right for such local storms to develop.

In the planning of offshore operations, every effort should be made to schedule critical operations for periods with low probability of storms or when those storms that may occur are of minimum intensity. Often there is a strong temptation to start early in the season or to work late just to finish up the project; these are frequently the times when storms occur and do damage that delays the project far more than a more prudent suspension of operations would entail.

It is customary in planning construction operations to (a) select the period during which the operations will be performed; (b) determine the maximum storms on a return period of at least five times the duration of work at the site (e.g., a five-year return storm for that time of year); and (c) develop procedures and plans and select equipment capable of riding out such a storm without significant damage.

1.9 Tides and Storm Surges

Tides result from the gravitational pull of the moon and the sun. Due to the relative masses and distances, the sun exerts only half the influence on the tides as the moon. During new

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and full moons, when the sun, Earth, and the moon are in approximate line, the highest tide ranges occur; these are called spring tides. When the sun and moon are approximately 90° apart, that is, at the first and third quarter of the moon, the ranges are lower; these are called neap tides.

The depth of the sea as shown on charts usually refers to mean lower low water (MLLW), which is the average of the low-water elevations during the spring tides. Some authorities use lowest astronomical tide (LAT) as the reference datum. Since land elevations usually use mean sea level (MSL) as the zero datum, structures in harbors and rivers may be keyed to that. This often leads to confusion: it is important to verify the reference datum.

Since the lunar month is one day shorter than the solar month, the times of tidal events are constantly changing. Normally, the tidal cycle moves back (later by clock time) by about 50 min each day. The time of high tide will be 50 min later tomorrow than it is today.

Typically, there are two tidal cycles each day. One of these tidal cycles will usually have significantly greater range (higher high tide, lower low tide) than the other. Some areas in the South Pacific (e.g., the Philippines and West Irian) have a prolonged high tide once each day, followed by a low tide 12 h later. These tidal cycles appear to follow the sun; hence the peaks occur about the same time each day; with high tide shortly after noon, low tide after midnight.

The times of tide and their height for the reference station are tabulated and published one or more years in advance. However, the time and height of tidal peaks at a particular location depend not only on astronomical conditions but also on the local bathymetry. Tidal tables are published for most coastal areas of the world, showing the time differences and height differences from the reference station for each locality. The influence of the ocean tides takes longer to reach to the head of estuaries and bays.

The tidal range in the deep ocean is relatively minor, usually less than 1 m. However, as one approaches the continental coasts, tidal ranges may increase radically, even though the site is still many kilometers offshore. This is especially noticeable off the west coast of North Africa and in the Bay of Biscay and on the northwest shelf of Australia. Within partially enclosed bays and estuaries, the tidal range builds up, reaching its maximum near the head of the bay.

Tidal ranges vary from approximately 0.5 m in some areas to as much as 10 m in others. Here are some typical values (approximate):

Location	Range (m)
Boston, Maine	2
Jacksonville, Florida	1
San Francisco, California	2.5
Puget Sound, Washington	5
Balboa, Panama (Pacific side)	4
Cristobal, Panama (Atlantic side)	0.2
Iceland	4
Mariana Islands	0.6
Cook Inlet, Alaska	9
Prudhoe, Alaska	0.5
Hangzhou Bay, China	6
Bay of Fundy, Canada	9

It can thus be seen that the tidal ranges vary significantly depending on location. In the cases of extreme tidal ranges, such as Cook Inlet, Hangzhou Bay, and the Bay of Fundy, the

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water may come in so fast that it forms a virtual wall of water, continuously breaking over its leading edge. This is known as a tidal bore.

The tidal cycles produce currents, which are discussed in more detail in Section 1.6. The flood stage is that with significant flow on a rising tide; the ebb is the flow associated with a falling tide. Periods of little or no flow are called "slack water." The times of slack water do not coincide exactly with the peaks of high and low water, since the water continues to flow for some period after a peak has been reached. Tidal currents are often stratified, with the current at the surface in a different direction from that at some depth.

Storm surges are changes in the level of the sea, which are superimposed upon the tide. They are caused primarily by the effect of the wind blowing for a long period in the same direction. A secondary cause is that of different barometric pressure. When low pressure exists at a site, the water level will rise to balance. Storm surges can be 1–4 m in height. They can also be negative, with a combination of offshore winds and high pressure depressing the surface of the sea.

Tidal currents within harbors are greatly modified by the topography and bathymetry. They often channel themselves into separate bodies of water, each moving at different speeds, producing a tidal rip at their interface. At the mouths of rivers, the current flow is amplified by a low tide and, conversely, may be reduced or reversed by a high tide.

1.10 Rain, Snow, Fog, Spray, Atmospheric Icing, and Lightning

Rain, snow, and fog are primarily a hazard to offshore operations because of their limitations on visibility. Fortunately, with the advent of radar, electronic position-finding, Global Positioning System (GPS), and other sophisticated instrumentation, these no longer constitute as serious a constraint as in the past.

Rain can occur during general storms and also during the tropical rain squall, which moves through quickly and intensely. Appreciable amounts of water can enter through open hatches or other openings in the deck; if unattended over a period of time, they can adversely affect stability due to the free-surface effect. Proper drainage must be provided, adequate to remove the rainfall at its maximum rate.

Fog is of two types. The summer-type fog occurs when warm air passes over a colder ocean. Moisture condenses to form low stratus clouds. Usually, there is clear visibility at the water surface. The second type is the winter fog, with cold air over warmer water. Here the fog is formed at the surface. Yet 15 or 20 m up in the air, it may be bright sunshine. Rain, fog, and snow will affect helicopter operations, which requires visual observation for landing. Where fogs are prevalent, as, for example, the winter type off the east coast of Canada, then the helideck should be elevated as high as possible. In the Arctic and sub-Arctic, there is almost always dense low fog at the ice edge, where the cold air above the ice condenses the warmer moisture evaporating from the open sea.

Snow presents the additional problem of removal; else it will accumulate and freeze. When the air temperature is at or above freezing, saltwater can be used to wash the snow away, melting it as well as removing it by jet action. When the air temperature is more than a few degrees below freezing, mechanical means of snow removal may be necessary.

Spray is created by a combination of waves and wind. Waves breaking against a vessel or a structure hurl the spray into the air where it is accelerated by the wind. A great deal of water per hour can thus be dumped onto a structure or vessel. Drains are often inadequate, having been sized for rainfall, not spray. In the Arctic and sub-Arctic, drains may become plugged by freezing.

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On the Tarsiut Island Caisson Retained Island, an exploratory drilling structure in the Beaufort Sea north of Canada, the wave energy was concentrated against the caissons by the submerged berm. Waves reflecting from the vertical wall formed a clapotis peak (standing wave) which the wind picked up as spray. Also, waves running around the island were concentrated at discontinuities into a vertical plume, which the wind then hurled onto the island. Several hundred tons of water were blown onto the island during a six-hour storm. The resulting runoff eroded the surface. Tanks were damaged by the impact, and equipment operation was impaired. The spray plumes reached almost 30 m into the air, endangering helicopters engaged in evacuation.

Spray in more serious quantities than rain can flood hatches and even minor deck openings of barges—for example, penetrations which were not properly seal-welded—leading to free-surface effects and loss of stability. Spray can also be so severe as to prevent personnel from working on deck.

It has been recently recognized that spray has been given inadequate attention in offshore operations, especially in the sub-Arctic. Spray during those transition periods when the sea is still largely free from ice but the air temperature is well below freezing can produce dangerous accumulations of ice on vessels, booms, masts, and antennas. Atmospheric icing ("black ice") is a notorious condition occurring in sub-Arctic regions, where the air is moist but the temperature is low. The northern North Sea and the southern Bering Sea are especially prone to this phenomenon. Icing accumulates very rapidly on the exposed portions of the vessel. As it builds up, it adds topside weight and increases wind drag. Small vessels and boats are especially vulnerable. Atmospheric icing can be minimized by the application of special low-friction coatings. In the design of booms, masts, and spars of construction equipment that may see service in the sub-Arctic areas, use of a minimum number of widely spaced, round structural members is preferable to a larger number of smaller, closely spaced latticed shapes. Some specialized equipment is even provided with heat tracers to keep the surfaces warm and thus prevent icing. Note that icing and frozen spray may occur in the same regions and under the same circumstances. Icing, if it occurs, should be removed promptly, by mechanical means and/or saltwater jetting. This latter can obviously only be used if it is free to drain.

"Whiteout" is an atmospheric condition which occurs in the Arctic, in which the entire environment becomes white: sea, ice surface, land surface, and atmosphere alike. All perceptions of distance and perspective are lost, and vertigo may occur. Obviously, this is a serious problem for helicopter operation.

Lightning is associated with storms, especially the tropical rain squall. The typical construction vessel and steel platform are well equipped to discharge from the masts or derrick and ground through their hull or piling. However, personnel should not be exposed on high decks such as helidecks or platform decks under construction during a lightning storm. Concrete marine structures should be grounded by providing an electrical conductor leading to grounds or into the sea.

1.11 Sea Ice and Icebergs

Sea ice is found year round in the Arctic and from winter to midsummer in the sub-Arctic oceans such as the east coast of Canada, Bering Sea, Barents Sea, and Greenland Sea. Maximum southerly extent of sea ice generally occurs in February; the minimum extent, the retreat to the polar pack, occurs in August, leaving a circumpolar annulus of open water.

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Sea ice is a unique substance, being a material that is near its melting point. It is saline, thus freezing at about −2°C. It occurs in many forms. (Reportedly, the Inuit language contains more than twenty different words to describe the different types of sea ice.)

1. Frazil ice is ice that forms within supercooled water, especially when that water comes in contact with foreign bodies that enable amorphous ice to congeal. Thus it can clog intakes. It also increases the frictional effect on the hull of a vessel. It may act to reduce the impact force from ice floes and icebergs.
2. Sheet ice is the horizontal layer of sea ice that forms in relatively calm sea water, freezing from the top down.
3. Leads are formed when the thermal contraction of the ice causes ruptures in the sheet ice. Leads may also be formed by currents and wind.
4. Rafting occurs when one sheet of ice is driven up over the top of another.
5. Ridges (pressure ridges) are formed by a combination of refreezing of water in an open lead, then closure of the lead with rafting and crushing.
6. A compressive ridge is formed when one sheet is forced normal to the face of the other, that is, normal to the lead. Rafting and crushing occurs, forming randomly oriented blocks of ice (Figure 1.11). Such a ridge may be from 100–500 m in length, but will average about 200 m long. Ridges often follow each other in closely spaced increments of 100–200 m (Figure 1.12).
7. A shear ridge is formed when two sheets interact laterally along the lead, crushing up a more rectangular pile of lesser size.
8. A first-year or annual ridge denotes a ridge that has been recently formed and is less than one year old.
9. A multiyear ridge is one that has survived one summer and has had time for the meltwater from the sail to drain down through the bulk of the ridge, refreezing the broken blocks into a more or less solid block of high-strength ice. Multiyear ridges can be distinguished from first-year ridges by the fact that their blocks are rounded and refrozen together instead of just randomly heaped individual blocks.
10. Sail is the part of the ridge that extends above water. Maximum sail height is 10 m or less.
11. Keel is the deepest portion of the ridge. First-year keels may be hard ice and extend to almost 50 m depth: multiyear keels usually have a bottom layer of weak ice (due to contact with the water) and extend to about 30 m maximum depth.

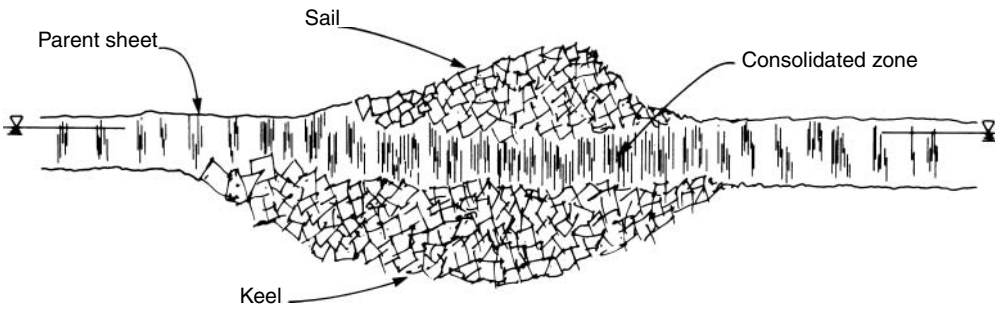

FIGURE 1.11
First-year pressure ridge.

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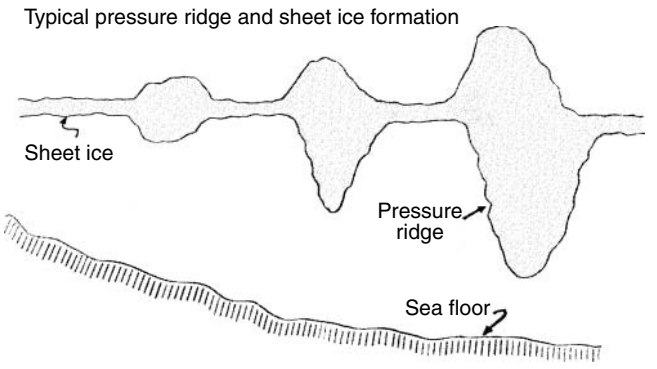

FIGURE 1.12
Typical pressure ridge profile.

Ice strength is influenced by the crystal orientation, salinity, temperature, and the rate of load application.

Rubble piles are found when sheet ice pushes up against grounded ridges. These rubble piles may reach 15 m in height and be quite extensive. They consist largely of broken blocks of ice. When storm surges occur during the subsequent summer, these large rubble piles, by now partially consolidated, float off, where they become "floe bergs," presenting many of the same hazards to navigation as small icebergs. Floe bergs are especially found in the northern Bering Sea. Rubble piles in the high Arctic may be formed by sheets being driven against ridges. In a compressive field, that is, a region in which the ice pack is exerting strong forces on the sheet and its ridges, a large rubble pile or series of rubble piles may be formed. These are known as a hummock or hummock field.

The polar pack itself consists primarily of multiyear ice. As it rotates slowly clockwise around the pole (as seen from above), it impinges against the shallow-water annual ice which forms each winter. This annual ice is relatively stationary and is called fast ice or shorefast ice. At the boundary between the fast ice and the polar pack, a shear zone is set up, which is highly dynamic, with much ridging and rubbing. The shear zone or stamukhi zone is usually located in 20–50 m of water depth, which unfortunately is also a region of high prospective activity related to the development of petroleum resources.

Large blocks of polar pack ice with embedded ridges may break off from the pack, becoming floes. Multiyear floes may be very large, several thousand meters in diameter, but usually are only 100–300 m in size. Their masses are very large. Since such floes may move with the general circulatory pattern, they may achieve velocities of 0.5–1 m/s.

The Arctic year can be divided into four seasons: the winter, when ice covers the entire sea; the summer, open-water season; the breakup; and the freeze-up. Within 20–30 km of shore, depending on the water depth, the winter ice is fast ice and hence can be used for access and transport. During the breakup, which may occur in June and July, the sea is filled with remnant ice floes and bits of ice. The percentage of ice coverage is then given either in tenths or octas, an octa being one-eighth. Maximum local pressure is usually about 4 MPa (600 psi), which is a value that is often used for design of barge and workboat hulls.

The summer open water varies with location and distance offshore. It may extend 300 km at times, but elsewhere may be only 20 km. Open water may last from as little as 11–90 days or so. Some years there is no open water at critical locations; this occurred in 1975 at Point Barrow.

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The fall freeze-up consists of thin surface ice forming, which initially is relatively easily broken up by icebreakers. Thus Canmar Drilling Company of Canada has been able to extend its working season for floating drilling and construction equipment in the Canadian Beaufort Sea to November by the use of icebreakers.

During the winter season, when the fast ice has reached a thickness of 2 m, ice roads and drilling islands can be built, enabling hauling of sand, gravel, and equipment. Airstrips can also be built on the fast ice. Dikes are built of snow and ice, and water is pumped up from holes in the ice and sprayed or flooded to thicken the ice. In the fast ice areas between the Arctic islands, insulating blankets of polyurethane have been placed on the early ice sheet, and then water pumped on top to progressively freeze. During early summer breakup, wave erosion undermines the edges and thermal fractures cause large shear cracks and break-off.

Early in the summer, when the rivers thaw and the snow in the mountains melts, the freshwater floods down over the shore fast ice. Flooded areas may be many miles in extent. Eventually the water thaws a hole in the ice sheet and large quantities of water pour through, eroding a cone in the seafloor to a depth of 7–10 m. This phenomenon is called strudel scour.

During the open-water season, floating equipment may be employed. Some ice fragments will still be encountered and present the hazard of holing of the hull of vessels. These ice fragments may have local "hard spots," for which their crushing strength exceeds the average values by a factor of two or more. Measurements on icebreakers indicate that the usual maximum local pressure is 4 MPa (600 psi), although, recently, extreme values of 6 MPa (800 psi) have been recorded. During this summer open-water season, one or more pack ice invasions may occur with multiyear ice floes moving at speeds of 0.5–1.0 m/s. At such times, all floating equipment must be towed to protected water for protection.

The Arctic pack shields the open water from storms. The fetch in the north–south direction is very limited. However, the fetch in the east–west direction can be several hundred kilometers. Therefore, the short violent cyclonic storms that spin off of the ice pack can kick up considerable seas, with an Hs of 2–3 m. Such storms are accompanied by storm surges of $+2$ to -1 m.

In sub-Arctic areas the ice duration is much shorter. The ice itself is less strong, due to the warmer sea temperatures. However, some areas may still experience severe sea ice problems. One such is Norton Sound, where during the winter, north winds break away huge sheets of recently frozen ice from the area south of the Seward Peninsula to float southward in the Bering Sea. New ice then forms and is in turn driven southward, rafting on the previous sheets.

The sub-Arctic regions of Davis Strait, and the Greenland Sea and Barents Sea are characterized by annual sea ice but, more dramatically, by the thousands of icebergs which are calved off as the glaciers discharge into the sea. These are blocky bergs. As they melt, they assume the dramatic shapes often seen in pictures, with pinnacles and saddles. Such icebergs may range up to 10 million tons; however, the normal maxima are about 1 million tons. About 70% of the mass of the berg lies below water and may extend out beyond the visible above-water portion, a phenomenon that led to the loss of the Titanic. Bergs can also roll over whenever their centers of gravity and buoyancy shift, resulting in a loss of stability.

Bergs are driven by a combination of wind, current, and wave drift force, and to a lesser extent by the Coriolis force. In their southernmost areas, especially, their paths are very erratic. Bergs may travel 20–40 km/day. They may safely pass a structure, only to return the next day (Figure 1.13).

Icebergs have been successfully lassoed and/or harpooned, using two tugs and Kevlar or polypropylene lines to tow them clear of an operating site. Considerable mathematical

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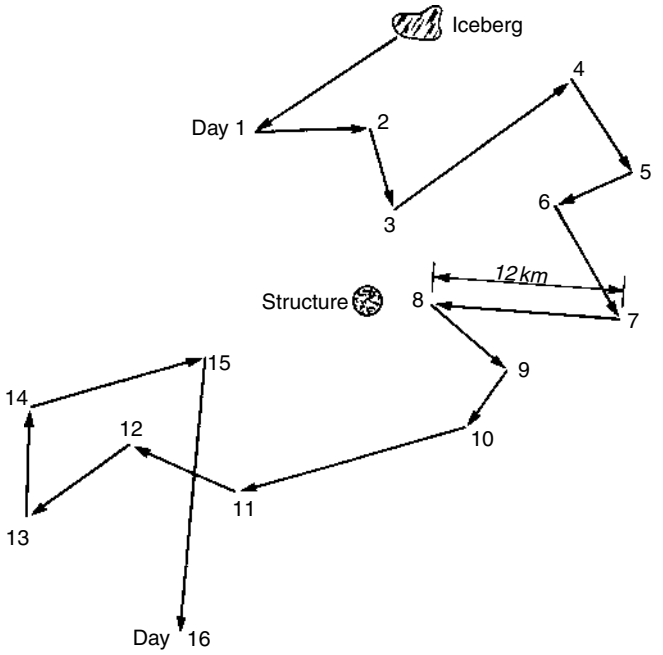

FIGURE 1.13
Typical trajectory of iceberg, illustrating unpredictability.

effort has been expended in developing computer programs to try to predict motions and to determine the optimum direction and extent of towing force to apply. The towing operation is often complicated by the fog, which frequently envelops the berg, and by uncertainties about its underwater profile.

Many berg fragments also clutter the water, especially in fall. Bergy bits are iceberg fragments from 120–540 tn. in size. They generally float with less than 5 m of ice above the water, extending over 100–300 m^2 in area. They are small enough to be accelerated by storm waves in the open sea, with the smaller bergy bits reportedly reaching instantaneous velocities up to 4.5 m/s.

Growlers are smaller pieces of ice than bergy bits. They are often transparent and usually extend only 1 m above the surface. They range in mass up to 120 tn. These smaller ice features are also driven by the waves and may follow their orbital motion just as might a similar-sized vessel.

One major problem when working in an iceberg-prone area is the ability to detect them. As noted above, visual observation is often limited by fog. Radar is unreliable; it may "see" a berg at a distance, and then suddenly lose it on the scope. Smaller features, largely submerged or even awash, may be missed altogether.

In winter the icebergs and fragments that remain unmelted are caught up in the sea ice, thus moving very much more slowly, but driven by the moving pack ice behind. Data are slowly being accumulated concerning iceberg types, masses, drafts, and profiles, to enable a probabilistic approach for design (Figure 1.14).

Tabular bergs are icebergs that break off a floating ice sheet extending out from a glacier over the water. They are the common icebergs of the Antarctic, where they may be 100–300 m thick and 100 km in diameter. Smaller tabular bergs break off the Ward Hunt Glacier on Ellesmere Island in the Arctic, where they become ice islands, trapped in the polar pack and circulating with it. They eventually ground and break into

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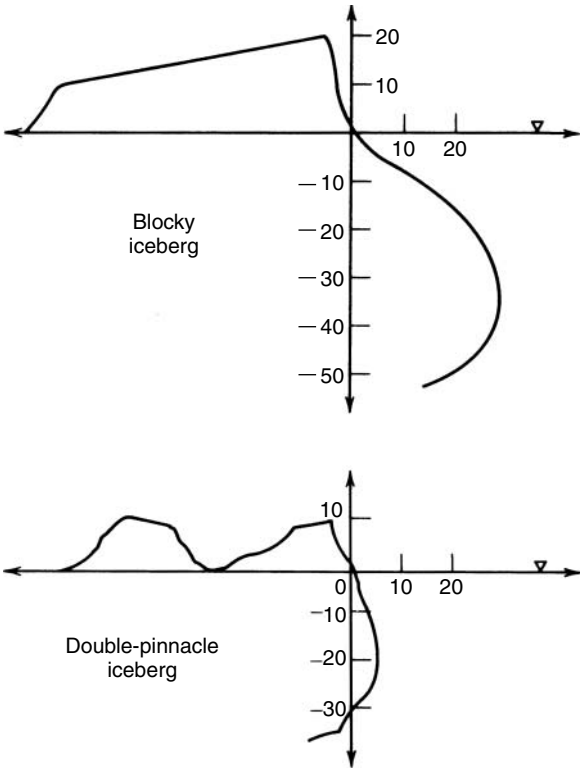

FIGURE 1.14
Typical underwater profiles of icebergs.

fragments, around 100 m in diameter and 40 m thick. Once every decade or so, an ice island is discharged into the Greenland Sea, east of Greenland.

Iceberg ice is freshwater ice, as opposed to the saline ice from the sea. Iceberg ice reportedly contains a significant amount of entrapped air. For icebergs that have drifted near their southern limits, their mass density is about 0.9 and their near-surface ice-crushing strength is 4–7 MPa.

1.12 Seismicity, Seaquakes, and Tsunamis

Earthquakes, although a significant load condition for the design of marine structures, are not normally a consideration during construction, because of their infrequent occurrence. The most serious earthquakes occur on active tectonic plate margins such as the ring which surrounds the Pacific. These vary in intensity up to such cataclysmic events as the Alaskan earthquake of 1964. The ground motions of these large earthquakes can cause significant structural response up to 200 km.

Earthquakes also occur within plates; these typically have a long return period, so that the area may have been popularly considered to be free from earthquakes. Their magnitude is usually limited to force 6 (very rarely force 7) on the Richter scale. But their effects spread for very long distances. Finally, local earthquakes of moderate intensity (force 4–5) are associated with volcanism, both subsea and on adjacent islands.

Earthquake accelerations are modified with distance, the higher frequency components being filtered out, so that only long-period energy reaches the peripheral areas. Many of

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the structures built offshore, especially those in deeper water, and on weaker soils have long natural periods (2–4 s). However, the amount of energy in this frequency range is usually well below the peak energy in an active-margin-type earthquake. In evaluating the effect of earthquakes on structures, the designer will consider added mass effects and the nonlinear soil-structure interaction. Current practice for the construction of major marine facilities is to consider the earthquake with a return period appropriate to the duration of the construction, e.g., five times the duration.

Although the above paragraphs would seem to apply primarily to the designer, they are of interest to the constructor because of three associated phenomena. The first is that of the seaquake, which is an intense pressure wave generated by the interaction of the seafloor and the water mass above. These short-period, high-intensity waves can be felt by ships, barges, and offshore structures for many hundreds of miles from the epicenter and usually are felt on the vessel as a shock similar to running aground or a collision. The overpressure generated is about equal to the hydrostatic pressure up to a depth of 100 m or so, and then remains essentially constant below that depth.

It is now being recognized that previously unexplained cases of ship damage may have been due to these seaquakes. While not currently considered for marine operations, they may have to be considered for deep-water construction in seismically active areas in the future.

Earthquakes also generate very long period waves, tsunamis, often mislabeled "tidal waves." The tsunami is so long and of such low height (100–200 mm) that it is rarely noticed on the open sea. When the energy is focused by the bathymetry and shoreline configuration, however, the kinetic energy in the wave is transformed into potential energy, resulting in disastrous lowering and raising of the water level. Such effects can spread completely across the ocean. The tsunami resulting from the Alaskan earthquake of 1964 did major damage in the Hawaiian Islands and California in addition to destroying the Alaskan cities of Seward and Valdez. South Pacific events have caused tsunamis in Chile and Japan.

Certain areas are known to be focal zones for tsunamis. Harbors and estuaries are especially vulnerable. The tsunami is characterized by an initial outrush of water from the estuary, followed by an inward massive surge, the so-called tidal wave.

Earthquakes have generated huge underwater landslides and turbidity currents, usually due to high pore-pressure buildup and liquefaction. Tsunamis also generate landslides near shore by the sudden lowering of the sea level. Earthquakes also generate landslides and rock slides on land, which, in places like western Canada, may slide into the sea, generating a huge solitary wave that may travel many kilometers.

1.13 Floods

Floods represent the major environmental phenomenon for rivers. They may be caused by torrential rains, such as those from the remnants of hurricanes that periodically cause deluges over the U.S. Midwest. These floods have apparently been amplified in recent years by the construction of levees that constrain the river channels and no longer allow dispersion over the adjoining farmlands and urban areas. Floods are also caused by warm rains that melt snowpacks upstream. The great floods of the Brahmaputra River System are caused by the warm monsoon rains falling on the snowfields of Nepal and Tibet.

An unusual flood is caused each fall in Lake George, north of Anchorage, Alaska, by the overflow of the lake waters over the glacial ice tongue, which dams the lake, thus discharging the entire into Cook Inlet within a few days. Such a phenomenon on a much larger scale is believed to have resulted in the torrential outflow of the Columbia

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River during geologic times, which produced the Grand Coulee and the Columbia River gorge.

The contractor working in the river is vitally concerned with floods. From historical records, the U.S. Corps of Engineers and others have produced probability statistics, such as the percentage of exceedance of various river elevations or discharge quantities. The constructor should be wary of these numbers because of recent physical changes that may cause these statistics to be only partially relevant, such as the construction of upstream dams or of levees.

Tributaries progressively feed more water into the main river. Where flood discharge occurs out of phase, their combined effect may be minimal. Where the phases overlap to augment the other flood discharges, the effect may be serious or even catastrophic.

The current velocity in a flood is affected not only by the quantity of discharge and the cross sectional area but also by the backwater elevation. When the Mississippi River is in flood, the backwater raises the river elevation and reduces the flow in the Ohio. Conversely, with a low river level in the Mississippi, the velocity increases and the river water elevations in the Ohio drop. Floods bring heavy sediments and floating debris, which may lodge against cofferdams, increasing the force on them. Floods currents cause scour and erosion, both general and local.

1.14 Scour

Scour undermines bridge piers. In the latter part of the twentieth century, more than 1000 bridges collapsed due to scour. Currents are accelerated past bridge piers and vortices form. Scour depths can be 2.5 times the diameter of the pier. Even weathered rock can be eroded if the currents are able to move gravel or larger stones during floods.

As noted earlier, scour is very difficult to model due to the scale effects when modeling sediments. Some approximate indications have been achieved using coal dust.

Scour is amplified when waves raise pore pressures, loosening the sediments, which are then swept away by its currents. A number of different scour patterns are considered, such as bend scour, where the currents are reflected at an angle, and confluence scour, where two currents join. The outer side of a bend in a river is deepest due to scour from the faster current.

Scour under bridge piers and box caissons is most serious structurally and can lead to catastrophic failure. Granular sediments, even small stones, are leached out by the flow of the current augmented by the raised pore pressure. Rocking under cyclic waves and the consequent liquefaction due to raised pore pressures has caused a number of seawalls to fail seaward.

Sheet pile cofferdams have failed due to deep scour at the leading corners or vortices at the rear corners.

Propeller-induced scour is common along quays, especially quay walls. The recent incorporation of bow thrusters on deep draft container ships has intensified this problem. Bow thrusters can also leach sand through open joints in the quay wall.

Riprap, articulated concrete slabs, and grout-filled bags, laid on geotextiles are used to prevent scour.

1.15 Siltation and Bed Loads

Rivers in flood can carry large amount of sediments, sands and gravels, silts, and clays. It is estimated that the Brahmaputra River in Bangladesh conveys 600,000 tn. of sediments

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per year. The Yellow River is notorious for its content of loess, eroded from the Tibetan plateau. Hangzhou Bay is heavily loaded by silt carried down the nearby Chiang Jiang (Yangtze). San Francisco Bay owes its thick mantle of recent bay mud on the seafloor to the sediments originally eroded by hydraulic mining of gold in the Sierra Nevada.

These sediments have become consolidated above the original seafloor and move as a colloidal mass. In tidal regions such as Hangzhou Bay, this bed load moves back and forth with the current.

The sediments drop out when the current slows below the critical value for that size; thus they tend to deposit in strata. They also drop out when they hit seawater's salinity.

Fast-moving flood currents, especially those occurring during episodic events, can move small boulders, concentrating them where the current diminishes. Lesser currents move cobbles in similar fashion.

1.16 Sabotage and Terrorism

The recent global spread of international destruction of facilities has required the introduction of an additional design consideration. These attacks are increasingly hard to predict as the enemy becomes more sophisticated and well-educated, and so he can be expected to search out plans and photographs to find the weakest spot in which to concentrate his attack. Fire and explosives are the chief weapons employed to date. Symbolic structures like bridges are favorite targets. Attacks on bridges may take place from deep underwater to the top of pylons. Both incident and reflected pressures must be resisted.

Less spectacular but still significant attacks may be carried out on projects during construction. The constructor must guard against this possibility.

Passive defense against such attacks can be achieved by providing structural redundancy and multiple load paths, by isolation, by shielding, by ductility, and by damping. Drainage can prevent the accumulation of petroleum products.

Complete safety against intentional attacks can never be achieved but structures can be rendered less vulnerable by proper design and construction, often without excessive additional cost. Sometimes temporary structures can be incorporated to give additional protection.

Section 4.4 addresses the increased impact resistance attainable by filling cells or grids of steel and concrete with rigid polyurethane.

1.17 Ship Traffic

The increase in size and speed of ships and their increasing number has led to a serious number of impacts with structures, to the point where statistically, they have become a major consideration in design and to a varying extent, a consideration in construction, also. Similarly, even the passage of a large vessel in a constricted harbor may cause damage to construction in progress. Tug boats at high speed can cause solitary waves that disrupt construction activities.

The constructor can attempt to get special restrictions on speeds in the area of construction. He may schedule his work so as to incorporate permanent ship impact barriers into temporary protection during construction. The constructor's own support craft (barges, tugs, and supply boats), are often the most likely to impact the structure under construction.

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Conversely, the constructor must take pro-active steps to avoid interference with navigation, especially that of ferry boats, where his responsibilities and liabilities are severe.

Temporary fenders and/or dolphins may be appropriate during construction.

1.18 Fire and Smoke

Some structures, especially submerged tunnels, including prefabricated submerged tunnels, are very vulnerable to fire, both during construction and in service. This also applies to shafts. Fire causes high-strength concretes to spall explosively, exposing the reinforcing and prestressing steel to high temperatures, causing them to lose strength rapidly. Fires can be caused by leaking hydraulic fluid. Non-flammable hydraulic fluid is recommended.

The spalling can be prevented, or at least minimized, by incorporation of plastic (polypropylene) fibers that vent the steam. Increased cover of concrete over the reinforcement increases the fire resistance.

Fire causes smoke, which is often the most serious consequence to humans. Proper planning and design for venting smoke is usually incorporated for service conditions but may need also to be considered for construction.

Safe refuge should be provided for personnel in the case of fire.

1.19 Accidental Events

a. Collisions
b. Dropped and swinging objects
c. Helicopter impacts
d. Explosions
e. Blowouts
f. Loss of pressure or ballast control
g. Unexpected flooding
h. Buoyancy loss due to subsea blowout
i. Loss of stability due to free surface inside.

Prevention, control, and mitigation:

a. A general arrangement to minimize events.
b. Protection of personnel—safety systems
c. Provision of a safe refuge
d. Protective energy-absorption devices
e. Blowout walls to relieve over pressure and barrier walls
f. Detailing of structural members for ductility (energy-absorption) redundancies
g. Protection of major structural elements
h. Essential safety systems

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1.20 Global Warming

Global warming is widely believed to be taking place. It is important that constructors recognize the probable potential effects that will occur over the design life. Chief among these are the following.

1. A general rise in sea level of about 1 m in the next one hundred years, with the consequent effects on the coast line and the run-off from low-lying lands.
2. Accelerated melting of glaciers and snowpacks leading to higher water volumes in rivers and increased flooding of lowlands.
3. The above combined with increased rainfall will increase river flow velocities, increase erosion upstream and increase deltaic deposits downstream.
4. Release of methane gas now frozen as hydrates in permafrost, thus giving positive feedback to global warming.
5. Changes in ocean currents due to melting of the Arctic ice sheets.

These will not only change the physical environment parameters but require extensive engineering works, especially in coastal areas.

I must go down to the seas again, to the lonely sea and the sky,
And all I ask is a tall ship and a star to steer her by;
And the wheel's kick and the wind's song and the white sail's shaking,
And a grey mist on the sea's face and a grey dawn breaking.

John Masefield, "Sea Fever"

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2

Geotechnical Aspects: Seafloor and Marine Soils

2.1 General

The seafloor of the ocean is highly complex, due to its geological history and the action of the various elements, especially in the relatively shallow waters of the continental shelves. These shelves vary in extent, depending on whether the margin is rising or slowly subsiding. Thus, the east coast of the United States has a very wide continental shelf, whereas that on the Pacific Coast of South America is very narrow. See Figure 2.1. Beyond the continental shelves are the slopes, averaging 4° of slope down to the abyssal plain. Submarine canyons, which cut through the shelf and slope, may have side slopes as great as 30°. They usually terminate in a fan on the deep seafloor.

The ice ages have had a very dramatic influence on the shelf areas. When the Wisconsin ice age was at its peak about 20,000 years ago, immense quantities of water were withdrawn from the sea, lowering the sea level as much as 100 m. This meant that the shelves were exposed to this contour: beyond the shoreline of that date, seas were shallower than at present. Rivers discharging from land cut troughs into that contour and a little deeper, which is why so many entrances to sounds and other large bodies of inland water are now approximately 100 m deep. On these coastal shelves, land erosion processes took place. The rivers were steeper and velocities higher; therefore, sedimentary deposits were coarser. As the oceans have risen, the velocities have been reduced, and finer sediments—sands and silts—have been deposited on the shelves opposite large rivers.

During this ice age, glaciers extended far out into what are now ocean areas, carving deep trenches such as the Norwegian Trench in the North Sea, Cook Inlet in Alaska, and the Straits of San Juan de Fuca between Washington and Vancouver Island. With the warming period in which the world now finds itself, the sea levels have been rising, slowly but inexorably flooding coastal areas, changing drainage patterns, and creating shoreline features.

Glaciers retreated, leaving morainal deposits. Shallow freshwater lakes were eventually inundated, trapping their land sediments. Rivers dropped their sediments sooner, creating deltas through which new channels were cut. Volcanic ash fell through the shallow waters, as did wind-blown (aeolian) sand. The loosely deposited sediments have been subject to mud slides, triggered by storm waves or earthquakes. Turbidity currents have removed millions of cubic meters of coastal sediments, which have flowed downward to form a fan on the abyssal plain, creating great submarine canyons during their flow.

The corals were actively extending or building new reefs as old ones were flooded. The skeletons and shells of billions of marine organisms slowly sank to the seafloor, to be

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FIGURE 2.1
Continental margins of the world.

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trapped in turn under new deposits. Successive reefs have become "caprock," consisting of dead coral, seashells, and sand, cemented together by lime secretions from marine organisms.

Sand dunes formed, migrated shoreward, were eroded, and were covered with the rising water. Major sand dunes in the southern North Sea and along the coasts of Holland have been submerged by the sea and now move back and forth as underwater sand dunes known as "megadunes."

The shallow-water deposits have been acted upon by waves, reworking and densifying the sands and silts. In the Arctic regions, near-shore silts have been subjected to cyclic freezing and thawing and to scouring by the keels of sea ice ridges. Off Greenland and Labrador, icebergs have reached down hundreds of meters to scour both sediments and rocks. Many such deep scours in rock can be seen on detailed underwater photos and acoustic images of the Straits of Belle Isle, between Labrador and Newfoundland. Fault scars have been identified with 5-m-high scarps in numerous offshore areas, and many more may exist, partially covered over by subsequent sediments.

Even today, an interesting process can be seen in Cook Inlet, Alaska. The steep fjordlike walls of Turnagain Arm have rock falls that deposit large boulders on the tidal flats. In winter, the high tide freezes so that cakes of ice form around the boulders. On a subsequent high tide, the ice raft floats away, carrying the boulder with it. As the raft moves into the southern portion of Cook Inlet, the increased salinity and warmer water melts the ice, dropping the boulder. This is one of the processes felt to be responsible for the many boulders found on the floor of the North Sea and Gulf of Alaska.

While the above is only a partial description of the many complex and interactive processes that have shaped, and continue to shape, the seafloor, there are certain specific problem conditions which have caused many constructional problems. These will be addressed in sections below.

Throughout these subsections, reference will frequently be made to the difficulties that geotechnical engineers have in obtaining proper samples and data in critical seafloor soils. While great progress continues to be made in improving sampling methods and in applying new techniques such as electrical resistivity, shear velocity, and geophysical methods, many types of seafloor soils continue to give difficulty. In many of these cases, the in-place strength will be greater than indicated by conventional sampling methods. With low-technology and crude sampling methods, critical constituents may not be recovered or identified. Construction engineers need to recognize these problems, so that they may adequately interpret the geotechnical reports and logs of borings and make appropriate decisions regarding their construction methods, equipment, and procedures. Failure to recognize these potential problem areas has led to a substantial number of cases of serious cost overruns and delays.

Most structures in the ocean extend over substantial areas. There may be significant variations in soil properties over this extent. Because of the cost and time required, it may not be possible to obtain a sufficient number of borings to show the true situation with its variations. There is a tendency to place undue emphasis on the few borings that may be available. Geophysical methods such as "sparker surveys" and a study of the site geology may help to alert the constructor to the range of soil properties that may be encountered.

The continental shelf is typically smooth and featureless, with a gentle slope. In contrast, the deep sea can be rugged and highly variable. The geologic process includes landsliding, active faulting, and seabed erosion. The topography of the deep seabed of the Gulf of Mexico is rough and irregular due to past and ongoing uplift of salt deposits. Vertical uplifts may be 2–4 m over a hundred-year period. Rocky seafloor caprock deposits have formed and gas hydrates have been trapped. Hydrate mounds 300–500 m across and 40 m above seafloor

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have been encountered. Biological communities form on the salt uplifts and at seeping hydrocarbons. The salt uplifts may be several kilometers across and up to 200 m above the surrounding seafloor. Fault scarps can be as steep as 45°. The pipeline crossing of the Strait of Gibraltar at a depth of 300 m encountered many abrupt scarps 10–20 m in height.

The properties of deep-water clays may be altered biologically and become much more sensitive than conventional marine clays. Brine lakes may have significantly greater fluid density. Chemosynthetic communities such as tube worms occur at seafloor vents and at hydrate mounds and faults from which hydrocarbons are seeping.

Estuarine and many harbor bottoms typically consist of very fine sediments such as muds, clays, and sands. The upper sediments are recent deposits, and therefore are loose and weak. They may be of variable depth, filling river and creek canyons that were formed during the post-glacial age when the sea level was lower. These loose fine sands and silts may be subject to flow slides during construction and dredging, apparently due to excess pore pressure building up on the uphill side, with failure initiated by a local vertical face formed during dredging, triggering instability. Flow slides may occur on slopes as flat as 1 on 6 or even flatter. It is very difficult to obtain undisturbed samples of loose sands.

River deposits vary from recent coarse sands to layers of gravel and cobbles, deposited during floods. Lake deposits are typically fine sands and clays.

2.2 Dense Sands

Sand deposits in the North Sea and off Newfoundland have been subjected to continuous pounding by the storm waves above. Pounding is perhaps an inaccurate term. What does happen is that the internal pore pressure in the upper layers of the sand is alternatively raised and then drained, only to be raised again. Pore pressure variations of 3.5 T/m^2 (35 kPa; 5 psi) have been measured. After millions of cycles, the sand becomes extremely dense, often with consolidation higher than can be reconstituted in the laboratory. Friction angles in excess of 40° may be found.

When sampled by conventional techniques, the sands are automatically disturbed; hence laboratory tests will often under-report their density and strength. Freezing may be necessary to obtain an undisturbed sample.

2.3 Liquefaction of Soils

Granular sedimentary soils, from gravel down to coarse silt, if saturated, are subject to liquefaction when strongly excited by earthquake. The pore pressure of the interstitial water rises faster than it can disperse, holding the particles apart so that they slide on a thin film of water. Liquefaction can also occur under the cyclic impact of storm waves and under the cyclic compression of the crushing of sea ice. The soil-water mass then behaves as a heavy liquid. This also occurs during construction operations with vibratory hammers or even under the repeated blows of an impact pile hammer. Liquefaction may be intentionally developed as a means of aiding penetration or unintentionally, without visible warning.

Once developed, until it is fully dissipated, it may be re-activated with less vibratory energy than it took to initiate, such as an aftershock. Liquefaction may also be produced by

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the impact of rapidly placed hydraulic fill on the previously placed sands, resulting in flow slides.

Loosely-packed sand is more susceptible than dense—an N_{sp} of 30 is a rough dividing line.

Liquefaction can be prevented and pore pressures dissipated by drainage, prior dewatering, and densification.

Under-consolidated clays may exhibit shear-strength degradation under cyclic loading.

2.4 Calcareous Sands

Calcareous sands exist through much of the warmer seas of the world—for example, along Australia's south and west coasts, in the eastern Mediterranean, and offshore of Brazil. They are sandlike deposits formed by the minute shells of microorganisms. In laboratory tests, they typically give indications of relatively high friction angles. Yet, their field behavior is far different from that of sands. High bearing values can be developed but only with substantial deformation. The friction on piling, however, may drop close to zero. The tiny shells crush and thus exert almost no effective pressure against the pile wall. Piles thus can be driven very easily, but develop little capacity in uplift. In one extreme case, the measured force to extract a pile driven 60 m into these calcareous sands was little more than the weight of the pile! These calcareous sands are relatively impermeable but grout under pressure may crush the grains at the contact surface, and hence lock to the fabric behind. Calcarenite, which is a calcareous "sandstone," and calcirubrite, which is calcareous conglomerate, while having initial rigidity and strength, are also subject to brittle crushing of their constituent calcareous sands, resulting in sudden loss of strength and large deformations. Uncemented calcareous sands are relatively impermeable, so that the potential for liquefaction exists.

Sampling techniques inevitably crush some of the grains. Scale effects appear to be important in any testing and evaluation. Methods of constructing suitable pile foundations in such soils are given in Chapter 8.

2.5 Glacial Till and Boulders on Seafloor

Boulders on and near the seafloor are typically found in sub-Arctic areas, where they may have been deposited by ice rafting. Another widespread process is that of erosion, occurring when the sea was shallower than at present. The weaker deposits eroded away, dropping the boulders down and concentrating them. A third process occurs in granitic soils, such as those of the east coast of Brazil and west coast of Africa, as well as those found in Hong Kong. As these rocks have weathered into residual soils, resistant cores have remained firm, thus becoming "boulders" formed in place. Borings made in such residual soils usually miss most of the boulders. When they do encounter one, they often erroneously report it as "bedrock." The practice in Hong Kong, after much distress due to excessive settlement, is to core 5 m below the top of "bedrock" in order to confirm validity.

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Boulders also exist in clay deposits. Some of these arose as morainal deposits from glaciers, discharging their bed load into shallow water muds, which have since been overconsolidated by subsequent advances of the glaciers. These are the boulder clays of the North Sea. Glacial till is a term used to describe these unstratified conglomerate deposits of clay, gravel, cobbles, and boulders found in many Arctic and sub-Arctic regions. The term is very nonspecific; some glacial tills have little binder and may be largely composed of gravel and cobbles, whereas others may contain large boulders. Perhaps the most difficult are the well-graded tills, with all the interstices filled with silts and clays, so that there is a very low percentage of voids. These deposits are usually heavily overconsolidated, resulting in a high unit weight and a structure superficially resembling that of weak concrete. Thus, unit weights have reached 2400 kg/m^3.

Geotechnical explorations, unless very carefully planned and carried out, often find only the finer sediments, and the samples will have been highly disturbed, resulting in the indication of much weaker material than that actually encountered. Wash borings bring back mud and sand. These tills are hard to drill; the material is very abrasive and hard, yet the bond is weak. However, high-pressure jets have proved effective in penetrating these tills. If relief can be provided in the form of holes or exposed faces so that the overconsolidation pressure may be released, then more normal construction activities such as dredging and pile driving may be carried out effectively.

Individual boulders and cobbles have not proved to be as difficult a construction problem as originally feared. A heavy-walled steel pile will usually displace them sideways through sedimentary soils. The same will occur when a large caisson with heavy steel or concrete skirts lands on a boulder. Clusters of boulders are a much more difficult problem. Where suspected, efforts should be made to locate them and remove them or relocate the structure accordingly.

Large boulders underneath the base of a caisson, for example, could exert a highly concentrated local force on that base. Large seafloor boulders have been successfully removed from platform sites in the North Sea by using trawler techniques, dragging the boulders clear. Another means used has been to place shaped charges to break them into smaller pieces. These boulders do not show up well on side-scan sonar or acoustic imaging. Work submarines (submersibles), taking video pictures using special lighting, have been the most effective in determining the presence and size of seafloor boulders. Below-surface boulders can be sometimes located by sparker survey and in shallower waters, by jet probes.

2.6 Overconsolidated Silts

Silts constitute one of the least-known types of soils, lying as they do in the size range between sands and clays and exhibiting properties different from both. Silts are typical of Arctic and sub-Arctic regions, although they also exist in temperate climates. They have been encountered in the Beaufort Sea, in Cook Inlet, in the St. Lawrence Seaway, and off the coast of California, this latter case being a weak siltstone.

One unique property is their overconsolidation. The overconsolidation of clays, for example, is usually due to their having been subjected to intense loads from overburden or ice (glaciers), which have subsequently been removed by erosional processes or melting. Silts, on the other hand, have frequently been found to be overconsolidated even when there is no prior geologic history of burial. Various hypotheses to account for this include freeze-thaw cycling in shallow water, wave action, and electrostatic

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attraction. Regardless of cause, these overconsolidated silts are extremely dense and resistant to penetration, pile driving, and dredging.

Sampling, and even in-situ vane shear tests, almost always disturb these silts. In many conventional borings, the silt will be reported as "mud." These silts are typically very abrasive to drills. Yet, they break up readily under the action of high-pressure water jets. Paradoxically, some of these silts remain in suspension for long periods, but when they do settle out, they become very dense once again. To the contractor they pose problems similar to a very soft rock that degrades when exposed to water.

2.7 Subsea Permafrost and Clathrates

It is now known that relic permafrost extends out under the Arctic seas. It has been trapped there since the ice ages and is now covered by more recent sediments and seawater. The Arctic seawater, with its shallow-water temperatures ranging from -2°C to $+8$°C in summer, has been slow to thaw the ice from the top. The overlying sediments, with temperatures about -1°C, act as effective insulation to the permafrost. Permafrost also underlies the beds of many Arctic and some sub-Arctic rivers and inlets.

The top zones of subsea permafrost may be sands. These may be only partially ice-bonded, due to the gradual thawing process. In other silt and clay deposits, ice lenses and frozen silt lenses may occur. Deeper down, fully bonded permafrost may be encountered. Typical depths below seafloor to the upper level of permafrost are 5–20 m. Underwater permafrost is generally not continuous.

Permafrost presents obvious problems in construction. Steam jets and high-pressure saltwater jets have been used to aid pile driving and excavating. Drills have occasionally been used, but the process is slow unless augmented by jets. When permafrost thaws, consolidation settlements occur.

Clathrates are methane hydrates, which are a loose crystalline bond of ice and methane, stable under appropriate combinations of temperature and pressure. They exist under the seafloor in the deep sea of temperate zones and at moderate depths of several hundred meters in the Arctic. When penetrated, they turn to their gaseous phase with a five-hundredfold expansion. They represent a potential problem for drilling of oil wells and for well casing but are generally too deep to affect construction.

2.8 Weak Arctic Silts and Clays

One of the worrisome problems of Arctic offshore construction is the presence of strata of apparently very weak silts and silty clays. Near the seafloor, these are probably due to their recent deposition and the constant plowing by sea ice keels. Much harder to explain are very low shear-strength measurements under 5–20 m of overlying stronger material. It is now known that Arctic silts are anisotropic, having much greater strength for bearing than shear. One potential explanation for the extremely low strengths is that the released water and methane gas from the thawing of subsea permafrost zones have been trapped under the surficial layer of impervious silts, creating a high internal pore pressure, which has broken down the structure of the silt. This phenomenon can also lead to serious sample disturbance.

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2.9 Ice Scour and Pingos

The Arctic and much of the sub-Arctic seafloor has been scoured by sea ice ridge keels and by icebergs. Turning first to the Arctic, these scours occur as a regular yearly event in water depths from 10 to 50 m. The keels plow erratic furrows, perhaps a kilometer or more in length, to a depth of 2 m normally, up to 7 m at the extreme. While at any one time these scours are directional, the directions change, depending on currents and winds. Thus the entire seafloor in the applicable depth range appears to have been regularly reworked. New furrows are typically 2 m deep and perhaps 8 m wide, with small ridges pushed up at the top of the slope. Furrows rapidly fill in with soft unconsolidated sediments, many of them derived from the plowing action. Walrus plow the shallow seafloor of the Bering Sea to uncover clams, thus creating new loose sediments.

There is a debate, concerning the ice scour marks in deeper waters, as to whether they are current or relics from a time when the seafloor was lower. Icebergs can also scour the seafloor but at greater depths. The individual scour marks are deeper and longer due to the greater energy in the berg. Icebergs can even produce surficial scour on exposed bedrock, such as in the Strait of Belle Isle, between Newfoundland and Labrador.

Another seafloor phenomenon occasionally encountered in the offshore Arctic is that of subsea pingos, which are hillocks raised in silty clay soils by progressive frost heave. They are a common feature of the onshore coastal landscape. When encountered offshore, in shallow water, they are believed to be relics from a time when the sea level was lowered during the ice ages. When ancient pingos have thawed and collapsed, they have left small craters in the seafloor.

2.10 Methane Gas

Methane gas occurs at shallow depths in deltaic sediments that contained organic matter. This can be released by geotechnical borings or even by pile driving and has caused minor explosions and injury to personnel. Methane gas also occurs at shallow depths in Arctic silts, which are not organic. It has been postulated that this gas was originally bound up in the subsea permafrost, perhaps as methane hydrates (clathrates), and has been gradually released as the permafrost warmed. The released gas has migrated upward and been trapped beneath the near-surface silts. The resultant rise in pore pressure significantly reduces the shear strength of the silty clay soils.

The presence of methane gas can be ascertained by seismic refraction and by drilling ahead with a small diameter hole. The presence of the gas can then be detected by special gas metering devices.

The result of a sudden large release, as by a large-diameter drilled hole or even a driven tubular pile, can be an explosion or fire at the top, with possible injury to personnel or in extreme cases, a loss of buoyancy due to gas bubbles in the water, which have caused drill rigs to sink.

2.11 Muds and Clays

The end weathering process of many rocks results in the formation of clays, which constitute the large bulk of many deltas. These clays consolidate under the overburden of later

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TABLE 2.1

Correlation between SPT and Stable Dredged Slope in Cohesive Soils (Clays)

Soil Consistency	Unit	Very Soft to Soft	Soft to Medium Stiff	Medium Stiff to Stiff	Stiff to Very Stiff	Hard
NSPT	bpf	0–2	2–4	4–8	8–16	16–32
Typical depth[a]	ft.	0.1–10	15–25	25–40	40–80	80–100
Shear strength (from NSPT of from unconsolidated, undrained laboratory tests or from field kPa vane shear tests.)	Ksf	0.25	0.5	1.0	2.0	4.0
(2000 psf)		(250 psf) (4000 psf)	(500 psf)			(1000 psf)
		12	25	50	100	200
Stable slope[b]		Requires special consideration	4:1	1½:1	1:1	3/4:1
Need to consider surcharge?		Yes	Yes	Possibly	Normally not required	

[a] The depth of normally consolidated clay associated with the shear strengths shown.
[b] This is the ratio of horizontal distance to vertical height.

deposits. These clays are highly impermeable and cohesive. They are often anisotropic, with greater horizontal permeability than vertical. Often there are thin lenses or strata of silts and sands embedded in the predominant clay body. Clays usually contain organic material. The behavior of clays is determined by their particle shape, mineralogical composition, and water content. Thin flat plates similar to montmorillonite possess dynamic lubricating qualities. Other types of clays may be sticky, "gumbo," plastic, or firm.

Mud is a term used to denote very soft, highly plastic, recently deposited clays. Typically, marine clays show shear strengths ranging from 35 kPa (700 psf) down to 14 kPa (300 psf), although some surficial muds may have only 1–2 MPa. These qualities of clay and mud present a number of problems to the constructor. Among them, the following deserve special attention (Table 2.1).

2.11.1 Underwater Slopes in Clays

Clays tend initially to stand at relatively steep slopes when the excavation depth is limited. The buoyant weight of submerged clay is much less than the air weight; hence, the driving force leading toward failure is much reduced compared to the same excavation in the same clay above water. With time, however, the clays strain (creep) and lose strength, failing in a typical curved shear plane. Their stable slope underwater may range from 1:1 (horizontal to vertical) to as flat as 5–1. Increased depth of excavation increases the tendency for slope failure; deep cuts should be stepped down with one or more berms.

A surcharge on the dredged slope increases the driving force and may lead to sudden, large-scale failure. In practice, surcharge often arises as dredge disposal, especially when

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the excavation is performed by a bucket-type dredge or when a levee is being built to contain a settling pond for hydraulic disposal. If the spoil pile is located so that its toe is farther back from the edge of the trench than the depth of the trench, the surcharge effect is usually negligible.

Clays are very sensitive to shock, such as that from nearby pile driving. The dropping of a large bucket-load of dredged material onto the top of a trench-edge spoil pile may trigger slope failure.

Waves cause cyclic strains in clay slopes. Strong continued wave action may lead to cumulative strain in the clays, a process similar to fatigue, leading to a reduction in shear strength of up to 25%.

Cyclic pounding of underwater seabed slopes as flat as 1° or 2° from hurricanes and cyclones can build up pore pressures and cause large scale mud slides, leading to failure of platforms and rupture of pipelines.

2.11.2 Pile Driving "Set-Up"

Clays typically are penetrated rather readily by a pile under the dynamic blows of an impact hammer. Their short-term cohesion against the side of a pile is low. However, with a short period of rest, the soil will bind to the pile with its full cohesion, a process called "setup" or "set." Thus in driving piling in clays, when a stop is made to splice on another section, the blow count will jump up considerably when driving is recommenced. On occasion, it will not be possible to get the pile moving again. A portion of this increased resistance usually remains as permanent resistance.

2.11.3 Short-Term Bearing Strength

The constructor is often interested in the bearing strength of clays, such as their ability to support a jacket on mud mats, for example. In the typical deep-clay deposit, the unit bearing value at the surface may be five times the unit shear strength, while at a depth below the surface, it may be nine times the unit shear strength.

When thin layers of clay overlie much stronger soils, the shear failure mechanism is inhibited and higher values may be sustained. This is also true when the base of the structure is large, whereas a concentrated local load will penetrate. Bearing strength may be increased by placing surcharge material around the loaded area to confine the material and force a larger area of clay to resist the shearing forces.

2.11.4 Dredging

Clays may present problems in dredging, due to their cohesive nature. In hydraulic dredging, clay balls may form. Flow will not be uniform. In bucket dredging, the clay may stick to the bucket and not discharge readily.

Once clay is in suspension, it becomes colloidal in behavior, and the discharge will be highly turbid. It requires a considerable length of time in relatively still water for clay particles to drop out of suspension. Where permitted, chemical flocculants, even seawater, will cause more rapid flocculation and dropping out of suspension.

2.11.5 Sampling

The sampling of muds and clays presents special problems to the geotechnical engineer. Physical disturbance can lead to remolding and a temporary loss of strength. Sample

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disturbance is often responsible for lower strengths being indicated than exist in place. Reference was made to the standard penetration test. Other tests are the field vane shear test and cone penetrometer (CPT) test. All such tests require correction factors for depth, strain rate, and the anisotropy of the clay.

2.11.6 Penetration

Structures designed to sit on the seafloor often have dowels or skirts, which are required to penetrate into the soil. Caissons of the bridge pier type and large-diameter cylinder piles also must penetrate to prescribed levels. The resistance to such penetration is a combination of bearing failure under the point or edge and side shear, the latter dominating. Bearing is greatly increased when the internal tip plugs, due to the internal shear. Recent studies show that internal plugs of clay may not form in large diameter (greater than 2 m), open-ended tubular piles when driven by rapid blows. Fatigue and remolding lowers the effective shear strength.

In some clays, an enlarged tip may create a temporary annulus around the outside and reduce the side-shear resistance, and consequently the total resistance, even though the tip-bearing area is increased. Short-term cohesion is often lower than long term, and any dynamic process usually results in local remolding and reduction in shear strength.

2.11.7 Consolidation of Clays; Improvement in Strength

Clays may undergo significant improvements in strength if they are drained. The reduction in water content causes consolidation, increase in shear strength, and generally beneficial gain in all properties. Such consolidation may be accomplished by surcharge (overburden), by provision of drainage (wick drains or sand drains), and by time. Due to this consolidation process, a heavy structure seated on clay soils will experience a gradual improvement in strength of the foundation. See Chapter 7.

2.12 Coral and Similar Biogenic Soils; Cemented Soils, Cap Rock

As skeletons of dead marine animals that live and extract calcium carbonate from the sea, coral and other calcareous deposits initially have a complex structure, which hardens through age. Storms wear off the weaker portions, exposing the older, harder portions, upon which new coral can grow and may become mixed with the old coral and embedded in it. New sediments may interbed with the coral. These may either be calcareous sand or silicious sand deposited by wind or shoreline transport.

Such rocks are usually highly, if irregularly, stratified. They are usually very hard, almost flintlike in small pieces, but having large voids, formed in many cases by seashells, so that the rock is very brittle. Cap rock exists in many tropical and subtropical areas; it is a recent near-surface coral deposit, usually containing embedded sand grains. It is called by many local names; for example, in Kuwait it is known as "gatch."

Other coral layers have been progressively submerged under the rising sea, so that it is not unusual to encounter numerous strata ranging from a recent coral reef or caprock on top, down through various strata of sands, coral, calcareous silt, and limestone. These soil profiles are highly variable from one location to another. It is not unusual to find a very weak lens of silty sand beneath a l-m-thick limestone stratum. Coral and limestone can be dredged mechanically with very heavy equipment, especially if it is possible to break through the overlying hard stratum. In other areas, the caprock may be too hard and

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thick. Off the east coast of Saudi Arabia, rock-breaking chisels are necessary. Off the coasts of the islands of Hawaii, surface blasting, using either shaped charges or large charges of powder, will frequently be sufficient to break the surface stratum downward into the unconsolidated sands below. Drilling and blasting are usually only effective if the charge can be located in the hard stratum. If placed below, it will result only in forming large, undredgeable slabs; if above, it may not break the stratum at all, but just blow sand and water into the air. Drilling of holes (e.g., for piling) may present major difficulties due to loss of drilling fluid in porous strata or voids.

Cemented and partially cemented sands are quite commonly found in subtropical and tropical seafloors. Usually, the cementing material is the calcareous deposit from marine organisms. These strata typically are highly irregular in their degree of cementation. They are often roughly stratified. Since the cementation process has usually occurred over long periods, the cemented zones may have been exposed to erosion when the sea level lowered, to be later filled with looser, uncemented deposits as the sea again rose.

2.13 Unconsolidated Sands

In many offshore areas, there are very extensive accumulations of sands, in some cases due to longshore sediment transport of sand discharged from rivers, in other cases due to ancient sand dunes, such as are found in the southern North Sea. Many river beds are comprised primarily of sands. Sands are extremely hard to sample during geotechnical investigations: they will almost always be disturbed by the sampling process. It may be necessary to employ special techniques, such as freezing, in order to get undisturbed samples; the geotechnical reports must be very carefully evaluated. These cohesionless materials are very mobile and sensitive to disturbance by construction activity. Under earthquake, storm wave pounding, or dynamic construction activity such as pile driving, the sands may locally liquefy, turning momentarily into a heavy fluid. Surface sands are readily disturbed by waves, which increase the internal pore pressures, causing the sand grains to tend to rise up, making them easily removed by currents. It is this that makes them so susceptible to scour and erosion.

Scour from wave action can occur when the water depth is less than half the significant wavelength. Whenever the depth is less than one-fourth the wavelength, substantial scour in sands is likely. Bottom currents, whether wave induced or from other causes (see Section 1.5) can move sand, especially if it is periodically raised by the pore pressure gradients induced by waves. The resultant scour holes tend eventually to stabilize at a condition where the rate of sand infill matches the erosion rate.

River and tidal currents adjacent to structures produce scour, especially around the corners and underneath a structure. Blocking of the river by equipment and structures such as cofferdams amplifies the velocity and scour potential.

The excavation of underwater trenches in sands is typically rather difficult and complex due to the varying densities of the sands. However, stable side slopes can be achieved in the absence of severe currents and wave action at shallow depths. NSPT values provide a guide. NSPT values obtained underwater must first be multiplied by 1.12 to account for the effect of submergence. In cohesionless sands, NSPT values must also be corrected for depth. See Table 2.2 and Table 2.3. SPT is the standard penetrometer test and NSPT is the measure of the density.

With the NSPT values so corrected, Table 2.3 can be used to evaluate the approximate behavior of underwater dredged slopes in sand. Where loose surficial sands overlie either

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TABLE 2.2

Correction Multiplier to Apply to Measured SPT Valves (as Adjusted for Submergence) to Account for Overburden Pressure at Various Depths

Depth Below Seafloor (ft.)	Correction Multiplier to Give Value under Standard Confinement Pressure
2	2.3
5	2.0
10	1.8
15	1.5
20	1.3

clay or denser sands, the joint action of the waves and bottom currents may readily move these sands into the trench, infilling it.

Medium dense sands in harbors can usually be dredged to a slope of 2:1 (horizontal to vertical). This is for a depth of 10–12 m and with a bank surcharge not higher than 4 m above water. Deeper dredging and/or higher surcharges lead to instability. Offshore trenches in water depths greater than 3 or 4 m have been successfully dredged with a slope of 2H:IV. However, there will be some raveling and infill, especially if the waves or tides angle across the trench.

The presence of silts and mica may significantly reduce the underwater stability of a dredged slope. On the Jamuna River in Bangladesh, the presence of up to 25% mica required side slopes of 5H:IV to 6H:IV for stability. Mica can also affect pile driving, reducing the skin friction well below that of pure siliceous sands of the same density. Sampling processes often miss the mica content because of washout of the particles.

Sands from the side slopes, especially the surficial loose sands, tend to move downward into an underwater excavation. If there is a prevailing current, the excavation will tend to fill in on the upstream side and erode on the downstream side. Thus a trench transverse to a current will tend to "migrate" downstream.

Trenches and other underwater excavations in relatively shallow ocean waters (50 m or so) have a diffraction effect on the waves, causing them to refract toward the edges and to leave the center of the excavation calmer. A long trench normal to the shore will not only be calmer, with the waves refracting to either side, but will form a channel for a return current, tending to clean loose sediments from the trench.

Sands also migrate along the shore, in the direction of the overall net current. This is the well-known longshore transport. During summer, when wave energy is reduced, sand accumulates near shore. In winter, when wave energy increases, the sands move further offshore, forming a bar with a typical depth of 10 m.

TABLE 2.3

Correlation between SPT and Stable Dredged Slopes in Cohesionless Materials (e.g., Sand)

	Very Loose to Loose	Loose to Medium	Medium to Dense	Dense to Very Dense
Corrected NSPT	0–4	4–10	10–30	30–50+
Relative density	0.15	0.35	0.65	0.85–1.0
Moist unit weight (lb/ft.^{3})	70–100	90–120	110–130	120–140
Stable slope	4:1	2.25:1	1.75:1	1.5:1

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In deeper waters, such as those of the northern North Sea, relatively thin lenses of surface sand, up to 1 m deep, occur over the stiff clay seafloor. Thin lenses of sand, although unconsolidated, form zones of high resistance, and therefore of high local bearing pressure, on the flat base of any large structure. These are the "hard points," which may develop local pressures as great as 300 T/m^2 (3 MPa; 66 ksf). It is partly because of these hard points that a detailed bathymetry is required at the site of such a structure.

The construction of a structure on sands modifies their behavior. Wave energy acting on the structure is transmitted to the foundation, increasing the pore pressure. Due to its cyclic nature, the pore pressure may be progressively built up until local liquefaction occurs under the edge. This eventually leads to a loss of material under the edge and the tendency for the structure to rock, aggravating the problem. This is why concrete caissons used as coastal seawalls usually fail outward under wave attack: they have lost the sand under their toe.

Coastal sands move laterally under the action of the prevailing current. Structures that interfere with that movement cause the sand to build up on the "upstream" side, and erode on the downstream side, as discussed in Section 1.2.4.

2.14 Underwater Sand Dunes ("Megadunes")

Under the action of strong currents, such as those found in the English channel and at the mouths of major rivers of South America and Southeast Asia, the sand bed may be formed into waves (sand dunes). These dunes move just as their land-based counterparts do, eroding from the back, redepositing on the front. Typical maxima heights are 3–15 m, lengths to 100 m. Thus in planning installations in such areas, it may be necessary to dredge the dunes down to, or below, the level of the troughs; otherwise, the structure or pipeline may end up exposed above the seafloor.

Sand dunes also form in the beds of rivers and estuaries, moving downstream with the prevailing or dominant current. Sand waves up to 10 m in height move downstream in the bed of the Jamuna River in Bangladesh at a rate of a few kilometers a day.

2.15 Bedrock Outcrops

Bedrock outcrops present a highly site-specific problem. In deep water or when the outcrop is partially covered with sand, such outcrops pose the problem of irregularities and hard points that are difficult to identify and map.

Rock outcrops may be fractured and weathered irregularly, so individual drilled shafts may need to extend to different lengths in order to found in sound rock. On the other hand, the weathered material near the surface facilitates the seating of casings for subsequent drilling and the penetration of pile tips. Weathered rock may give lateral support to piling.

There are several alternative methods by which such outcrops may be treated:

1. Softer rocks may be dredged to remove all significant irregularities and backfilled with properly graded gravels or crushed rock. Alternatively, a pipeline plow may be used to rip the irregularities.

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2. Harder rocks may require underwater drilling and blasting to enable them to be dredged. In the case of isolated high points of rock, shaped charges may be used.
3. Drilled shafts may be constructed, each penetrating to sound rock.
4. A blanket of rock (underwater embankment or berm) may be placed to cover all irregularities to a sufficient depth (e.g., 3 m) to give a uniform bearing.

Pipelines and structures may be designed to span between irregularities. Suitable anchors may be required to prevent movement, abrasion, and pounding under the action of waves and currents.

Drilling of piling and shafts into rock outcrops may present difficulties in getting the hole started, especially if the rock is steeply sloped, highly irregular, or covered with hard but fractured material. Rock outcrops that have been exposed in ancient geological times will have been weathered. Weathering may have been quite variable within the formation, proceeding down fractures and other discontinuities, so drilling may progress through hard rock into softer material below. To initiate drilling on a hard rock outcrop, the casing must be seated far enough to seal off the return flow from drilling and to prevent run-in of sands under the tip of the casing. The best method is to seat the casing on the surface, then use a down-the-hole or churn drill to obtain 300 mm or so penetration, and then reseat the casing by driving before drilling. Placing a clay blanket may also be a means to enable sealing of the casing.

In some weak rocks, such as mudstones, it may be possible to install piling by driving if there is a relief of the confinement. For example, a pilot hole, or holes, of relatively small diameter may permit a larger-diameter pile to be driven.

Karstic limestones are characterized by solution cavities that have subsequently been filled with silts, clays, or sand. Foundations of major structures either have to extend below them into sound rock or must be designed to span over them. While grout may often be pumped into such cavities, it cannot fully displace the infilled material. Because of their erratic and uncertain configurations, as well as connections to adjoining cavities, use of techniques such as jet-grouting are of doubtful reliability, requiring confirmation by borings at closely spaced intervals, not only directly under the bearing area but also in the zone surrounding the footprint of the structure.

A somewhat similar problem is posed with all basic rock types that have relic faults filled with gouge or fractures along which significant weathering and degradation have occurred. These are often near vertical and therefore difficult to discover. The weak material or gouge zone may be from 100 mm to 1 m in width. Typically, the depth of the weathered seams is limited to that which was exposed during the lower sea levels of the ice age.

2.16 Cobbles

Some seafloor areas that have been subjected to high currents or wave action are "paved" with cobbles, closely packed, with or without sand in the interstices. These cobbled areas are difficult to excavate because most conventional equipment has difficulty getting a "bite" into the material. Once a trench or hole is begun, the slopes may become very loose and unstable, taking a rather flat angle of repose, 2:1 or even flatter, for example, depending on the current. Their rounded surfaces give a low angle of friction. If a drilled shaft is to be constructed, it may be necessary to grout below the tip of the casing in order

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to stabilize the cobbles so they can be cut. Percussion drilling (churn drill or down-the-hole drill) may prove useful.

Sampling may require large diameter (e.g., 600-mm) holes.

2.17 Deep Gravel Deposits

In sub-Arctic areas, deep deposits of gravels are found. These have been eroded by glaciers and rivers from the mountains and discharged into the sea to have a minimum of stratification and fines. Although composed of sound material, they often develop very low frictional resistance, due to their rounded surfaces and high void ratio. Thus they are quite unstable and dredged slopes will be very flat, 3:1, for example.

Piling driven into such material has typically failed to develop the desired skin friction. End bearing may also be less than expected, due to the large void ratio. It has proved necessary in many cases to provide enlarged tips or deeper penetration of piles in order to obtain adequate bearing. Such gravel deposits are inherently difficult to sample by conventional means; any samples are more or less completely disturbed, and it is difficult to determine the degree of packing (consolidation). Freezing may be required.

2.18 Seafloor Oozes

Seafloor oozes are relatively thin, flocculant layers, usually of organic sediments lying on the surface of the seafloor in many of the deep basins of the seas. They are readily displaced, so that conventional objects sink readily through them. They create turbidity clouds upon the slightest disturbance and thus interfere with vision, positioning, and control. In many cases, they are so soft that a diver or an object (e.g., a Remote Operated Vehicle [ROV]) will sink through them just as though they were a fluid of slightly heavier density than water.

Seafloor oozes do not normally show up on echo-sounding or sparker surveys and may easily wash out of sampling tubes; hence their presence may not always have been ascertained prior to construction. For example, a 25-m layer of silty ooze was discovered on the floor of a Norwegian fjord by sampling with a special grab bucket, whereas conventional acoustic soundings had penetrated through it without reflection. A special basket has had to be developed in order to support a vane shear sampling rod in weak sediments of the deep seafloor.

2.19 Seafloor Instability and Slumping; Turbidity Currents

The offshore areas of principal interest to the petroleum industry are great sedimentary basins. Although many are ancient deposits and relatively stable, others are still active deltaic areas. The great freshwater rivers—the Mississippi, the Amazon, the Orinoco, and the Congo—are transporting huge quantities of silts and clays in colloidal suspension. Contact with the saltwater causes flocculation, and the highly dispersed soil particles settle to the seafloor. Periodically, huge blocks of these recent sediments slump off and flow seaward. This process is prevalent even far out to sea on the sides of submarine

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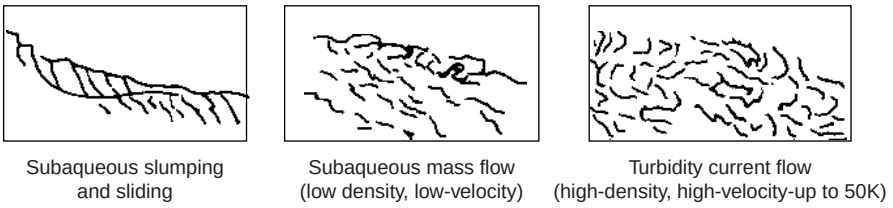

FIGURE 2.2
Seafloor instability phenomenon. (From U.S. Navy Civil Engineering Laboratory Report R 744-1971, Port Hueneme, CA.)

canyons such as the Baltimore Canyon of the U.S. East Coast and the outer edge of the continental shelves. Evidence of widespread slumping exists near the break of the continental shelf of the Alaskan Beaufort Sea (Figure 2.2). In the Gulf of Mexico, they occur as mud slides.

Underwater sand deposits may also be very loosely consolidated. If the internal pore pressure is raised by any of several mechanisms, the sand turns into a heavy liquid. These underwater flows and turbidity currents have occurred in both sands and clayey silts. Some of these occur frequently, and are the mechanism by which shore sands are fed down submarine canyons to be deposited in the fan at the bottom of the continental slope. During this downward flow, they erode the canyon itself. Others are more infrequent, being triggered by an intense storm or by earthquake. In clay areas, these slides are aggravated by entrapped methane gas in the silty clays. These failures often occur on very flat slopes, which superficially would appear to be stable.

2.20 Scour and Erosion

Currents and waves can cause serious erosion around structures and construction on the seafloor. In the case of cohensionless materials, they will erode them but at the same time fill in, whereas in cohesive material such as clay, it can more gradually erode the soil, but it will not normally refill.

A trench dredged across the cement in sand will tend to migrate slowly downstream, eroding on the downstream side as well as rounding off the upstream edge. Scour of clay banks on the outer bends of rivers often proceeds by undermining and collapse.

Scour under caissons has been especially serious in the case of caissons. As the base nears the bottom, the current speeds up underneath, sometimes eroding material as fast as the caisson is sunk.

Scour around sheet pile cofferdams and bridge piers occurs at the upstream and downstream corners, sometimes extending far downstream.

When large stones are placed directly on sands, the sand is leached out by the current and eventually drops the stone to a lower elevation.

Scour around structures is caused primarily by vortices that form. In rivers, scour is due to bends imposed by the bank and by confluence of two currents. Sand waves may be formed by high currents and then slowly migrate downstream.

The susceptibility of stones to scour is proportional to the cube of their underwater density, hence the advantage of high-density stone. The scour is also affected by the gradation and packing of the stones and their exposed surface roughness.

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2.21 Concluding Remarks

Bathymetry at and adjacent to the construction site is extremely important since it affects initial setdown of jackets and caissons as well as the landing of more flexible structures like pipelines. Adequate surveying methods and locating systems must be employed to ensure that the bathymetry, geotechnical borings, and actual installation are all controlled to the same positioning relative to each other. Because of tolerances and systematic errors (e.g., night effect) on many electronic survey systems, a method that marks the true position of the site—for example, Global Positioning System (GPS), acoustic transponders, or, in shallow water, articulated spar buoys—will usually be found desirable.

The site survey should also identify carefully the relative position of any foreign or artificial objects—pipelines, abandoned anchors, dropped casing, and the like—which often litter the seafloor in the vicinity of an offshore site due to the prior conduct of exploratory drilling operations. Side-scan sonar and ROV video are usually the most effective means of detection of these objects.

Scour and erosion are addressed in specific sections where their occurrence is most likely. The potential for scour exists at depths up to 100 m and even greater, where wave action may build up internal pore pressures and where eddy currents may have vertical as well as horizontal components. Recent studies indicate that scour potential may even exist in certain areas of the deep ocean where periodic eddy currents are superimposed upon steady-state unidirectional currents.

Since scour may occur during or immediately after the installation of a structure, it is essential that monitoring is carried out and adequate protection placed as soon after landing as possible. Scour protection methods are described in Chapter 7. In some cases, it may be necessary or desirable to place scour protection prior to the installation. When a large caisson is being seated on the seafloor, for example, the water trapped underneath must escape, thus creating a scouring action. At this same time of installation, when there is a relatively small gap under the structure, current or wave action may induce a high flow rate under the structure, causing scour.

A similar phenomenon is that of piping. Piping is the formation of a channel or tunnel under a structure due to pressure gradients that erode the soil locally. Such piping not only weakens the foundation, but it also may prevent subsequent construction operations which require the maintenance of an overpressure (e.g., for removal of the structure) or underpressure (e.g., for aiding penetration of skirts).

Certain soils may degrade and soften when exposed to the changed conditions they experience during construction, especially if they have been previously blanketed by impervious material or were loosely cemented.

In summary, it may be fairly said that seafloor geotechnics represents the area of greatest concern to, and difficulty for, the constructor. Problems of instability, inability to penetrate, and slope failure continue to plague marine construction activities. Overconsolidated silts in the Arctic and calcareous sands in the subtropics are perhaps the most demanding problems facing the constructor in today's offshore operations, while loose silty and micaceous sands pose great difficulties for river and harbor construction. Thus, a closer relationship between the geotechnical and construction engineers should lead to more effective and economical offshore construction.

And I have loved thee, Ocean! and my joy
Of youthful sports was on thy breast to be
Borne, like thy bubbles, onward: from a boy

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I wanton'd with thy breakers—they to me
Were a delight; and if the freshening sea
Made them a terror—'twas a pleasing fear
For I was as it were, a child of thee
And trusted to thy billows far and near,
And laid my hand upon thy mane—
As I do here.

Lord Byron, Childe Harold's Pilgrimage

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3

Ecological and Societal Impacts of Marine Construction

3.1 General

Recent years have seen a revolutionary growth in society's concerns about the impact of activities, especially construction, on the ecology as well as on the health and quality of life of humans. Marine construction activities take place in an extremely sensitive environment, since water readily conveys local discharges and effects to the wider area, encompassing, in extreme cases, an entire estuary, bay, or even a sound, An example is the mammoth oil spill in Prince William Sound, Alaska. Public concern has focused on marine activities, resulting in a host of regulations intended to eliminate or mitigate damages to the ecology and to minimize the danger and disturbance to human communities.

It has become of great importance to incorporate these rules and precautions in the planning stage rather than, as in the past, attempt to correct or mitigate the negative effects during actual construction. Constructors need to consider these rules as an inherent requirement to their work, similar to the specifications, but with the added force of law.

The concept of ecology is that of an all-inclusive living system, ranging from microorganisms to whales and including humankind. Any disruption of this system may have a chaotic effect, causing extensive negative effects throughout the system. While a complete discussion of ecological and societal constraints is beyond the scope of this book, some of the more prominent concerns relevant to marine construction are presented in the following sections.

3.2 Oil and Petroleum Products

The construction contractor is generally not involved with drilling for oil but may very well be conducting operations in the vicinity of live oil lines. Thus, the constructor may well have the potential for causing an oil spill. The constructor's operations themselves involve the use of fuel oils (diesel, gasoline, etc.) and lubricants. Leaking equipment, errors in transfer of fuel, and failure to close and seal valves may all produce the "sheen of oil on the surface," which is prohibited by regulations in many coastal waters and is unfortunately highly visible from the air. The amount of oil that is tolerable is, of course, the subject of highly emotional debates. However, there is no question that this is a matter to

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which construction contractors must give attention or that they must take active steps to prevent oil spills.

The most harmful immediate effects of oil spills are the contamination of the feathers of seabirds. Oil may travel long distances, eventually ending up on a beach where it has serious aesthetic, as well as some harmful biological, effects. Fortunately, these latter do not seem to persist for long on active shoreline beaches. Oil in estuaries, marshes, and the like appears more harmful. Another serious effect is the contamination of the beaches and shoreline rocks where lesser marine organisms such as mussels, sea anemones, and algae thrive. Since these are an essential part of the food chain, oil deposits are harmful. However, the use of steam cleaning and detergents may be even more harmful.

Gasoline and diesel oil are more toxic than crude oil. Oil, being an organic substance, biodegrades in the open water, due to a combination of bacterial activity, oxygenation, and sunlight.

In the Arctic, there has been widespread concern over the consequences of an offshore oil spill in and under the ice. A number of tests have been run by the Canadian Offshore Oil Spill Research Association (COOSRA), the Alaska Oil and Gas Association (AOGA), and the U.S. Coast Guard. In the winter, when there is full ice coverage, the spilled oil tends to collect under the ice. Its spread is limited by the keels of ridges. It tends to coagulate. Some balls drop to the seafloor. Being thus naturally contained, the oil will eventually degrade. In the open-water season, an oil spill in the Arctic is similar to that in the more temperate zones. The effect if it reaches the beaches is considered more serious because the beaches are flat and low-lying and heavily populated by breeding birds.

It is during spring breakup when the effects of an oil spill are potentially the most uncertain and inherently serious. Cleanup operations are impeded by the ice floes. The oil tends to concentrate in the open-water leads, which are the areas in which phytoplankton growth normally starts earliest and which the sea mammals use as entry routes. It also migrates up through brine channels in the ice to form melt pools that can then be burnt on the ice surface if permits can be obtained. Considerable effort is currently being expended to develop effective oil spill cleanup capability under broken ice conditions.

Liquified Natural Gas (LNG) and Liquified Petroleum Gas (LPG), while free from direct pollution, are highly inflammable and explosive, so quality control of construction to ensure strict compliance with design of containment structures becomes of paramount importance. Similarly, the quality control of radioactive processes is essential. Since absolute dimensional and performance requirements cannot be achieved, the constructor should ascertain the allowable tolerances and adhere to them.

3.3 Toxic Chemicals

Strict prohibitions are placed by international law on the disposal of toxic chemicals at sea. The constructor would rarely become involved in such a situation, and then more or less inadvertently if the constructor were to have surplus or waste chemicals, such as coal tar epoxy or solvents. Arrangements must be made for their containment and return to shore for disposal in accordance with regulations.

However, the contractor may be faced with the need to contain and dispose of bentonite slurry. When working in the marine environment, it is often best to use polymer slurry instead. Although it is more costly, it is nontoxic and biodegrades.

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3.4 Contaminated Soils

Construction in harbors frequently involves work at sites where the seafloor soils have been contaminated by heavy metals or other toxic chemicals. Sampling and evaluation are required prior to the start of construction operations. When underwater contaminated soils are present, a determination must be made regarding the best means for either containment (e.g., blanketing with clay) or removal. Often the only appropriate means are dredging and disposal at some designated site. The requirements for containment of the contaminated soils during dredging and transport vary, depending on the degree and type of contamination, but usually require that hopper barges be tight against leakage. In confined waters, they may require excavation inside cofferdams or behind silt curtains. In other cases, they must be transported to distant disposal sites or even encapsulated in sealed barrels.

The costs can be severe, along with significant delays. While these are usually the contractual and legal responsibility of the property owner, the constructor is required to comply with the regulations.

3.5 Construction Wastes

Bentonite is much used in both construction and oil well drilling. Being colloidal, it causes high local turbidity. Many of the additives used in oil drilling, such as barites, are toxic. In most inland waters, the discharge of bentonite is prohibited. Discharge of surplus concrete is prohibited in inland waters because of contamination of the water by raising of the pH, which can adversely affect fish.

3.6 Turbidity

Dredging, filling, blasting, and trenching are typical marine construction activities that churn up the sediments and cause the finer particles, such as clays, to go into colloidal suspension. The resultant turbidity may be quite persistent, and along with the resultant deposition of fine sediments, it may seriously affect the growth and reproduction of oysters, mussels, and microorganisms.

Dredging in a river can cause turbidity that extends far downstream. In an estuary or harbor, these operations typically create a plume that is highly visible from the air. When excavating material from a specific site, like a bridge pier or intake, disposal on the adjacent seafloor may be permitted. The least mixing will be obtained by clamshell bucket, raising the full bucket a few meters above the seafloor, swinging, lowering to the seafloor, and only then opening it for discharge. Similarly, when excavating small quantities by airlift or similar means, discharging through a down-running pipe or casing at depth or at the seafloor will minimize turbidity.

For dredging operations in inland waterways, especially when in the vicinity of intakes, silt curtains may be required. These are constructed of a series of floats, moored in position at intervals. Plastic membrane curtains are weighted by chain or similar weights at their lower end. These curtains are affected by waves and currents, so the design must be tailored for the specific site conditions. Silt curtains impede the dredging operations. It

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may be necessary to provide a gate that can be opened to permit passage of a dump scow or other floating equipment.

Disposal sites for major dredging operations in inland waters are usually strictly regulated. Generally, it has been found that dumping a bottom-dump or side-dump barge load in a single mass produces the least turbidity. The mass falls to the seafloor more or less intact and then disperses radially. Very little goes into suspension.

For hydraulic dredging, disposal in open water is either prohibited or restricted. Some success has been obtained by discharging through a tremie pipe to the seafloor. A cone-value fitting at the bottom end has been used to slow the velocity of discharge. Most hydraulic dredging is discharged on shore. A system of temporary dikes and levees is constructed to contain the water and allow the silt and clay to settle out. Final discharge back into the bay is monitored by a turbidity meter.

The settling-out of sediments may be accelerated by application of flocculants, but care must be taken to ensure that they are nontoxic and their use is approved. The salinity of seawater aids flocculation.

3.7 Sediment Transport, Scour, and Erosion

In most shallow-water and coastal areas, sediments are in constant movement. In addition to the local response to the orbital motion of waves and wave-induced currents, there is the offshore displacement due to storms, alternating with the onshore replacement during calm periods. More important is the general longshore transport due to the net current, which moves vast quantities of sand along the coasts. The disruption caused by major works, such as breakwaters, has long been recognized in civil engineering practice. The sand tends to build up on the "upstream" side and erode from the "downstream" side. The constructor can create similar if short-term changes through the construction of trestles, cofferdams, or jetties for service boats or for pullout of submarine pipelines. The design of such temporary facilities can mitigate this problem by providing large, relatively free openings for the free movement of the sand. It may be necessary to provide for augmented bypass of the sands, either mechanically or hydraulically. Jet eductor systems have been developed for this function, but they are easily clogged by algae, such as kelp, or by a local fall-in of sand. Construction in a river of a bridge pier or cofferdam will cause locally accelerated currents. Since the erosive power of a current increases exponentially with the velocity, deep holes may form around the corners and for a short distance downstream.

The construction of a bridge across the Jamuna River (Brahmaputra River System) in Bangladesh, with its approach embankments and many piers, resulted in changing the pattern of this braided river downstream for several kilometers. Because the native sediments in this braided river are loosely deposited fine sands and silts, temporary islands (chars) and sandbars, were eroded away, to reform downstream.

3.8 Air Pollution

In certain critical areas, notably Los Angeles Harbor, the use of diesel dredges is restricted in order to prevent smog. Only electrically powered dredges are permitted.

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Similarly, smoke from diesel pile hammers or other operations may be restricted in certain areas.

3.9 Marine Life: Mammals and Birds, Fish, and Other Biota

Many laws and regulations restrict operations that can endanger the breeding sites of marine mammals and the nesting sites of birds. Operations that interfere with migratory routes of fish will be restricted. Seafloor disturbance of colonies of shrimp or mussels is a concern. Many of the above are seasonal; others are specific as to water depth and location.

Use of explosives underwater is severely restricted because of fish kill. Surface blasting (bulldozing) is the most damaging. Shaped charges may produce less kill if they are mounted in a frame or are otherwise secured so that they do not become dislodged and turned over. The best method, with minimal fish kill, is by drilling and blasting, with packing to keep the explosion contained below the seafloor.

In recent years, the trend to driving large steel piles with heavy offshore-type hammers in inland water, as foundation piles for bridges and terminals, has led to significant fish kills in the immediate vicinity, due to pressure waves. To minimize this problem, air bubbling has been used. However, the bubbles are rapidly disposed by the currents. In critical locations for fish, therefore, enclosures, like steel cylindrical shells that contain the bubbles, have been provided. These, of course, must be of large enough diameter to allow the driving head and the pile hammer to pass.

Turbidity can ruin oyster and mussel beds. Oil is very damaging to birds and to a lesser extent to fish. The subsequent cleanup by steam cleaning and detergents may be even more damaging.

Several endangered species of birds, such as the least tern, nest on coastal shorelines and in wetlands adjacent to rivers and harbors. Construction during the breeding and nesting season may be severely restricted or prohibited.

Airborne noise and underwater acoustic waves affect marine mammals such as seals as well, especially during the breeding season. Concern has been expressed that pile driving, and especially hydraulic dredging, can affect the communication and navigation of the bowhead whale in the Arctic. The roll of cobbles and gravel through the pipeline generates the objectionable underwater noise. Seasonal restrictions have been applied by governmental authorities and watchers have been posted to alert and shut down operations until the herd of whales has passed.

In many areas, fish migration is very close to shore, in relatively shallow and protected waters. An example is along the west and north coasts of Alaska. In other areas, fish and migrating mammals often use a relatively narrow channel. In rivers, there are seasonal restrictions on work in shallow water, through which the salmon and other androgynous fish migrate. In some shallow-water areas, algae and sea grass grow, which is the food of endangered species of fish.

For such limited and constricted areas, even relatively minor disturbances can have important consequences. For example, an offshore jetty would obviously interfere, but so also might an overhead trestle, since many fish, such as salmon, reportedly are reluctant to swim under a shadow.

Most countries today require the filing of an environmental impact statement or report prior to the undertaking of a major marine project. This will usually have been filed by the client. Included will be sections dealing with the impacts during the construction phase and the marine and onshore impacts of marine operations. It is important for the offshore construction contractor to become familiar with these documents and the constraints,

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restrictions, and mitigating procedures set forth in them. Compliance is not only legally required but, as a practical matter, is essential in order to assure that the construction operations may proceed without interruption or delay. Lack of strict compliance may involve the constructor in legal disputes, even criminal charges, and in today's social and political environment, may stir up public opposition and interfere with the operations.

Environmental protests have led to the shifting of the location of a pipeline from an offshore platform off Sakhalin in order to avoid disturbance and possible pollution of the spawning grounds of the Western Gray Whale.

3.10 Aquifers

Constructors in inland waters must consider the potential contamination of aquifers by allowing saltwater ingress. Obviously, deep, unprotected dredging could expose the aquifer. The penetration of piling may allow communication between sea water and fresh water aquifers.

3.11 Noise

High-speed outboard motors, helicopters, low-flying aircraft, discharge of dredged gravel through submarine or floating pipelines, pile driving, drilling, sparker and seismic surveys, and even echo sounding are examples of construction operations that create noise in the water column.

Noise appears both to attract and repel sea animals. Low-frequency noises travel farther in water. Concern has been expressed that wideband noise spectra may interfere with the navigation used by the bowhead whale. There is concern among the Inuit hunters that the noise may drive the whales and seals farther offshore, to the edge of the polar pack, where hunting is more difficult and dangerous. Several experts believe that the distance over which construction and drilling noise will have a significant effect is of the order of 1000 m.

Noise may be isolated by an air gap, such as that created by intense bubbling of air around the resonator in contact with the water. Ma, Veradan, and Veradan in OTC Paper 4506 (Offshore Technology Conference Preprints, 1985) have shown that gas or air bubbles in water and sediments can attenuate long-range, low-frequency underwater sound propagation very effectively.

Many larger animals (caribou, geese, ducks, etc.) appear to become accustomed to the noise of helicopters if they are not too close, although it is generally believed that loud noise such as that from low-flying aircraft is injurious to breeding birds. Airborne noise may disturb calving of sea mammals on adjacent shoals.

Noise can also be a significant nuisance to inhabitants of nearby shores, since noise, especially low-frequency noise such as that from pile hammers, travels long distances (2000 m and more) over water. Typical restrictions on noise in harbor construction are 65 DB for five minutes every hour from 6:00 a.m. until 9 p.m.; for the remaining period, 55 DB is the maximum. Near waterfront hotels or residences, pile driving may be prohibited at night. Noise from gravel traveling through the disposal pipes of hydraulic dredging appears to be especially detrimental to marine mammals, interfering with their communication.

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Noise is, of course, a serious concern for workers at the site. Earplugs are required for workers in the vicinity of pile hammers and diesel engines. A number of means have been tried to minimize the noise of pile hammers. Wood or plastic cushions between the pile hammer striking block and the pile head are partially effective. Curtains and boxes have been tried; they are not very effective. Unfortunately, sound absorption requires mass.

Similar limits may be placed on bright lights near shore prefabrication sites. For example, bright lights were a major issue during the construction of the concrete offshore platforms in Stavanger, Norway, resulting in the construction of shields to protect nearby residences.

3.12 Highway, Rail, Barge, and Air Traffic

Highway traffic, as well as rail traffic, may be impacted by the new construction. This may result in detours, limited hours of work, and establishment of sequences for the several phases. Safety for the users and for the construction personnel must be a priority.

Navigation must be allowed to continue with minimum interruption and with minimal impact on safety. This usually constrains the scheduling of construction of overwater bridge piers and requires the construction of fenders, buoys, and navigation lighting. Notices to Mariners are issued by the regulatory bodies such as the U.S. Coast Guard to warn shipping.

In rivers and estuaries, the restriction of the river by cofferdams or other structures and by moored construction equipment will cause a small backwater rise in water level above and more important, an increase in river velocity in the remaining cross section of the water flow. This may adversely affect navigation, especially upstream traffic. Downstream barge operations are also affected, making it difficult for the vessel to thread the narrowed channel accurately, especially if upstream traffic is passing.

When working in the vicinity of airports, the height of booms and towers will be strictly regulated. Care has to be taken in the lofting (pitching) of piles that they do not extend into the critical air space, even momentarily, without specific permission from the air traffic controller in each case.

During construction of the Overwater Runway Extension of La Guardia Airport in New York, as well as more recently during the construction of the Øresund Tunnel in Denmark, a boundary cone was prescribed in three-dimensional air space. The airports changed their normal landing patterns to use other runways whenever practicable, but takeoffs were directly over the construction operations.

3.13 Protection of Existing Structures

Many offshore construction operations must be carried out in the vicinity of existing structures and facilities. For example, it is increasingly becoming the practice for the oil company to have wells drilled prior to the installation of the jacket and platform. Subsea satellite wells may similarly have been completed ahead of platform construction. Flow lines and pipelines may be in the vicinity. It is, of course, essential that these not be damaged by the construction contractor through carelessness, such as allowing an anchor line to be wrapped around a subsea well completion or an anchor to be dragged

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or dropped onto an existing pipeline. These incidents have occurred, with serious financial cost for repairs. There is always the possibility that oil will be released to the sea. Submarine cables may be snagged and broken by dragging anchors.

Particular care has to be taken when in the vicinity of seafloor well completions. Pipelines and moorings laid in an active bottom-fishing area have to cope with trawl boards and nets. Although these can damage the line, most often it is the fishing gear that is lost (or claimed to be lost), with resultant claims for reimbursement.

In the vicinity of saltwater intakes for onshore facilities such as LNG plants and power plants, sediments, especially sand, may be a hazard. Sand particles, for example, swept into suspension in an intake may clog spray nozzles in the plant. Operations, therefore, will have to be planned to minimize stirring up of the seafloor sediments. In extreme cases it may be necessary to install barriers on the seafloor (e.g., steel frames with filter fabric curtains) to prevent sand movement just above the seafloor.

Considerations of existing installations require that very careful surveys be made prior to the start of operations and that their relative position be tied in to visible structures or acoustic transponders, so that they may be a guide to subsequent operations. Side-scan sonar or more sophisticated profiling systems are the usual means for location of underwater structures. On occasion they may need to be supplemented by underwater visual or video means using a submersible or ROV, or by diver surveys.

River and harbor construction is commonly required to be carried out in close proximity to existing structures and may even connect with them. Usually they will be in operation and only limited incursion and shutdown will be permitted. New structures may be required to connect with existing structures.

It is important that the constructor of the new project have both as-built drawings and accurate survey information on the existing structure, since many older structures were not built to exact tolerances and field changes may have been made. Careful records should be kept of all anchor locations and the survey plots used in setting them to provide verification of the contractor's work and to protect the contractor from claims for damage that may have been caused by others also working in the vicinity.

The billion-dollar Chicago Flood was caused by driving a timber pile through the roof of an unused tunnel that unfortunately connected with an extensive network of tunnels and basements.

The client or others may be carrying on other operations in the vicinity of the new construction: tanker loading, offshore supply, drilling, and other construction. One common source of problems is anchor line interference. In such cases, carefully planned schedules and layouts should be agreed upon by all parties and then adhered to. If it becomes necessary to adjust them, all parties should be notified promptly, as more than one may have planned operations in the same "weather window" and space.

When working in the vicinity of operating petroleum facilities, especially offshore terminals, there may be very strict limitations on operations (e.g., welding) involving the possibility of fire or explosion. These may be conditioned upon the direction of the wind. Similarly, work near an operating flare may be dependent on wind direction in order to avoid excessive radiant heat. At loading and unloading terminals, the constructor may be required to shut down all welding and burning, or even all operations, while the transfer of petroleum products is taking place.

Dredging and excavation plans must insure against the undermining of an existing embankment, levee, or seawall. Unequal removal or deposition of material may cause lateral displacement or settlement of an existing facility. When conducting blasting operations near or adjacent to an existing structure, constructors may attenuate the shock by using air bubbling. Timber mats may be placed to protect valves. Blasting mats can be deployed to prevent airborne fragments.

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3.14 Liquefaction

Saturated loosely densified sands and sandy silts (sometimes even gravel) can be suddenly transformed into a heavy liquid if the pore pressures are increased to a critical amount. The energy can be delivered by explosives, earthquake, or pile driving, especially with heavy vibration, or even by sudden dumping of a load of soil by impact. If a critical mass has the pore pressure raised high enough, the soil mass may flow and in turn generate progressive liquefaction and flow slides. Large disasters have resulted during construction on coasts (Monterey, California), river banks (Trondheim Fjord) and the Brahmaputra River (Bangladesh). Overpressures leading to failures due to the fluid load have occurred in cofferdams (Lower Mississippi River System).

Piezometers should be used when performing construction in suspected areas of loose saturated soil. A density of N_{SPT} 30 or higher will generally assure against liquefaction. Where there is concern, drains should be provided to rapidly dissipate high pressures.

3.15 Safety of the Public and Third-Party Vessels

Ships and vessels not under the control of the contractor may be operating in close proximity to the construction site. The constructor may be encroaching into the normal navigation channel. Ships, especially large ships, may have limited maneuverability at slow speeds. The pilot may be confused by the lights from the construction equipment. A rain squall may impair visibility. The radar screen will be confused by the congestion of construction equipment.

Constructors, recognizing these problems, should take special steps. They can arrange to have a Notice to Mariners issued by the proper regulatory agency (e.g., the U.S. Coast Guard). For operations that will go on for a long period, they can arrange for a danger area to be marked on the navigational charts. Unfortunately, many ships keep neither their Notices to Mariners nor their charts up-to-date.

Provision of bright floodlights and/or horns on the platform has been found effective, but it must be cleared by the local regulatory agency to ensure that these devices do not conflict with navigational aids. Otherwise, contractors may increase their potential liability by using them.

Protected zones, usually 1000 m in diameter, have been established around offshore platforms. All vessels, large and small, are required to keep clear. In areas of serious potential hazard, the contractor may keep a boat full-time to warn fishing boats and sightseeing boats away from the platform. However, these are of little use in event of a ship off track in bad weather. For these cases, contractors may operate their own radar and use voice, radio, bullhorn, whistles, or signal lights to attract the attention of an approaching vessel. Constructors may be prohibited from placing their anchors in a shipping channel or fairway.

In addition to the normal requirements for protection of the public against injury and loss of life, the marine contractor frequently encounters special risks and extreme liability. For example, ferryboat operations may be in close proximity. They typically operate day and night, in clear water weather and in fog and storms. They carry large numbers of people.

Sightseeing boats present a special risk. Some enterprising boat operators advertise when a particularly dramatic marine operation is to take place and run tours. The

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potential liability for a constructor is enormous, as well as the more likely interference with the operations. Close liaison, scheduling, and personal communication with sightseeing boat operators is essential. The local harbor police or the coast guard may be willing to aid constructors in the interests of safety.

On major rivers, barge traffic may be a special hazard. Barge operators should be individually warned of the restrictions necessary for construction. When such relatively unmanageable vessels are moving through a congested zone, subject to currents and winds, potential hazards inherently exist. Protective dolphins, buoys, spars, booms, and moored barges are types of protection that have been effectively employed.

3.16 Archaeological Concerns

Historically, civilization arose mostly along coasts and rivers. These seas were 100 m lower 15,000 years ago. The coasts that existed even as late as 2000 BPE are now submerged. Ships sank in the shallow waters, to be preserved by mud. The Roman trade of 100 BPE to 300 PE was voluminous, and many relics lie buried in coastal sediments. Whenever any are found, the national laws require that work be ceased until the local archaeologists have removed all they wish.

Similarly, river margins were often the campgrounds of American Indians or other ancient peoples. When their presence is discovered during excavation, as for a graving dock, work must cease until the local authorities give permission to resume.

"For all at last returns to the sea—to Oceanus, the ocean river,
like the ever-flowing stream of time, the beginning and the end."

Rachel Carson, The Sea Around Us

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4

Materials and Fabrication for Marine Structures

4.1 General

The principal materials for offshore structures are steel and concrete. Composites (plastics) are a recent addition. The fabrication and/or construction contractor is generally responsible for their procurement and quality control, although in some cases, especially pipeline steel, the basic material may be separately purchased by the client (operator) and made available to the constructor.

These materials must perform in a harsh environment, subject to the many corrosive and erosive actions of the sea, under dynamic cyclic and impact conditions over a wide range of temperatures. Thus, special criteria and requirements are imposed on the material qualities and their control.

Fabrication is especially critical for both steel and concrete in order to assure that the structure will perform properly under both service and extreme loads. The cyclic nature of the loading combined with the corrosive environment tends to propagate cracks; hence, improper fabrication details and procedures may grow into serious problems. Fabrication is also rendered more difficult because of the large sizes of offshore structures. Spatial dimensions are difficult to measure and maintain, and thermal strains cause significant temporary distortions. Details of fabrication become highly important.

4.2 Steel Structures for the Marine Environment

Durability is of prime importance in the marine environment. Structural steel is subject to external corrosion, both general and pitting, while internal corrosion may occur inside of tubulars. Corrosion is especially severe in crevices and fractures. The inside of steel tanks may be subject to corrosion from stored liquids and other substances, H_2S may be liberated from drill cuttings in the presence of salt water, leading to embrittlement. The rate of corrosion may be amplified by abrasion and by the current. The rate is also increased by an increase in temperature, by an increase in oxygen availability and by the presence of chlorides.

Corrosion is greatest in the splash zone, especially when the current is high and the water temperature is low, since cold water contains more dissolved oxygen. The splash zone accumulates salt due to evaporation in the drying cycle.

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4.2.1 Steel Materials

Steel materials are characterized by the following parameters:

Minimum yield strength
Minimum ultimate strength
Minimum elongation at rupture
Notch toughness at low temperatures
Through-thickness properties
Weldability
Fatigue endurance
Chemical composition
Resistance to corrosion

The American Petroleum Institute (API), the American Institute for Steel Construction (AISC), and Det Norske Veritas (DNV) have prepared documents with classifications of steel materials as well as limitations on their use.

The contractor is not only responsible for their procurement but is concerned about minimizing his costs for fabrication and installation, so the constructor is especially concerned about tolerances in thickness and length and weldability. For tubular members that must be spliced, the constructor is concerned about out-of-roundness and diameter tolerances. For piping, the contractor is concerned about length tolerances, since semi-automated processes aboard a submarine pipe-laying vessel do not permit the random length variations that are accepted in cross-country pipe laying.

Steels to be used in structures or components that will be utilized in cold climates must have adequate toughness at those temperatures. Welding materials are equally critical in assuring proper strength and ductility in service. The weld metal has to be compatible with the base material as regards heat treatment and corrosion. Crack-opening displacement tests or other fracture mechanics tests are normally conducted for selection of the welding consumables.

High-strength bolts and nuts, when used as structural elements, should have Charpy V-notch toughness values as required for the structural steel members being connected.

4.2.2 Fabrication and Welding

Welding procedures should be prepared, detailing steel grades, joint/groove design, thickness range, welding process, welding consumables, welding parameters, principal welding position, preheating/working temperature, and post-weld heat treatment. Stress relieving is normally not required for the range of wall thickness used in the jackets and piles of offshore jackets in moderate environments such as the Gulf of Mexico, but it is frequently required for the thicker members of large deck structures and for the joints (nodes) of the thicker-walled jackets of North Sea platforms.

The qualification of welding procedures is based on nondestructive testing (NDT) and mechanical testing. These latter include tensile tests, bend tests, Charpy V-notch tests, and hardness tests. A macro-section cut through the weld should show a regular profile, with smooth transitions to the base material and without significant undercuts or excessive reinforcement. Cracks and cold lap (lack of fusion) are not acceptable. Porosity and slag inclusions are limited. Fracture mechanics toughness of heavy welded joints should be verified by crack-opening displacement tests.

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Nondestructive testing may include x-ray (radiographic) testing, ultrasonic testing (UT), and magnetic particle (MP) testing. Both the weld itself and the heat-affected zone should have notch toughness properties equal to those specified for the members.

Requalification is required for a welder who has interrupted his or her welding work for more than 6 months.

Manual welding of all higher-strength steels and of normal-strength steel having a carbon equivalent greater than 0.41 should be carried out with low-hydrogen electrodes. For "special structural steels" and for all repair welding, DNV requires the use of extra-low-hydrogen electrodes. It is recommended by this author that all piling be welded with low-hydrogen electrodes in order to prevent fracture under impact.

Welding consumables should be kept in sealed moisture-proof containers at 20°C–30°C, but in any event at least 5°C above ambient. Opened containers should be stored at 70°C–150°C, depending on type of electrode. When electrodes are withdrawn for use, they should be kept in heated containers and used within 2 h. Consumables that have been contaminated by moisture, rust, oil, grease, or dirt should be discarded. Surfaces to be welded should be free from mill scale, slag, rust, grease, and paint. Edges should have a smooth and uniform surface.

No welding should be performed when surfaces are humid or damp. Suitable protection should be arranged when welding is performed under inclement weather conditions. Heating of the enclosed space can be used to raise the temperature above the dew point. The groove should be dry at the time of welding, and moisture should be removed by preheating. The joint should be at a temperature of at least 5°C.

Fit-up should be checked before welding. Misalignment between parallel members should not exceed 10% of the thickness or 3 mm. If the thickness of abutting members differs by more than 3 mm, the thicker member should be tapered by grinding or machining to a slope of 1:4 or flatter (see Figure 4.1). Each welding pass and the final weld are to be deslagged and thoroughly cleaned. Certain completed welds that are critical for fatigue endurance may be required to be ground to a smooth curve. This also reduces the probability of brittle fracture.

Welds that are essentially perpendicular to the direction of applied fluctuating stresses in members important to the structural integrity are normally to be of the full-penetration type and where possible, they should be welded from both sides. Intersecting and abutting

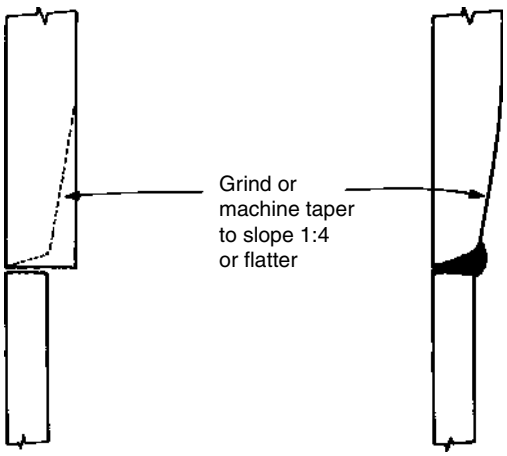

FIGURE 4.1
Tapering of thicker steel plate for full-penetration weld to thinner plate.

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members for which the welding details have not been specified in the design should be joined by complete-penetration groove welds. This requirement includes "hidden" intersections, such as may occur in overlapped braces and pass-through stiffeners.

The construction contractor should detail all lifting plates, padeyes, and so forth that are subject to dynamic impact stress, so that welds are not perpendicular to the principal tension. Welds acting in shear are much less sensitive to cracking than welds in tension. Where this is not practicable, then full-penetration welds must be used. All temporary plates and fittings should be subjected to the same requirements for welding procedures and testing as the material of the member to which they are affixed. Lest this seem unduly conservative, remember the case of the Alexander Kjelland floating hotel, which capsized due to a fatigue crack initiated at the attachment of a minor sonar device to a principal structural member.

Permanent steel backing strips are permitted when properly accounted for in the design analysis. These are especially useful for piling and other members that are to be welded in the field and that are not accessible from both sides. Temporary backing can be provided by internal lineup clamps. Special skill is required for single-side welding of complete joint penetration tubular welds without backing. The interference of these backing strips with other operations such as drilling must be considered.

Temporary cutouts should be of sufficient size to allow sound replacement. Corners should be rounded to minimize stress concentrations. Fillet welds for sealing purposes are required by DNV to have a leg length of at least 5 mm, whereas API-RP2A requires only 3 mm. If such welds are perpendicular to the principal tension of a member subjected to dynamic impact, then great care must be taken to avoid undercutting.

Where welds are found to be defective, they should be rectified by grinding, machining, or welding as required. Welds of insufficient strength, ductility, or notch toughness should be completely removed prior to repair. If arc-air gouging is used to remove a defective weld, it should be followed by grinding. Whenever a discontinuity is removed, the gouged and ground area should be examined by MP testing or other suitable methods to verify complete removal. Repair welding should use extra-low-hydrogen electrodes and an appropriate preheating temperature, usually 25°C above the level used for production welding and at least 100°C.

All welds should be subjected to both visual and NDT as required by the specifications as fabrication and construction proceeds. All destructive testing should be properly documented and identified so that the tested areas may be readily retraced during fabrication and construction and after completed installation of the structure.

Accurate cutting and beveling, while taking more care and consequently more time, will more than pay for itself in reducing welding costs and ensuring high-quality welds.

Increasing use is being made of computer-controlled cutting and beveling that ensures that all intersecting tubulars will fit properly. In many cases, the welding can then be carried out by semiautomatic welding equipment.

Because of the growing importance of documentation and the political sensitivity of many offshore structures, the contractor should make special effort to set up a quality assurance system that will ensure proper records of all testing.

Welding machines must be properly grounded to prevent underwater corrosion damage. Since welding machines are normally DC, the discharge to ground may otherwise occur underwater at piping penetrations or other similar points of concentration.

Provision must be made for the inspection of welds. This includes adequate access. *API-RP2A*, Section 13, and *DNV Rules*, part 3, Chapter 3, Section 4 address both visual and NDT, or examination (NDE), by such means such as UT and radiography (RT). References are given to other means of examination like MP and liquid penetrant.

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Fabrication of offshore steel structures should follow applicable provisions of codes (the AISC specification for the design, fabrication, and erection of structural steel for buildings, for example) for the fabrication and erection of structural steel for buildings.

API-RP2A currently requires that tubular piles be fabricated with longitudinal seam welds and circumferential butt welds. It states that spiral welded pipe cannot be recommended.

Spirally welded pipe piles are widely used on land foundations and in some harbor structures, due to their significant savings in cost. There are limitations on wall thickness and diameter, which generally preclude their application to offshore structures and deep water marine projects, especially where heavy hammers will be employed.

However, advances in the control and reliability of the technology are continually being made.

Spiral-welded piles of 2.0-m diameter and 28-mm wall thickness have been successfully installed for the Hangzhou Bay Bridge in China.

Additional requirements are given in API-RP2A. These include the following.

Beams, whether of rolled shapes, tubulars, plate, or box girders, may be spliced. In cantilever beams, there should be no splice located closer to the point of support than one-half the cantilevered length. For beams within a span (continuous span) there should be no splice in the middle one-fourth of the span, or in the eighth of the span nearest a support, or over a support.

The fabrication of an X-joint of two or more tubulars is especially difficult. In most normal practice, the larger diameter and thicker member should continue through the joint and the smaller member frame into it. In a number of recent large and important jackets, the intersection node is specially fabricated, so that several or all intersecting members are continuous through the joint. In this case, the node is fabricated separately, so that it can be properly treated in the shop, and the members framing it are joined to the node by simple full penetration butt welds (Figure 4.2 and Figure 4.3).

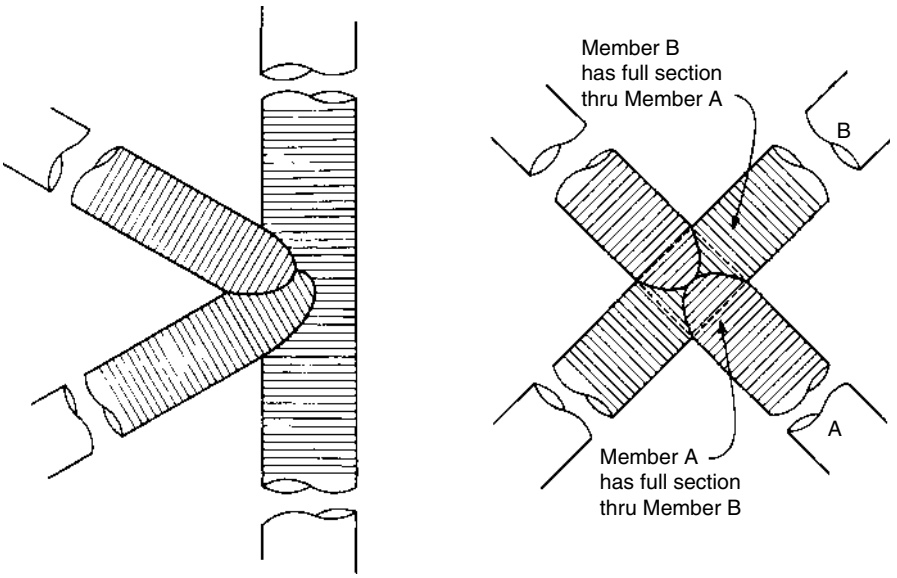

FIGURE 4.2
Prefabricated nodes.

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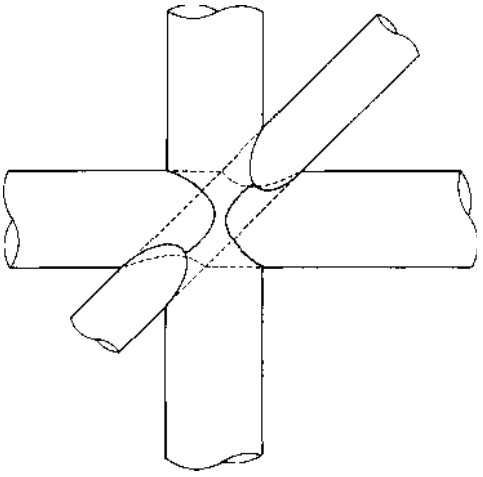

FIGURE 4.3
Intersecting joint of Hondo platform, with full sections carried through the joint.

This same procedure has been employed quite effectively for jackets that have to be completed at remote areas. The nodes are fabricated separately and shipped to the site. Then the main legs and braces are joined by butt welds. Cast steel nodes are being used increasingly, in order to eliminate the critical welding details.

Typical details for the proper bevel and weld for tubular members framing into or overlapping another member are shown in Figure 4.4. Grinding the external profile of the weld may be required in order to improve the fatigue endurance.

For plate girders, the web-to-flange connection may usually consist of continuous double-fillet welds. Welds should have a concave profile and a smooth transition into flange and web. The connection between flanges and plates for stiffening the flanges should be a full-penetration weld made from both sides.

Stiffener plate-to-web connections may usually be continuous double-fillet welds. Weld metal and heat-affected zone notch toughness should not be less than the minimum toughness requirements for the girder.

High-strength bolts may be effectively employed in temporary construction and in many cases for permanent construction where the connections are made offshore. They are especially suitable for field connections where spray and wave-induced vibration

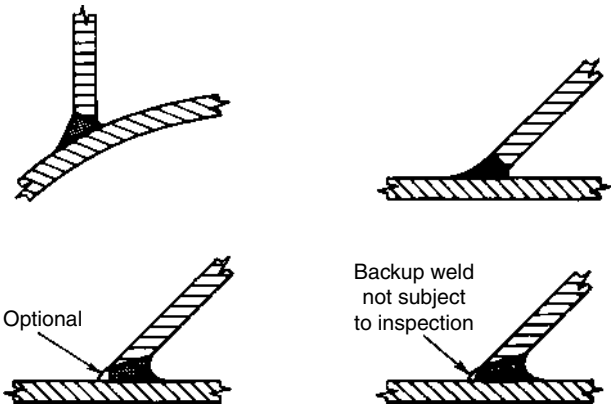

FIGURE 4.4
Welded tubular connections for shielded metal arc welding.

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make it difficult to obtain high-quality welds. They are also an effective means for making connections under cold conditions.

The "turn-of-the-nut" method appears to be the most reliable method of ensuring that adequate torque has been applied. In a large joint, with multiple bolts, either the abutting plates should be pre-milled or shims should be employed to ensure a tight fit.

4.2.3 Erection of Structural Steel

The spatial relationship of structural elements is critical for offshore structures that are to be assembled in the field or where major components are to be mated. API-RP2A provides specific tolerances for final fabrication. For jacket and deck section columns, in any plane critical to field assembly, the horizontal distance from the centerline of the adjacent columns should be within 6 mm of the design dimension. The same tolerance should be applied in the other planes to working points on the outside of the columns.

Angles between corner columns should be within 1 min of the design angle. Diagonals of rectangular layouts should be within 18 mm of each other. Alignment of jacket columns should be maintained within 6 mm. For jacket and deck section bracing, all braces should be within 12 mm of the design dimension. The deck beams and cap beams at their ends should also be within 12 mm of design position.

The assembly of a jacket frame, often having a spread at the base of 60 m and a length (height) of 300 m, places severe demands on field layout and survey and on temporary support and adjustment bracing. A tunnel laser has been utilized to provide accurate levels and alignment on some recent platforms. Such large dimensions mean that thermal changes will be significant. Temperature differences may easily be as great as 20°C–30°C from before dawn to afternoon, and half that between various parts of the structure, resulting in significant distortion (e.g., 30–40 mm). On platform Cerveza, members were cut and shaped to the cold-side dimensions in the morning and then held for the midday sun to tightly fit metal to metal for welding.

Elastic deflections are also a source of difficulty in maintaining tolerances in the location of nodes. Foundation displacements under the skid beams and temporary erection skids must be carefully calculated and monitored.

Jacket frames are typically laid out flat and then rolled up by the use of multiple crawler cranes (see Figure 4.5 and Figure 4.6). Because of the great distances and heights involved, some of the cranes may have to walk with their load. To coordinate such a rigging and lifting operation requires:

1. Thoroughly developed three-dimensional layouts
2. Firm, level foundations for the cranes
3. Trained and rehearsed operators
4. Proper communications
5. Central control

In all, 24 cranes were involved in the two major side-frame lifts during the erection of platform Cerveza, which was 1000 ft. (300 m) in total length. The Hondo platform was fabricated in two halves, joined during erection but subsequently split apart for transport (see Figure 4.7). For the Magnus platform, a different procedure known as "toast rack" was employed. The jacket, which was being fabricated lying on one side, was divided by vertical planes to form five stages. Subassemblies weighing up to 1150 tn. were erected to complete each stage. On other projects, the "slices of toast" have been completely

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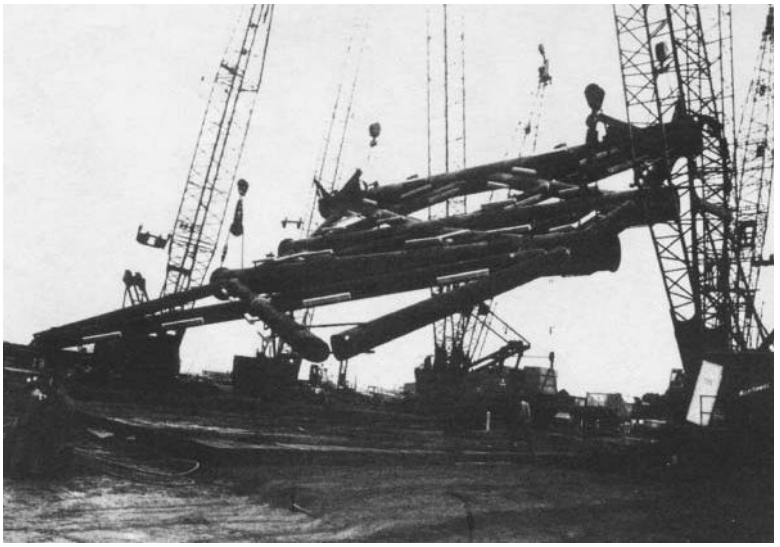

FIGURE 4.5
Roll-up of jacket framing. (Courtesy of Shell Exploration and Production.)

FIGURE 4.6
Roll-up of jacket framing. (Courtesy of Shell Exploration and Production.)

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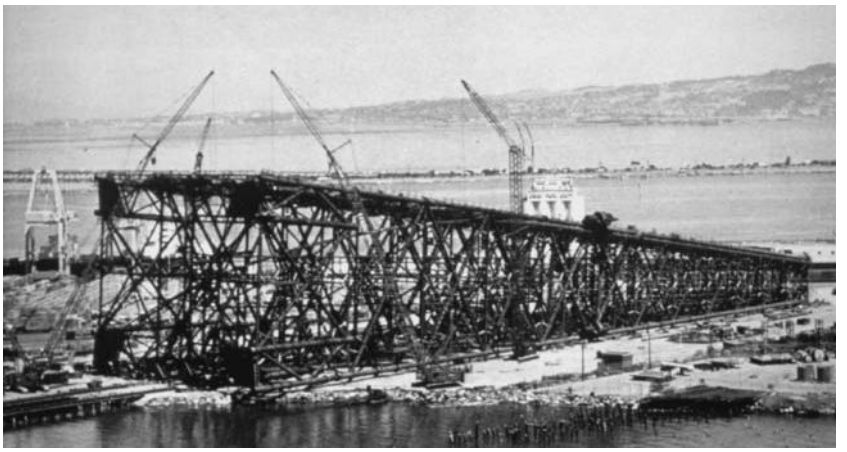

FIGURE 4.7
Both halves of Hondo platform were fabricated together, later separated for transport and launching.

fabricated off site, then barged to the erection ways, skidded ashore, and joined to their neighbor.

For the Bullwinkle jacket, sections of the jacket were fabricated in Japan, transported by barge to Texas, and assembled by the use of jacking towers, which rolled up the sections to heights as great as 460 ft. (140 m). After the jacket frames have been rolled up, the final assembly required staging and support high in the air (see Figure 4.8). Safety of workers was of paramount importance. Carefully planned scaffolding and staging was attached prior to rollup. Attachment plates were provided, to secure the staging when it was subsequently erected. Adequate wind bracing must be provided. Usually this is done by means of guy wires (stays) and turnbuckles secured to deadmen or the skidways. Communication can be provided by voice radio. Tools and supplies should be prepackaged and hoisted up as a unit. Power cables should be laid out to avoid interference with other operations and to minimize chances for snagging.

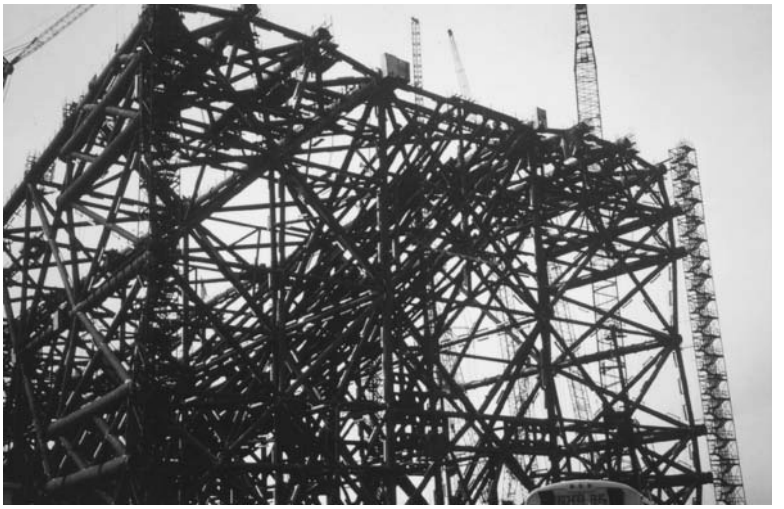

FIGURE 4.8
Platform Eureka being fabricated.

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For jackets destined for shallow water, where the height is of the same order as or less than the plan dimensions, erection is usually carried out vertically, in the same attitude as the final installation. Such jackets may be lifted onto the barge, if within the capacity of the cranes, or skidded out. In this latter case, adequate temporary pads and braces must be provided under the columns to distribute the loads for skidding. Jackets destined for deeper water, in which the height is significantly greater than the plan dimensions, are usually erected on their side. Such jackets are loaded by skidding out onto a barge. Another method used for very large jackets and self-floating jackets is to assemble them in a graving dock, on blocks, similar to shipbuilding practice. The load-out operations themselves are described in Chapter 11.

Large-diameter piles are fabricated from pile segments (cans) of rolled plate. The length of the can should be 1.5 m (5 ft.) or more. The longitudinal seams of two adjacent cans should be at least 90° apart. The pile should be straight, with a tolerance not greater than 3 mm in 3 m, nor 12 mm in 12 m or more.

Out-of-roundness is often a problem with pile segments. The cans may have to be rotated and/or selected to match properly for welding. Outside diameter (O.D.) and out-of-roundness tolerances for adjacent segments should meet the requirements of the API specification 2B (Specification for Fabricated Structural Steel Pipe). As a general statement, the inside circumferences should match within 15% of the thickness of the thinner wall. For joining pile segments of different wall thickness, if the thicker wall is more than 3 mm thicker than the thinner wall, the thicker wall should be tapered as shown in Figure 4.1.

Steel surfaces of piles and the inside of skirts or jacket legs where the connection is to be made by grout bond should be free of mill scale or varnish. Mechanical bond transfer devices such as weld beads or shear rings may be installed during fabrication to enhance the effective bond shear with grout.

4.2.4 Coatings and Corrosion Protection of Steel Structures

Steel is subject to a variety of corrosion phenomena: atmospheric corrosion, splash zone corrosion, crevice corrosion, etc. Recently, many steel structures in service in seawater have been corroded by microorganisms spawned by the interaction of aerobic and anaerobic bacteria.

Typical rates of corrosion of uncoated steel in seawater are 0.15 mm/year in the splash zone; 0.07 mm/year in the submerged zone, except more, up to 0.3 mm/year, in cold fast running tides carrying silt or other abrasive sediments. Other studies for uncoated steel in seawater give rates of 0.127 mm/year. Rates in fresh water are about half of those in seawater.

Painting and coating of the steel members, where specified, should be carried out as far as practicable in the shop, under appropriate conditions of humidity and protection from extremes of weather. The joint surfaces should, of course, be masked to permit welding. Field coating of the joints and touchup of shop coats should be done only when the surfaces are dry and at the proper temperature. In some locations, portable tents or other protection will have to be provided. Heaters and/or dehumidifiers may be required. Coatings may delay initiation of corrosion by 10–20 years.

Steel sheet piling will normally have its highest rate of corrosion from high tide to 2 ft. above. However, if the sheet piling is capped by a concrete wall extending to -1.0 m MLLW, the highest rate of corrosion will be just below -1 MLLW. Sheet piles should be coated or else sacrificial steel thickness should be provided. Interlocks are usualy not coated and reliance is then placed on sacrificial steel.

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It is extremely important that surface preparation be thorough and in accord with the specified requirements. The offshore environment will quickly degrade any coatings placed on damp steel, or over mill scale, or rust. The morning dew can quickly degrade a well-prepared surface.

DNV rules require that the provisions for coating include:

1. A description of general application conditions at coating yard
2. Method and equipment for surface preparation
3. Ranges of temperature and relative humidity
4. Application methods
5. Time between surface preparation and first coat
6. Minimum and maximum dry film thickness of a single coat
7. Number of coats and minimum total dry film thickness
8. Relevant drying characteristics
9. Procedure for repair of damaged coating
10. Methods of inspection—for example, adhesion testing and holiday detection

Surface preparation and application of coating should be carried out when the surface temperature is more than 3°C above the dew point or when the relative humidity of the air is below the limits recommended by the coating manufacturer. Coatings are usually applied to steel in the splash and atmospheric zones and to internal spaces that are exposed to seawater. In the case of sealed internal spaces permanently filled with seawater, corrosion inhibitors may be added to the water prior to sealing.

The most effective coatings seem to be organic coatings over metallized zinc: vinyl mastic on urethane in temperate zones over zinc or zinc silicate and phenolic over the zinc primer in arctic and subarctic.

The U.S. Corps of Engineers is currently providing corrosion protection to lock gates in West Virginia by shot blasting, a single coat of zinc primer 0.625–0.1 mm thick, followed by two coats of zinc rich vinyl immersion coating to 0.175 mm. Underwater and in the splash zone they apply Copoxy Shop Primer, followed by top coat epoxy to 1.0 mm.

Sacrificial anodes or impressed current cathodic protection are normally used to protect steel below water. Anodes must be carefully installed in accordance with the specifications to ensure that they cannot become dislodged during transport, launching, installation, pile driving, and service. An adequate electrical connection between sacrificial anodes and the steel structure is essential (see Figure 4.9). Impressed current is believed more effective because it is less likely to be shielded, but requires continued monitoring and adjustment. If compressed current is turned off and on frequently, as happens on offshore platforms on football days, corrosion is actually accelerated. It is prohibited in closed spaces or where water flow is restricted because of the possibility of generation of hydrogen. Sacrificial anodes discharge their ions on a line of sight through water. The anode demands on the exposed face and the back side of sheet piles are much different.

Coatings may be applied to members that will be underwater in service in order to minimize the requirements for cathodic protection, provided the coating has adequate resistance to cathodic disbondment. Zinc-based and aluminum-based alloys have been applied by thermal spray. Titanium-clad steel tubular piles were used on the Trans-Tokyo Bay Bridge (Figure 4.10).

In the splash zone, additional protection may be provided by means of Monel wrap, copper nickel, austenitic stainless steel, or carbon steel plate wrap or simply by

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FIGURE 4.9
Sacrificial anodes hung between braces. Hondo Platform.

allowing an added steel thickness in order to provide for some corrosion. Allowances of 0.1–0.3 mm/year are made, with the higher values being used in locations where silt or ice in the water tend to remove the protective corrosion products, exposing new surfaces to attack, and in aggressive areas such as the Arabian Gulf.

Recently, pre-coated steel tubular members have become available from Japan. Polyethylene coatings are applied in the plant. Spray-applied dense polyurethane coatings, dense epoxies, and zinc-enriched epoxies have been developed for application to tubulars and structural steel. These not only give good corrosion protection but also possess good resistance to abrasion.

FIGURE 4.10
Structural lightweight concrete. CIDS Platform, Arctic Ocean.

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4.2.5 High Performance Steels

These are available with yield strengths of 500–700 MPa and acceptable elongation. Care must be taken in their use, however, with thorough consideration of buckling and vibration. These steels also have high fracture toughness. Two to three times the resistance to corrosion in the splash zone can be achieved with ASTM A-690.

4.3 Structural Concrete

4.3.1 General

Prestressed and reinforced concrete has been used for more than 25 large offshore platforms, mostly in the North Sea. Concrete lends itself to the gravity-base box caisson-type of structure, especially when developing a large field and when offshore storage is required. Recently, a large concrete platform designed to resist iceberg impact as well as North Atlantic storm waves was installed off Newfoundland. Smaller oil production platforms have been installed off Australia and Brazil. Structural concrete has also been used in the Arctic Ocean north of Alaska and Canada, as mobile exploratory drilling structures. Two concrete floating platforms are in the northern North Sea.

Smaller concrete box caissons have been used for more than 100 piers for overwater bridges and as the foundations for offshore wind-power generators in the waters surrounding Denmark. Concrete box caissons have been extensively used for breakwaters, seawalls, loading terminals, and quays for the berthing of ships. Large concrete barges are in service off Indonesia and West Africa. Concrete is also used in conjunction with structural steel in hybrid and composite designs. Cement grout is used in conjunction with steel platforms, to bond the piles to the skirts and jacket legs.

Structural concrete itself is a composite material consisting of aggregate with a cement mortar matrix, reinforcing and prestressing steel. Structural concrete should conform to the best practices of concrete construction and codes as set forth in building codes and recommended practices for bridges and marine structures, as applicable. As in the case of steel offshore structures, the harsh environment and the special loading combinations and operating requirements make it necessary to supplement such general documents with recommended practices and rules for marine and offshore concrete structures. The principal recommended practices used are listed in the introduction and in the bibliography. Excerpts relating to construction are presented in the appendices.

Structural concrete as a whole and its individual components must be designed to work together in effective composite action. It must be durable under exposure to the sea and the air. Emphasis is placed in design and construction on quality assurance to assure a long life with minimal maintenance (Figure 4.10).

The splash zone, with its wetting and drying, heating and cooling, is the zone most vulnerable to seawater attack, while the submerged zone shows low risk and the below-mudline zone essentially no risk, except if drained. The atmospheric and splash zones have high susceptibility to chloride and CO_2 corrosion of the reinforcing steel.

4.3.2 Concrete Mixes and Properties

For modern offshore construction, the desired properties are often complex, demanding, and occasionally conflicting to some degree, thus requiring development of an optimal solution. Compressive strength has historically been the controlling parameter by which concrete quality has been measured. We now know that it is not necessarily an accurate or

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adequate measure of other qualities. Recent advances in concrete technology have led to significant increases in concrete compressive strength, and the trend continues.

Tensile strength determines the onset of cracking and shear strength and influences fatigue endurance. Reinforced concrete, and especially prestressed concrete, shows excellent fatigue endurance in the air, as long as the concrete is not repeatedly cycled into the tensile range to a greater level than half its static tensile strength or into its compressive range more than half its compressive strength. These limits are normally met in practical design.

When submerged, conventional concrete (not prestressed) shows a reduction of fatigue endurance, apparently due to high pore pressures generated within the microcracks. Interestingly, structural lightweight concrete using modern high-strength, lightweight aggregates, special cementitious admixtures (microsilica), and adequately reinforced shows little such reduction.

Fortunately, even the reduced S–N curve or equivalent for submerged prestressed concrete structures of normal-weight concrete is still fully adequate for water depths of current interest. For greater depths, cyclic loads and increased abrasion resistance, addition of microsilica has been shown to be beneficial.

Permeability is an extremely important property. Low permeability to seawater and chlorides is desirable to minimize the occurrence of corrosion. The use of cement having a moderate tricalcium aluminate (C3A) content is beneficial in that it combines with the seawater chloride ions to form an insoluble compound that blocks the pores. Permeability in concrete occurs primarily along the interfaces between the aggregate and the cement paste matrix. It can be minimized by selection of a mix with minimum bleed, by the use of aggregates having surface characteristics that promote physical or chemical bond, and by adopting a low water–cement ratio. Impermeability is enhanced by the addition of fly ash and microsilica to the mix.

Low permeability to water and especially water vapor, plus the incorporation of entrained air, is important to the prevention of freeze–thaw damage in cold environments. Both the quantity and the quality of entrained air (its pore or bubble size and its spacing) are important for ensuring durability in low-temperature areas. Because the concrete is being used in the marine environment, it has the potential for water absorption and saturation; when such a condition exists, the number of freezing and thawing cycles to cause freeze–thaw disruption is significantly reduced. The sea itself, rising with the tide with waves splashing over the cold concrete surfaces, thaws the concrete, thus drastically increasing the number of cycles of freezing. Verification of proper air entrainment requires petrographic examination of hardened concrete test specimens.

Abrasion resistance formerly was felt to be determined solely by the hardness of the aggregate. Now it is recognized that the strength of the cement paste and the bond with the aggregate are also prime factors. The use of microsilica is especially advantageous in critical applications.

Sulfate resistance was formerly believed to be almost a non-problem in seawater when rich and impermeable concrete mixes were employed, except in areas having extremely high sulfate contents (the Arabian Gulf, for example). For such cases, the addition of pozzolans is especially useful in both reducing permeability and in combining with the free lime to reduce the chemical attack of the sulfate ion. Now the changing constituents of cement make the addition of fly ash or other pozzolan to replace 20%–30% of the cement desirable in essentially all mixes for marine structures. For structures in freshwater containing significant sulfate content (greater than 1300 ppm), the use of pozzolanic admixtures, such as fly ash, are essential and the C3A should be limited to less than 4% maximum.

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Resistance to petroleum compounds and crude oil is provided by normal high-quality concrete. Resistance can be enhanced by the inclusion of pozzolan (e.g., fly ash) in the mix. This also enhances the resistance to the anaerobic sulfate/sulfide-producing bacteria such as *Theobacillus concretivorus*, which is present in oil.

A high modulus of elasticity increases stiffness; a low modulus enhances energy absorption and ductility. The modulus of concrete is proportional to the square root of the strength.

Creep has long been considered an undesirable property because it reduces the effective prestress and produces permanent deformations such as sag. However, it also is beneficial in enabling concrete to adjust to sustained locally concentrated loads, thermal strains, and differential settlement. Thus, it often reduces the onset of cracking.

Fire resistance is important in the structural portions embodying operating facilities such as utility or riser shafts or where hydrocarbons may be accidentally released. On the typical marine structure, the principal elements of fire resistance that are of interest are spalling, thermal conductivity, and creep at elevated temperatures. Spalling can be limited by the inclusion of reinforcing ties through the thickness.

Bond properties are of special importance when mortar or grout is used to transfer shear from piling to skirt or jacket sleeves. Bond is also of importance in the anchoring of reinforcing steel of prestressing tendons, ground anchors, and cast-in-drilled hole (CIDH) piles.

Heat of hydration must often be limited in order to reduce the temperature gradients, which later arise when the outer surface cools or when the element as a whole cools but is restrained. Too high a heat of hydration can lead to thermal cracking. Heat of hydration may be reduced by using coarser grind cement, by controlling the cement chemistry (e.g., using blast furnace slag cement), by replacing 20% or more of the cement by fly ash, and by cooling the mix. The mix is usually cooled by one or more of the following methods:

1. Water soaking of aggregate piles, to cool by evaporation
2. Shielding of aggregate piles from the sun
3. Shielding of batch plant, cement, silos, delivery trucks, conveyors, and pumping pipes from the sun or using reflective surfaces
4. Mixing with ice instead of water
5. Introducting liquid nitrogen into the aggregate pile or the concrete mix.

Temporary insulation of the forms and outer surfaces may be used to reduce thermal gradients.

Thermal cracking occurs at an early age, typically 7–20 days after casting. If there is sufficient reinforcement crossing the crack, this will be under tension and subsequently pull the crack closed. However, if there is too little steel area, so that the steel is stretched beyond yield, then the crack will stay open. The critical steel area is given by the formula:

$$A_s = \frac{f_{ct} A_{ct}}{\phi f_y} \tag{4.1}$$

where f_{ct} = tensile strength of the concrete at age 7 days, A_{ct} is the area of concrete in the tensile zone involved, and f_y is the yield strength of the reinforcing. The tensile zone, A_{ct}, is usually determined as the sum of the thickness of cover plus seven times the diameter of the outer reinforcing bar, times the unit length. The area of steel, A_s, is required over this same tensile zone. For marine structures, it typically calculates as 0.8%–1.0%.

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Thick concrete members and masses need to be individually evaluated in order to prevent internal laminar cracking due to differential cooling after heat of hydration has expanded the fresh concrete. Other properties may become important for specific applications offshore, such as floating or submerged cryogenic storage. The concrete mix will typically consist of the following ingredients:

1. *Cement*. This should be similar to ASTM Type II, except that a C3A content of 8%–10% seems optimum to minimize chloride attack. With very thick sections or large masses, fly ash should be used to replace a part of the cement, or blast furnace slag-Portland cement in the ratio 70:30 may be employed. Alkali content should be limited to 0.65% ($Na_2O + 0.65\ K_2O$).
2. *Coarse aggregate*. Natural or crushed limestone or siliceous rock (gravel), maximum size 20–25 mm for normal sections, but may be as small as 10 mm for congested and thin sections and for flowing concrete. Aggregates should be non-alkali-reactive. For lightweight concrete, use structural lightweight aggregate, normally of sealed-surface type, having minimum water absorption characteristics.
3. *Fine aggregate*. Natural or manufactured sand conforming to standard grading curves.
4. *Pozzolanic additions or replacements*. Use pozzolans, ASTM Class F (fly ash) or N (natural) with limitations on free carbon, sulfur, and CaO. It can replace a portion of the cement. Measure strength at 56 or 90 days instead of 28 days.
5. *Water*. For all reinforced and prestressed concrete, use only fresh water, with appropriate limits on chloride ion and sulfate ion.
6. *Water–cementitious material ratio*. Normal maximum, 0.42. Practical target for high-quality structures, 0.33–0.37.
7. *Water-reducing admixtures*. Use high-range water-reducing agents (superplasticizers) for flowing concrete, and where reinforcement is very congested or where reduced permeability and thus enhanced durability is required.
8. *Slump*. With conventional water-reducing agents, 50–150 mm; with high-range water-reducing admixtures (superplasticizers), 150–250 mm. "Slump flow" (radial flow) is a better measure than slump (vertical flow) for high workability because it is more sensitive for these mixes. (For a given mix, slump flow is usually about twice the slump numerically.)
9. *Retarding and/or accelerating admixtures (as required)*. Do not use $CaCI^2$ as accelerator in reinforced or prestressed concrete.
10. *Air entrainment agent*. To give proper amount of entrained air and proper pore size and spacing in hardened concrete.
11. *Silica fume*. Enhances both early and long-term strength; also bond. It requires more mixing time. Metakaolin is similar. They increase impermeability and durability.
12. *Anti-washout admixture*. Limits wash-out of cement underwater.
13. *Calcium nitrite admixtures (such as DCI)*. Reduce the corrosion of reinforcement.
14. *Limestone powder*. Sometimes used to promote workability while having very low cementitious action and almost no heat of hydration.

In recent years there has been a revolution in concrete technology, resulting in the ability to design concrete mixes specially for specific performance characteristics and environments.

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Thus, the state-of-the-art concrete mix requirements have become a recipe, including not only mix proportions but also sequences of addition.

Both fly ash and slag cement are recycled materials and hence, environmentally and technically acceptable. Aggregates, however, must be from hard rock or manufactured structural lightweight aggregate for use in marine structures; recycled aggregates are unsuitable.

Fresh water should be used for all structural concrete that is reinforced or prestressed. The chloride content of the mix is an important factor in ensuring protection against corrosion of the steel. However, saltwater can be used in unreinforced concrete, such as breakwater armor units (dolos, Tribar, Tetrapod, etc.). The concrete mix should be verified by trial, since the saltwater tends to accelerate set and since it may be incompatible with certain admixtures. Added dosage of retarding admixture may be necessary. Salt water can also be used with the unreinforced underbase concrete for gravity-based structures such as the concrete offshore platforms of the North Sea. There, saltwater has been used with heavy doses of retarder. Salt water may be suitable also for mass underwater concrete (unreinforced), provided adequate retardation of set is achieved.

With so many components, compatibility is essential. Some of the problems that have developed can be traced to incompatible trace chemistry in the various components. For this reason, trial batches are always recommended, duplicating insofar as practicable the placing and curing conditions and temperatures.

Because of the above considerations and because of the worldwide spread of marine construction, no attempt has been made to tie the components to specific national specifications or to specific quantitative values. These can be obtained in general terms from handbooks on concrete technology, and from the governing codes, also in specific terms from consultants specializing in this field and knowledgeable of the construction area and the service environment.

Special warning is given concerning application to new environments or requirements. The concrete mixes, which have given satisfactory service in temperate zones, usually require important modifications for use in the Arctic and the Middle East.

4.3.2.1 High Performance Concrete—"Flowing Concrete"

High-performance concrete (HPC) is generally classed as that developing a strength at 28, 56, or 90 days greater than 7000 psi (50 MPa). Strengths above this are consistently obtainable only with well-graded hard rock aggregates, over 400 Kg/m^3 of cementitious materials (usually cement plus fly ash), high strength water-reducing admixtures, and the addition of condensed silica fume. Water–cementitious material ratios need to be below 0.37.

To ensure proper dispersal of silica fume, since it comes commercially in a granulated form, extended mixing or special mixing procedures are required. Both silica fume and fly ash need early and extended periods of water curing in order to prevent surface cracking.

Flowing concrete is generally similar to HPC but by use of high-strength, water-reducing admixture, adequate fine and small coarse aggregate, and a viscosity admixture such as AWA, a slump equivalent of 25 cm (10 in.) is obtained (slump flow of approximately 50 cm). Water–cement ratio is kept to 0.35 maximum.

Flowing concrete can be properly consolidated by gravity alone—no vibration is required. It is especially suited for filling the interstices between bundles of bars.

Due to the rich mixes, care must be taken to avoid excessive heat of hydration in thick and massive placements. Increased percentage of fly ash, up to about 30% replacement of cement, helps. Limestone powder has very little heat of hydration and can promote fluidity. In very massive placements, Type IV cement can be used.

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TABLE 4.1

Typical Mix for Self-consolidating Concrete in Marine Environment

		lbs./cy	kg/m^3
	Cment type 30	689 in.	409
	Fly ash class F	122	72
	Fne aggregate	1314	781
	Coarse aggregate	1420	842
	Total water	308	183
	Corrosion inhibitor	5 gal	24.8 <
	HRWR	74 fl. oz.	2.86 <
	Viscosity modifier	23 fl. oz.	0.89 <
As needed:	Set retarder	54 fl. oz.	2.09 <
As needed:	Air entrainment	13 fl. oz.	0.50 < 5.5%–7.5%
	W/CM ratio	0.38 max.	0.38 max.
	Strength f′c—28	6000 psi	40 Mpa
	Density	143$^{\#}$/cf	2290 kg/m^3

The contractor should run trial batches, preferably full size, to verify behavior and properties, such as time of initial set and final set, rate of strength gain, etc.

Table 4.1 gives the proportions of a typical self-consolidating (flowing) concrete mix.

4.3.2.2 Structural Low-Density Concrete

Structural low-density (lightweight) concrete is based on the use of ceramic aggregates such as expanded clays and shales to produce aggregates less than a density of 1.0. The best aggregates have surfaces sealed during manufacture, giving absorption values less than 10%. Both coarse and fine aggregates are produced but for structural grade concrete, only coarse aggregate particles are used, along with natural sand fines. The resultant concrete can be produced to give both high compressive strength (60 MPa) and tensile strength, along with acceptable creep and shrinkage. Modulus is lower and thermal properties differ from those of conventional concrete.

Structural lightweight concrete has 100% lightweight coarse aggregate, while modified density concrete replaces only 40% or so of the coarse aggregate.

Among the favorable properties of structural lightweight concrete are low density (sp. gr. of 1.7–2.0), very low permeability (especially when silica fume is added), and lower thermal conductivity. Tests in the Baltic Sea show that structural lightweight aggregate has resistance to ice abrasion equal to that of conventional concrete.

Modified-density concrete has properties such as modulus in between those of all lightweight and conventional concrete. Negative properties include the absorption of water into the lightweight aggregates, especially under pressure such as placement by pumping, and the need to restrict vibration to prevent flotation of the aggregates. Either moderate vibration or flowing concrete are effective in consolidating the fresh concrete.

The lower density is of great value in floating structures of all types, since the submerged density is only 0.6 times that of conventional concrete. Even with the high proportions of reinforcing steel typically incorporated in large floating concrete structures, they will float with a draft about half their depth.

Structural lightweight concrete was selected for the Global Marine platform known as CIDS, built in Japan, towed to the Beaufort Sea north of Alaska for 10 years service, and now moved to Sakhalin Island off the coast of Siberia.

With GBS and other mammoth offshore structures, the dead weight tends to grow during construction, due to a variety of causes: added reinforcement, increased tolerances, etc. For these, reduced density concrete can be used. An example is the Hibernia Iceberg

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Resistant platform in Newfoundland where increases in reinforcement threatened to increase the draft beyond that available. Use of modified density concrete, in which 40% of the coarse aggregate was replaced by structural lightweight aggregate, solved the draft problem while maintaining modulus, shear, and creep at their original design values.

Modified density concrete was selected for construction for the four concrete caissons for the Tarsiut Caisson-Retained Island in the Beaufort Sea north of mainland Canada. This reduced the weight and draft to the values required for transport and installation.

4.3.2.3 Ultra-High Performance Concrete (UHPC)

This mix was originally developed by Buoygues for use in sophisticated structures where very high strength and low permeability are required. It is now available commercially. The mix contains no coarse aggregate. It is based on the addition of very finely divided silica to a rich cement mix, along with a limestone powder. The reaction is initiated by heat, generated by the hydration of cement, and often augmented by steam curing. A number of admixtures are added. The water–cement ratio is very low, necessitating the addition of high-range water-reducers. Two-percent steel fibers are added. These are short (12.7 mm) and only 0.2-mm diameter fibers with slightly enlarged tips. They are added to the mix in random orientation and hence prevent microcracks from growing. Care must be taken to ensure dispersion of the fibers. The result is a highly workable concrete that is very dense and impermeable and has both high compressive and much higher-than-normal tensile strength. This latter property minimizes the need for transverse reinforcement normal to the post-tensioning. The concrete is highly resistant to freeze–thaw deterioration.

While the use of VHPC permits thinner structured members, care must be taken to ensure resistance to transverse and punching shear as well as to vibration.

Buoygues has successfully utilized this material for the hull of the N'Kossa offshore petroleum floating platform, more than 300 m long, now moored off the West Coast of Africa. It has also been used for the anchor plates on an anchored retaining wall on Reunion Island in the Indian Ocean, where corrosion rates are abnormally high and is now available commercially in the United States.

4.3.3 Conveyance and Placement of Concrete

Concrete, properly mixed, may be conveyed by a wide variety of means to the site for placement. It is important that its properties not be significantly altered during conveyance. If conveyed in trucks, continuous agitation of the entire batch is normally required. Mixer blades must not be excessively worn and there must not be significant buildup of hardened concrete inside the mixer.

If conveyed by conveyor belt, it may require covering in intense heat to prevent premature stiffening or flash set or, in rainy weather or extreme cold, to prevent excessive bleed and slump loss. In all cases, segregation must be minimal. If conveyed by pumping, the pressure may be so great as to cause water absorption into the aggregates, causing a loss of slump. Mixed concrete, suitably retarded, has been conveyed in hoppers in boats for short distances.

Entrained air content may be seriously reduced, although experience shows that spacing factors may usually remain satisfactory. All the above means of conveyance have been used satisfactorily, but only when recognition has been given to the various factors and when appropriate steps have been taken.

Concrete, when placed, must be properly consolidated. In general, internal vibration is required, even when high-range water-reducing admixtures are used, in order to ensure

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complete consolidation and filling of all spaces and interstices between reinforcement. External vibration is limited as to the depth to which it is effective.

When concrete is placed in hot weather (above 30°C; 90°F) or in cold weather (below 5°C; 40°F) adequate procedures must be implemented to ensure not only a proper mix temperature when placed but protection until it has gained adequate maturity. Recent research by the U.S. Corps of Engineers' Cold Regions Research Laboratory and by the Russians have developed antifreeze admixtures which enable mixes to be successfully placed in temperatures as low as −5°C.

4.3.4 Curing

Current research has greatly revised requirements for curing. Because most of the mixes used for sophisticated offshore structures are highly impermeable, emphasis today is primarily on sealing the surface against loss of moisture and heat rather than supplying additional water. Concrete with silica fume, high percentages of blast furnace slag, or greater than 15% fly ash require the availability of external water to ensure full hydration of the surface concrete to prevent surface crazing and salt crystallization, and to ensure impermeability.

Membrane-curing compounds represent one form of sealant, the white pigmented variety being especially suitable for reflecting heat in hot climates. However, heat, whether from the sun, the internal heat of hydration, or steam curing, degrades the curing compound, and so one or more additional applications may be necessary during the first day. Where coatings (such as epoxies) are otherwise required, these may often be applied to the concrete as curing membranes, provided they are formulated to adhere to damp concrete.

4.3.5 Steel Reinforcement

Reinforcing steels are generally uncoated deformed steel bars 10–50 mm in diameter with a yield strength of 40 MPa. Higher yield strengths are available; however, due to the need for crack-width control in the concrete of marine structures, yield strengths beyond 50 MPa are generally not usable in tension. Weldable steel is often specified for offshore structures. Brittle steels, such as those from re-rolled rail, are not suitable—adequate elongation after yield is required.

Reinforcing steel is protected against corrosion by the semi-impermeability of concrete cover and by its alkalinity. Carbon dioxide from the air and chlorides from seawater or intentionally applied salts degrade this protective encasement. Corrosion is especially severe in the splash zone but can extend up well into the atmospheric zone where spray periodically deposits salt by evaporation, which then becomes concentrated.

Corrosion of reinforcement is the leading cause of degradation of marine structures. In the Mid-East, structures have been rendered unusable even before they are completed! The best and most economical protection against corrosion is impermeable concrete of adequate thickness of cover. Where additional longevity is desired, corrosion-inhibiting admixtures such as calcium nitrite C_aNO^2 may be incorporated.

Epoxy-coating of the reinforcing steel has become widespread as a means of preventing corrosion. Both hot-dip and fusion-bonding have been applied. Stainless steel reinforcement, while expensive, is growing in use for structures where 100 years or longer design life is required. Corrosion-resistant steel is also available.

Epoxy coating, especially fusion-bonded epoxy coating, eliminates the adhesive bond and depending on its thickness, may reduce the mechanical bond of the deformations. Therefore, longer development lengths are required, or the use of mechanically headed

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bars. Due to this lack of adhesive bond, when strains approach their ultimate value, delamination may occur. Epoxy-coated bars do not perform well under impact, for example, as spiral or hoops for piling due to lack of adhesive bond.

Most epoxy-coated bars are hot-dip coated as straight bars, with any bending or welding performed subsequently. Bending of epoxy-coated bars, especially tight bends such as are typical for stirrups, can cause minor cracking on the outside of the bend. This requires inspection and touch-up by hand. Some facilities are now available that can electrostatically fuse an epoxy coat on prefabricated bent bars and welded cages. This is called fusion bonding. Hot-dip epoxy coating can also be applied to prefabricated units.

Conventional reinforcing steel is also called passive steel because it is nominally under a state of zero stress when enclosed in the fresh concrete. Actually, it usually ends up being in mild compression due to shrinkage of the concrete after hardening. The typical marine structure uses very heavy concentrations of reinforcing steel, far more than the usual land structure. To provide adequate space for concrete placement, the bars may be bundled in groups up to four. Special consideration should be given to the design of the concrete mix to ensure that adequate "paste" (cement–sand mortar) is available and that thorough vibration, preferably internal vibration, is employed, to fill the interstices between bars completely. Coarse aggregate size may be reduced. Alternatively, flowing concrete may be employed.

Cover over the reinforcement is important for durability in the marine environment. Too much cover increases the width of cracks, while too little leads to easier access by chloride and oxygen and in extreme cases to loss of mechanical bond.

Reinforcing steel requires an adequate development or bond length at the end of each bar, in order to be able to develop the full strength of the bar without pullout. Because of the dynamic and cyclic nature of loadings on offshore structures, bond lengths are often greater than those specified for static loads, up to as much as double. The positioning of the ends thus becomes of importance, to ensure that the anchorage is in a zone that is able to develop bond, that is, within a compressive zone. Provision of confinement reinforcement in zones of anchorage and splicing is important. Tie wire for steel reinforcement should be black (uncoated), soft iron wire, never copper or aluminum; otherwise local pitting may ensue. Welded fixing of bars may lead to pitting corrosion and loss of fatigue endurance. Flash welding (shop fabrication) appears to avoid these problems and is therefore acceptable.

Stirrups are extensively used in offshore structures. It is desirable that the tails of all stirrups be anchored back within the confined core of shells, slabs, and beams, but this is often impracticable. Tying of the free end helps. Because of this problem and the fact that at high loads, the concrete under the bend of stirrups crushes, stirrups rarely develop their full yield strength. Because of the bends in stirrups, their fatigue strength is limited. Therefore, headed bars, such as forged or fabricated T-bars, anchored behind the in-plane reinforcement have been developed to provide a more efficient tie and shear-resistant element (see Figure 4.11). These have been extensively employed on recent offshore platforms, primarily for stirrups but also for longitudinal bars as a replacement for hooks, in order to obtain a positive anchorage.

Electrical contact should be avoided between coated and uncoated bars since the uncoated bar is exposed to oxygen and becomes a cathode, which then provides the electrical current to corrode the anode. An abraded area on the coated bar may become the anode.

Recent developments in durable reinforcement have been the growing use of stainless steel and also of a new steel with controlled crystallization (MMFX-2), which has been shown to be highly resistant to corrosion. Stainless steel reinforcement is currently being specified on several important structures designed for a service lifetime of 100 and more years.

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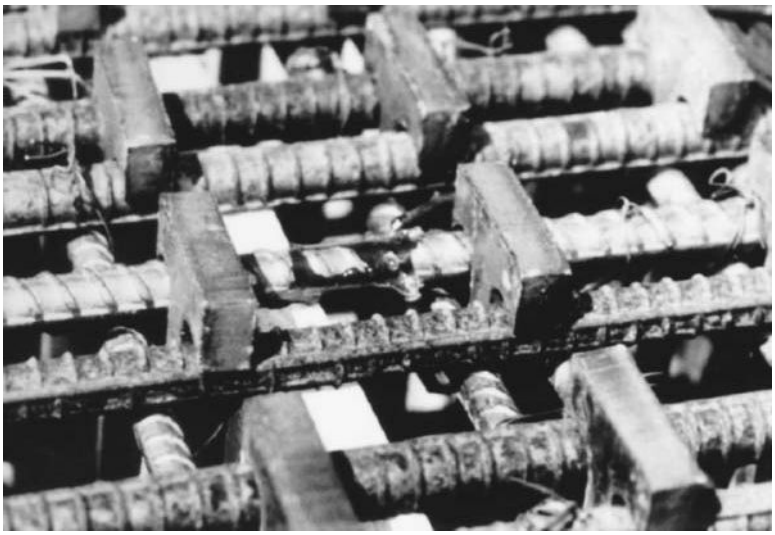

FIGURE 4.11
Headed reinforcing bars.

Not all alloys of stainless steel are suitable. Some stainless steel alloys can be strongly cathodic to any carbon steel to which it is electrically connected, causing the latter to corrode rapidly.

Splices of reinforcing bars may be made by lapping; however, the long splice lengths and the congestion when trying to lap-splice bundled bars raises many difficulties. Mechanical splices are desirable for the larger bar sizes. Welding of reinforcement, if used, must employ low-hydrogen electrodes. Working drawings must show splice details so that congestion may be evaluated and the weights of steel accurately calculated, the latter for purposes of draft and stability control. Splices of bars destined to work only in compression may, according to the code, be end bearing. However, as a practical matter, in most cases the reinforcing steel in offshore structures must work in both tension and compression under the variable loading conditions. Therefore, it is considered good practice to design splices to transmit both. If lap splices are used, the bar size should be limited to 32-mm (1¼ in.) diameter and the laps should be well tied at both ends and confined. Splices in adjacent bars must be staggered. Mechanical or welded splices are preferred for the larger-diameter bars.

Corrosion of reinforcing steel is the most common form of degradation in the marine environment (see Figure 4.12). Corrosion protection for the reinforcement has traditionally been provided by selection of the proper concrete mix for low permeability and cement chemistry, along with an adequate cover and good consolidation and curing. The constructor is responsible for ensuring that the specified cover is achieved by use of appropriate chairs and spacers. Figure 4.13 shows that highly durable concrete can be obtained even in severe exposure.

Corrosion-resistant reinforcing steel is commercially available as MMFX-2. Tests and limited experience in seawater have confirmed its resistance to corrosion, although not absolute protection.

To prevent abrasion damage, bars should be handled with fiber slings. Abraded areas and all cracks should be touched up in the field.

In recent years, the efficacy of epoxy coating bars for marine structures has been questioned. The experience with straight bars in bridge decks subject to de-icing salts has been generally excellent, whereas the experience with bent bars in marine construction and

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FIGURE 4.12
Example of corroded reinforcing steel in concrete quay, Kuwait.

especially the splash zone has been varied. Epoxy coating has the disadvantage that it prevents adhesive bond, thus giving reduced bond capacity and leading to delamination under severe loading. There have also been a number of cases where the bond of the epoxy coating to the bar has been destroyed by saltwater, resulting in corrosion under the coating.

FIGURE 4.13
Example of durable concrete after 30 years exposure, Kuwait.

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Therefore, this author recommends that the use of epoxy-coated bars be evaluated in each specific case of marine construction and not be just generally adopted, as required by some existing codes and guidelines. In any event, the quality of the concrete, and especially its impermeability, is the most important factor in ensuring durability.

Stainless steel reinforcing is being increasingly used on critical structures in the marine environment, despite its high cost.

Fiber reinforcing bars and grids have potential advantages due to corrosion-resistance, good bond, and enhanced stiffness. However, the grids are very difficult to hold in place to close tolerances during concreting. Flowing concrete should be used.

A practical problem in construction arises when bars of different steel grades or both coated and uncoated bars are to be placed in the same general regions. Coding must be employed or else all the steel should all be of the highest grade. Clearly identifiable color coding is preferable to the marks stamped on the bars' deformations; the latter are of use in the warehouse but not in the field for installation and inspection. The supports for reinforcement should be concrete "dobe" blocks or plastic chairs. Stainless steel chairs may lead to local pitting of the bar in a saltwater environment. Therefore, when stainless steel reinforcement is used in conjunction with conventional steel, the proper alloy must be selected or the conventional steel epoxy coated. Similar precautions may be necessary when MMFX-2 steel is used.

Practicable methods of cathodic protection of embedded reinforcing steel have been developed and applied where serious corrosion is anticipated, e.g., saline water conversion plants and seawater cooling systems for power plants. Cathodic protection by sacrificial anodes can be used underwater: impressed current CP is required for above-water applications.

Bending in the field must follow carefully detailed procedures. Reference is made to the ASME/ACI Code for Nuclear Containment Structures, where provisions for field bending are carefully prescribed. Small tie bars, 10 mm and less, of weldable steel may be field-bent as required.

4.3.6 Prestressing Tendons and Accessories

Prestressing tendons usually consist of seven-wire strands for long lengths and threaded bars for short lengths or where the tendons must be successively extended (see Figure 4.14). The tendons are placed within ducts, stressed and anchored, and then grouted. A number of reliable anchors are available commercially with which to anchor the stressed strand. These are proprietary. They are generally furnished with an entering or transition trumpet and confining spiral. Anchorage zones must have adequate confining reinforcement.

Ducts should have thick enough walls to prevent local sag, be tight against the entry of mortar from concrete mix when it is vibrated, and have smooth interiors so as not to snag the strands when they are pushed or pulled in. Ducts for post-tensioning tendons are of several types. Thin-walled steel pipe or conduit, with screwed couplings, is often used for vertical ducting. Semirigid corrugated steel ducts are extensively used. They are grout tight although not fully watertight. Couplings are sleeves with either waterproof tape or, even better, with heat-shrink tape. The recent development of corrugated plastic ducting is of major importance for marine structures. Splices are fused. Combined with plastic anchorage caps, these plastic ducts can ensure complete electrical and moisture insulation of the tendons and hence effectively eliminate potential chloride corrosion. The ducts should be tied to the reinforcing steel to maintain a smooth profile in accordance with the design and within the limits of the prescribed tolerances. Ducts must be protected

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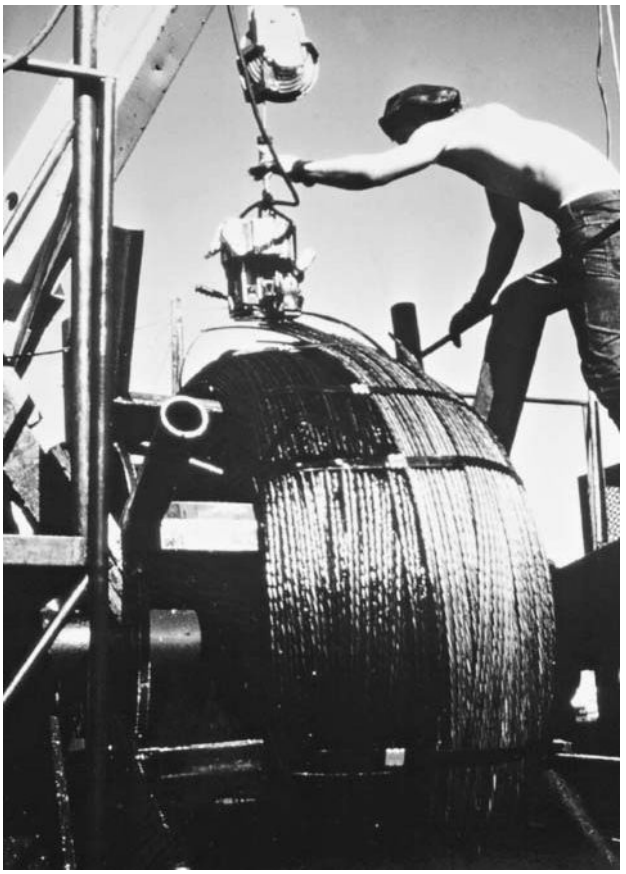

FIGURE 4.14
Prestressing strands delivered in coils. (Courtesy of VSL.)

against the accidental entry of debris and concrete materials. Ducts should be delivered with temporary plastic caps attached. Prior to the general use of temporary plastic caps, ducts have been blocked by such miscellaneous waste as screwdrivers, concrete aggregate, rags, and even soda bottles! Duct ends must be protected from burrs. Steel ducts should be sawed, not burnt, and should have a cap attached. Vents should be provided at all high points in the tendon profile.

The seven-wire strand is high-tensile steel and must be treated with care. It should be stored in a dry, weatherproof warehouse. German specifications require that the warehouse be heated to reduce the relative humidity well below the dew point. When tendons consisting of multiple strands are inserted in the ducts, they should be entered through a smooth, abrasion-free trumpet or funnel. In most current practice, individual strands of a tendon are pushed in, one at a time. The present practice is to procure strand that is coated with water-soluble oil. During insertion, more water-soluble oil may be daubed on. The ends of the duct are then sealed against moisture entry until stressing. As an alternative to the use of water-soluble oil, a vapor-phase inhibitor (VPI) powder may be dusted on.

Stressing is carried out normally from one end for straight tendons, from two ends for curved tendons. For curved tendons, friction losses may be minimized by one or two cycles of pulling, first from one end, then from the other. If the measured elongation differs by more than 5% from that calculated, the reasons should be ascertained. Note

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that the moduli of elasticity of strand from different manufacturers varies. Excess friction loss may be overcome by cycling the stressing.

The grouting of ducts can be carried out in accordance with standard practice. The grout mix should be selected for minimum bleed. The ducts should not be flushed with water. Where water-soluble oil has been employed, the grout should be pumped through until all oil-contaminated grout is expelled from the far end. Vents should be progressively closed and the grout forced out through the strand ends.

Vertical ducts and those having significant vertical components require special consideration, especially for offshore structures. What happens is that after the duct has been pumped full, the head of fresh grout will force the water into the strand interstices, which will act like a wick, expelling water through the strand ends at the top. This allows the top surface of the grout to settle, leaving a hidden void at the top, which may be several meters in length. Several means are available to help prevent this:

1. Minimize bleed in the grout by selection of mix and admixtures.
2. Use a thixotropic admixture, which causes the grout to gel as soon as pumping stops.
3. Use a standpipe to maintain a reservoir of grout above the top of the upper anchorage.
4. Top up the grout a few hours after initial grouting.
5. Use vacuum grouting.

Steps 3 and 4 require an extra hole in the anchorage plate. Usually two or more of these steps will be found necessary for complete filling of long vertical duets.

In cold weather, the grout must be protected from freezing. While the grout may be heated by use of warm water, this is not adequate to offset the loss of heat when the massive concrete structure is below freezing. Prior to grouting, the temperature of the concrete structure should be at least 5°C (40°F). As noted earlier, antifreeze admixtures are available to enable the grout to set and gain strength at temperatures as low as −5°C. The anchorage zones are zones of high three-dimensional stress, with transverse tension, especially between multiple anchorages. Transverse confinement on both orthogonal planes is required to prevent splitting parallel to the tendons. Less well recognized is the fact that where the tendon is applying stress to only a portion of the concrete, that portion tends to be pulled away from the remaining concrete behind the anchorages. Hence, anchorages in the concrete should be staggered and passive reinforcing steel provided to distribute the tension back into the concrete structure. Anchorages should normally be of the recessed type. For offshore structures, these are the zones most likely to show corrosion. The provision of proper anchorage patches requires a high degree of care in detailing and workmanship (see Figure 4.15).

In temperate climates, coating the pocket with epoxy bonding compound or latex, pouring a high quality grout, employing the window box technique to ensure complete filling, and provision of two or more steel ties, bent down from the structural concrete into the pocket patch, represent the best practice. In some cases, a strand extension may be used for this latter purpose (see Figure 4.15). With plastic ducts, a plastic cap can be placed to encapsulate the anchorage fully; then, the pocket can be concreted as described above.

For structures which will be subject to a freeze–thaw environment, the epoxy should be omitted. Instead, the surface of the pocket should be carefully cleaned by wire brushing and latex applied as a bonding agent (Figure 4.16).

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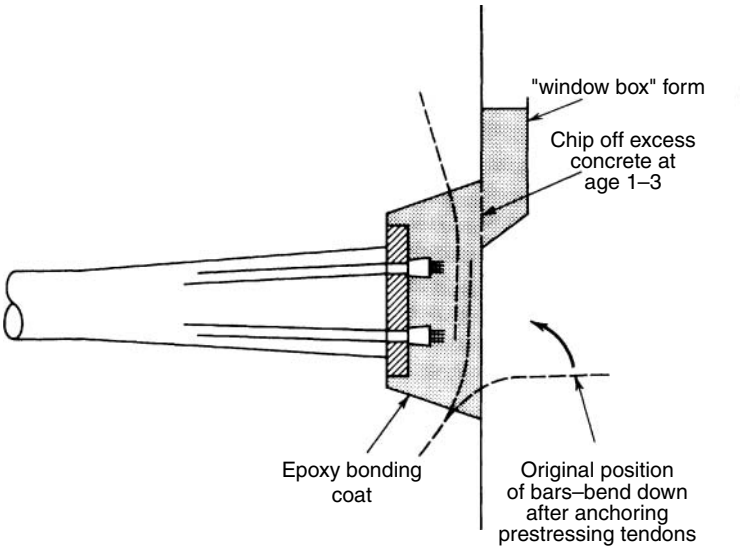

FIGURE 4.15
Anchorage pocket details to provide protection to prestressing anchorages.

4.3.7 Embedments

Embedment plates should be accurately installed prior to concreting. They must be sufficiently fixed to avoid dislocation during slip forming and vibration. In some current practice, it is required that embedment anchors be electrically isolated from the reinforcing steel. This is usually done by tying on concrete dobe blocks, using fiber or plastic cord.

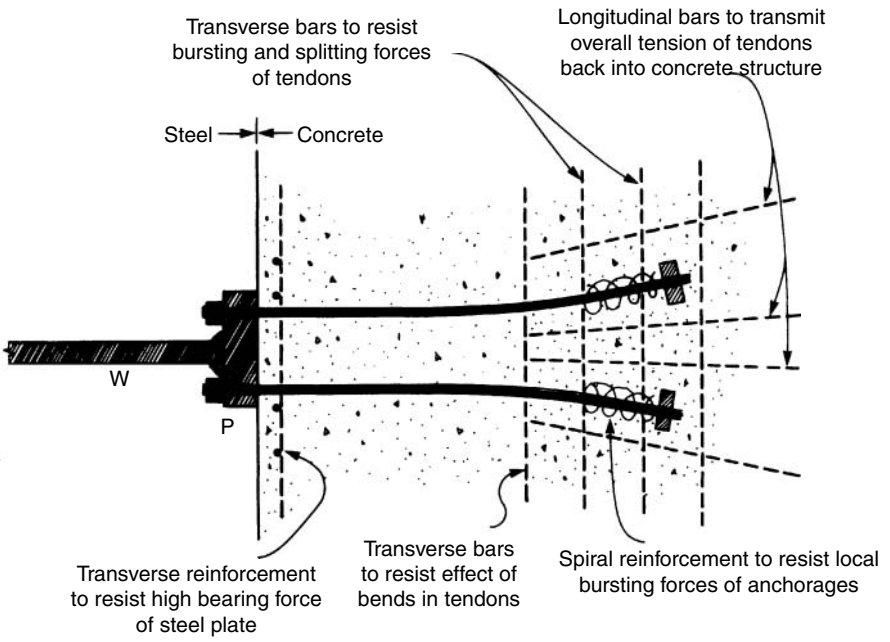

FIGURE 4.16
Connection of steel structure to concrete structure.

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After concreting, the embedment plates should be sealed at the edges to prevent crevice corrosion behind them. This is especially necessary if padeyes or similar attachments have been welded to the plates, as this causes heat distortion. Sealing is facilitated if a rubber or wood strip has been attached to the plate all around; this then forms a recess to receive the epoxy sealant.

Dissimilar metals must be avoided. Stainless steel of many alloys is highly cathodic to steel. A very serious and expensive error was once made in which stainless steel lugs were used to position cast steel embedments of carbon steel. The latter corroded seriously in a matter of months. Copper and aluminum embedments can also lead to corrosion of the steel.

4.3.8 Coatings for Marine Concrete

If coatings are specified, it is important that the surface condition of the concrete be properly prepared. This may involve filling of air or water "bugholes" or honeycomb, and light (not heavy) sandblasting of the overall surface. Primers and coats should be applied and cured in accordance with the manufacturer's recommendations. Because of the prevalent damp conditions around offshore structures, use of a hydrophobic epoxy that permits placement on damp (not wet) surfaces will be found desirable. Many otherwise satisfactory coatings develop "blowouts" and "pinholes" a week or two after application due to water vapor pressure from underneath the coating. Procedures and materials should be selected to minimize these and where they occur, to fill them. Firmly bonding coatings tend to "mirror" any cracking in the concrete beneath, whereas more elastic coatings with reduced bond, such as polyurethane, can span over cracks.

Silane, applied to a concrete surface, will prevent or minimize water penetration while allowing water vapor and air to escape. Unfortunately, it degrades over a period of several years, due to being slightly soluble in water; hence it must be re-applied.

Methacrylate is a thin polymer with high capillarity, so it penetrates cracks. It can be obtained both clear and black. The clear coating does not disfigure the concrete surface significantly while the black has better UV resistance.

Polyurea is currently favored for coating concrete. It is applied over an epoxy prime coat. Polyurea is very different from polyurethane, despite the similarity in names. Various coal tar epoxies and coatings are also available, but many are prohibited for environmental reasons. Formulations of calcium nitrite are commercially available. These penetrate hardened concrete and provide a measure of corrosion resistance to the reinforcing steel.

4.3.9 Construction Joints

Water stops may not be necessary in most cases where reinforcing steel extends across the joint. For horizontal joints, rough screeding and cobbling of the top surface, followed after initial hardening by a water jet to clean off all laitance, can produce a sound, watertight joint. The indentations should be sufficient to engage the coarse aggregate, normally 6 mm deep. The subsequent concreting should start with a 200-mm lift of the regular mix, minus the coarse aggregate, then be followed by the regular mix. The two should be vibrated internally so that the first lift is well engaged by the second.

Vertical construction joints can be prepared after hardening by wet sandblasting or high-pressure water jet, to expose the coarse aggregate to a depth of about 6 mm. Reasonably good success has also been experienced by placing metal wire mesh against the form of the first pour and then stripping it when the form is removed. Latex and epoxy bonding compounds have proved effective in preventing a shrinkage crack at this joint.

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4.3.10 Forming and Support

Vertical elements of offshore structures—slabs, shells, walls, and soon—usually constitute the major portion of offshore concrete structures. The basic forming systems employed are panel forms, slip forms, and flying or jump forms.

Slip forms enable the steel to be installed as the forms are raised. Thus, the concrete is placed at deck level where it is accessible for internal vibration. The rate of rise of the forms is often controlled by the rate of production of the concrete and the rate of placement of the reinforcement, prestressing tendons, and embedments. The time of initial set of the concrete is then controlled by the use of admixtures, so that the concrete emerging from the lower end of the slip forms will not slump or fall out. The temperature of the mix and of the air will affect time of set. Because the slip-form operation is carried out continuously (round-the-clock), it results in rapid construction of vertical and near vertical walls. The length of reinforcing steel bars should be set to match the rate of rise.

When reaching the top of a slip-forming operation, there is no longer the weight of fresh concrete to hold the last lift down: jacking up of the forms after initial set may cause horizontal cracking. One solution is to stop them until after final set and then loosen them before jacking up a short distance. During concreting of slip-formed structures, the vibrator should be marked so that it will always be inserted through the present lift and extend into the previous lift; this is to prevent horizontal cracking and to ensure aggregate interlock.

The use of pozzolans and especially silica fume often makes the concrete adhere to the slip forms. Methods being currently tried as a means of overcoming this include special coatings on the forms and possibly the incorporation of a pumping-aid admixture in the concrete to reduce friction. In any event, steel slip forms should be coated to prevent rusting, since rusting greatly increases surface friction. Stainless steel slip forms, coated with dense polyurethane, have recently been adopted by a leading offshore concrete contractor. Slip forms have been used on offshore structures to construct tapered sections and to construct inclined sections. For the casting of the cell walls of the Oseberg A platform, Norwegian contractors constructed a giant slipforming yoke, giving more room for installation of reinforcing bars. When walls with intersecting beams or slabs are slipformed, blockouts may be installed, with bent-up dowels exposed, so that the slipformed wall construction may continue without interruption. The horizontal slab can be constructed later.

"Flying forms," that is, panel forms that are progressively raised after a lift has gained sufficient strength, usually at age one or two days, and are employed where the congestion of reinforcement steel, prestressing ducts, and embedments is so great that an impracticable number of workers would be required to maintain the minimum rate for slipforms. Typical flying-form panels are 3–6 m high; they may be equipped with window boxes to minimize the height of fall of the concrete (hence, reduce potential for segregation) and to facilitate vibration. Typically, inserts are installed at the top of each lift to support the lower edge of the form in its next lift. The forms are sealed by gaskets along the bottom edge to prevent mortar leakage.

Alternatively, precast panels may be employed, joined by either match-casting or by a cast-in-place column. It has been found that, contrary to accepted belief, dropping concrete in free-fall down a vertical joint or connection column does not result in segregation, especially if flowing concrete is employed. "Match-casting" is obtained by casting concrete segments against the construction joint of an adjoining member. Thus on re-assembly, the joints fit perfectly. They are coated with epoxy-glue (both faces preferably) and then temporarily stressed until the glue hardens. If accelerated heat or steam curing is employed in fabrication, the two segments should be heated together so as not to distort the joint due to warping.

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The provision of continuity of reinforcement through the joints between segments can be attained by the insertion of reinforcing bars or even unstressed prestressing strand through pre-formed ducts; subsequently, they are bonded by grouting. As noted earlier, for marine structure, subject as they are to dynamic, cyclic loads, the total steel area at the joint should be equal to:

$$A_s = \frac{A_c f_{ce}}{\phi f_y} \tag{4.2}$$

Typical scaffolding may be used for horizontal slabs, but in many cases the slab will be high above any supporting structure. Then, trusses may be used, supported on and spanning between walls. They may be embedded in the concrete. Stay-in-place forms may be employed to eliminate the need for stripping. The reinforcing itself may be prefabricated to serve as an internal truss, with stay-in-place forms hung from it.

Horizontal and sloping surfaces may be formed using precast segments set in place, aligned, and joined with reinforcing and cast-in-place joints or with prestressing and epoxy joints. Special care must be taken to ensure bond, such as use of an epoxy bonding agent just ahead of the new concrete, so as to prevent delamination under differential water pressure. Where heavy craneage is available, this enables very rapid construction of complex elements. Match-casting techniques may be effectively employed, to ensure accurate fit of mating surfaces and tendon ducts. Precast elements may also be cast as half-depth segments, to be completed with a top pour of cast-in-place concrete or as merely stay-in-place forms.

If such half-depth segments are designed to work in full monolithic action, then their surfaces must be roughened and reinforcing ties provided at relatively close spacing to prevent laminar cracking.

4.3.11 Tolerances

Construction tolerances will generally be given for at least the following:

1. Geometry of cross-section
   a. Deviation from true position along vertical axis
   b. Deviation from true circle or polygon on a horizontal plane
   c. Deviation from best-fit circle or polygon
2. Verticality: Deviation from vertical and horizontal axis and planes
3. Distances and bearings between vertical elements such as columns or shafts
4. Thickness of members
5. Positioning of reinforcement (through thickness and along wall)
6. Cover over reinforcement and prestressing ducts
7. Positioning of embedments
8. Deviation of prestressing ducts from design profile
9. Deviation in fresh unit weight of concrete

4.4 Hybrid Steel–Concrete Structures

The combination of structural steel and concrete in offshore structures is an obvious development that has finally emerged as the synergistic benefits of combining the two

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materials have been recognized. Two forms of steel–concrete construction offer potential advantages.

4.4.1 Hybrid Structures

The first form is where elements of structural steel are joined to elements of concrete. Each works in its own way, but the connection must transmit structural forces in such a way to preserve its integrity. Examples of hybrid structures are

1. Structural steel superstructures joined to concrete substructures,
2. Structural steel frames supporting exterior concrete walls and slabs,
3. Steel articulated hinge providing the articulated joint between a concrete base and a steel or concrete column of an articulated loading buoy.

For these hybrid structures, the principal concern is the working of the joint under cyclic-dynamic loads. This can properly be taken care of by prestressing the steel element to the concrete element (see Figure 4.16). To ensure successful performance of such a prestressed joint, several aspects have to be given special consideration.

1. Because the prestress tendons are usually relatively short, the effect of take-up and seating losses will be significant. For this reason, prestressing bars have been developed in which the final securing is done by threaded nuts or couplings rather than by wedges.
2. The anchorage zone of the prestressing tendons within the concrete must be carefully detailed to ensure that cracking and progressive degradation cannot occur around and behind the anchorage. The concentrated bearing of the anchorage plate produces high strains, which in turn lead to radial tension: the well-known bursting forces associated with a high-capacity anchorage. For this phenomenon, transverse reinforcement is required.
3. Full bearing is achieved between the steel bearing plate and the concrete. The steel plate (P) of Figure 4.16 must be sufficiently thick to prevent local warping under the highly concentrated forces imposed on it by the tendon anchorages. Obviously, the weld (W) must have had adequate weld procedures, and may require pre- and postheating, to ensure full development of the connection. The plate must have proper through-thickness to prevent laminar tearing. As noted, the bearing of plate on the concrete must be uniform. Two methods are used. In the first, the gap between the plate and the concrete is filled with cement mortar or a special bearing compound (noncorrosive) or is injected with epoxy. In the second, the concrete is ground and the plate milled, both to a fine tolerance such as 0.1 or 0.2 mm, the allowable tolerance depending on the size of the plate and its stiffness (thickness).

4.4.2 Composite Construction

The second form of steel–concrete construction is where steel plates are joined with concrete so that the two form one structural element in themselves. The two must have mechanical means for developing shear between the steel and concrete components.

Typical arrangements are:

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1. A steel beam is joined to the concrete by welded studs. The concrete takes compression; the steel plate takes tension and transverse shear. This is the scheme commonly employed in bridge construction.
2. Special shear connectors, in the form of transverse bars or vertical plates (with holes), are welded to the steel.
3. Two steel plates, spaced apart, are filled with concrete. The two plates are tied together with steel diaphragms, or bolts, or perhaps even short bars. The concrete ensures plate and shell action to distribute local loads and transmit horizontal shear.

   A prefabricated version of arrangement 3 above is now commercially available under the name CORUS Bi-steel.

The external steel plates provide tensile capacity. The internal concrete provides compressive capacity, and the diaphragms, studs, or bars provide shear capacity. With such sandwich construction, the steel plates will, of course, be forced to carry any compressive strain imparted to the concrete. Buckling inward is prevented by the concrete fill; buckling outward is thus the typical mode of failure and can be restrained by the through-thickness ties. Thus not only the size of the shear connectors but their spacing and welding become important. Figure 4.17 indicates some forms of composite construction.

Where welded studs are used, the amperage and procedures must be in accordance with the manufacturer's recommendations. These welds are heavily stressed in shear and bending at points of high beam shear. The surface of the steel should be clean to develop good bond with the concrete.

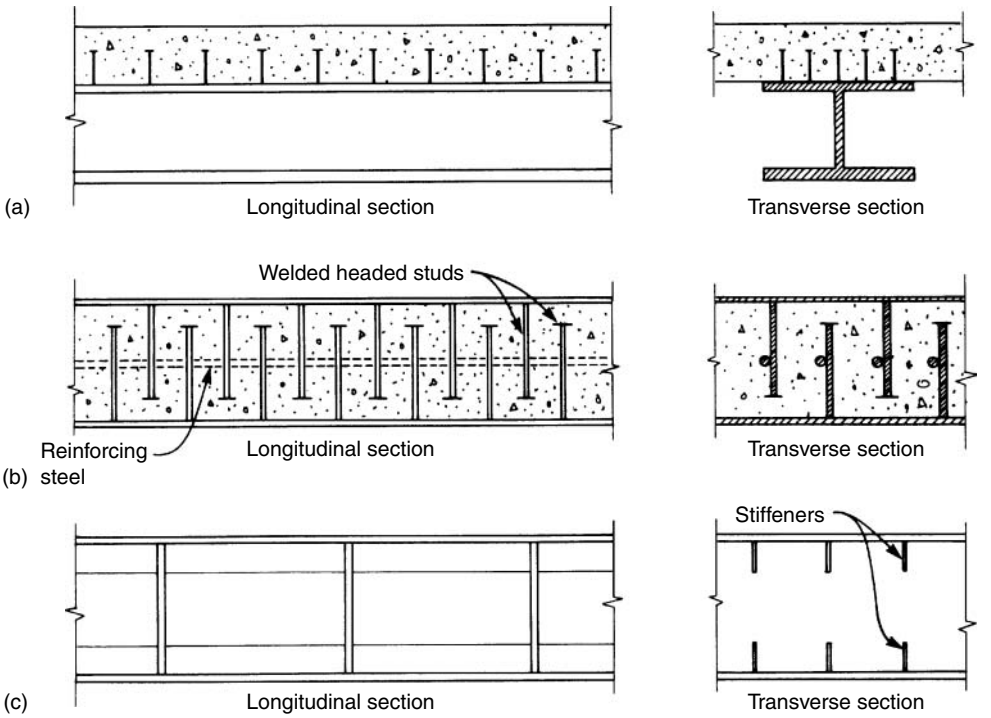

FIGURE 4.17
Forms of composite construction.

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Placement of concrete into sandwich composite systems requires careful planning and procedures to ensure complete filling. Self-consolidating "flowing" concrete is recommended. Small holes may be necessary in internal stiffeners and diaphragms to prevent trapping of air or bleed water. It will frequently be found practicable to leave temporary (or sometimes permanent) holes in one of the plates for access for placing the concrete by pump. The hydrostatic head of fresh (unset) concrete, especially when placed under pressure by pump, may deflect the plates.

Composite construction appears especially attractive where offshore structures must resist high local impact forces, such as those due to ice or ship and barge collision and for the external walls of concrete structures where cracks and minor leakage are unacceptable and unrepairable. Composite construction is increasingly used to provide stiffness and ductility to steel tubular piling. In many marine installations, the maximum bending occurs just under the pile cap. Thus, concrete fill can provide ductility, provided it is detailed to act in composite action with the steel tubular.

4.5 Plastics and Synthetic Materials, Composites

Increasingly, plastic and similar synthetic materials are being utilized in the marine environment. Glass, carbon, and aramid fibers are embedded in a resinous synthetic polymer. Uses range from glass fiber-reinforced plastic for pipelines to neoprene and natural rubber fenders and bearings, to polyethylene bags for slope protection and polyurethane foams for buoyancy. Porous geotextile filter fabrics are extensively employed under riprap to prevent leaching of sand. Epoxies are injected for repair or applied as jointing and coating compounds. Polyethylene pipes (HDPE) have been used for cold water pipelines in depths up to 2000 ft. (600 m) off the island of Hawaii, and most recently, on the 11-km outfall at Montpellier, France. This latter line was fabricated in 550-m lengths in southern Norway and towed afloat for 2000 miles to the Mediterranean, where concrete weight collars were slid over the HDPE pipes to form a continuous protection. The pipe was then floated out, put under tension and sunk into a predredged trench by a combination of water filling and internal air pressurization. The pipeline was then covered by articulated concrete mattresses to protect it from fishing trawlers.

Kevlar, nylon, and carbon fiber mooring lines are in common use in floating offshore operations. Glass fiber-reinforced plastic is used for fender piles to protect wharves. Fiberglass and carbon tendons have been employed as prestressing tendons on an experimental basis. Ductility of concrete piles and columns has been increased by encasement in aramid fibers. Carbon fiber sheets, affixed to the bottom of beams, increase the bending capacity while carbon fiber sheets, affixed to the sides, increase the shear capacity. Aramid fibers (Kevlar) are increasingly utilized in deep-water mooring systems. Poltruded fender piles (fiberglass reinforced polyethylene) are being used by the U.S. Navy on berthing facilities.

Shapes and bars of composites, some of which contain carbon fiber, and others, fiber glass, are being experimentally applied to provide corrosion-free reinforcement. Carbon fiber mesh is available.

Carbon fiber and fiberglass wires are being experimentally tested as prestressing tendons. The most difficult problem is the provision of slip-free anchorages.

From the constructor's point of view, certain aspects must be considered.

1. Where plastic is to be applied in the field, the joint surfaces must be clean, of proper texture (roughness), dry, and at the proper temperature to permit curing. Epoxies are especially sensitive to moisture and dilution by water, unless they

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are especially formulated with a hydrophobic component to be used underwater or on damp surfaces.

2. Many jointing and sealing materials have a closely controlled thickness over which they are effective, with a narrow tolerance range.
3. Extreme temperature changes may lead to delamination of steel–neoprene bearings and fender units, due to change in properties and differential thermal contraction–expansion.
4. Many plastics, especially polyethylene, are subject to ultraviolet (UV) degradation, unless they are formulated with a pigment to give them added UV protection.
5. Most plastics are positively buoyant in water; this makes them difficult to place underwater. Some geotextile filter fabrics are now purposely manufactured to have negative buoyancy and thus facilitate placement. Fiberglass pipe may require supplemental weighting, e.g., by precast concrete saddles, to ensure stability during placement and service.
6. Geotextile membranes and fabrics are generally supplied in rolls and sheets, which are to be overlapped in installation. The laps are a cause of many difficulties both in placement and service. Adequate, even excessive, overlap is usually a wise precaution for the constructor in order to accommodate irregularities in the seafloor and the tolerances associated with work underwater and under wave action.
7. Rigid plastics are susceptible to damage from impact and abrasion. They generally require special softeners underneath concrete saddles.
8. Plastic pipe such as polyethylene is subject to internal fatigue at locations of concentrated stress. Therefore, all attachment points must be reinforced and care taken to ensure a distributed, rather than a concentrated, bearing force. A polyethylene pipe for an OTEC project was under tow near Hawaii. Its weighted end was being held up by wire rope, which in turn was attached to the barge. Under sustained load, with cyclic stresses due to waves, the pipe ruptured, allowing the line to break loose. A major salvage effort was required for recovery.
9. Polysulfide sealants can be rapidly disintegrated by bacteria under some conditions. In one case in the Arabian Gulf, the disintegration took place within a few months, before the construction contract was completed and accepted.
10. Flexibility properties change rather dramatically with temperature, so that a material that is very pliable at 15°C may be stiff and brittle at −10°C.
11. Plastics are generally anisotropic in their properties, depending on the orientation and concentration of the fibers.
12. Plastic mooring lines, especially those of nylon, degrade over time in saltwater.
13. Epoxy coatings may delaminate in saltwater.

Having cited some of the problem areas, it is important to look at the many advantages offered by the use of such materials. They are free from corrosion. They generally have low friction factors. They are light in weight. Dense polyurethane and dense epoxies are used on icebreaker hulls to provide corrosion protection and reduce ice friction. Teflon pads are used to reduce friction when skidding heavy jackets onto barges and again during launching. The light weight of these materials eliminates many handling problems. Many elements can be floated. Mooring lines of plastic are of special benefit in the deep sea and also in short lines, where flexible stretch is required to absorb impact. Polyethylene

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pipes can be "buckled" transversely during installation and later regain shape, all without damage. Their positive buoyancy permits them to be installed above the seafloor and anchored down at intermittent points. Coflexip pipelines, composed of specially bonded steel-reinforced neoprene, are extensively used for flexible risers and connections for offshore oil production.

Kevlar is being widely adopted for mooring lines and light-duty hoisting lines, especially in deep water, where its almost neutral buoyancy is highly beneficial. Kevlar can be obtained in either highly flexible or relatively stiff form. Nylon is also used for such lines but has a very low modulus. Nylon and some other plastic mooring lines are subject to internal fatigue when immersed in water. Carbon fiber mooring lines are being considered because of their strength, light weight, and stiffness, despite their high cost.

Filter fabrics can be laid as mats underwater, generally ensuring more complete, economical, and effective protection against sand migration than can be attained by graded rock filters.

Carbon fiber sheets may be bonded by epoxy to the surface of concrete to prevent opening and closing of active cracks in critical zones. In some cases, polyethylene liners are bonded to steel or concrete pipe in order to provide corrosion protection, reduce ion transfer into the fluid, and reduce friction. It can be wrapped in overlapping strips. This is also one way to affix vortex-shedding strips to riser pipes, mooring cables, and cables of cable-stayed bridges.

Polyethylene is also being utilized to provide corrosion protection to steel pipe piles and to wire rope cables. The guys for the guyed-tower structure Lena are encased in polyethylene for long-term corrosion protection. Rigid polyurethane incorporated in grids of steel plates or reinforced concrete cells increases the impact resistance. This is used to buffer the impact of ship collision and terrorist attacks. It appears certain that the use of plastics in the marine environment will continue to grow. Because of the wide variety of properties of the various materials, the constructor must take special care to verify the special requirements for installation under the ambient conditions involved.

Carbon fiber (CFRP) reinforcement can increase the ultimate strength compared with conventional steel welded wire fabric, reduce crack width to less than 0.25 mm at a load level up to 60% of ultimate, and reach strains up to 0.2%. Thus, it would allow the service load of these concrete panels to be increased by a factor of two, while reducing the required cover(LaNier 2005), as noted earlier. These CFRP grids are very difficult to hold in accurate place while placing the concrete.

4.6 Titanium

Titanium is the "ultimate" material for marine applications, due to its strength and freedom from corrosion. However, it is very expensive.

Titanium is used in critical marine installations that are subject to rapid corrosion, such as saltwater ballast lines that are in frequent use. Titanium cladding was applied to the steel shafts of the Trans-Tokyo Bay Bridge.

Titanium structural elements can be rolled with the following properties:

Strength	800–1200 MPa (120,000–160,000 psi)
Endurance limit under cyclic loading	400–500 MPa (60,000–70,000 psi)
Unit weight	48 kN/m^3 = 4.8 T/m^3 (300 lb/cu. ft.)
Cost	Five times that of steel

Future developments in metallurgy may make titanium more available at lower costs.

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4.7 Rock, Sand, and Asphaltic-Bituminous Materials

Rock is, of course, extensively used in coastal construction for seawalls, breakwaters, and revetments. For offshore construction, it is used to protect the foundations of structures from scour, to protect submarine pipelines from current-induced vibration, trawler boards, and impact, and to protect the slopes of embankments from wave and current erosion.

The four principal properties specified in the design relate to specific gravity, size, abrasion resistance, and durability. Increased density of the rock permits use of smaller-sized rock while still maintaining stability against erosion. The stability of rock under wave and current action is approximately proportional to the cube of the underwater density:

$$S = (\text{sp.gr.} - 1)^3 \tag{4.3}$$

Thus, the contractor may be able to obtain the required results with denser material that is easier to handle and place and to penetrate with piles. Other requirements for rock relate to soundness in seawater and impact resistance.

Size of rock fragments is usually specified as a nominal maximum dimension or weight, with a gradation of smaller rock fragments. In most cases, the most difficult and expensive rock for the constructor will be the larger size. In quarrying, therefore, the constructor will choose methods designed to maximize the production of the larger rock. Usually the constructor will still end up with an excess of finer material, some of which may be used elsewhere on the project, some of which may be used for temporary roads, and so forth. Conversely, to prevent sand migration through embankments, filter rock must be well graded. This usually requires screening and, in some cases, crushing. A significant problem for the constructor is to ensure that when a range of sizes is specified (a gradation curve), each area and zone in the competed structure ends up with a reasonable approximation to that gradation. Although the constructor may have complied with the specified gradation in total, it requires experience and skill to ensure that each batch as placed is properly graded.

Thus in many practical applications, placement of a graded mix in sufficient thickness will achieve the intended results adequately. Alternatively, appropriate geotextiles may be used to act as a filter.

When rock is transported, there is always some abrasion and breakage. Thus, there will be a layer of fines that will accumulate on the deck of a barge. In many cases, this can be placed or wasted without harm; in other cases where porosity and permeability are important, precautions must be taken to ensure that this fine material is not allowed to contaminate the specified rock. Especially when filling sheet pile cells to be densified and when placing aggregate for later filling by intruded grout, care must be taken not to place the fines in one layer where they will prevent water and grout flow.

Durability of rock is normally specified to ensure against disintegration in seawater, primarily due to sulfate expansion. Rock produced from arid and desert regions is particularly suspect, as it has not been subjected to normal weathering.

Sand is generally specified by internal friction angle and gradation, the two, of course, being related. Sand is often produced hydraulically, which allows washing overboard of fines in order to achieve the prescribed gradation. The principal problem in sand production is that of excessive fines, which prevents proper densification and makes the embankment subject to liquefaction and, above water, to frost heave.

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Asphalt and bitumers are used to bind rock and sand into flexible yet scour-resistant mattresses. Great technical advances have been made in this regard by Dutch engineers, who have perfected the ability to place asphaltic and bituminous materials underwater, including placement while hot. They have also developed various gradations and percentages of binder material to permit or restrict permeability as desired for the particular application.

Rubber asphalt, for which old tires have been blended into asphaltic mixes, has a number of desirable properties for marine application, since it has greater extensibility and flexibility over a range of temperatures. A rubber asphalt layer was used on Global Marine's Super CIDS platform between the concrete and steel elements to provide uniform bearing in the warm waters of the Japanese summer, while later providing high lateral shear resistance in the cold Arctic waters.

Roll on, thou deep and dark blue ocean, roll!
Ten thousand fleets sweep over thee in vain.

Lord Byron, "Childe Harold's Pilgrimage"

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5

Marine and Offshore Construction Equipment

5.1 General

The demands of the marine working environment, coupled with the demand for large-scale structures, have led to the development of a great many types of specialized and advanced construction equipment. The response of equipment manufacturers and constructors has been rapid and effective. The availability of construction equipment of greater capabilities has in turn played a major role in altering construction methods and in making it technically feasible and economically justifiable to construct complex structures in extremely demanding environments. These advances will continue as industrial development, principally the offshore petroleum industry, military requirements, and maritime commerce, continue their current rate of growth.

This major construction equipment has been designed to work in and under the sea and has drawn heavily on naval architecture to ensure serviceability and stability as well as limited and predictable motion response under the prevailing marine and offshore conditions. This extension from conventional barges and ships, intended primarily for transport, construction, drilling, and dredging operations has in turn forced the naval architectural profession to develop a methodology adaptable to a wide variety of configurations and dynamic forces.

Safety must be paramount in offshore operations. The nature of the work is inherently demanding and dangerous. The equipment must be designed not only for serviceability but also for safe operations.

Marine equipment, and especially offshore equipment, is very expensive: each hour has a high value in ownership or rental, plus high operating costs. Therefore, the equipment must be designed with reliability and redundancy. As a general rule, it should be capable of efficient operations in 70% or more of the days in the working season. Construction engineers must understand the capabilities and limitations of the equipment they use. They must be alert to detect early signs of problems before they develop to catastrophic proportions. Thus, a full understanding of equipment performance is essential. In subsequent subsections of this chapter, principal generic types of marine and offshore construction equipment will be discussed.

The marine construction industry has been subject to dramatic cyclic variations, from overdemand to recession. In times when the demand for large specialized equipment exceeds the supply, two responses have developed. One is the placement of orders with shipyards and crane manufacturers for new construction of the standard offshore equipment, upgraded to allow its use in deeper water and in exposed environments. The other, very interesting development has been that in which existing equipment is being modified and new procedures are being developed in order to perform tasks which hitherto were

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only possible with large conventional equipment. These latter are making extensive use of the newly developed hydraulic jacks, with long strokes, high capacity, and the ability to accommodate transverse relative motion by means of rollers and low-friction materials such as Teflon. Greater use is being made of buoyancy; gantry barges and float-over techniques supplementing the large crane barges. For inshore marine operations, such as bridges and locks and dams, these same two contrary approaches are being employed. This is especially true where physical constructions limit maneuverability and draft limits access.

There are a number of basic considerations applicable to all offshore construction equipment. These are motion response, buoyancy, draft, freeboard, stability, and damage control.

5.2 Basic Motions in a Seaway

A typical floating structure has six degrees of freedom and hence six basic response motions to the waves: roll, pitch, heave, surge, sway, and yaw (see Figure 5.1).

Wave action on vessels is exhibited by two effects. The first-order effect is the oscillatory force. The second-order force, often called the wave drift force, is a relatively steady force in the direction toward which the waves are propagating. In irregular seas, it varies slowly, with a period of 1 min or more. In a real sea, the vessel will be responding to a complex set

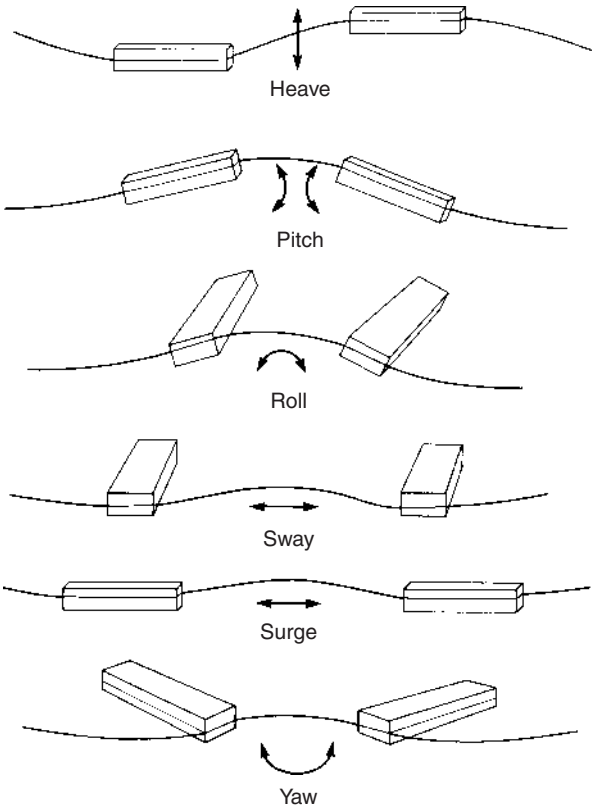

FIGURE 5.1
Six degrees of freedom of floating structure.

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of excitations, differing in direction, frequency, phasing, and magnitude. The responses are therefore a combination of all the above. The typical construction vessel is moored with mooring lines of rope or chain, which give nonlinear restraint to the movements of the vessel. Energy is stored in these lines, to be subsequently released to the vessel as the restoring forces return the vessel to its mean position. Although mooring lines are normally designed to prevent low-frequency displacements in surge, sway, and yaw, they also act to impart both low- and high-frequency excitations into the vessel. Some highly sophisticated offshore construction vessels employ dynamic positioning to enable station keeping.

The various types of motion interact to reduce or amplify the motion of any individual point on the vessel. A point of special interest in derrick barges, for example, is the boom tip. Due to the interaction of the six response motions, the boom tip may describe a complex three-dimensional orbit in space.

Vessel motions are affected by hydrodynamic interaction forces, especially when the vessel is in close proximity to a boundary—for example, when in shallow water, so that there is little water between the hull and the bottom. Vessel response is highly frequency dependent. Each vessel has its natural period of response in each of the six degrees of freedom. Note that in addition to a resonant period of the vessel itself at moderately high frequency, there may be a natural period at low frequency for the full vessel-mooring line system.

Maximum response in pitch will occur when the effective wavelength parallel to the vessel longitudinal axis is two to three times the vessel's length. Then the vessel will be riding the slope of the wave. Under such a condition it will also have maximum surge; in effect, it will be trying to surf. The trend in design of offshore construction vessels is to make their length equal to or larger than of the maximum wavelength in which they are expected to work.

This is also why such great difficulty is experienced when using conventional offshore floating equipment in areas such as off West Africa or the southwest coasts of Africa and Australia, where very-long-period waves from the southern ocean, even though of moderate height, cause excessive response of the vessel.

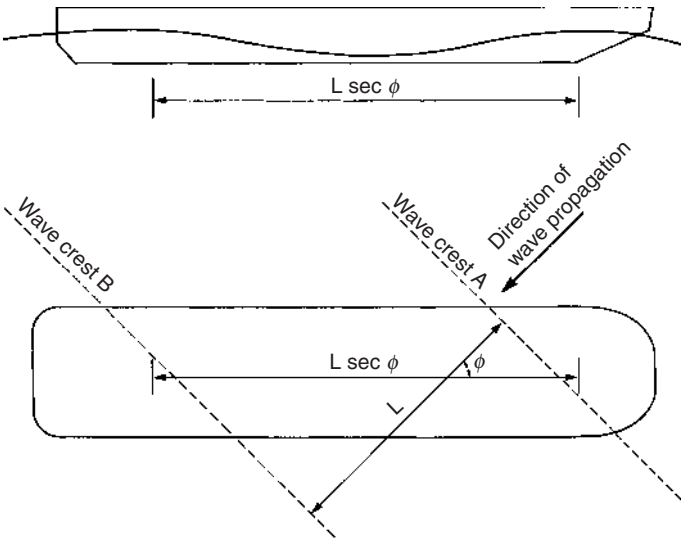

FIGURE 5.2
Effect of quartering waves on long offshore construction vessel in producing both bending and torsion.

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The effective wavelength is the distance between crests as measured parallel to the longitudinal axis. A sea of shorter wavelength, acting at an angle to the vessel's axis, can produce a significant pitch response. The directional spread of waves can also cause a response in pitch even in a beam sea (see Figure 5.2). Similarly, even when the barge is headed directly into the sea, there can be a significant response in roll.

In shallow water, the long-period swells are shortened in length, so that they may approach critical length in relation to response of a typical barge. For example, with a barge of 120 m length and a swell period of 18 s, the deep-water wavelength will be $1/2T^2$ or 500 m. In water depths of 15–20 m, these waves will shorten to 400 m or less, resulting in high pitch and surge.

In roll, most barge-type construction vessels have a natural period of 5–6 s. The average wind waves in a workable sea have a 5- to 7-s period, giving a wavelength of 40–70 m. This is the reason offshore construction vessels are usually designed with beams greater than 25 m.

5.3 Buoyancy, Draft, and Freeboard

One of the oldest engineering principles of history is Archimedes' principle that the displacement will be equal to the weight. The same results can be reached by integrating the hydrostatic pressures acting on the vessel under still-water conditions.

Weight control is always a concern during the fabrication of structures. A check on weight can, of course, be kept by measuring the displacement. There are, however, a number of factors that may act to reduce the accuracy of simple draft measurements and displacement calculations:

1. Variations in the density of the water
2. Deflections and deformations of the structure
3. Tolerances in the underwater dimensions, and hence in the displaced volume
4. Inaccuracies in calculation of ballast water and inadvertent drainage water
5. Absorption of water, for example, into concrete.

Draft is determined by geometry and displacement; it is the depth below sea level of the lowest point of the structure, as measured in still water. Phenomena acting to increase draft are "squat," the hydrodynamic pull-down force acting on the hull when the vessel is moving through shallow water; "heel," which is the list of the vessel under wind and eccentric loading; and variations in trim. Pitch, roll, and heave can also, of course, reduce the bottom clearance.

Reduced bottom clearance can adversely affect the vessel's movement through the sea. Water pressures build up underneath the vessel, tending to increase the added mass and reduce speed: more water is pulled through the surrounding sea. The vessel loses its directional stability and starts to make rather extreme excursions in yaw and sway.

Freeboard is the deck height above the still-water level; like draft, it is modified by squat and heel in shallow water, by list and trim, and temporarily by pitch and heave. Freeboard is usually designed to minimize the frequency of waves which can build up and overtop the deck. The wave height adjacent to the vessel's side is built up above normal levels by refraction and mach-stem effects, the latter being the result of accumulated energy raising the wave crest. Freeboard may also be increased in order to reduce the amount of spray on deck under high winds. High winds can blow the tops off the standing wave (clapotis), onto the deck.

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5.4 Stability

There are three major parameters controlling stability: the center of gravity, the center of buoyancy, and the water plane moment of inertia. See Figure 5.3. Submerged vessels, with no water plane, depend on the center of gravity remaining below the center of buoyancy.

The formula for stability of a surface floating vessel is

$$\overline{GM} = \overline{KB} - \overline{KG} + \overline{BM} \tag{5.1}$$

where $\overline{BM} = I/V$. K is the geometric centerline at the hull bottom, G is the center of gravity, B is the center of buoyancy, M is the metacenter, I is the transverse moment of inertia of the water plane area, and V is the displaced volume.

The vessel will have inherent stability at small angles of roll as long as $\overline{GM}$ is positive (see Figure 5.4). The righting moment is $\overline{GM}$ sin q times the displacement, where q is the angle of roll. For small angles of roll, sin q can be assumed equal to q (in radians).

The transverse moment of inertia of the water plane of a typical barge or other rectangular vessel is given by

$$I = \frac{b^3 l}{12} \tag{5.2}$$

where b, beam and l, barge length. Since V is bld, where d is the draft, I/V reduces to $b^2/12d$. The easiest practical way to find the approximate location of B for other than rectangular configurations is to lay a typical transverse cross section out on graph paper and count the squares. This is especially useful for a vessel with a complex below-water configuration, for example, a semisubmersible. The moment of inertia of the vessel's water plane area is diminished by the sum of the moments of inertia of the water planes of any partially flooded spaces. The reduction in metacentric height and stability is known as the free-surface effect.

For structures having columns or shafts that extend through the water plane, the moment of inertia, I, is approximately proportional to Ar^2, where A is the area of each shaft and r is the distance from each shaft's centerline to the vertical axis of the structure. Thus, the most efficient columns, insofar as stability is concerned, are those located farthest from the axis.

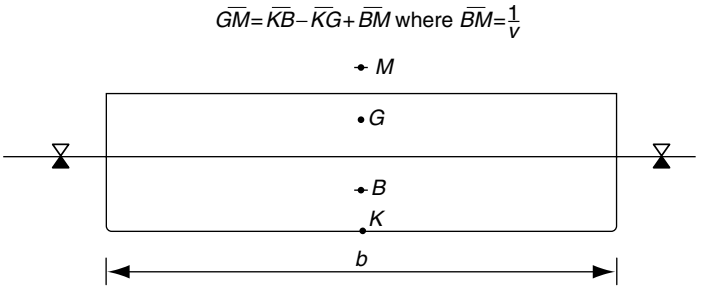

FIGURE 5.3
Relationship between centers of buoyancy, gravity, and metacenter.

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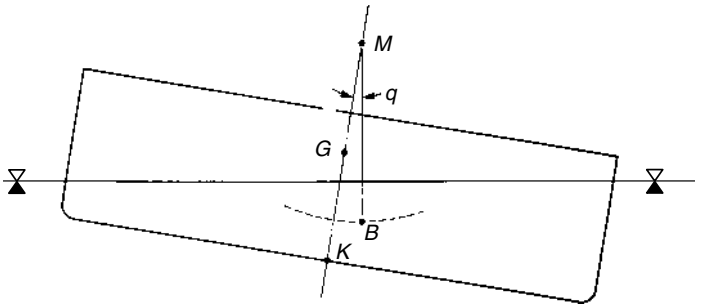

FIGURE 5.4
Effect of metacentric height.

The above criteria are of value only for small angles of roll. At large angles of roll, the geometry may change rapidly. The following are typical areas of such sensitivity:

1. When the edge of deck goes awash: The waterline plane diminishes rapidly, and the moment of inertia diminishes rapidly.
2. When the center of gravity is very high, as in a jack-up with legs raised; then, the stability becomes excessively sensitive to the instantaneous transverse moment of inertia.
3. Special care must be taken when the waterline area reduces materially with an increase in draft (see Figure 5.5). Such a stability problem occurs with semisubmersible vessels, with conical structures such as those used in some Arctic offshore platforms, and with gravity-base structures which have a large base but only shafts extending through the waterline.
4. When computing stability of a crane barge or derrick barge, the load picked is computed as though it were placed at the boom tip. The center of gravity for the vessel must include this load.

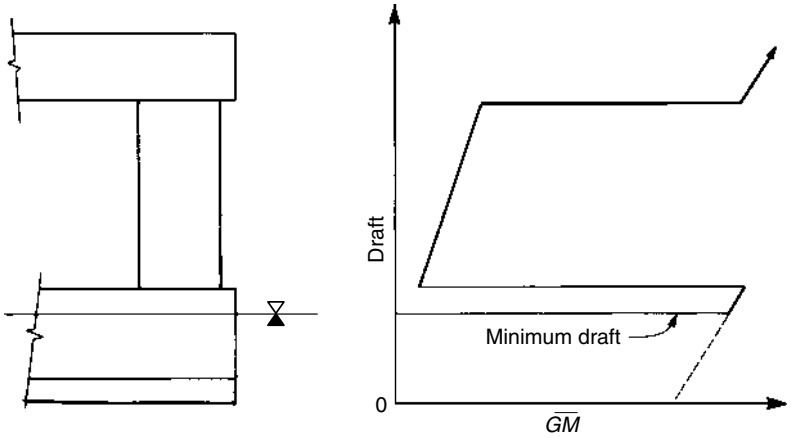

FIGURE 5.5
Variation of metacentric height $\overline{GM}$ with draft of semi-submersible.

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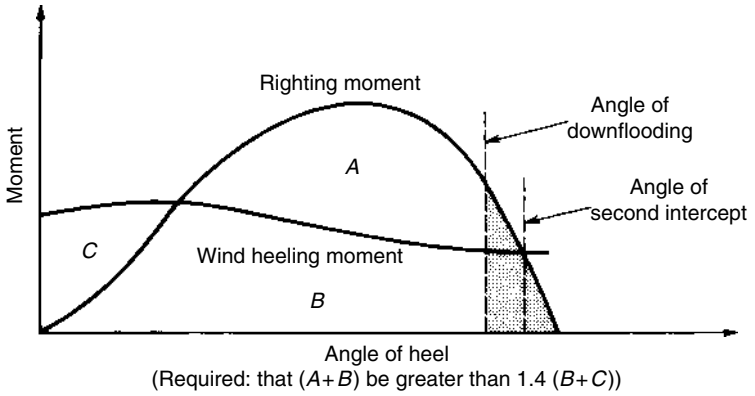

FIGURE 5.6
USCG and ABS intact stability criterion.

Righting moment is given by $\overline{GM}$ sin q D where D, displacement. Righting moment may be augmented by a supplemental lift from a crane barge P multiplied by the height of the picking point (hook) above the metacenter M.

A meaningful guide to stability is provided by the curves of righting moment vs. wind heel. A typical curve for a relatively stable vessel is shown in Figure 5.6. Many offshore organizations have adopted this U.S. Coast Guard and American Bureau of Shipping (ABS) rule, which is essentially as follows:

The area under the righting moment curve to the second intercept or downflooding angle (or angle which would cause any part of the vessel or its load to exceed allowable stresses), whichever is less, is to be not less than 40% in excess of the area under the wind heeling moment curve to the same limiting angle.

The above would apply, for example, to a barge carrying a large module, and the limit on allowable stresses would apply to the tie-downs as well as the barge and module.

Often a minimum $\overline{GM}$ for example, +1.5 m, is also required. When moored, the effect of mooring and anchor lines should also be considered as an alternative loading case. However, dependence should not be placed on the mooring lines for basic stability. A relatively critical situation exists for a vessel with a righting moment curve like that shown in Figure 5.7, which may be found with some jack-up barges in transit with legs raised

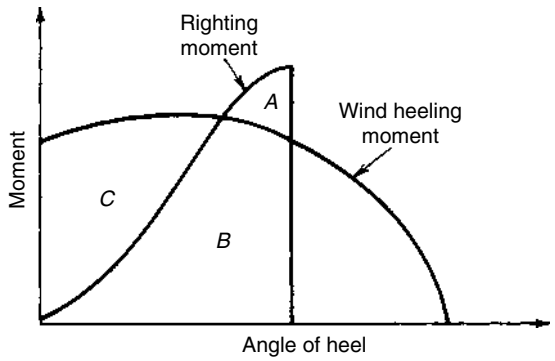

FIGURE 5.7
Unacceptable response of jack-up with legs raised, despite high $\overline{GM}$.

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because of shallow water. A catastrophic accident occurred with a construction barge carrying concrete materials. As the materials from deep in the hull were depleted, the $\overline{KG}$ increased, the draft lessened, and the KB decreased. The $\overline{GM}$ became negative and the barge capsized, with tragic loss of life.

Serious problems of stability as well as potential collapse of the boom may occur if the load is allowed to swing with the roll. Use of tag lines, properly sized and secured, can reduce this tendency. In the event that a load gets away—that is, swings wildly free—the only solution may be to lower the load into the water in order to reduce its weight and dampen its motion. The act of submergence will also lengthen the period of swing, bringing it out of resonance with the natural period of roll of the barge.

5.5 Damage Control

Marine construction vessels are subjected to collision from barges and boats to a far greater degree than normal vessels engaged in transport. The latter avoid close proximity to other vessels, whereas an offshore construction vessel must work with these other craft alongside. The marine construction vessel is frequently picking and setting anchors; it is not unusual for a fluke to rip into the side. Finally, this rig must also work adjacent to platforms and other structures. Every precaution is taken to avoid collision with the structure because of the danger to the equipment, facilities, and wells; for an operating petroleum platform or terminal there is also the danger of fire from hydrocarbon release.

Damage control considerations require that vulnerable areas be subdivided into smaller compartments, that all manholes and most doors be equipped with watertight gaskets and dogs, so that they may be kept closed except when actually in use, and that areas where anchors will rub or boats lie alongside be armored or fendered as appropriate.

During tow or when moored in heavy weather, green water will come over the decks of most barge-type vessels. An inadequately closed manhole will let in a large amount of water within a short time (see Figure 5.8).

FIGURE 5.8
Riding out a storm at sea during installation of outfall.

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Temporary attachments and supports are frequently welded to the deck. If welded only to the deck plating, they may pull free; the welds are in tension normal to the deck plate, and the deck plate may be unsupported below that point. Therefore, holes are frequently cut in the deck plating so that attachments may be welded in shear to the bulkheads below. These must subsequently be seal-welded to the deck plate to prevent water entry. These temporary attachments, especially padeyes, winch foundations, and mooring attachments, are often subject to extreme lateral impact loading as well as low cycle, high amplitude fatigue. The connection should be detailed so that failure occurs in the connection, not in the vessel's structure.

Construction vessels often take a sudden list due to a shift in the load, such as a crane swinging or a heavy deck module moving to one side or lifting off. These sudden lists may coincide with a roll and temporarily submerge an above-deck door, which has been left open, or a vent, or other opening below the oncoming wave. Other flooding accidents have occurred due to broken portholes (or ports left open).

Workboats and small barges are often pulling heavy mooring lines, whose weight may cause a temporary trim down by the stern or bow, resulting in wave overtopping. Boats pulled or running astern are especially subject to taking water over the stern sheets. Flooding into the stern well may have several severe consequences. It may enter the engine room or control room spaces and short out the power. Even a small amount of water in a compartment gives a free surface that reduces the metacentric height; that is, the righting moment available at the waterline of the vessel is reduced by the free-surface effect of the partially flooded compartments.

Watertight closures designated to be closed during operations must be kept closed and dogged. Two serious accidents occurred to workboats during the 1970s when, because of the calm seas and warm weather, engine room doors were left open for ventilation. An operational event caused water to be shipped over the stern; this in turn flooded the engine room spaces. Both boats sank rapidly in each case with serious loss of life.

In a somewhat similar case, in dead calm weather, a semisubmersible derrick barge was making a heavy lift. The load swung to one side, the vessel heeled, and the swing engines and brakes were unable to hold the crane, which rotated to the beam, causing a very extreme heel, so that the upper deck was awash. Doors on the upper deck, supposed to be closed at all times, were wide open. In this case the experienced crane operator prevented a catastrophe by lowering the load into the sea.

Construction vessels are usually equipped with ballast tanks to enable the list (heel) or trim to be controlled. These ballast tanks typically have vents that extend in a gooseneck above deck and are equipped with a flame-arresting screen and a flap valve. One purpose of these vents is to prevent accidental overpressurization of the tank, which might rupture a bulkhead and flood an adjacent space. However, these vents become plugged or may even be intentionally blocked off. For example, cargo might be stored on the deck space in question. The result may be overpressurization and a rupture of an internal bulkhead.

The more sophisticated offshore equipment of today—derrick barges, pipe-laying barges, launch barges, and semisubmersibles—have complex ballast systems to enable their list and trim to be rapidly controlled, even as operations are being carried out. Accidents, even capsizing, have occurred when the controls were short circuited. Therefore, emergency manual controls are also provided. The crew must be trained regarding procedures after a malfunction.

Valve stems sometimes break loose, so that they appear "closed" when actually the valve gate itself is still partially open. Critical valves should be equipped with remote indicators. Critical valves have been opened for testing and then inadvertently left open afterward. These should be equipped with locks or tags as appropriate.

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Steel working under cyclic or impact loads can be subject to fracture, especially at low temperatures when it drops below the transition value. Usually these cracks start and propagate with repeated cycles. Careful inspection of critical areas can locate these incipient cracks before they have propagated to a dangerous degree. They can then be repaired or a crack-arresting hole drilled or strap installed. Decks of barges are especially vulnerable since they are exposed to low temperatures and high stresses.

Closed compartments inside steel vessels can be very dangerous to human life. The steel corrodes slowly but continuously, using up the oxygen in the compartment. In other cases, heavier-than-air gases such as carbon monoxide may accumulate in the lower areas and bilges. Therefore, all compartments that have been closed and all tanks should be thoroughly ventilated before entering.

Fire aboard a vessel at sea is one of the traditional worries. For many fires, the most effective way of fighting them is to close off all air supply and cool down the adjoining bulkheads and decks by water spray. Fires can jump across steel bulkheads by igniting the paint on the other side. Electrical fires and hydrocarbon fires should not be fought with water. Acetylene tanks must be chained or strapped tight to prevent "falling, fracture, and ignition."

Materials, such as casing and all separate units stored on the deck of barges, must be secured against displacement in the event of a sudden list. Casing and pipeline pipe are especially dangerous because of their ability to roll and the large tonnages involved. Shifting loads can cause the vessel to capsize. Steel sheet piles on the deck of an inland marine barge have shifted as the crane barge listed while picking a load.

Often a large module is transferred from a cargo barge or a shore base to sit on the deck of a derrick barge. Even though the duration of exposure is relatively short, the module should be promptly secured with chains or wire lines so that it cannot shift even if the derrick barge lists.

All lifesaving equipment should be maintained in full operating condition at all times. When a life-saving capsule or raft or firefighting gear must be removed in order to carry on operations, it must be relocated or reinstalled immediately afterward. Emergencies can occur at any time, and according to "Murphy's second law," will occur at the worst possible time.

5.6 Barges

An offshore construction barge must be long enough to have minimal pitch and surge response to the waves in which it normally works, wide enough in beam to have minimum roll, and deep enough to have adequate bending strength against hog, sag, and torsion, as well as adequate freeboard. The deck plating must be sufficiently continuous to enable it to resist the membrane compression, tension, and torsion introduced by wave loading. Side plates must carry high shear; they must be stiffened against buckling.

Impact loadings can come from wave slam on the bow, from ice, and from boats and other barges hitting against the sides. Unequal loads may be incurred in bending of the bottom hull plates during intentional or accidental grounding and of the deck plates due to cargo loads. Corrosion may reduce the thickness of hull plates.

The internal structure of a barge is subdivided by longitudinal and transverse bulkheads. Because of the relatively high possibility of rupturing of a side plate, with consequent flooding of the adjacent compartment, the longitudinal bulkheads are usually spaced at the middle third of the beam. A single centerline bulkhead could allow flooding of one entire side, causing excessive heel and possible capsizing.

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Longitudinal bulkheads plus the two sides provide the longitudinal shear strength of the barge. The transverse bulkheads are usually spaced with one just aft of the bow (the collision bulkhead), one forward of the stern, and one or more in the midships region. These provide the transverse shear strength. Quartering waves produce torsion as well as bending in both planes. The torsional shear runs around the girth of the vessel: sides, deck, and bottom.

Typical offshore barges run from 80 to 160 m in length. Width should be one-third to one-fifth the length. Depth will typically run from 1/15 the length. Such ratios have been found to give a reasonably balanced structural performance under wave loadings. Inland barges, subjected to minimal wave loadings and required for operations in shallow water may have depths as low as 1/20 of the length. They may be stiffened by external trussing. Shallow-depth barges are often used in rivers and lakes; these can be very hazardous in exposed locations where not only high quasi-static bending stresses can develop but also dynamic amplification and resonance.

Offshore barges typically have natural periods in roll of 5–7 s. This is unfortunately the typical period of wind waves; hence resonant response does occur. Fortunately, damping is very high, so that while motion in a beam sea will be significant, it reaches a situation of dynamic stability. The corners of barges are subject to heavy impacts during operations, so they must be heavily reinforced. Fenders should be provided on the corners to minimize impact damage to other craft and structures. Fenders should be provided along the sides to minimize damage to the barge itself from other boats and barges as they are docked. These may be a combination of integral fender strakes plus renewable fenders. Bitts are provided at the corners and at intervals along the sides to enable the securing of the barge and any other craft that come alongside. Towing bitts are provided on both bow and stern.

Consideration must be given to the need to temporarily weld padeyes to the deck in order to secure cargo for sea. These padeyes must distribute their load into the hull; proper strength cannot be achieved by just welding them to the deck plate. They will be subjected to fatigue and to impact loads in both tension and shear. In modern offshore barge design, special doubler plates are often affixed over the internal bulkheads so that padeyes may be attached along them. Low-hydrogen electrodes should be used. Alternatively, posts may be installed, running through the deck to be welded in shear to the internal bulkheads.

The deck is often protected by timbers to absorb the local impact and abrasion of the load. This is especially needed for barges that will carry rock that will be removed by clamshell or dragline bucket, or upon which a tracked crane or loader will operate. Manholes are provided in the deck for access to the inner compartments. These must be watertight. There should be a heavy coaming to protect the dogs or bolts that secure the manhole. Once again, the warning must be made about entering inner compartments that have been closed for a long period. They are probably devoid of oxygen and must be thoroughly aerated before entry.

Marine barges are often intentionally grounded (beached) in order to load or unload cargo. The beaching areas must be well leveled and all boulders, and even large cobbles, removed in order to avoid holing or severely denting the barge. Once beached, the barge should be ballasted down so that it will not be subjected to repeated raising and banging down under wave action at high tide.

When heavy loads are skidded on or off a barge, they punish the deck edge and side because of the concentrated loading. Skid beams are often arranged to partially distribute the load to interior bulkheads. A timber "softener" may be temporarily bolted to the deck edge. The barge must be analyzed structurally for each stage of loading to ensure that a side or bulkhead will not buckle under the temporary overload.

Cargo must be secured against movement under the action of the sea (see Figure 5.9). Thus, sea fastenings are designed to resist the static and dynamic forces developed under

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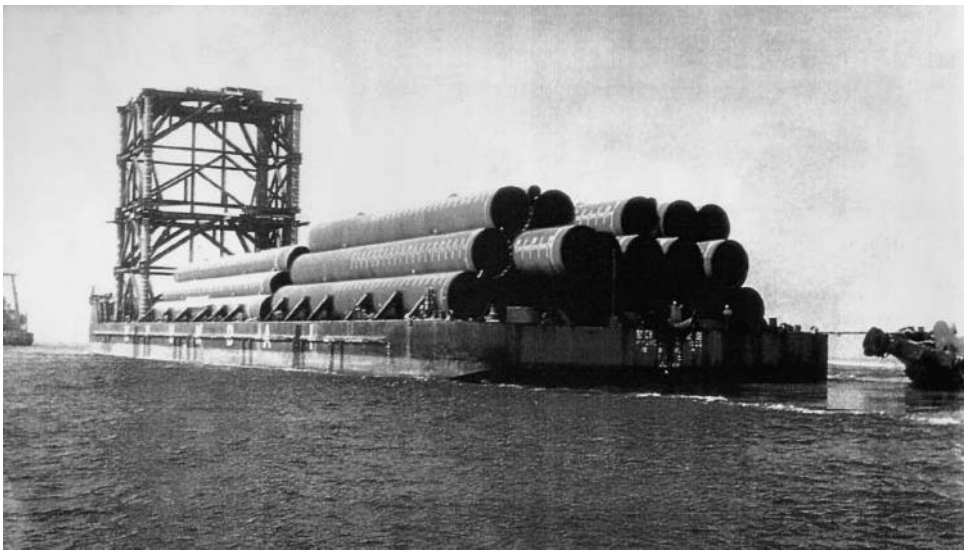

FIGURE 5.9
Steel piles being transported from Korea to Bangladesh. Note structural supports.

any combination of the six fundamental barge motions (roll, pitch, heave, yaw, sway, or surge). The dynamic component is due to the inertial forces that develop due to acceleration as the direction of motion changes. Roll accelerations are directly proportional to the transverse stiffness of the barge, which is measured by its metacentric height. Since barges typically have large metacentric heights, accelerations are severe. Conversely, if due to high cargo, the metacentric height is low, the period and amplitude of roll and the quasi-static force imparted by the load are greater, but the dynamic component may be less.

These loads are cyclic. Sea fastenings tend to work loose. Wire rope stretches; wedges and blocking fall out. Under repeated loads, fatigue may occur, especially at welds. Welds made at sea may be especially vulnerable because the surfaces may be wet or cold. Low-hydrogen electrodes will help. Chains are a preferred method for securing cargo for sea, since chain does not stretch. If structural posts are used, they should be run through the deck to be welded in shear to the internal bulkheads. The slot through the deck should then be seal welded to prevent water in-leakage.

The effect of the accelerations is to increase the lateral loading exerted by the cargo due to the inclination of the barge by a factor of two or more. Flexing of the barge can also have a significant effect on support forces and the sea fastenings. Therefore, deeper, and hence stiffer, barges will experience a smaller range of loads than shallow, less stiff barges.

With important and valuable loads such as modules or jackets, sufficient freeboard should be provided to ensure stability, even if one side compartment or end compartment of the barge has been flooded.

Barges are normally designed to the standard loading criteria of the classification society. These criteria are usually based on submergence of the hull to the deck line, plus an arbitrary load of 3 m of water on deck.

Proposals are often made to build a structure on a barge and then submerge the barge by ballasting and float the new structure off. Actually this has been successfully carried out in a number of cases: the construction of the pontoons of a dry dock in northern Spain, the manufacture of several hundred shallow draft concrete hulls and posted barges near New Orleans, and the construction of Arctic offshore caissons in Japan (see Figure 5.10).

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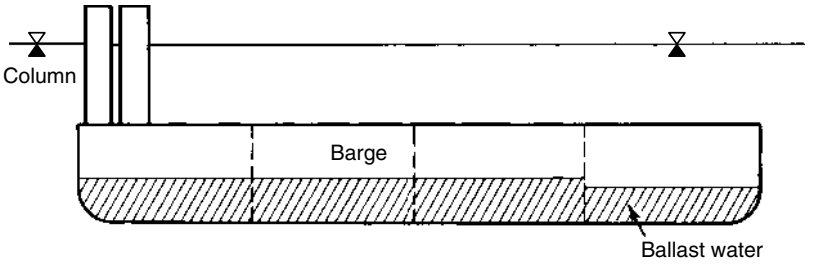

FIGURE 5.10
Submersible transport vessel maintains stability during submergence by superstructure at bow. (Courtesy of Chevron Resources Canada, Ltd.)

However, there are three key items that have to be taken care of:

1. When a barge is submerged by partially filling compartments with ballast, the external pressures are essentially the same as if the barge were empty and submerged to that depth. The hull must be designed for the deepest submergence.
2. Once the deck of the barge is submerged, stability of the barge itself is lost, although the structure on deck may give an effective water plane and thus provide stability during submergence. Once the structure floats free, while the barge itself may initially have stability due to the low center of gravity of the ballast water, as it is pumped, the barge may rise up in an unstable mode, out of control. Stability and control can be provided by columns at the ends which always extend above the waterline (see Figure 5.11).
3. The third problem is that of depth control. The support barge must be neutrally or negatively buoyant in order to launch the structure. Control of depth can then only be achieved by:

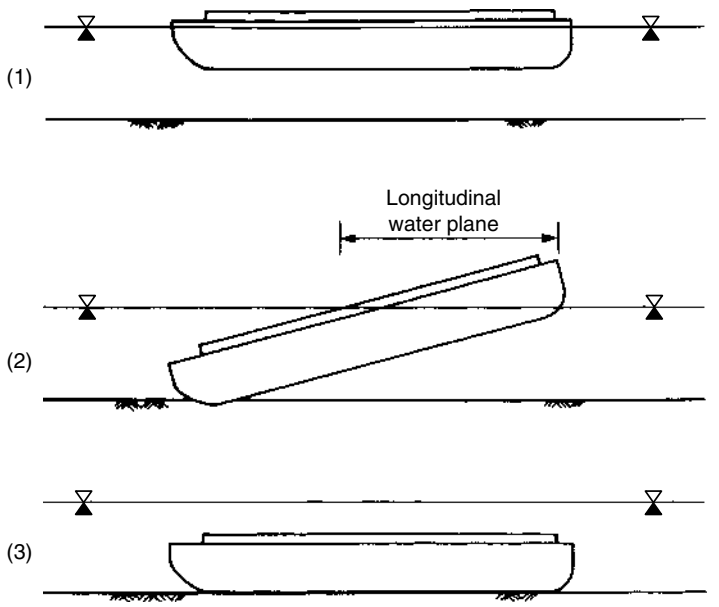

FIGURE 5.11
Tipping a barge to seafloor in shallow water.

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a. Carrying the operation out in shallow water of known depth;
b. Use of lowering lines from surface barges; however, the load distribution may shift, causing lines to fail successively—even small waves may develop large forces in lines due to the heave of the lowering vessels;
c. Use of columns penetrating the waterline, as recommended for stability control; these will tend to regulate the load distribution.

If a barge is to be seated, fully submerged, on an underwater embankment or on the seafloor in relatively shallow water, then it can be tipped down, one end first. Thus, the beam of the barge and the inclined water plane provide stability at this stage. Then, the barge end touches bottom. Now the barge may be fully submerged, gaining its stability from the end of the barge reacting against the bottom (see Figure 5.11). This practice is normally limited to a water depth about one-third that of the barge length.

Note that as the barge is tipped down, the transverse water plane area and moment of inertia are reduced to about one-half of normal. Therefore, transverse instability can develop before the end touches the bottom, and the barge can roll. This acts to limit the depth of water suitable for such an operation. To recover the barge from the seafloor, the reverse procedure is followed, raising one end first.

A barge seated on a mud or clay seafloor develops a suction effect, consisting both of adhesion and a true suction due to differential water pressures. Breaking the barge loose requires that full hydrostatic water pressure be introduced under the bottom and that the adhesion of the clay to the barge be broken.

Extensive experiments by the Naval Civil Engineering Laboratory at Port Hueneme, California, and confirmed by practical experience in the Gulf of Mexico show that breaking loose can best be accomplished by sustained water flooding at a low pressure, less than the shear strength of the clay. Higher pressures will just create a piping through to the sea and prevent development of any pressure. Periods of several hours may be required to develop a fully equalized head of water under the structure. Then by debal-lasting one end first, the barge can be lifted clear. This method was used to free the GBS-1 (Super-CIDS) exploratory drilling platform, after one year's service seated on a clay foun-dation. It proved very successful. It has also been successfully used in initial flooding of graving docks in which a flat-bottomed caisson or barge has been fabricated.

As with ship salvage, a fully submerged barge must be given only limited positive buoyancy; otherwise it may break loose suddenly. If compressed air has been used to displace water in open compartments, the vessel will achieve additional buoyancy with every meter of rise due to the expansion of the air inside and may become uncontrollable. For all the above reasons, submergence of standard barges must be considered only to shallow depths. For deeper submergence, special construction and internal pressurization may be required; these are described in Chapter 22.

5.7 Crane Barges

The term crane barge is used to denote an offshore barge equipped with a sheer-legs crane or fully rotating crane. A sheer-legs crane can pick loads and luff but not swing. The sheer legs consist of an A-frame made up of two heavy tubulars or trussed columns held back by heavy stays to the bow (see Figure 5.12).

The sheer-legs barge is maneuvered by deck engines, tugs, or mounted outboard engine propellers. The crane barge positions its stern at the side of the cargo barge, picks the load,

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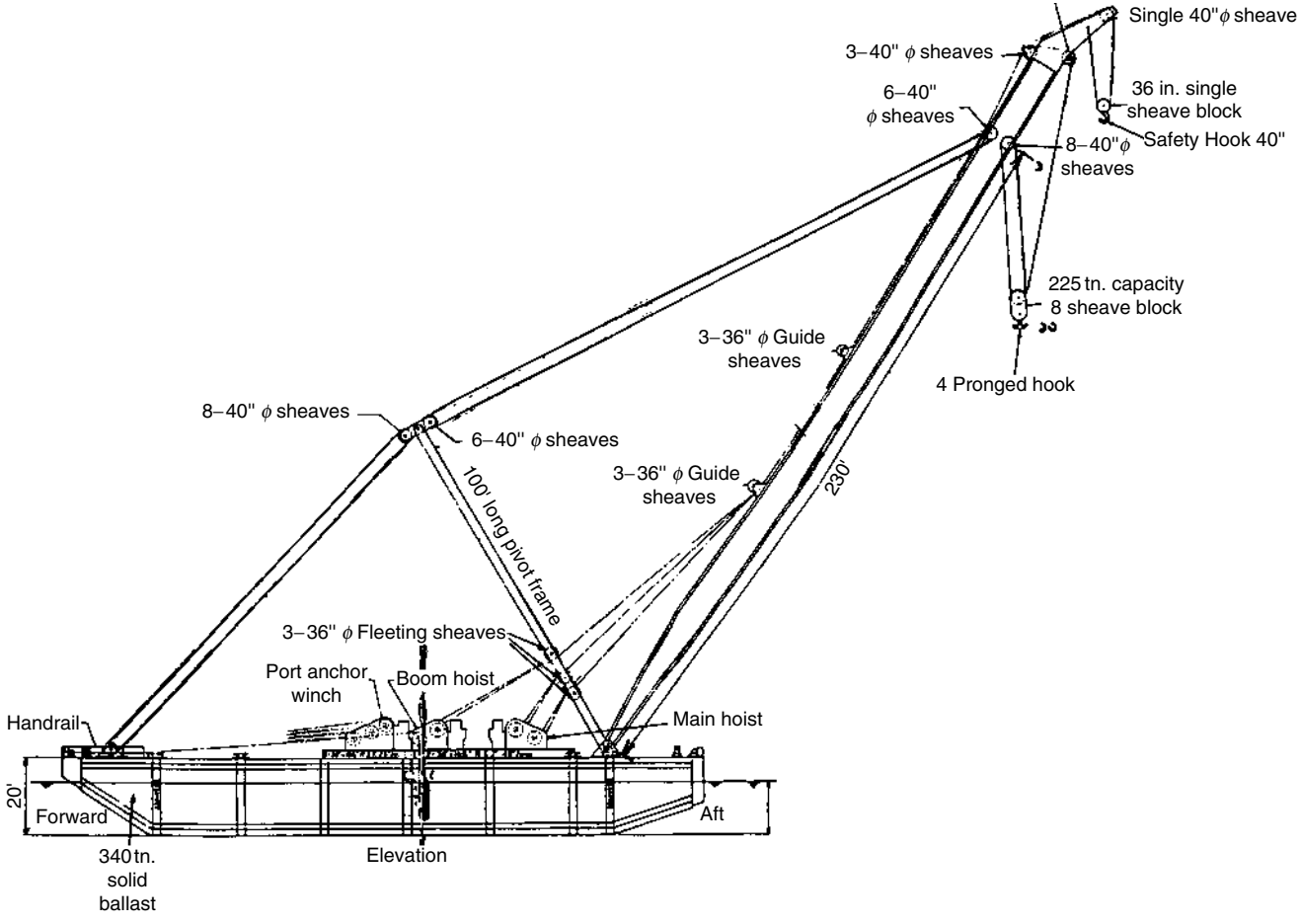

FIGURE 5.12
Crane barge for construction of offshore terminal, Australia.

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and then moves as necessary to set the load in exact position. Modern torque-converter deck engines and propellers with variable pitch allow a high degree of accuracy in positioning to be obtained, for example, of the order of 50 mm. One of the advantages of a sheer-legs crane barge over a fully revolving derrick barge is that the load is always picked over the stern end, hence preventing list from the swing of the crane.

The sheer-legs crane is also much less costly than a full-revolving crane, both in first cost and in maintenance. Because of the need to move the entire barge to proper position to set the load, its operations are slower than those of a fully revolving derrick barge. Further, it cannot choose its heading to minimize motion response to the sea. A sheer-legs crane barge is normally capable of ballasting down by the bow, to offset the trim induced by picking of the load over the stern. The barge must, of course, be designed to resist the hogging moment, which then occurs when the load is picked.

The ability of a sheer-legs barge to lift a module or other large spatial load to a height (for example, in order to set it on a platform deck) is limited by the necessary length of slings and by the interference between load and the sheer legs themselves. The load cannot be allowed to swing into the sheer legs or boom or it may buckle them. Swinging of the load due to pitch will, of course, increase this danger. To prevent such fore and aft swing, tag lines should be used to suck the load slightly in toward the stern; this will prevent it from swinging in this direction. Swinging transversely can be snubbed by the use of tag lines as well.

Typical tag lines for offshore crane barges are $1/2''$ to $3/8''$, 6×37 wire lines to give flexibility, and are controlled by air or hydraulic hoists. Care must be taken to prevent their chafing as the load is moved to new positions in three-dimensional space. Softeners should be provided as necessary.

To pick loads from a barge at sea and then set them on a platform, the sheer legs are usually fixed at the appropriate orientation to serve both. Luffing of the sheer legs under load, that is, raising the sheer legs themselves, is awkward and slow and should normally be avoided. The load should be hoisted from the barge at the top of the heave (of the barge) so that six seconds later, on the next cycle of heave, the load will be clear of the barge. The operator (and foreman) will watch and try to catch a relatively higher wave on which to start the pick. Hoisting speeds depend on the number of parts of line in the blocks and, of course, on the rated speed of the engine and the amount of wire on the drum.

When it comes to setting the load, the problem is reversed. The load will tend to first make contact while the barge is near the bottom of the heave cycle; three seconds later, before the hoist engine can overhaul to slack the lines, the crane barge may lift the load up again. Under any significant sea state and pitch response, the load becomes a battering ram. Therefore, the crane barge should be fitted with a free overhaul capability to allow the load to remain seated once it has landed. In any event, the skillful operator will try and set the load during a period of minimum motion and as close to the top of the crane barge's heave cycle as practicable, to give time for overhaul.

The slings used to lift typical modules and other heavy loads are very heavy and awkward. A whip line, single-part, is run over a sheave at the boom head to help lift the eye of each leg of the slings over the hook (see also Figure 6.21).

The deck engines of a sheer-legs crane barge must be adequate to control the barge's motion in yaw, sway, and surge to a very close tolerance despite the state of the sea. This requires an excess of power as well as torque-converter controls or equivalent. Fairleads must be carefully laid out to ensure a proper fleet angle from the winch and to ensure that they will properly follow the changing position of the barge (see Figure 5.13).

Sheer-legs crane barges were used to set the 200-tn. precast concrete dome and breakwater segments of the Ninian Central platform. Sheer legs crane barges have been used to set the 3000-tn. approach caissons of the Great Belt Eastern Bridge and those of the

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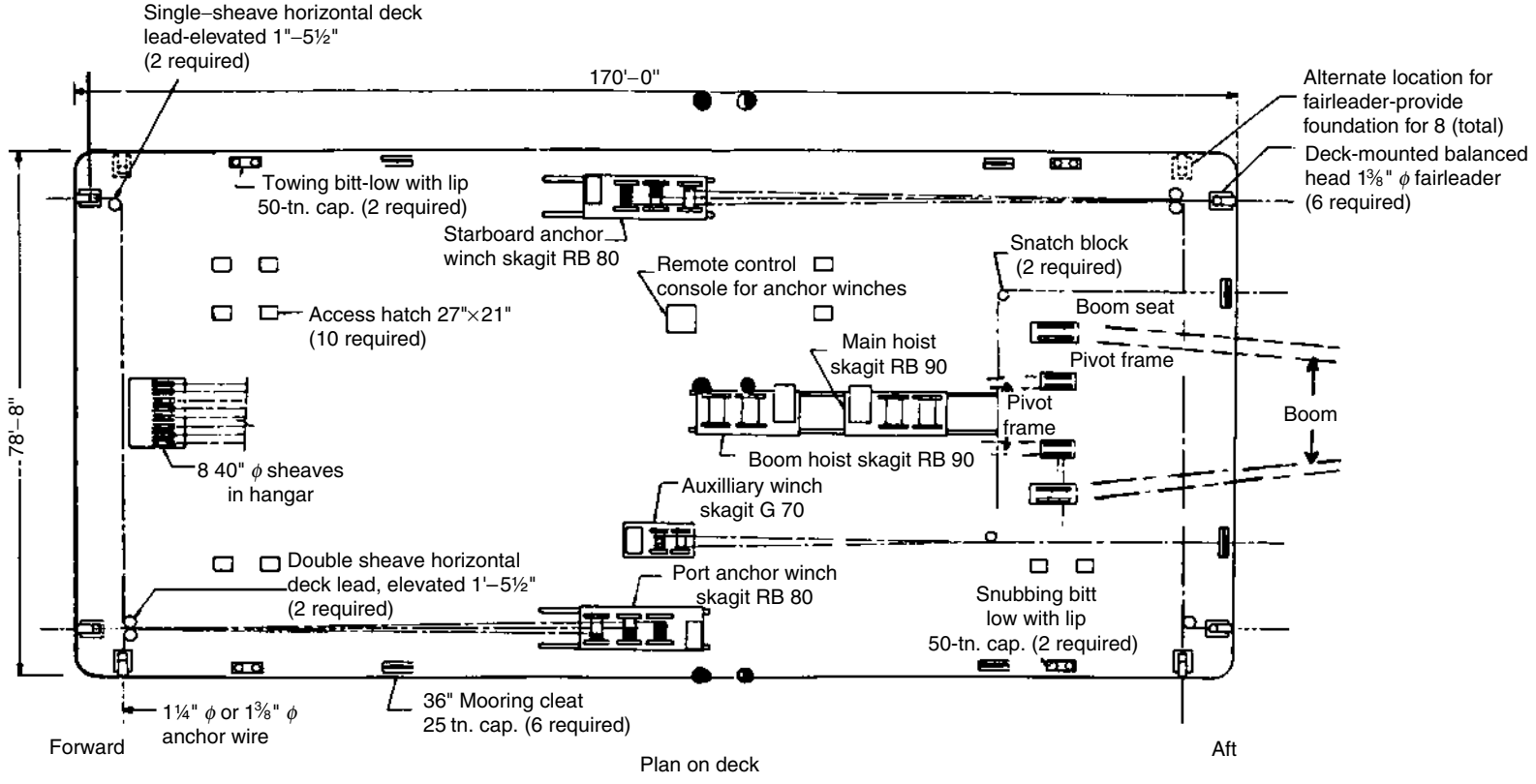

FIGURE 5.13
Layout of equipment on crane barges.

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FIGURE 5.14
Heavy-lift hammerhead crane barge setting 8000-tn. concrete bridge girder, Confederation Bridge, Canada. (Courtesy of Stanley Construction Co. Inc.)

Øresund Bridge. Three crane barges, rigidly positioned relative to each other by multiple lines, were used to lift the 1200-tn. quarters modules onto the Statfjord A platform. This is an inherently hazardous operation, but it was carried out successfully because of the provision of oversize lines, the interconnection of adequate deck engines to a single control station on the central barge, and the tying of the three barges together rigidly to prevent relative movement.

A crane barge was successfully used on the Port Latta, Tasmania, offshore terminal to set jackets into pre-installed frames with a tolerance of only 50 mm. A job-built sheer-legs crane barge was used to set the superstructure of a similar iron-ore terminal in Queensland. Thus, the capability of a crane barge for extensive use offshore should not be eclipsed by the currently more popular but much more expensive fully revolving offshore derrick barges.

Hammerhead crane barges have fully fixed hammerhead cranes. They operate in the same way as sheer-legs crane barges, but cannot luff. The SVANEN has a capacity of 8000 tn. It was used to get the piers, shafts, and girders on the Great Belt Western Bridge, the Prince Edward Island Bridge, and the Øresund Bridge (see Figure 5.14).

5.8 Offshore Derrick Barges (Fully Revolving)

Fully revolving derrick barges are the workhorse of offshore construction. As with the sheer-legs crane barges, they are fitted with deck engines and full mooring capability, only here the emphasis is on stabilizing the barge's position rather than close control in positioning, since the derrick barge normally remains stationary during any particular operation.

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The typical inland marine derrick barge has a capacity of 50–300 tn., whereas an offshore derrick barge has a crane capacity of 500–1500 tn. To handle ever larger modules and deck sections, capacities have been rapidly increased in recent years, with the latest offshore derrick barges having two cranes, each rated at 6500 metric tn. each, or a total of 13,000 T.

The derrick barge represents a compromise (or optimization) of opposing demands. Structural and naval architectural considerations require it to be located forward of the stern a distance 20%–25% of the length, that is, at the one-quarter or one-fifth point. The barge should be wide enough to minimize list as the crane swings and to provide adequate distribution of the structural load.

On the other hand, the effective reach of the crane and its load capacity is diminished by the distance from the boom seat to the stern or side of the barge. One way to meet these two contrary demands is by the use of a large swing circle that moves the boom seat closer to the barge end while maintaining the center of rotation and support well back. A major consideration is the list of the barge under fully loaded or no-load conditions. The counterweight is usually designed to limit the list under half load, hence under no-load the barge may list opposite to the boom. This list can be reduced during operations by booming down while swinging under no-load. The swinging is carried out by swing engines driving the bull wheel. Due to list, the crane is often forced to swing "uphill" under load. Offshore cranes are therefore provided with two and sometimes three swing engines. The list also places heavy structural loads on the crane tub, which forms the structural connection to the barge. Hence, its design must provide proper structural reinforcement for bending and to prevent buckling under inclined compression loads. Land cranes mounted on barges often fail as a result of collapse of the tub or center pin. The advantages in operations of a fully revolving derrick are many: the ability to pick off a barge or boat alongside or even from the deck of the derrick barge itself, the close control of positioning to be able quickly to reach any point in three-dimensional space with one set of controls, the ability to follow the surge motions of a boat or barge alongside in order to pick a load from off it, and the ability to orient the derrick barge in the most favorable direction to minimize boom tip displacements and accelerations.

When setting loads, it is the boom tip motions that control. These are affected by motions of the barge in each of the six degrees of freedom. When working far out over the stern, pitch amplitudes will be amplified. When working over the side, it is roll that causes the most difficulty. Computer programs have been developed to assist in selecting the proper heading, which treat the barge and load as a coupled system. A skillful barge superintendent and crane operator will take advantage of the "groupiness" of waves to perform a critical pick or setting operation during a succession of low waves.

As with a crane barge, tag lines must be used to control the swing of the load. As contrasted with the sheer-legs crane barge, the position of the load relative to the barge is constantly changing; hence the tag line engines are fitted to the crane body and revolve with it.

A load suspended from a boom tip is a pendulum. While the load line length is usually too long for direct resonance, the load may tend to get dynamic amplification from lower-frequency energy. The practical solution is to raise or lower the load quickly through those positions that develop amplified response.

Marine cranes are usually designed to work under their rated loads up to a 3° list. The load capacity ratings for marine cranes are based on 2° roll at a period of 10–12 s, which equates to an acceleration of 0.07 g. The swinging of the load develops lateral forces on the boom. Hence, offshore crane booms are designed with a wide spread at the heel (usually 1/15 of boom length or more). This in turn means that the boom lacing (bracing) members will be subject to buckling; they must be properly designed to prevent this mode of failure. Booms today are made of high-strength steel, usually round or square tubulars. This

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makes them lighter and hence increases the effective load capacity of the crane and reduces the inertia in swing. However, it means that welds are more critical and that buckling becomes a common mode of failure. Good design and fabrication will take care of these. It also means that the boom is much more sensitive to lateral impact from the load itself or to failure under an accidental lateral loading. It means that attachments such as padeyes for snatch blocks and so forth must be affixed to the boom only after careful engineering and with fully controlled welding procedures suitable to the grades of steel involved.

One of the potential hazards with offshore derrick barge operation is that, although the lifts have been carefully engineered for load and reach, in the actual situation, the derrick barge surges farther away from the platform and moves laterally. The operator, intent on the load and the landing site, booms out and swings beyond the crane's capacity. This may result in a direct failure of the boom or may result in a loss of swing control, which accelerates as the barge lists. Offshore derrick barge cranes are fitted with automatic warnings to alert the operator when allowable load-radii combinations are being exceeded, but swing control is normally a matter of judgment.

To snatch a light load out of a supply boat, a single line, the whip, is preferred. It can raise the load fast enough to prevent an impact on the subsequent heave cycle. Raising a heavy load from a barge is more difficult since there may be twenty-four or more parts in the line and the barge will rise as the load is lifted, increasing the risk of impact of load and boat deck.

A similar problem occurs when setting a heavy load. When setting on a platform, the deck will usually be above the sight lines of the crane operator, the operator is working blind, dependent on signals. Hence, one or more guiding devices are needed. Tag lines from the crane barge may bend over the edge of the platform deck; if they chafe, they may part at the worst possible time. Softeners should be provided. Structural guides may be pre-installed on the platform so that the load, once set within 0.5 m or so of position, automatically guides down to the correct location. These guides must have sufficient height so that the load does not ride up out of them on the next pitch-heave cycle. If that were to happen, they could puncture the load rather than guide it. Taut guide lines can be employed to help pull the load to the correct position. A system of guides that often works well is to use two columnar guide posts. Suspended loosely from the load are two pipe sleeves of larger diameter. These can be hand-fitted over the posts; when the load is lowered, the sleeves will guide the load into place.

Alternatively, loosely hanging pins (smaller-diameter pipe) may be entered into the tubular posts. Tag lines and winches may be installed on the platform to assist in guiding the load into place. Another solution is to set the load only to an approximate location, landing it on softeners such as timber or rubber fender units or used earthmover tires. After it has been landed in approximate position, it can be skidded to final exact position by hydraulic jacking equipment of the type commonly employed on oil-drilling rigs. This procedure is often adopted when setting trusses or other unwieldy superstructure elements of bridges.

There is an arbitrary functional division that exists between offshore construction crews and offshore drilling crews. Neither seems to fully appreciate the problems of the other. This has resulted in much needless work and several accidents. Close coordination and communication are essential.

The lower (traveling) block and hook of a large offshore derrick can weigh 20–30 tn. or more. As it is brought up close to the boom housing, it may get into resonance with the roll of the barge. A special hook control tag line is required. The traveling block-hook combination should never be left hanging at short scope. A sea may come up that excites the hook and makes it impossible to secure. Thus, except when the crane is being used, the

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block should always be fully stowed and the boom lowered into the boom cradle and secured. This will also reduce fatigue wear on the swing gear.

When a derrick barge is working alongside a platform, the moorings are laid out in a pattern that allows the barge to reorient and relocate as necessary to reach as many parts of the platform as possible. Care must be taken that during a reorientation, the mooring lines are not allowed to cross one another. Although there are exceptions, as a general rule, mooring lines should never cross; it prevents retrieval of the underneath line, and it may lead to erratic reactions from the lines as the load in one changes its catenary and affects the other. Worst of all, one line may snag the anchor of the other line.

5.9 Semisubmersible Barges

While the standard barge, whether serving for cargo transport or as support for a crane or other operational equipment, has good stability and load displacement characteristics, unfortunately it has excessive response to the wind-driven waves and swell. These then limit the workability of the vessel.

In areas such as Bass Strait, Australia, and the northern North Sea, where the persistence of low-sea states is short, a conventional barge may encounter excessive weather down-time, which may extend the construction schedule beyond the summer "weather window" and thus require an extra year for completion.

The semisubmersible concept was first developed for offshore exploratory drilling but has since been extended to both derrick barges and pipe-laying barges. It is a simple concept: a large-base pontoon or pontoons that are fully submerged during operations, supporting four to eight columns that extend through the water plane and in turn support the deck. Thus there is a large submerged mass and large displacement combined with minimum water plane. The vessel is therefore subject to minimum exciting and righting moments. Some have referred to the concept as "transparent" because the waves sweep right through between the columns or shafts, with little effect on the barge motion (Figure 5.15 and Figure 5.16).

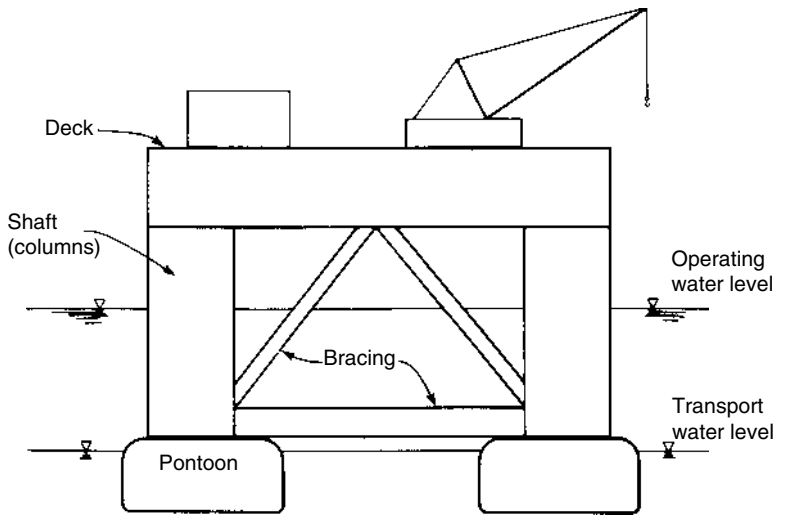

FIGURE 5.15
Semisubmersible concept.

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FIGURE 5.16
Semisubmersible crane barge.

This lack of response to the typical wind-driven seas is due both to the relatively small change in gross displacement and to the much longer natural period of the vessel, especially in roll, pitch, and heave. Whereas the standard barge has a natural period of 5–6 s, the typical semisubmersible has a natural period of 17–22 s (see Figure 5.17).

There are three penalties to pay for this favorable performance:

1. The semisubmersible has much greater response to externally applied loads such as weights, loads, and ballast. Another way of stating this is to say that its righting moment and metacentric height are much lower than those of a standard barge.
2. The semisubmersible has much reduced topside cargo capacity. It relies on a low center of gravity to maintain stability.

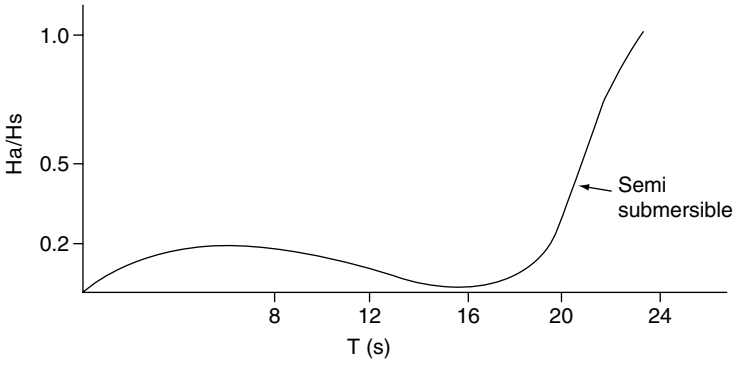

FIGURE 5.17
Response of semisubmersible vessel to heave.

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3. The semisubmersible costs more to build and to operate. Ballast controls are similar to those for a submarine.

However, semisubmersibles are increasingly used in drilling construction, for floatels (floating quarters vessels), and even for floating production because of their ability to carry out their operations over extended periods without interruption due to weather downtime.

As indicated above, the semisubmersible must have a very complete and effective ballast and drainage system, with high-capacity pumps and quick-acting controls. The system must have a high degree of reliability; failure of a valve to close can cause a catastrophe. There must be positive valve position indicators, tank level indicators, and sensitive list-trim indicators in the control room. Redundant venting systems must be provided to prevent accidental overpressurization.

The semisubmersible normally rides upon its base pontoons for transit, going into the semisubmersible mode only after arrival on station. As with all vessels having a drastic change in water plane area, at the draft when the water plane crosses over the base pontoons there is a sudden loss in metacentric height to almost zero. This is further compounded by the action of the waves breaking across the tops of the pontoons and impacting the shafts, so this stage is one of unpredictable response and instability. Therefore, no other operations should be attempted during submergence until there are 2–3 m of water depth over the pontoons.

The effect of accidental holing of a shaft, for example, can be much more serious than for a similar holing of the side of a standard barge. Therefore, the shafts of modern semisubmersibles are double-hulled and protected by heavy timber and rubber fenders. Because of low topside capacity, deck winches are usually mounted low in the shafts. Mooring lines leave the barge through swivel fairleads located on the base pontoons; this keeps them well below the keel of attendant boats and barges. The safety of a semisubmersible against capsizing can be immeasurably improved if appropriate damage control systems are built into the vessel and enforced in operations. For most semisubmersibles, the deck is watertight and has the structural strength to act as an upper barge hull. Then, if the vessel should heel over so that the deck enters the water, the righting moment increases significantly.

However, operational carelessness often negates this in practice. Watertight access doors and portholes and vents are left open, especially in warm climates. Internal subdivisions are modified, so as to lose their watertight bulkhead capabilities. Gear is left loose on deck to shift with the list. These have contributed to the loss of two semisubmersible vessels, the Alexander Kjelland in the North Sea and the Ocean Ranger off Newfoundland. Operational mishaps, structural defects, and dragging anchors have apparently also been involved.

The semisubmersible is subjected to high stress concentrations and cyclic loadings at the connection between shafts and pontoons and bracing. When sea and operating conditions permit, the semisubmersible should be ballasted up so that these can be visually checked to detect any cracks.

There is another advantage to the semisubmersible, and that is the high elevation of its deck, especially when ballasted up onto the pontoons. From such an elevated situation, a crane can reach farther out over the deck of a platform, and thus interior modules can be more easily placed.

Because of its low heave response, the semisubmersible can be utilized as a tension leg construction platform, that is, with vertical mooring lines to clump weights or anchor piles on the seafloor. Pulled down against these reactions, this temporary tension leg platform (TLP) can hold itself accurately in the vertical direction, thus enabling it to carry out heave-

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sensitive operations such as screeding, setting, and fitting large individual pipes or underwater vehicular tube sections. This principle has been adopted for marine operations in inland waters, such as screeding for submerged tunnels (tubes) and prefabricated box caisson bridge piers. It has been used in this manner for the setting of the San Francisco BART tubes, and those crossing the Chesapeake Bay entrance channel.

These favorable properties of the semisubmersible concept have been adopted by offshore contractors and operators for a variety of small special-purpose rigs. These may in turn be tended by large offshore derricks, since the small semisubmersible has limited versatility, usually being intended for one specific operation. Moored, for example, as a TLP and working in conjunction with a derrick barge, it can carry out operations on the seafloor that require minimum or no heave.

Although the semisubmersible itself is stable in moderately high sea states, it has problems in transferring piles, pipe lengths, and deck equipment from a standard cargo barge because of the latter's motions. The semisubmersible provides little lee; the waves sweep right through its columns. Large supply boats are therefore often used to deliver such items to the extent practicable. The supply boat can run a stern line to the semisubmersible, and then run ahead slowly, with its bow headed outward. Thus, it can lie alongside but still be free from direct contact.

For deck modules and the like, it may occasionally be most practicable to tow the semisubmersible into protected waters, load the module, and tow back to the offshore site. If the load is hanging on the boom(s), over the stern, then the load must be blocked to the barge and held in with taut tag lines of adequate size to prevent swing and heave during transport. Long piles of large diameter may be delivered to the site afloat, to be picked from the water by the derrick barge.

The semisubmersible concept has been the preferred choice for the proposed U.S. Navy Mobile Offshore Base System, a combined floating airport and supply base for all-ocean operations, as well as several other proposed offshore airfields.

5.10 Jack-Up Construction Barges

The jack-up barge has proved to be a very useful construction "tool," especially when working in turbulent sea areas, or breaking waves such as shoal or coastal waters, and in swift currents. Where a great many operations must be carried out at one location—for example, at an offshore terminal or bridge pier—the jack-up construction barge is especially valuable (see Figure 5.18). The barge is outfitted with four to eight large jacks and legs, built either of tubulars or fabricated steel. The barge is towed to its work position and jacked up free of the waves to perform its work.

The typical sequence starts with the barge moving to the site with its legs raised. Upon arrival at the site, it is moored with a spread mooring. Construction jack-ups can operate only in relatively shallow water, 30–60 m, with 150 m as an extreme, so the use of a taut mooring is practicable, although up to 300 m may be adopted for drilling rigs.

With the sea state being calm (waves and swells must usually be less than 1 m), the legs are lowered to the seafloor and allowed to penetrate under their own weight. In some soils, penetration can be aided by jetting and vibration. Using the jacks on one leg at a time, the barge acting as the reaction, the legs are forced into the soil. With all legs well embedded, the barge is jacked up clear of the water. This is the most critical phase, since wave slap on the underside of the barge may cause impact loads on the jacks and may shift the barge laterally, bending the legs. To cushion the impact, special hydraulic cushioning may be connected to nitrogen-filled cylinders; alternatively, neoprene cushioning may

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FIGURE 5.18
Jack-up construction barge, Brazil. (Courtesy of HV Anderson.)

be employed. Once well clear, the barge is raised up to its working height. Then the legs may be cut loose, one at a time, and a pile hammer used to gain even greater penetration. Since uneven settlements may take place as a result of time, operations, and wave energy input into the legs, the jacks have to be periodically reactivated to equalize the load at each. This is especially necessary during the first few days at a site.

To leave a site, the sea must again be calm, with waves and swells usually less than 1 m. The mooring lines are reattached, slack. The barge is then jacked down until it is afloat. Once again, the critical period is when the waves are hitting the underside. The mooring lines are tightened. Then the legs are jacked free, one at a time. If legs do not pull out easily, several techniques can be applied. The fastest is jetting. In clays, a sustained load may eventually free the leg. Also in clays, water injection at low pressure to break the suction may be more useful than high-pressure jetting, which leads to the formation of escape channels. The same process may be used to free legs in sand, also at low sustained pressure. In no event should an attempt be made to free the legs by lateral working of the barge. A bent or jammed leg may result, with very serious consequences. In one case in Cook Inlet, Alaska, the leg was jammed by the high current working on the barge side during the refloating operation. Then the tide rose some 6 m, flooding the jack-up barge itself.

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High currents may create local eddies around the legs, leading to scour and loss of lateral capacity. Steel mats are usually built onto the bottom of the legs, so that when the legs are jacked down they take their temporary support from the seafloor. A short stub pipe sleeve may penetrate below the mat to provide shear resistance against sliding. Then the main leg is jacked down through the sleeve. Since jack-up performance is so highly dependent on the seafloor soils, it is essential that a thorough geotechnical evaluation, including at least one boring, be made at each site. Of particular concern are layered soils, in which a leg may gain temporary support but then suddenly break through (see Figure 5.19).

In clay soils, where jack-ups have previously worked around the site, holes will have been left that now may be partially empty or filled with loose sediments. If a leg is now seated adjacent to such a hole, it may kick over into it, losing both vertical and lateral support and bending the leg. A general rule of thumb is to plot the previous leg positions (if known) and to space the new leg locations 4–5 diameters away. This, of course, is another advantage of the mat-supported jack-up legs: the mats can span local anomalies. Walking jack-ups have been built, varying in size from a small test-boring rig capable of walking through the surf to a monstrous dredge hull on jack-up legs. These rigs are equipped with two sets of legs (six or eight in all) supporting a double framed hull (or segments) so that it can successively launch forward, lower the legs, take its full support forward, pick up the legs behind, and retract the rear into the forward section. The rear set of legs is now lowered to give added support during operations and then to enable the forward set to be picked up once again. Such walking jack-ups eliminate the need for the hull to be lowered into the sea in order to move. The smaller walking jack-up rigs are especially useful for taking borings in the surf zone. Unfortunately, the large walking jack-up dredge proved too costly and slow, and it has been taken out of service. The smaller walking jack-up has proved quite successful.

FIGURE 5.19
Jack-up construction barge foundering due to erosion around legs, California.

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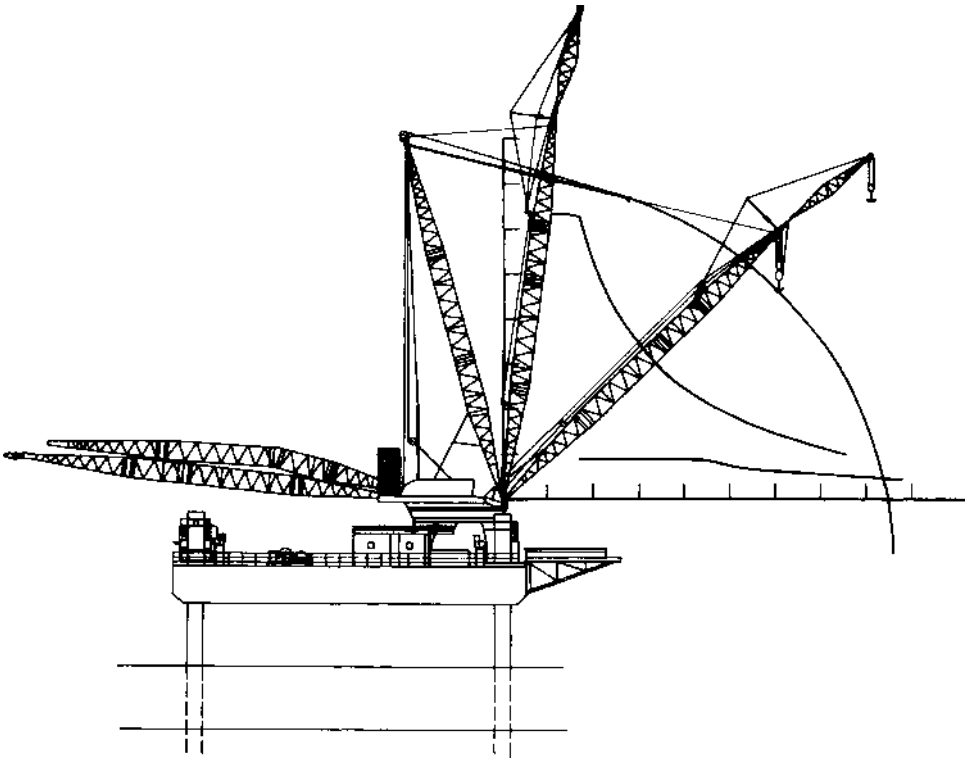

FIGURE 5.20
Jack-up construction crane barge. (Courtesy of HAM Dredging and Marine Construction.)

Large jack-up construction rigs are most applicable where the sea conditions are highly variable, with frequent periods of calm, so that the rig may find convenient times to move. On the other hand, if numerous moves are required—as, for example, in laying outfall sewer pipes—then persistently rough seas may delay moves so long as to render the jack-up uneconomical. One disadvantage of the jack-up occurs during the transfer of loads from barges or supply boats. Here the jack-up concept again becomes weather sensitive, for the barges must not be allowed to contact the legs or they may damage them.

Jack-ups provide a fixed platform, free from motion response to the seas (see Figure 5.20). Hence, they are ideal for carrying out operations such as grinding a rock foundation in order to seat a caisson, as was done on the Honshu-Shikoku Bridge (Koyama-Sakaide Route, Pier 7A). They are also ideal for screeding the foundation site. Statistical studies covering both jack-up drilling rigs and jack-up construction rigs show that they have been six times more likely to suffer serious damage or loss during relocation and transit than they are when on location. This is primarily due to the barge having its legs fully raised, thus creating a very high center of gravity. Some jack-ups, therefore, have telescoping legs.

As with the semisubmersible concept, the jack-up principle has been applied to smaller, special-purpose construction rigs tended by an offshore derrick. The derrick barge with its large mooring system can be used to position the jack-up and if necessary to help in penetrating its legs and later to help in retracting them. Meanwhile, the jack-up rig forms a vertically stable work platform for such sensitive operations as coring and sampling operations or for submarine pipeline repairs.

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The jack-up has been used to set heavy loads. In this case, a barge carrying the load is floated between the legs. The load is then lifted by direct hoisting from the jack-up deck above, the barge removed, and the load lowered to the seafloor. This operation, with a barge between the legs, is obviously highly hazardous and should only be attempted under ideal sea conditions, with adequate lateral controls to ensure that the barge cannot hit the legs. This concept was used to set the 600-tn. precast concrete caissons for the Columbia River (Oregon) bridge at Astoria.

5.11 Launch Barges

One of the most dramatic developments in offshore construction practice is the use of the launch barges for the transport and launching of jackets. They have also been utilized to deliver and launch subsea templates (Figure 5.21). The typical launch barge is a very large and strongly built barge, long and wide, subdivided internally into numerous ballast compartments. Since it must support a progressively moving jacket weighing thousands of tn., it must have strong longitudinal and transverse bulkheads. Heavy runner beams or skid beams extend the length of the barge (Figure 5.22). These girders distribute the jacket's load to the barge structure. The stern end of the barge, over which the jacket will rotate and slide into the water, requires special construction.

First, for a short period of time the stern will have to support the full weight of the jacket. Second, since this reaction force has to be transmitted into the jacket, it must distribute the reaction over as long a length as feasible to avoid a point reaction. The jacket will be sliding on its specially reinforced runners; even so, they need a distributed rather than a point reaction. Hence, the stern of the barge is fitted with a rocker section that rotates with the jacket as it slides off (Figure 5.23). The operational aspects of launching of a jacket are described in Chapter 11. There is one other structural aspect to a launch barge: intentional grounding. For loading out the jacket at the fabrication yard, the usual method is to ground the launch barge on a screeded sand pad at the appropriate depth so that the barge deck matches the yard level. Then, the jacket can be skidded out onto the barge with no change in relative elevations. This means that the hull bottom must withstand high local pressures from irregularities in the prepared sand bed. Not only must the bottom be

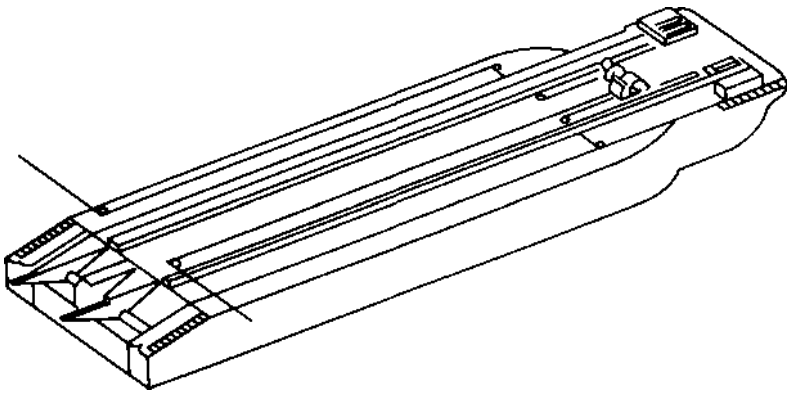

FIGURE 5.21
Launch barge.

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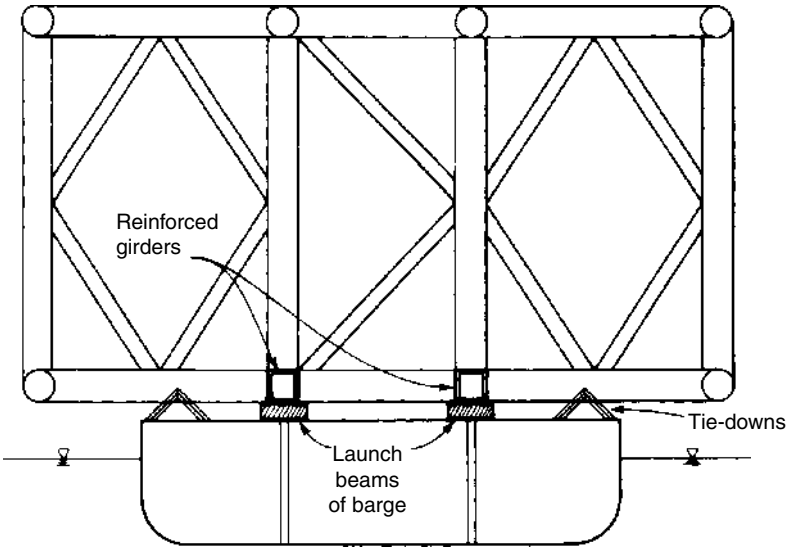

FIGURE 5.22
Jacket loaded on launch barge.

of heavy plate, but the stiffeners must be adequate to prevent buckling as the jacket is moved onto the barge.

When the load-out is performed with the barge afloat, then ballast must be rapidly adjusted to maintain the relative elevation at the barge deck as the load of the jacket comes on. Step-by-step adjustments or computer control are used to adjust deck elevation and trim. A launch barge is also fitted with heavy winches or linear jacks on the bow to pull the jacket onto the barge and later, by re-rigging through sheaves on the stern, to pull the jacket off the barge during launching. The beam width of a launch barge is often less than the base width of the jacket. The base of a deep-water jacket may be 60 m wide, overhanging the sides of the barge significantly. Several large launch barges are 196×52 m and can carry and launch a jacket of 40,000 tn. A 300-m-long launch barge was fabricated

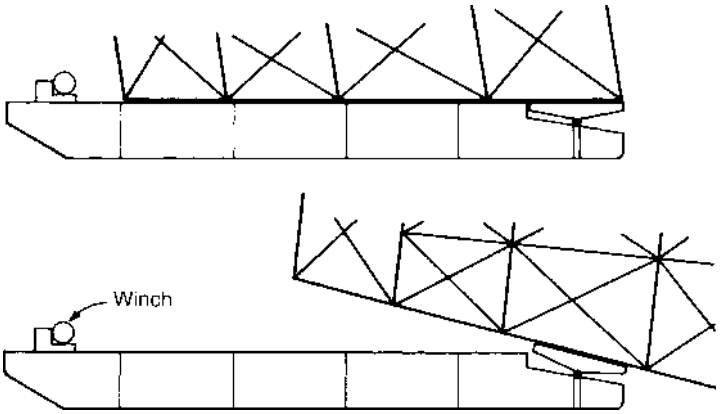

FIGURE 5.23
Rocker arm on stern of launch barge.

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in Japan to transport the 55,000-tn., 415-m-long Bullwinkle jacket. During transport the barge must have enough freeboard to prevent the outside legs of the jacket from dipping into the waves as the barge rolls. The beam width of the barge is designed to give stability transversely during launching. This is often the critical condition during launch; if the barge lists and the jacket rolls sidewise, it may buckle a jacket leg.

5.12 Catamaran Barges

For heavy lifts in harbors and on rivers, especially for submerged prefabricated tunnel (tube) segments, catamaran heavy-lift barges are frequently employed. These consist of two long barges, spread apart, and joined over-the-top by gantries. Usually, but not always, the gantry legs are pin-connected at the centerlines of the barges, so as not to impose any listing moment on the barges, thus necessitating additional ballast and consequent increased draft. This also allows the barges to undergo a small degree of roll independently without affecting the gantry trusses (Figure 5.24).

Gantries are usually equipped with twin gantry trusses, one at each end of the barges, to enable lifting and/or lowering of long prefabricated segments such as tubes. Two lifting devices are arranged on each truss, making a total of four lift points for the system. To resist fore-and-aft differential movement of the two barge hulls, horizontal connecting trusses are installed at one or both ends. Catamaran barges are also used for screeding of underwater foundations. To maintain exact grade and to be unaffected by waves and swells, they may use the semisubmersible concept (discussed in the previous section) in which the hulls are ballasted down below water, with columns or shafts extending up to support the superstructure. To offset the effect of tidal changes as well as those of waves, precast concrete clump weights may be lowered to the bottom to maintain a constant elevation, offsetting the increased buoyancy of the rising water on the columns.

FIGURE 5.24
Catamaran Barge transports and installs underwater structure. (Courtesy of Morrison–Knudsen.)

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The catamaran principle can be achieved by four large independent pontoons joined in pairs, using the self-floating element such as a precast tunnel section to maintain relative position of the pontoons in the horizontal plane.

In the case of the Øresund Bridge between Denmark and Sweden, two large barges were secured directly to the precast concrete shell of the main pylon pier, one on each side, by multiple post-tensioning bars, enabling the combined units to float out of the construction dry dock, float to the site, and lower onto the prepared base.

In the Versatruss concept, two barges are used as in the catamarans above, but the function is to lift a load up from below, rather than lowering it. Inclined struts of large tubulars are sloped upward from each barge and connected to the load to be raised, typically the complete deck structure of an abandoned offshore platform. Powerful winches at the deck level draw the two barges closer together, which causes the inclined tubular struts to rotate and raise the load. Once the load is lifted, the catamaran can float free of the platform substructure and by slowly allowing the winches to back down, to lower the load onto a barge.

5.13 Dredges

For the dredging of harbors and channels, a number of quite different types of dredges have been developed. Each has its special capabilities, depending on extent and quantity, type of soil to be removed, disposal requirements, and depth.

Clamshell dredges use a bucket on vertical lines, which enable them to operate close to existing structures. They gain their digging ability through the weight of the bucket concentrated on their teeth. They are not depth limited.

Dragline dredges are especially suitable for digging trenches in relatively shallow water in soft and moderately hard soil. They gain their digging ability through their near-horizontal pull on the bucket, which in marine projects, requires a corresponding reaction from the mooring lines.

Ladder or continuous flight bucket dredges can dig deeply (more than 40 m) and in hard material. Their small buckets exert the weight of the entire flight and ladder on a single cutting edge at a time. They are much used in channel maintenance dredging in existing harbors because of their ability to control the discharge into barges alongside, as well as to dig to an accurate elevation with a smooth cut. They can also be used to dig trenches in firm material. A ladder dredge was employed to dredge the pier sites for the Great Belt Western Bridge in Denmark.

Hydraulic suction dredges, as the name implies, suck up the material through a pipe to a pump mounted in the hull, then discharges it through a pontoon line to the disposal area. The material may be loosened by jets at the lower end of the suction line. This dredge is widely used in maintenance dredging and soft material. When these hydraulic suction dredge systems are fitted to a self-propelled vessel with storage capacity, enabling it to transport the dredged material for discharge elsewhere, it becomes a hopper dredge, used primarily for maintenance dredging. It can have rippers or jets mounted at the dredge head of an inclined suction arm ladder to enable it to dig relatively consolidated and hard material.

A cutter head may be mounted at the end of the suction line, with both supported by a ladder. This enables the dredge to cut the material mechanically for suction up the line to the pump. This is the cutter-head hydraulic suction dredge, extensively employed for large-scale dredging and landfill.

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Some large hydraulic cutter head dredges are capable of dredging to 60 m depth and of excavating very dense and hard material, even rock up to 140 MPa (20,000 psi) strength. Thus they are effective in digging trenches for pipeline burial and glory holes for subsea well heads, for example, the extremely dense sandy gravels offshore Newfoundland.

Dredging offshore is rendered extremely difficult because of swells, depth, and position control. Water is usually deeper than traditional harbor dredging. Quantities are usually substantial. Disposal is severely restricted in many cases and distant from the excavation site. There are also the usual problems of trying to work with auxiliary craft, each responding in its own way to the waves. Nevertheless, efficient and effective dredging systems have been developed for use offshore. Dredging operations themselves will be discussed in Chapter 7, as will the minor dredging equipment used in construction, such as airlift and eductor devices. Ocean mining will be described in Chapter 22. Use of jet sleds and plows to bury pipelines will be discussed in Chapter 17.

The trailer suction hopper dredge is the workhorse of ocean dredging. This is a self-propelled vessel of standard ship hull configuration. It supports a long "ladder," a pipe with structural framing capable of reaching to the seafloor at an angle of about 30°. At the end of the ladder is the entry port, equipped with jets to break up sand so that it can flow freely into the ladder or with ripper teeth to break up cemented or hard material. On board, a suction pump moves a column of water and dredged material up the ladder at a velocity sufficient to keep the material in suspension. The pump discharges the material into hoppers of the vessel; the solids settle out and the surplus water overflows. By controlling the flow into the hoppers, the material can be roughly graded, with silts and other fines being washed overboard.

The trailer suction hopper dredge can now transport the material as far as desired, discharging it by dumping from the hoppers by opening the gates. Alternatively, the trailer suction hopper dredge may agitate and slurry the material in the hoppers and pump it ashore. When dredging harder material, the dredge may use the power of its propellers to give added thrust to the rippers attached to the lower end of the ladder (the dredge head) or even use the momentum of the vessel. For example, when dredging soft and fractured rock, it may circle around to gain speed and then make a run at the dredged area. As it reaches the area, it lowers its ladder and rips and sucks up the rock. A trailer suction hopper dredge performs its work in successive long runs. Thus, it is ideally suited to excavation of a seafloor trench.

One of the problems facing the operation of a trailer suction hopper dredge in the open ocean is that of swells, which raise and lower the dredge head and may damage it, as well as giving an uneven dredge depth. Therefore, the suspension system, by which the dredge ladder is suspended from the hull, may have a heave compensator installed in it, capable of rapid response. In its most sophisticated installation, the heave compensator may be activated by a sonic depth indicator to maintain an appropriate elevation relative to the seafloor regardless of the heave or pitch of the vessel.

The hydraulic suction dredge is an extremely efficient piece of machinery for moving large masses of material for moderate distances. In the Beaufort Sea, for example, a hydraulic dredge has moved up to 100,000 m^3 of sand per day to deposit it in an underwater embankment for an offshore drilling island.

The standard hydraulic dredge suspends its ladder from an A-frame over the stern. This places high bending moments in the hull. The lower end of the ladder is fitted with a cutter head to cut the soil so that it can be swept up into the flow of water up the ladder. This cutter head has traditionally been a set of blades rotating around the axis of the ladder. More recently, wheel excavators have been employed, rotating around the axis normal to the ladder. These latter are driven by a submersible motor, either electric or hydraulic. As with the trailer suction hopper dredge, swells cause the dredge head to raise and lower.

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This may severely damage the cutter head. A heave compensator is therefore necessary when working in the open sea. One form of heave compensation may be obtained by the use of an articulated arm and spar buoy to support the end of the ladder; this eliminates the effect of pitch of the hull and minimizes the heave response due to the swell. Powerful cutter heads can dredge material which would be classified as soft rock. In very soft or loose material, on the other hand, the cutter head may be replaced by a system of jets.

In deep dredging the pump on board the vessel loses efficiency. For depths over 20–25 m, therefore, additional means are employed to keep the water column moving at high velocity. Eductor jets ("booster jets") angled upward may shoot water at high velocity into ports located at several elevations along the column in the ladder. This overcomes the frictional and entrance losses in the suction pipe and accelerates the water to working velocity so that the dredge may function at full efficiency despite the greater depth. Alternatively, the dredge pump may be submerged or even located down on the ladder to minimize the suction length.

Spuds are often used to position dredges in harbors and to develop the needed reaction to the thrust of the cutter head into the underwater bank. In the depths and sea conditions found offshore, spuds cannot usually be used. The hydraulic dredge must operate off wire lines to anchors. To allow the dredge to sweep laterally without undesired translation, the three aft-leading lines are brought in through a "Christmas tree" fairlead system, with the three fairleads arranged on one vertical axis (Figure 5.25 and Figure 5.26).

Special concepts for cutter head dredges have been developed to enable mining and heavy-duty dredging at depths of 100 m and even more.

Toyo of Japan has developed heavy duty dredging pumps which can move high concentration of sediments such as sand at a low head of about 10–15 m. These pumps can be suspended from a crane boom and are especially useful in dredging local areas such as a cofferdam or trench.

The clamshell dredge is another tool that can be used effectively in the marine environment, especially when digging rock or other hard materials or when excavating within limited areas. Its cutting ability is not affected by pitch, heave, or roll, but vertical control is rendered difficult. Surge, sway, and yaw will, of course, make the horizontal control more difficult but do not affect the ability to dig. The clamshell bucket is designed to penetrate by its own weight, acting on the lips of the bucket or on the teeth. It is closed by raising on the closing line or by hydraulic activators.

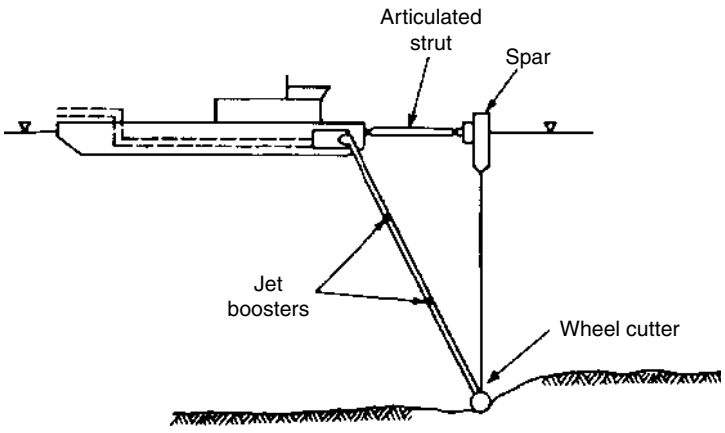

FIGURE 5.25
Wheel cutter head hydraulic dredge.

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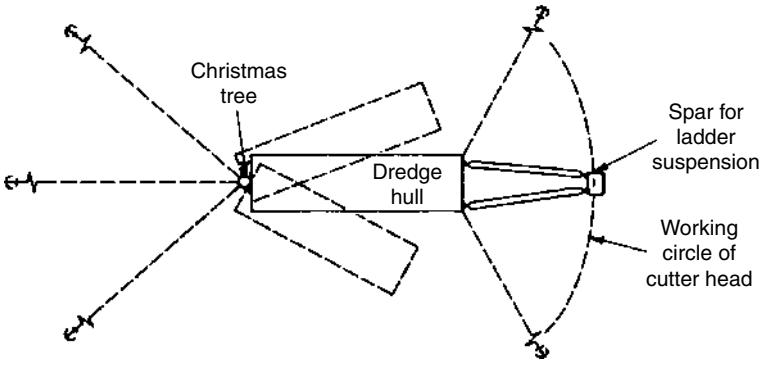

FIGURE 5.26
Mooring of cutter-head hydraulic dredge.

For practical reasons, offshore clamshell buckets are large and heavy, weighing up to 100 tn., as in the case of the deep dredging for the piers of the Honshu-Shikoku bridges (see Figure 5.27 through Figure 5.29). A 50 m^3 bucket was employed to dig the trench for the prefabricated tunnel segments of the Øresund crossing between Denmark and Sweden. A 100 m^3 hydraulically operated grab bucket has been developed in Japan, for dredging to a depth of 30 m.

After closure, the bucket is lifted to the surface; in deep water, this is the slowest part of the cycle. Then, conventionally, the boom swings to starboard for disposal into hopper barges alongside. Alternatively with long boom and favorable currents, the material may just be sidecast when the boom is abeam. Then the boom must swing back. The bucket is allowed to fall. The velocity of fall is restrained by friction of the wire lines over the multiple sheaves or else braked as necessary. In deep water the bucket may be lowered under power. The cycle then repeats.

FIGURE 5.27
Clamshell bucket discharging into bottom-dump barge. (Courtesy of Kajima Engineering and Construction Co.)

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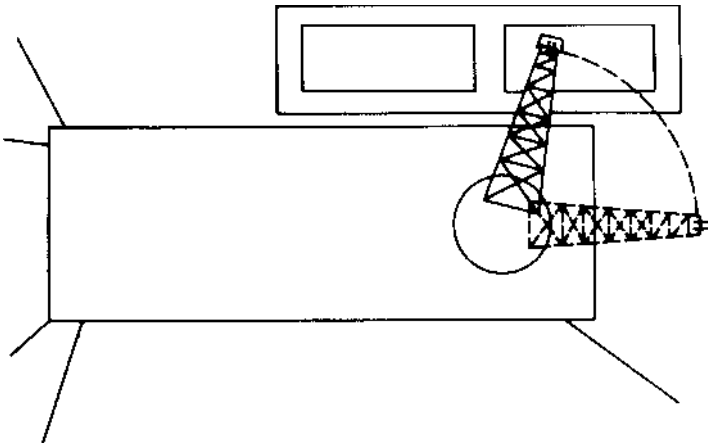

FIGURE 5.28
Clamshell derrick discharging to hopper barge alongside.

Swinging also is time-consuming, due to the inertial effects in starting, stopping, and reversing the swing. One operator minimizes this by swinging in a 360° circle, without slowing at discharge, that is, discharging into the hopper barge as the boom and bucket pass abeam, and then continuing on around.

Another solution is not to swing at all, but to position the barge transversely at the stern. With the boom well out, the bucket is raised and then pulled in to discharge into the disposal barge. This can reduce the cycle time significantly, but in turn requires a long boom.

To help the clamshell bucket teeth penetrate hard material, vibrators have been fitted; these have so far been plagued with maintenance and practical problems. In one case in very stiff coral, the teeth penetrated so far that the bucket was unable to close and the bucket could not be retracted. In other cases, the vibrators have led to fatigue failures. Clamshell buckets have been widely employed for excavation of bridge pier sites and in cofferdams. They are very useful in digging rock after it has been broken by blasting.

The hydraulic backhoe has been adapted to moderate dredging depths, primarily in harbors, where barge motion is minimal. They can reach up to 25 m below sea level. This tool is especially effective in removing ledges or strata of limestone and caprock, such as are found in the Arabian Gulf. It has excellent power, applied optimally in a leverage fashion. In calm water, it also gives accurate depth control. However, this equipment is not able to work effectively in swells and is, of course, severely restricted as to depth.

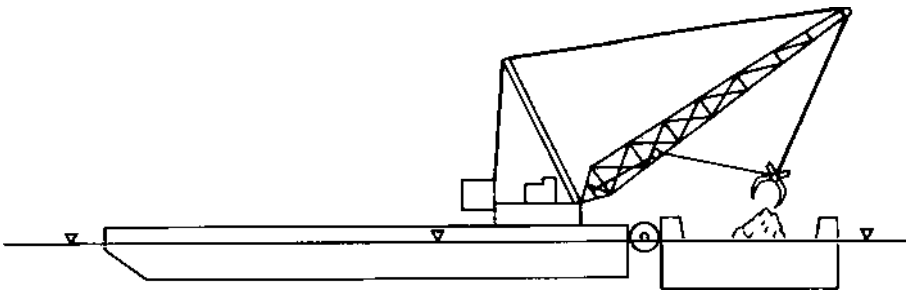

FIGURE 5.29
Clamshell derrick discharging to hopper barge at stern.

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Control of dredging operations has long been a serious problem, especially when dredging a slope, as for an embankment or a marginal wharf (quay) and where the water elevation is varying due to tide or river fluctuation. Recent advances in electronic controls, connected to GPS, have greatly facilitated accurate excavation. Sensor devices mounted on the end of the dredge ladder or arm enable real-time three-dimensional control of the dredge head; accuracies of 30 mm are claimed.

For further discussion of dredging operations, see Chapter 7.

5.14 Pipe-Laying Barges

The pipe-laying barge is a highly sophisticated vessel which constitutes the key element in an offshore submarine pipeline installation system. The system itself and its operation are discussed in detail in Chapter 15. In this section, attention will be devoted to the pipe-laying barge proper (Figure 5.30).

The functions of the barge are to receive and store pipe lengths, assemble and weld them into a single length, coat the joints, and lay the pipeline over the stern to the seafloor.

Operations involved in accomplishing the above include:

1. Positioning the barge
2. Handling pipe lengths from a barge or supply boat to the barge deck
3. Double-ending (optional)

FIGURE 5.30
First generation pipe-laying barge.

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4. Lining up and completing the initial hot pass weld
5. Completing the welds
6. X-ray
7. Applying tension to the pipeline
8. Coating the joints
9. Laying the line out over the stern, usually by means of a stinger
10. Moving the barge ahead on its anchors
11. Shifting anchors continuously ahead
12. Recording positions of laid pipe accurately
13. Radio communications to boats, shore, and aircraft
14. Helicopter and crew boat personnel transfer
15. When weather conditions dictate, "abandoning" pipeline onto the seafloor in an undamaged, unflooded condition
16. "Recovering" an "abandoned" line and recommencing pipe-laying operations
17. Davits to permit supporting a section of the line uniformly for riser tie-in or repair
18. Diving support for inspection
19. Housing and feeding of up to 300 people.

In the above listing, the word "abandonment" is to be construed as a temporary cessation of work and laydown due to the real or threatened onset of a storm.

Such a long list of requirements inevitably requires a large offshore barge. Both heavy-duty standard offshore barges and semisubmersible hulls have been used. The length of the barge is further dictated by the number of welding stations required in order to maintain the desired rate of progress. Since deep-water pipelines inevitably have thick walls, many passes are needed in order to complete full-penetration welds. The more stations there are, the less time needs to be spent at any one station, and hence the lay rate increases (see Figure 5.31).

To move the barge ahead requires many mooring or anchor lines. Large two- and three-drum waterfall winches are mounted along the sides of the barge. The mooring lines lead from the winches over direction-changing sheaves to submerged fairleads and thence to the anchors.

To handle the pre-coated pipe lengths onto the pipe-laying barge, a large crawler crane usually is used, one which can quickly snatch a 40-ft. length from a tossing supply boat or barge at the top of the heave cycle.

A number of heave-compensating devices have been tried, with a signal line from the hook to the boat, for example; however, the snatch method still seems most effective. Once the pipe is stored on board, the next operation may be double-jointing. This usually does not speed up the overall pipe laying, but does reduce the number of specially skilled welders required.

The pipe length, single or double, is then conveyed end-0 and sideways to the lineup station. The pipe rolls onto the rack, which is hydraulically controlled to line up and position the pipe accurately. An internal lineup clamp is applied to join the new section to the previous one so that the first "hot pass" weld may be made. The joint then moves iteratively to the several welding stations, where the weld is chipped and cleaned and new metal deposited.

The weld, once completed, moves to the x-ray station, where pictures are taken, reviewed, and approved. In the case of a reject, a cutout must be made and the

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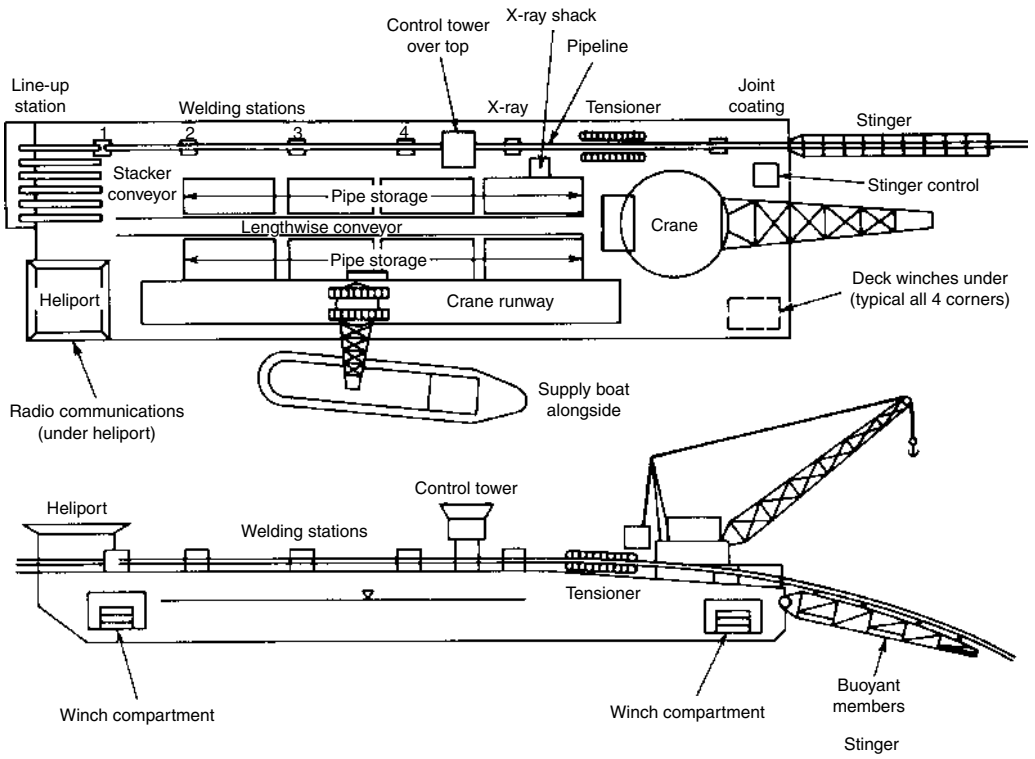

FIGURE 5.31
Second generation pipe-laying barge.

removed weld rewelded and reinspected. Aft of the x-ray station, the tensioner is installed. Tensioners are usually of the caterpillar track type, using polyurethane tracks pushed tight against the rough coating by multiple hydraulic jacks. The tensioning force is thus applied to the pipeline by friction. At the next station, the joint is coated with bitumastic on cement mortar, and then an electric connection is established. The pipeline is now ready to move down an inclined ramp and out over the stringer. Early (first-generation) stingers were long, hinged ladders, partially buoyant, not unlike a dredge ladder in concept. They in effect formed a ramp down which the pipeline ran to the seafloor, with minimum bending stress. Wheels or rollers were provided to reduce friction and to prevent abrasion of the coating.

Second-generation stingers were articulated to accommodate the higher-frequency wave motions and thus reduce the stress in the pipe. These stingers were also buoyant, some even employing the semisubmersible or spar principle to minimize heave response to the waves. Third-and-forth-generation pipeline layers are discussed in Chapter 15.

With the development of improved tensioners has come the third-generation stinger, a curved cantilevered ramp, supported on the barge. It guides the overbend of the pipeline down to its point of departure. The fourth-generation pipe-lay barge uses a high angle for assembly and the J-lay procedure.

Early welding lines, ramps, and stingers were also put on one side of the barge, usually the starboard side, originally as an appendage to an offshore derrick barge. With higher tensioner forces, the tensions in the anchor lines leading forward became critical. The most recent pipe-laying barges therefore have the welding line and stinger on the centerline of the vessel.

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Control of the pipe on the stinger and consequent control of the tensioner force require the use of load cells or similar devices on the stinger so that the pipe reactions and point of departure may be read out in the control room.

For abandonment and subsequent recovery, a large, constant-tension winch is required, positioned so that it can lead its line down the pipe-laying alignment.

Finally, there must be provided all the housing, feeding, and support functions: cabins, mess room, recreation hall, machine shop, power generation, pumps, and winches. A large crane is on the stern. The original cranes were there to enable the pipe-laying barge to also double as a derrick barge. However, a long-boom crane capacity is also needed for setting risers and for installing and removing the stinger.

5.15 Supply Boats

A supply boat is a boat having a large open bay astern, as wide and as long as feasible, to enable the boat to deliver cargo and supplies of all kinds. The "well" or open bay should be long enough to accommodate pipe lengths which, although nominally 12 m in length, may run 2 m or more additional. So a 15–20-m well is common. Supply boats are constantly increasing in displacement and capacity; 1000-tn. displacement was a large boat until recent North Sea needs, sea states, and distances led to increases to 1500-, 2500-, and even 3500-tn. displacement (see Figure 5.32).

While the boat is designed primarily to transport cargo, it must have maneuvering ability for close-in work alongside. It also needs reinforced gunwales and heavy fendering to absorb the impact of contact with other vessels.

FIGURE 5.32
Supply boat moored at Platform Cognac in Gulf of Mexico.

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5.16 Anchor-Handling Boats

The anchor-handling boat is specially designed to pick and move anchors, even in a rough sea. Therefore, it is a short, highly maneuverable vessel. Its stern is open and armored so that wire or buoys can be dragged in over the stern as required. It has a winch at the forward end of the well so that, by means of a line, a wire line pendant or buoy can be quickly dragged on board. Hydraulic-assist equipment is available.

5.17 Towboats

Towboats are of several basic types. The large oceangoing, long-distance towboat is capable of operating to 20–30 days without refueling. It is designed to move to any part of the world to carry out a major towing job. Such vessels may be up to 80 m or more in length and carry a crew of 16–20. They can run light at speeds of 12–15 knots. Harbor and other inland tow boats are smaller and more maneuverable.

Towboats are often described in terms of horsepower, but this can be misleading. Indicated horsepower (IHP) measures the work done at the cylinders of the engine. Shaft horsepower (SHP) is the work actually delivered to the propeller shaft and may be 15%–20% less than IHP. Long-distance towboats typically have IHP ratings of 4000–22,000 HP.

Bollard pull, a much more meaningful measure, is the force exerted by the boat running full ahead while secured by a long line to a stationary bollard; that is, the boat is making no headway through the water. A rough relationship exists between IHP and bollard pull: a 10,000 IHP boat can exert 100–140 tn. of static bollard pull. However, the relationship varies with the size of the propeller(s), whether single- or double-screw, and the draft of the towboat. The effective bollard pull falls off as the speed through the water increases (see Figure 5.33). The largest tugs have static bollard pulls of over 300 tn.

Large oceangoing towboats are fitted with the latest in navigational equipment: GPS, Loran C, radar, electronic positioning, and sonar. They can communicate by voice radio anywhere in the world. These boats may be fitted with a towing engine to enable them to maintain a constant tension on the towline, despite the varying response of the boat to the

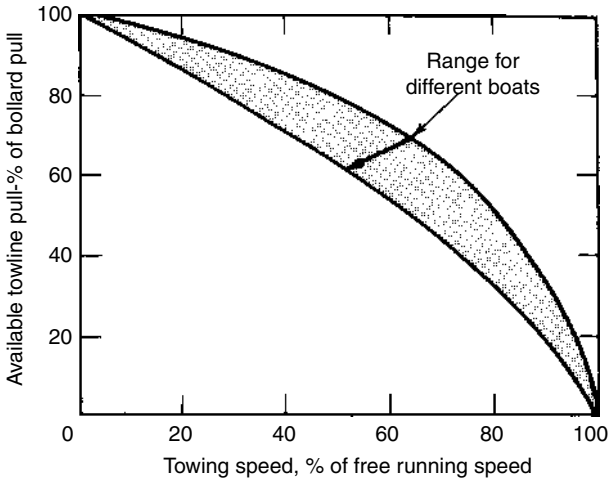

FIGURE 5.33
Effect of speed on towline pull.

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FIGURE 5.34
Five ocean-going tugboats towing a concrete gravity-base platform in North Sea.

waves. Other operators prefer to rely on the use of a long catenary, adjusted during tow to span a full wavelength or more. Boat length should be 11 or more times the expected maximum H_s for safe and efficient operation. In major storms, the boat may have to cut loose and subsequently recover its tow after the storm has passed.

Shorter in length but still powerful are the boats designed for general operations in a specific theater of operations such as the North Sea. These boats are highly maneuverable, often fitted with a variable-pitch propeller that enables them to keep the engine running at full speed during critical positioning operations. They are usually equipped with bow thrusters to enable them to turn up into the wind while making no headway.

Ocean towboats range from 4000 HP for use in moderate seas to 11,000 HP for all-weather ocean tows to 22,000 HP for towing the largest offshore platforms. Up to eight large boats have been used in tandem to tow a platform displacing 600,000 tn. (see Figure 5.34). Inland marine towboats range from 200 to 1000 IHP.

The smaller harbor and coastal tugs are short-range, maneuverable boats, designed for short-term jobs near to port. They may have a heavily reinforced and fendered bow to enable them to push as well as tow on a line. River tugboats on the major rivers typically push their string of barges. Since with a pusher tug, the discharge is unimpeded astern, pushing is more efficient than pulling with a short line. Using the boat as a pusher also enables more accurate control in restricted waters.

A few towboats have been ice-strengthened to enable them to tow through broken ice.

5.18 Drilling Vessels

Normally one does not think of the drill ship or semisubmersible vessel as construction equipment. However, such vessels are often available at the site, having been used for

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exploratory drilling. They are large offshore vessels, fully equipped, including appropriate mooring gear. They have heavy-lift equipment for direct vertical pulls on the drill string, and they have a central moon pool (open well) which provides direct and partially protected access to the sea below with minimal wave action at the interface. They have the ability to work at great depths. Thus they have been used for many offshore construction tasks, from setting subsea templates to pipeline repair and seafloor modifications.

The offshore drilling vessel may be a semisubmersible, with response characteristics as previously described, or may be a large ship hull, especially configured to minimize roll. Nevertheless, such a ship-shaped vessel does have inherently more roll response than a semisubmersible. The drilling derrick is equipped with a large hoist with perhaps 500 tn. (5000 kN) or greater direct lift and is usually equipped with a heave compensator.

5.19 Crew Boats

Crew boats are used to transfer personnel from shore to offshore operations wherever sea conditions permit this to be carried out in a reasonable and practical manner. Crew boats are seldom used in the North Sea; distances are too great and weather conditions unpredictable. Helicopters are used instead. Crew boats are used in the Gulf of Mexico and offshore Southern California. Economics dictate that the boat should have as high speed as practicable. For nonplaning boats, the required horsepower is proportional to the square of the velocity. Consideration has to be given to the boat's motions en route: one does not want the entire crew change to arrive seasick. Generally speaking, accelerations should be minimized by adopting a boat with as low a metacentric height as is consistent with safety. A high $\overline{GM}$ means a quick roll response and physical discomfort to the passengers. A boat may get into pitch resonance with head-on or nearly head-on seas. This may be modified by changing the speed or heading or both. If the boat's length exceeds the wavelength, pitch response is reduced; however, this is usually only practicable in the Gulf of Mexico, not in the Pacific or North Atlantic, with their longer waves.

Discharge and transfer of personnel at sea will be discussed in Section 6.4. In relatively low-sea states, direct transfer can be made to a large derrick barge or pipe-laying barge by coming alongside the lee side or stern while heading into the sea, thus using the derrick barge as a floating breakwater.

5.20 Floating Concrete Plant

These plants have been mounted on large, heavy-duty barges which are equipped with mooring lines or spuds to hold the barge in position. The on-board plant typically has large bins for coarse and fine aggregates, silos for cement, fly ash, and blast furnace slag, water tanks, a batching plant, concrete mixers (usually turbine mixers), and concrete pumps (see Figure 5.35).

The weighing and batching plant is the heart of the operation. The scales should automatically compensate for list (heel) of the barge. Provision should be made for precise

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FIGURE 5.35
Floating concrete plant.

dosing of admixtures. Full recording devices are essential for proper quality concrete. Moisture meters feed data to the batching plant to enable automatic compensation for free water in the aggregates.

To load the bins, cranes with buckets or conveyors are used. Conveyors transport the aggregate from bins to batcher; pumps and screw conveyors deliver the water and cementitous materials. Aggregates, water, and cementitous materials are delivered to the floating concrete plant by barge. At any one time, there may be two or three barges moored to the floating concrete plant. Hence, its mooring system must be sturdy enough to hold the fleet in position. A crane on board may be used to set and relocate tremie pipes or, alternatively, this may be done by a separate crane barge. The plant may discharge the fresh concrete by bucket or by pump.

5.21 Tower Cranes

These are being increasingly used to construct the piers and superstructures of overwater bridges. While limited in capacity, they provide a means of lifting and setting at relatively long reach and height (Figure 5.36). They are also useful in the construction of concrete caissons; offsetting their limited mobility by their accuracy in placement of lifted items.

They are usually mounted on fixed foundations on the partially completed structure. However, they can be mounted on a barge, provided it is sufficiently large and stiff. Foundations must be adequate to resist tipping, uplift, bearing, and torsion.

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FIGURE 5.36
Multiple tower cranes support construction of "Skyway" on East Bay Replacement, San Francisco–Oakland Bay Bridge Seismic Retrofit. (Courtesy of General Construction.)

5.22 Specialized Equipment

Other arrangements and combinations of marine equipment can be assembled to carry out specific tasks. Under specific conditions, such unique assemblies must be thoroughly engineered, including details of connections, and load-tested, in order to ensure safe operations in the marine environment. Stability and buoyancy must be maintained. Many otherwise brilliant innovations have failed due to lack of thoroughness and misguided attempts to save costs. The failure is often in the components, rather than in the basic concept.

The waves tell of ocean spaces,
Of hearts that are wild and brave.

Robert Service, "The Three Voices"

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6

Marine Operations

In this chapter, commonly encountered marine and offshore operations will be described. These include towing, mooring, ballasting, handling heavy loads at sea, personnel transfer, surveying, and diving.

6.1 Towing

Certain basic principles apply to towing. One is that the attachments to the structure or barge must always be sufficiently strong that they do not fail or damage the structure under the force that parts (breaks) the towline. The actual breaking strength of wire rope is typically 10%–15% greater than the guaranteed minimum breaking strength. Actual breakage will usually occur under a dynamic load rather than a static load. It is important that under overload, the structure or vessel being towed remains undamaged. A usual requirement is that the ultimate capacity of any towline attachment to the unit be at least four times the static bollard pull and at least 1.25 times the breaking strength of the towline from the largest tug to be used on that attachment. At least one spare attachment point, with pennant, should be fitted for towing ahead, to be used in case of emergencies. A second principle is that the towing force must be able to be resisted through a significant range of horizontal and vertical angles, thus imparting shear and bending, as well as tension, on the towing attachment.

If a towline does break at sea, it is desirable that it fail at a known "weak link" so that it may readily be reconnected, even in high sea states. A typical arrangement when a single boat is towing with a bridle is shown in Figure 6.1. If the towline is subjected to a high-impact overload, the short pendant between B and C breaks, the shackle at B is pulled back on deck by means of a fiber rope pendant, a new pendant fitted (BC), and the towline reconnected. To reduce shock loads in the towline, either an elastic fiber pendant catenary may be used or a length of chain installed in the belly of the catenary (see Figure 6.2).

During passage through restricted waters and during final positioning, the towline may be shortened in scope to permit better control. If it is too short, however, the thrust of the propeller's wash will react against the towed vessel. When one of the large GBS caisson structures (Gravity-Base Caisson) was being moved in Stavanger Fjord, Norway, the lines had very short scope in order to control movement between rock islands. The thrust of the propeller wash against the 120-m-wide and 50-m-deep projected area of the caisson resulted in inability to get the structure to move. The solution was to place the primary tugs at the rear of the caisson, pushing in notches fabricated of steel and timber. Thus, the full efficiency of the propeller's thrust could be developed (see Figure 6.3).

The inertia (momentum) of a towed structure, especially a large one such as an offshore caisson, is tremendous. It tends to keep moving ahead long after pull has ceased. A constant concern of boats when towing in congested traffic conditions or in ice is that if

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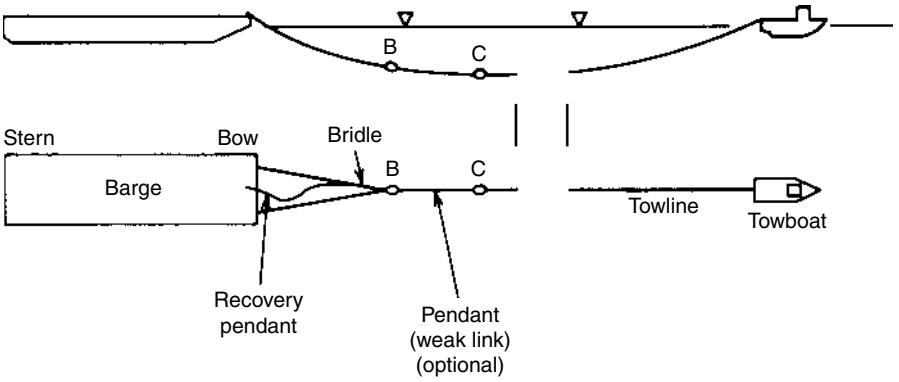

FIGURE 6.1
Typical towing arrangement for barge on ocean tow. (Courtesy of Aker Maritime.)

the boat is stopped, the towed vessel or structure may overrun it. Further, due to the inertia of the towed structure, it is difficult to slow it or change direction (Figure 6.4). In a narrow channel, therefore, additional boats may be used alongside and also astern. The boats located astern are being dragged backward; when needed, they can go ahead on their screws and thus slow the towed structure. However, being dragged astern, there is a tendency for them to be pulled down and swamped, so special stern sheets are usually fitted and special attention paid to watertight closures on the boat, since otherwise the engine room door may be left open, regardless of the published instructions (see Figure 6.5).

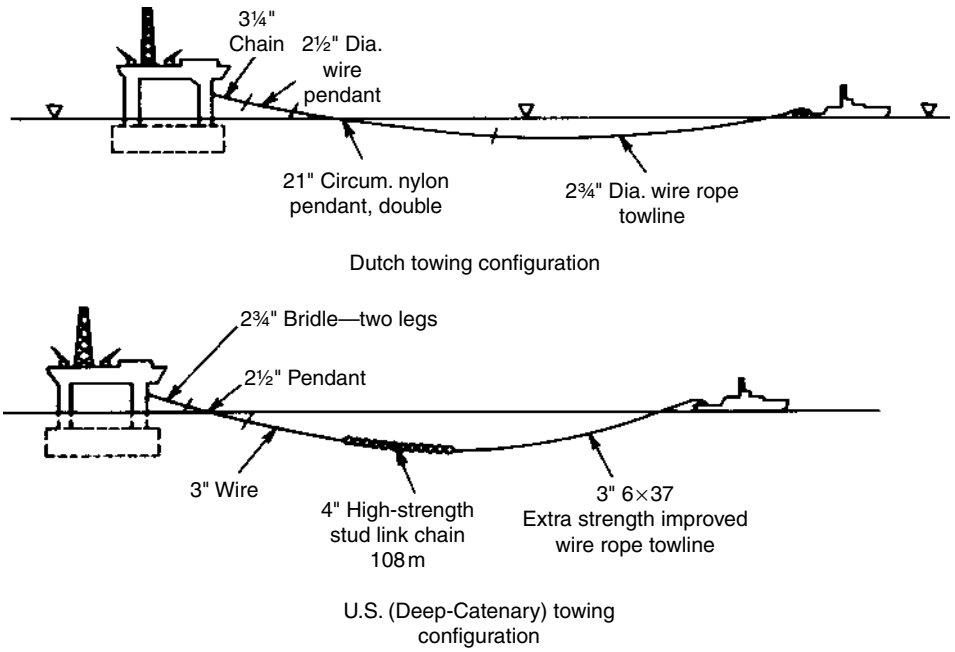

FIGURE 6.2
Ocean towing configurations.

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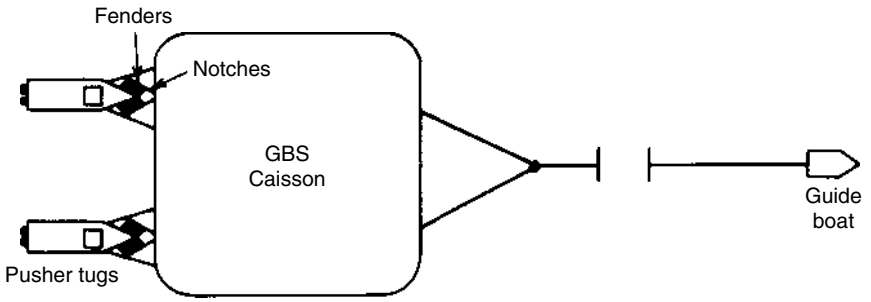

FIGURE 6.3
Pusher tugs moving massive caisson.

When towing out in the open sea, the boats lengthen out their towlines to offset the wide range of loads in the lines due to the waves and swells. When towing a very large structure in coastal waters, a single lead boat may run ahead to verify route, confirm depths by forward-looking sonar, and pick its way through underwater obstructions, or ice. Such a lead boat can also warn other shipping, which may be disregarding published Notices to Mariners.

If the towed structure is a deep-draft vessel (some of the offshore platforms in the North Sea have drawn 110–120 m), then the towline, if attached to the structure below water near to the center of rotation, may have a steep inclination. This will tend to pull the stern of the boat down into the water. Therefore, the towline may be led up to a pontoon or buoy, which will resist the vertical component of the towing force. Such a buoy should be foam-filled to prevent flooding in event of a leak or hole (see Figure 6.5). Such a system may also be useful when towing through broken ice to minimize the shock loads in the towline itself.

In the case of the Dunlin platform and the Ninian Central platform, flotation units shaped like sausages were clamped to the towlines. These floaters were filled with

FIGURE 6.4
Towing concrete platform out of Norwegian fjord. Note small tugs pulling astern in order to provide control. (Courtesy of Aker Maritime.)

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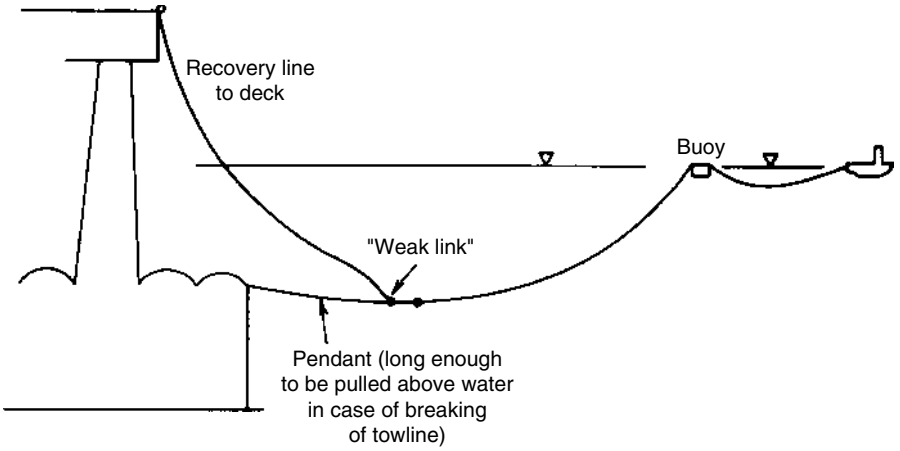

FIGURE 6.5
Towline arrangement for deep draft tow.

polyurethane, each giving approximately 5 tn. net buoyancy. In Figure 6.5, note the pendant leading up to the deck, to permit reconnection in case of a towline breakage.

Most channels, harbors, and shallow offshore coastal areas have been extensively surveyed, with the results published on hydrographic charts. Unfortunately, the depths of interest were those that pertained to ships having drafts of 10–20 m, so when the survey ship got into deeper waters, it usually only recorded depths on a grid, with no interest in determining possible rises, shoals, or pinnacles as long as they did not present a hazard to shipping.

A similar problem arises when towing vessels or structures in areas not normally used by shipping. Hence, careful surveys need to be made, using sonar, side-scan, and profiler acoustic equipment so that both the route and its full swath, including sway excursions, are thoroughly scanned.

Required channel widths in sheltered areas are usually twice the beam, but this must be considered in relation to the environmental conditions and navigational accuracy. For exposed areas, the required width will depend on currents and navigational accuracy and thus may vary from about 600–1500 m for relatively short distances of 12 km or so. During the tow between islands or shoals, accurate electronic position-finding systems need to be set up. Unlike the case of a towed ship or barge having a draft of perhaps 8–10 m and a width of 30–40 m, an offshore structure such as a deep-water caisson may have a draft of over 100 m and a width of 100–150 m. Therefore, it is not enough to plot only the position of the "bridge"; the extremities must also be charted. Detailed current surveys, both on the surface and at depth, must be made in restricted areas.

A structure under tow will experience sway and wander somewhat on its course. In confined waters, a band may be plotted, shaded in color, within which the structure is safe. Then as the edge of the structure approaches the band edge, corrective action can be taken. This will eliminate excessive "hunting" back and forth, trying to stay exactly on a course line (see Figure 6.6).

Towed vessels and shallow-draft structures may have an actual draft greater than their mean draft. This may be due to trim, squat, list, or wind heel. It may be due to the lower density of fresh water discharging from a river into the adjacent sea: fresh water reaches

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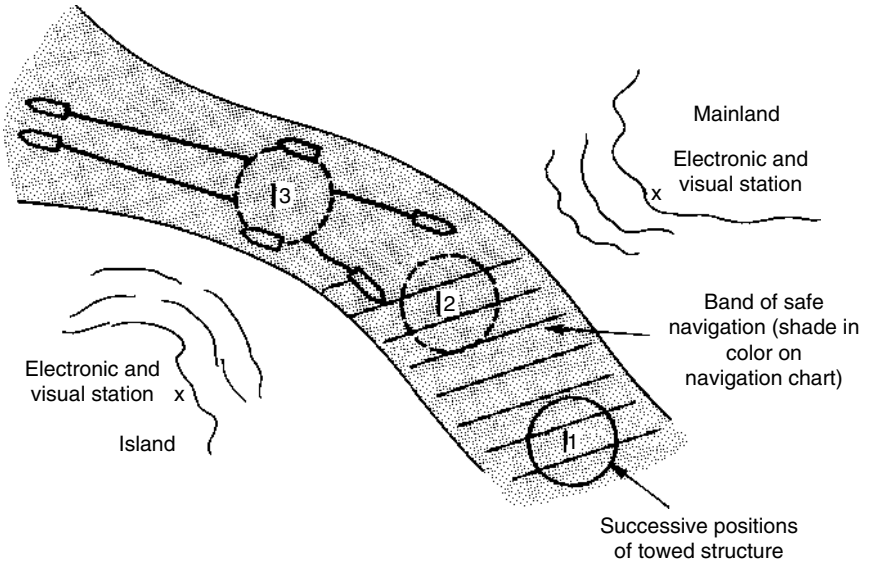

FIGURE 6.6
Swath or band for navigating in constricted channels.

long distances from the mouths of such rivers as the Orinoco, Amazon, and Congo. In some cases, especially if crossing a bar, heave response may need to be considered.

The usual requirement for underkeel clearance is that the distance between maximum static draft and minimum water depth should not be less than 2 m or 10% of the maximum static draft, whichever is lesser, plus an allowance for motion. The maximum static draft should be the actual measured draft at the deepest point with allowance for errors in measurement, initial trim, and water density change. The motion allowance should include the maximum increase in draft due to towline pull, wind heel, roll and pitch, heave and squat. These values can best be determined by model tests. In practice, most of the structures under consideration in this book will be governed by the 2-m minimum clearance.

Air cushions may be used to reduce draft when crossing local areas of limited water depth. In general, the use of air cushions should be employed only to increase underkeel clearance above the theoretical minimum value to ensure that the structure will still not hit in event of loss of air. It is important that the reduction in metacentric height and stability due to an air cushion be considered, since the air cushion acts like a free surface in reducing the moment of inertia. After the crossing of the shoal area, the air cushion should be completely vented. When using an air cushion, an adequate water seal height between the skirts must be left to prevent loss of air. The height of the seal will depend on the speed, since this will cause some air to be sucked out. Typical water seals vary from 0.5 to 2.0 m.

To optimize the reduction in draft, large air bags, each $11 \times 11 \times 4$ m, of PVC-coated polyester fabric, were inserted under the base of the Andoc Dunlin platform. This enabled the full depth of the skirts to be utilized for buoyancy, without need for a water seal. Free air was also used in the compartments in the small spaces around the bags.

Communication between multiple boats during a critical positioning operation is all-important. Voice communication is used exclusively; however, it must be remembered that the tug skippers are of all nationalities. While English is usually the common language, misunderstandings and lack of full comprehension have led to serious mistakes.

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To obviate these, a carefully agreed set of common procedures should be adopted and reviewed so that there will be a clear understanding of all commands. If there are one or more captains who are not fluent in English, it may be desirable to have an interpreter available.

Procedures should be adopted to handle the case of a broken towline. The boat in question must take in the line and circle back. The towed vessel or structure should recover the bridle or pendant. As the towboat returns alongside, a messenger can be passed and the towline brought on board and made fast. All this is simple with one boat in a calm sea. It is very complex when it occurs at night in heavy seas and wind and the boat is one of three or four, each with its towlines under strain. For very long tows, provision must be made for refueling enroute. One boat at a time can be fueled from a spare boat.

The dynamic accelerations of the towed structure should generally be limited to 0.2 g, to minimize forces acting on the tie-downs and to minimize adverse effects on personnel. Emergencies that must be included in the planning for a tow are fire, flooding, and man overboard. While in congested areas near the exit port, a special fast boat "guardship" should be employed for the dual purpose of picking up a man overboard and warning away sightseeing boats. When the Condeep Statfjord A sailed from Stavanger, private cruise boats advertised an excursion alongside. This involved hundreds of people whose safety was paramount. News photographers fall into a similar category. Imagine what would happen if a tour boat was overturned by running up on a submerged towline! Harbor police can often be engaged to keep the route clear. In the case of the Statfjord A Condeep, the ceremonies were held on the advertised day, the flags flown, the cruise boats ran around to take pictures, and the towboats blew their whistles. Meanwhile the caisson was securely moored. Two days later, with no fanfare, the flotilla actually got underway, with no unwanted interference.

When large and valuable structures are towed (Statfjord B and C each had a value of approximately U.S. $2 billion), the insurance surveyors require a full manning, with adequate pumping capacity, power generation, and firefighting capabilities. Manning of large and important structures under tow may require a crew up to ten or more. This crew will probably contain not only marine and ballasting crews, but also a meteorologist, sonar specialist, and navigator.

When towing in thin or broken ice, an icebreaking vessel will usually open a clear channel. Crowley Maritime has utilized an ice-strengthened barge, with a pusher tug, to open a channel around Point Barrow into the Beaufort Sea. Other similar tows have been preceded by several icebreakers. Even when a channel has been made through the ice field, there remains the problem of ice clearance. An offshore platform for the Arctic may have a beam width of 100 m. Underkeel clearance will usually be minimal. Somehow the broken ice must be forced to clear around the sides so as not to jam the tow. Boats at the sides can clear ice. Towing in ice has the further problem of fog obscuring visibility. Radar is fine for locating other vessels but usually cannot distinguish floating ice growlers, bergy bits, and other broken ice floating half-submerged from wave-reflected echoes. If a lead boat, towing ahead, is on short scope to facilitate maneuverability, it is in danger of being overrun by the tow in the event it is stopped by ice. For these reasons, the use of stern pusher tugs may be an appropriate method of moving an offshore structure through heavy ice (see Figure 6.7).

Some of the structures proposed for the Arctic have a conical shape. In the open sea, waves can run up over the lower sides, leading to erratic response. The waterline diminishes rapidly with immersion; hence stability can be significantly reduced. Tows of such structures may require trim down by the stern, thus increasing draft. In broken

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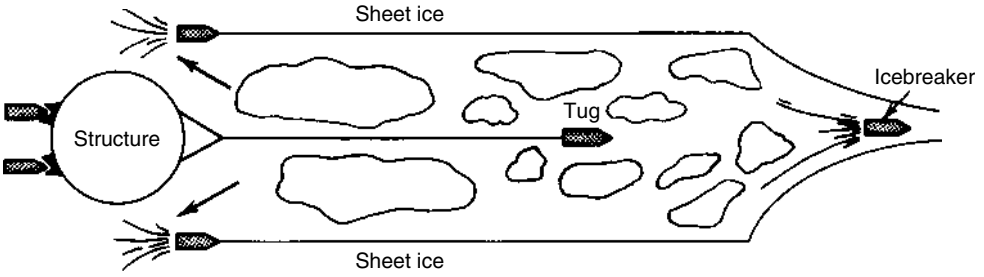

FIGURE 6.7
Towing a large structure through heavy ice.

ice, masses may ride up over the sides. The effect of all these can best be evaluated by model tests.

Tows of lesser value may be manned or unmanned. If manned, the Coast Guard will require adequate life rafts and communications to ensure the safety of those on board. If unmanned, trailer lines of fiber should extend from the four corners or quarters to enable a boat to pick up the tow and put personnel on board.

When towing a deep-draft structure through shoal water, the tidal conditions must be carefully plotted. Delays of a few hours, common to marine operations, must be anticipated; otherwise the structure may arrive at the critical zone at low tide. On the other hand, an advantage of traversing a shoal at mid-tide is that, if the structure does ground, it will probably raise off at the next high tide. Of course, shifting ballast or deballasting may also be attempted, but not so much as to endanger stability once the structure floats free. Currents affecting the tow of large structures are principally tidal in nature. When towing the Ninian central platform through the Minch, northeast of the Isle of Skye, the necessary height of high tide was unfortunately always preceded by an adverse flood tide. The current was such as to permit almost no headway to be made against the flood. Therefore, the boats had to catch the slack water at the top of the tide and move over the shoal at maximum speed.

Summer storms may arise despite the best long-range and short-range forecasting. They may turn out to be so severe as to make it necessary to cut loose the tow. Lay-by and standby areas along the route should be identified and marked on the chart. The tow can proceed to point A and then, based on the current sea conditions and the short-range forecast, continue to B, and so forth. At each such station, the alternative of standing-by can be considered. These standby areas are selected for having adequate sea room to lee.

When positioning a structure at an offshore site, it is customary for the tugs to fan out in star fashion. Then, the positioning is controlled by going ahead on some tugs more than others; that is, all lines are kept taut. Such an arrangement has been used on the North Sea offshore concrete platforms (see Figure 6.8). Note the use of buoys in the lines to prevent pulling the sterns under. Bow thrusters are very desirable in enabling a boat to turn into the wind without exerting an increased pull on its towline.

As noted earlier, towboats are usually rated by their indicated horsepower (IHP), whereas a more meaningful figure is the bollard pull that they can exert. A tug will not, of course, be able to maintain its static bollard pull under continuous running conditions at sea, since the bollard pull decreases with speed.

The towing horsepower selected should be sufficient to hold the towed structure against waves of Hs = 5 m, 40-knot sustained wind, and 1-knot current. Obviously, these arbitrary parameters have to be adjusted to the region involved.

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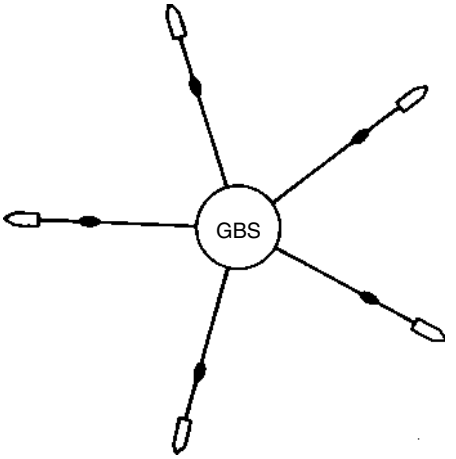

FIGURE 6.8
"Star" positioning of tugs for installation at site.

Limitations and requirements are placed on stability under tow by the marine surveyor. Typical requirements are the following:

1. The metacentric height should have a positive value, typically 1–2 m for a large offshore structure.
2. The maximum inclination of the towed structure under conditions of Hs = 5 m, wind 60 km/h, and full towline pull is not to exceed 5°.
3. The maximum inclination of the towed structure under the 10-year storm for the season involved, with no towline pull, does not exceed 5°.
4. The static inclination under half the total towline pull, in still water, does not exceed 2°.
5. The static range of stability should not be less than 15° at the draft during tow or installation.
6. To insure dynamic stability, the area under the righting moment curve to the second intercept or down flooding angle, whichever is less, is to be not less than 40% in excess of the area under the wind-heeling moment curve to the same limiting angle. The wind velocity is that associated with a 10-year seasonal return period, sustained for 1 min.
7. Model tests are usually required to verify the motion of the towed structure in both regular and irregular (random) seas. These can be used to determine directional stability and any tendency for excessive yaw. Model tests can also be used to investigate the towed structure's behavior under damaged (flooded) conditions, and to determine towing resistance.
8. Inclining tests to verify the $\overline{GM}$ (metacentric height) must be carried out shortly before the tow, after all superstructure modules and consumables have been loaded.

In May 1978, the Ninian central platform of prestressed concrete was towed from the Inner Sound of Raasay on the west coast of Scotland to the Ninian Field in the United Kingdom sector of the North Sea. The structure had a base diameter of 140 m, a draft of 84.2 m, and a displacement during tow of 601,220 tn. The draft was severely constrained by the water depth in the Minch at the exit from the Sound of Raasay. The towing distance

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was 499 nautical miles (925 km) and took 12 days. A fleet of five tugs, totaling 8600 IHP with a combined bollard pull rating of 585 tn. (5850 kN) was employed.

In August 1981, the Statfjord B concrete platform was towed from Vatsfjord, near Bergen, Norway, to the Statfjord Field, a distance of 234 nautical miles (433 km). The displacement during tow was 825,000 tn. and the draft 130 m. Beam was 135 m. Five tugs, totaling 86,000 IHP, with a combined bollard pull rating of 715 tn. (7150 kN) were used. The tow required 6 days. Detailed data on the tows of early concrete platforms in the North Sea are given in chapter 12 (see Figure 12.31 and Table 12.1).

The Thistle Platform was one of the largest self-floating steel jackets ever installed. It was built at Graythorp on the River Tees and towed 420 nautical miles (773 km) to the Thistle Field in the United Kingdom sector of the North Sea. Its displacement was 31,000 tn., its dimensions 110 m wide $\times$ 184 m long. Two tugs, having a combined bollard pull of 195 tn. were used to tow at a mean speed of 3.8 knots.

In September 1975, a slightly smaller self-floating tower was towed from Tsu, near Nagoya, Japan, to the Maui field off New Zealand. The two tugs had a combined horsepower of 15,200 IHP. The structure survived a typhoon in the Pacific with only minor damage.

The steel caisson Molikpaq and the composite steel and concrete caisson GBS-1 were both towed from Japan to Point Barrow, Alaska, and thence to sites in the Beaufort Sea. For the Molikpaq, three tugs with a total of 48,000 IHP were used; for the GBS-1, two tugs totaling 44,000 IHP. Displacements were 33,000 and 59,000 tn., respectively, and the beams were 100 and 110 m. Drafts were about 10 m. Each tow took approximately 50 days.

Four steel gravity platforms were towed 4250 nautical miles from Cherbourg, France, to the Loango field offshore Congo. Each structure resembled a jacket, to which three stabilizing bottle-shaped towers had been affixed in order to give stability. Draft of each was 16–19 m; displacement was between 7000 and 8000 tn. Towing speed averaged 3.2 knots, using two boats for each platform, developing 30,000 IHP. Several 5-tn. polyurethane foam-filled "floaters" were inserted in each towline.

Further specific information on towing is presented in Section 11.3 and Section 12.2.13 (see especially Table 12.1 and Table 12.2).

6.2 Moorings and Anchors

6.2.1 Mooring Lines

Vessels working at an offshore site must be held in position despite the effects of wind, waves, and current. The current forces are relatively constant in direction in the offshore zones; in closer-in areas and opposite the mouths of great estuaries they may vary with the tidal cycle. The wave forces can be considered as comprising an oscillatory motion plus a steady, slow drift force. Both the mean forces of a quasi-static nature and dynamic forces must be resisted.

The standard means of mooring is by way of a mooring system that connects the vessel (or structure) to the seafloor by means of laterally leading lines to anchors. Moorings must be thought of as a system that includes the vessel, the anchor engines, fairleads, mooring lines, buoys, and anchor. In deep water, they can be of the catenary type, extending from the vessel in a catenary to the seafloor and thence laterally to the anchor. In shallow water, taut moors may be employed in which the mooring lines are tensioned to run relatively straight from the vessel to the anchor or fixed structure. Recently, taut moors have also been used in deep water.

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The dynamic portions of the mooring force must be absorbed. The most often used method is by means of the catenary: the dynamic surge raises the line, using up the kinetic energy in geometric displacement. This concept can be further exploited by including extra weight in the belly of the catenary—for example, a shot or two of chain or a clump anchor weight.

Another means of absorbing energy is by the elastic stretch of the line itself. Wire rope has an initial modulus of about 100,000 MPa (15,000,000 psi), which increases with use. As the tension force increases due to surge of the moored vessel, the line stretches. As the vessel returns, the line contracts. The energy absorbed is proportional to the line length and the effective modulus. In practical moorings, both the catenary and the elastic stretch participate.

Very-low-modulus materials are available in the form of nylon and polypropylene. Nylon is widely used for very short lines; unfortunately, it is so elastic that it stores great amounts of energy. If a nylon line breaks, it may not only develop a sudden shock loading but whip back dangerously. Higher-modulus fiber lines, such as Kevlar, are available. Steel wire rope is the standard material for mooring lines for construction. Wire lines made with a fiber core and close pitch have somewhat lower moduli than those with a wire core. This is why most mooring lines are wound around a fiber core.

The third method of absorbing energy is by the use of some form of compensator in the system. Hydraulic and steam constant-tension winches are available. Hydraulic compensators may be placed just ahead of the winch. These can be procured with the desired amount of surge accommodation and force displacement characteristics. Even job-fabricated systems with rubber fender units have been used successfully. These latter are effective for very short mooring lines of a temporary or single-time use.

Some large semisubmersible drilling vessels and drill ships use chain mooring lines, with winches specially fitted for this use. Longer-term moorings may use a hybrid combination of steel wire and anchor chain.

The sheave diameter of fairleads should be at least 20 times the diameter of the wire rope. When mooring lines break, they usually do so at the fairlead, for this is where bending stresses are added to direct tension.

6.2.2 Anchors

6.2.2.1 Drag Anchors

Anchors are of a number of basic types (see Figure 6.9 and Table 6.1). First are the reusable drag anchors, which have evolved from ship anchors; they include the stockless type used by the navy, the Danforth, and the newer Bruce, Stevin, and Doris anchors. These anchors are designed so that, as a horizontal force is applied, they dig down into the soil and mobilize it as resisting force. They are often rated on the multiple of their holding power to their air weight. This is an oversimplification, since it is the soil that they must mobilize, and the resistance varies with the characteristics of the soil and the configuration of the anchor. Some anchors are specially designed for soft clays, others for sands.

All these anchors require that the pull be horizontal. In fact, they are purposefully designed so that a vertical pull breaks them loose with little more force than their weight. This means that the portion of the mooring line immediately ahead of the anchor must be heavy enough to stay seated on the seafloor even when the line is under full tension. One or one and one half shots of chain are usually placed in this segment of the line.

Sometimes two anchors are used (“piggybacked”) by joining them with one-half shot of chain. Tandem anchor arrangements can frequently develop more than twice the capacity

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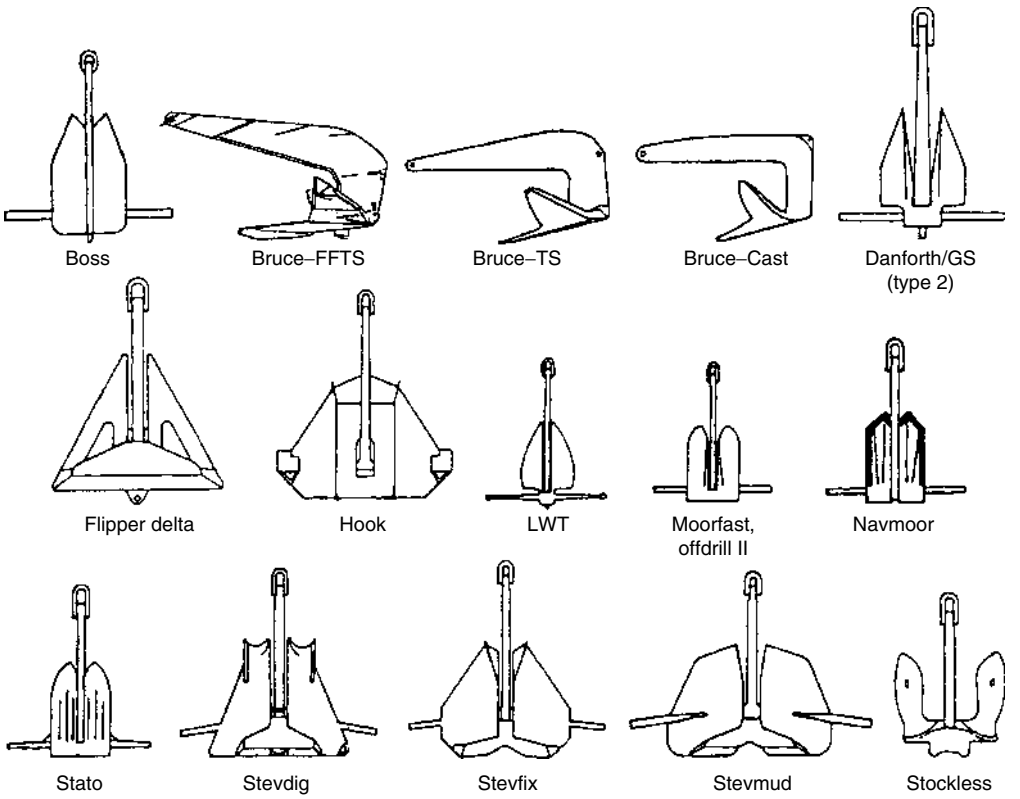

FIGURE 6.9
Drag embedment anchors.

TABLE 6.1

Drag Embedment Anchor Efficiencies

	Efficiency[a]	
Anchor Type	**Sand**	**Mud/Slit**
Navy stockless	8:1	3:1
Stevfix	18:1	15:1
Stevfix	31:1	15:1
Stevdig	29:1	—
Stevmud	—	20:1
Hook	12:1	18:1
Bruce	25:1	—
Bruce twin-shank	—	12:1
Doris mud anchor	—	20:1[b]
Danforth	15:1[c]	15:1[c]

[a] Ratio of horizontal holding power when fully "set" to weight.
[b] Exact value unknown but believed to be about 20:1.
[c] Exact values unknown but believed to be about 15:1.

Source: Based on reports by the Naval Civil Engineering Lab, Port Hueneme, California.

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of an individual anchor. There is an exception, however, for the cases where frequent moves are required. Chain cannot usually be accommodated through the fairleads and onto the winches. Therefore, wire lines may be used to within a few meters of the anchor, with extra length of line to ensure a horizontal pull. Anchor system holding capacities are portrayed in the form of nomographs on Figure 6.10 and Figure 6.11.

Drag embedment anchors of the Danforth, Bruce, Stevin, or Doris type typically weigh 10,000, 30,000, and even 40,000 lb (4, 13, and even 17 metric tons). They are usually placed on the seafloor by lowering directly from the mooring line or from a pendant. The pendant line leads more or less vertically upward to an anchor buoy. This enables an anchor-handling boat to pick up the anchor and move it to a new location. The anchor-handling boat typically raises the anchor only a few meters clear of the seafloor, carries it to the new position, and lowers it back.

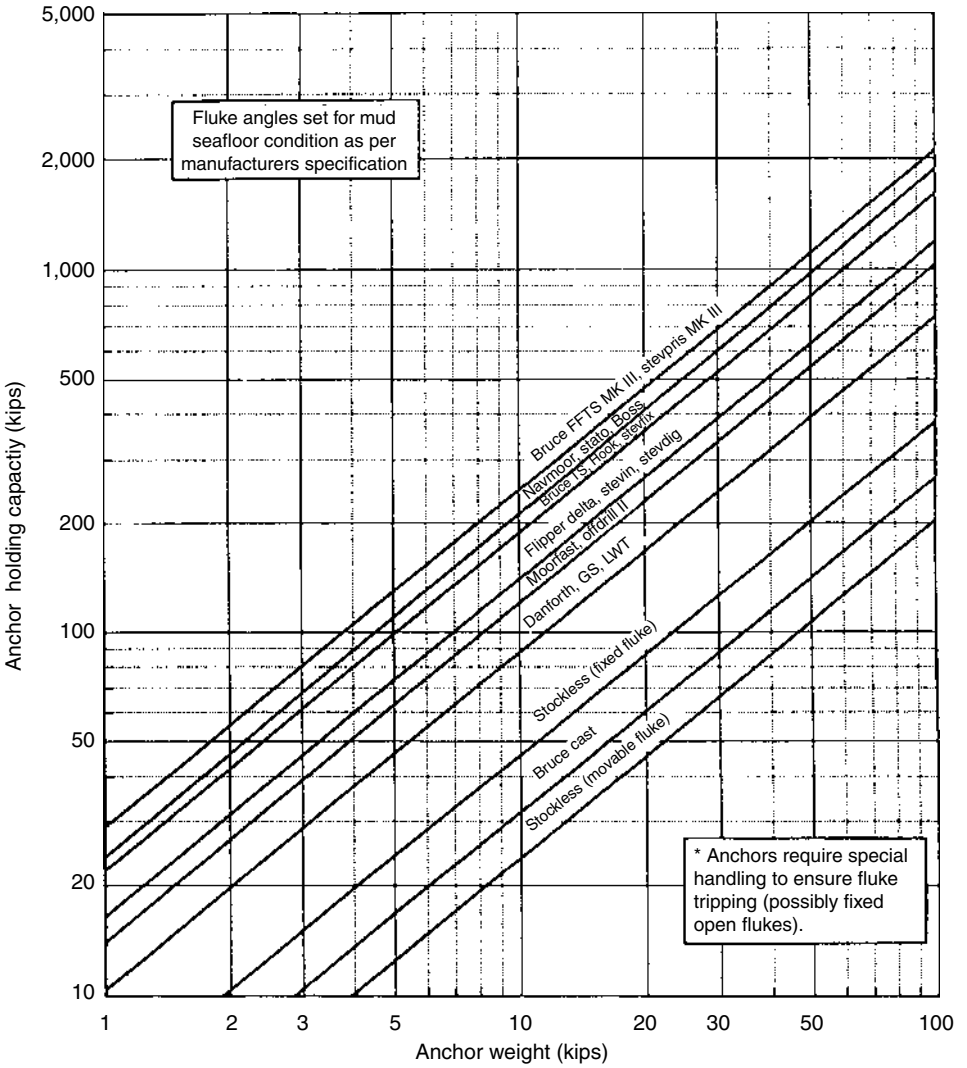

FIGURE 6.10
Anchor systems holding capacity for drag anchors in soft clay.

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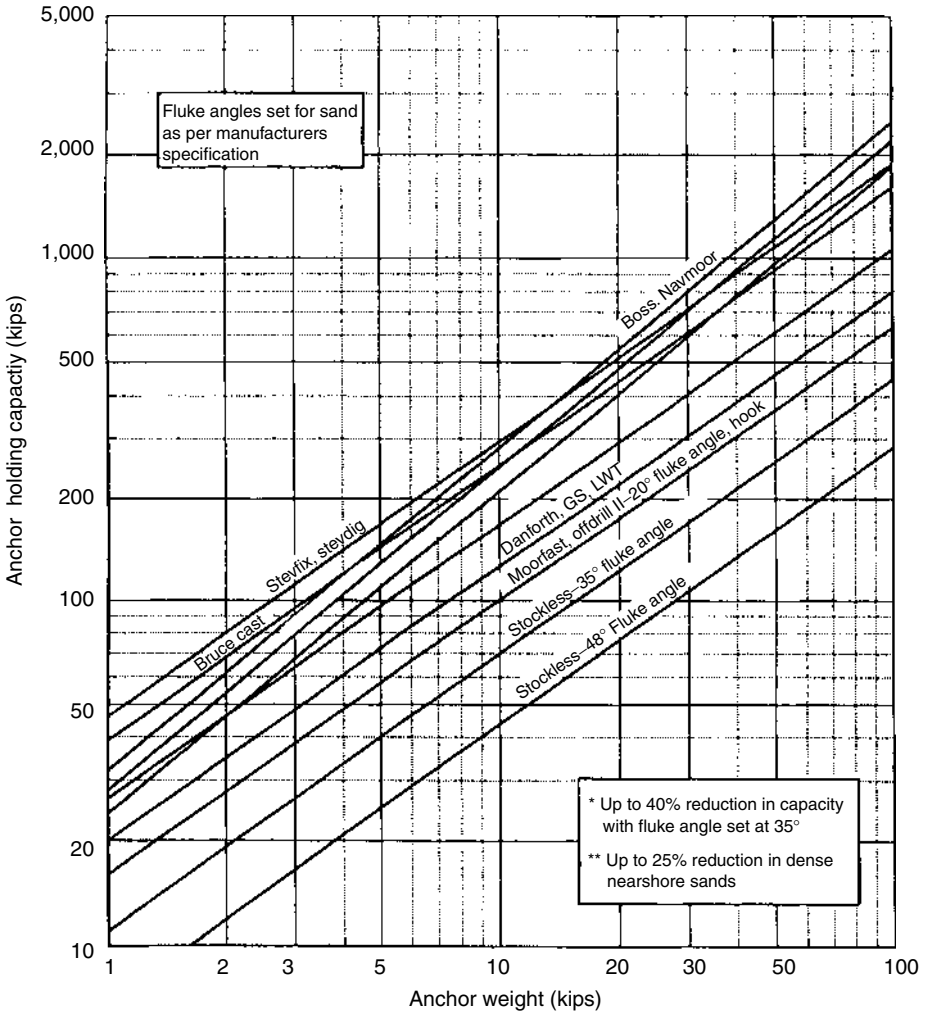

FIGURE 6.11
Anchor system holding capacity for drag anchors in sand.

Factors of safety for drag anchors are intentionally set lower than those for wire rope and chain, in order that they will drag before the line breaks. Refer to API Standard 2SK. For long-distance moves, the vessel takes in on the mooring line until, as it approaches a vertical pull, the anchor breaks loose and swings in under the fairlead. The anchor is now raised until it is housed between plate housings, or else is picked by a crane or davit and placed on deck. The anchor cannot be left dangling just below the hull; it might swing into the hull under wave action and hole the plates.

Drag embedment anchors are ineffective on rock and erratic on layered (stratified) seafloors. So for these conditions, a clump or gravity anchor is used. These develop their resistance primarily from dead weight times a friction factor. Such a clump anchor is best used on hard soils (boulder clay or rock, etc.) where the friction factor will approach 1.0 and the holding force therefore is approximately equal to the dead weight. This type of anchor is used on hard bottoms such as rock, boulders, and conglomerate. The navy stockless anchor can be used as a deadweight or clump anchor, although the larger

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deadweight anchors are usually concrete. Semipermanent deadweight anchors may be open boxes filled after placement with rock or concrete. These anchors may be placed in a dredged hole and backfilled with stone.

Deadweight anchors can also be used for permanent moorings where the long-term characteristics of the soil are little known. They can be used where the direction of pull changes radically from time to time, since they are omnidirectional, whereas the Danforth-type anchor will just pull out when the pull reverses direction. Deadweight anchors can be utilized where the seafloor is bedrock or has very little overburden.

Drag embedment and deadweight anchors have the useful property that they can be designed to drag through seafloor sediments before the mooring line parts. Deadweight anchors, sliding across the seafloor, maintain a large portion of their static holding power.

Drag-in plate anchors can be used for both vertical and lateral anchorage.

6.2.2.2 Pile Anchors

Pile anchors are very effective in many soils. The pile can either be drilled in and grouted, using an offshore mobile drilling rig, or driven in with an underwater hammer or a follower. The anchor line, usually a shot of chain at this location, can lead from the top or from a point a few meters down the pile. The anchor pile resists pullout by a combination of bending plus passive resistance (the P/y effect) and skin friction shear. In some cases, in rock, a chain has been grouted into a drilled hole, connecting directly to the mooring line. This system was successfully installed off Tasmania to serve as permanent moorings at an offshore iron ore shipping terminal.

Of special concern are soils that have unsuitable characteristics. One of these is calcareous soil, for which little skin friction is developed. Any vertical force applied will lift the pile. Even a straight horizontal force may lead to crushing of the calcareous grains and a degradation of holding power. Extensive grouting of an anchor pile in such soils has greatly improved its capacity as compared with a driven anchor pile. Gravity anchors can also be used.

The most difficult anchoring soil of all is a soft mud, silt, or loose sand overlying a hard material such as conglomerate (off Taiwan) or very dense sand and silt (in the Canadian Beaufort Sea). For these soils the conventional drag embedment anchors tend to skid on top of the hard stratum. Drilled-in anchor piles are not practical if many moves are involved. Deadweight anchors may be used if placed by jetting to seat them firmly on the hard material. Conventional (navy stockless) anchors may be placed in holes excavated by clamshell bucket and then backfilled with dumped rock. This type of anchor was effectively used in northern Queensland, Australia, where soft muds overlay hard volcanic tuff.

6.2.2.3 Propellant Anchors

The U.S. Navy has been active in the development of propellant-type anchors, in which the anchor shaft is driven into the soil by either free fall or explosive force. Once penetrated, its flukes resist pullout. These propellant anchors have been extensively used to hold the mooring buoys at the naval base at Diego Garcia, in the Indian Ocean, where hard limestone layers overlain by sand make conventional anchors ineffective.

Propellant embedment anchors are rated up to 150 tn. long-term capacity in soft mud and clay soils. Actual values are higher in sand and coral, ranging up to 300 tn. These anchors are multidirectional, are installed rapidly, and function best where drag anchors are least effective.

For use in the deep sea (over 200 m), the anchors must resist primarily vertical forces. Very heavy concrete deadweight anchors may be used. Recent developments include drag

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anchors shaped to develop high vertical capacity. They are seated by horizontal pull, then rotated (or the flukes rotate) to resist uplift.

6.2.2.4 Suction Anchors

Suction anchors gain their vertical capacity by the weight of the plug inside and the friction (shear) on the outer surfaces, and in addition, the negative end-bearing, that is, the force required to separate the lower end of the soil plug from the undisturbed soil. Since taut moorings impose large lateral loads, chains are attached about half way down the length. This is a point of very high stress and potential fatigue on the cylinder, which must be well-reinforced (Figure 6.11).

Typically, suction anchors are larger than 5 m in diameter and 20–30 m in length. The suction anchor is lowered to the seafloor with the top valve open and allowed to penetrate under its own dead weight. Then the top valves are closed and water pumped out to create an underpressure in the cylinder. This gives an additional driving force equal to the net differential hydrostatic pressure over the cylinder's area. The underpressure is limited by soil heave, which plugs the cylinder and prevents further penetration. If the top is sealed in service, the capacity is increased with time.

For removal, the water is forced into the top of the anchor. By keeping the pressure on for several hours, the pore pressure in the soil will be raised and the shear reduced. While the suction anchors are most effective in deep water (see chapter 22), they can be used in depths as shallow as 100 m.

6.2.2.5 Driven-Plate Anchors

Driven-plate anchors have been developed by the U.S. Navy for use in coral lagoons. For the Second Tacoma Narrows Bridge in 50-m water depth, these were upgraded to a 500T tested capacity. A flat plate is connected to the lower end of a steel pile with a stout hinge, and a shot of chain is attached. Then the pile is driven into the seafloor, the steel plate in a vertical alignment being pulled down deep in the soil. When the lateral pull is taken on the chain, the plate rotates as it pulls upward and develops the full weight and shear resistance of the soil above (Figure 6.12).

6.2.3 Mooring Systems

In very deep water, the weight of the anchor line itself is excessive. Composite lines of steel wire and aramid fiber (Kevlar) or polyester are often used to reduce self-weight. A submerged spring buoy installed in the middle of the catenary will support it and give more horizontal control during operating conditions. In deep water, the lines will be predominantly vertical, and hence exert uplift on the anchors. For this, the suction anchor or deep embedment anchor that mobilizes the weight of soil appears best.

Conversely, in shallow water such as rivers, a significant concern is to keep the anchor well below the mudline so as not to be a hazard to the boats and barges above. This may require prior excavation of a pit in which to bury the anchor, covering it with small stone riprap.

While suction anchors are most effective in deep water (see chapter 22), they can be used in depths as shallow as 100 m. Drag-in plate anchors are used for taut anchors with a large lateral component, especially in shallower water.

Mooring buoys are especially valuable in deep water and where a number of moves of equipment or structures, on- and off-station, are required. They may be positioned by a single mooring line plus a vertical pendant to the seafloor, or by a group of three mooring lines. The mooring buoy should be designed so that the force is directly transferred from

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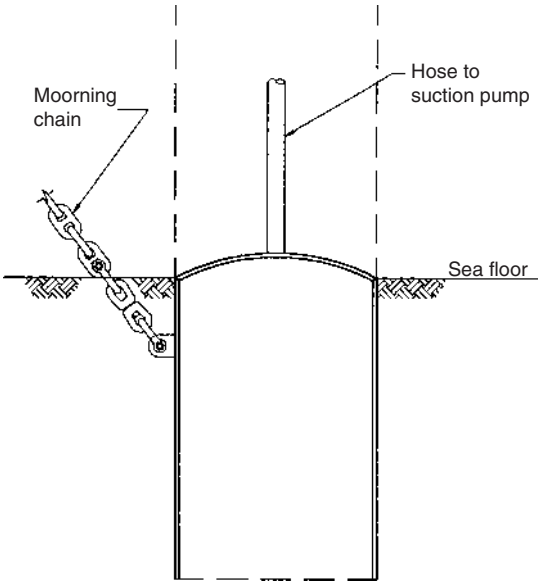

FIGURE 6.12
Suction anchor.

the anchor leg to the barge leg, for example, by running one line through a pipe sleeve. Otherwise, the maximum forces may damage the buoy. The buoy should be designed to resist the maximum hydrostatic head if it is pulled underwater. It should be foam filled. Many buoys have been sunk by rifle shots from fishermen. The Mini-OTEC riser buoy was not filled: it was sunk by a rifle shot and the entire riser was lost.

Although a mooring buoy may be held in approximate position by a vertical pendant to a clump anchor, to hold it in a fixed position requires three mooring lines. For offshore work, the effective pull that the buoy must resist is usually directional, within a spread of 30°–45°. Therefore, two legs of mooring lines are led out to anchors. The third leg is a short leg, either attached to a clump anchor more or less directly below the mooring buoy (it will never be directly below) leading with short scope toward the mooring position.

API Standard API-RP2A suggests that if a fully safe anchorage and mooring arrangement cannot be implemented at the site, due, for example, to constraints imposed by adjacent structures or operations, the orientation should be such that if the anchors slip, the derrick and supply barges will move away from the platform.

Mooring with a single mooring line, thus allowing the vessel or floating structure to weathervane around it in response to the currents and wind, requires that the mooring system have omnidirectional holding capability. A pile embedment anchor or a large clump anchor, both using swivels, are suitable, provided the swivel does not foul. Better yet is a mooring buoy with three or four legs to preset embedment anchors. Such a single mooring may allow the vessel to swing so far in yaw that it develops its highest forces. Thus some restraint on swing, such as an anchor dragging underfoot, will reduce the maximum force.

When the very large steel jacket for Platform Eureka was being towed on its launching barge to the site, it was temporarily moored with a single drag embedment anchor in San Francisco Bay. When the tide changed, the barge drifted back over the anchor and the line snagged it, breaking it loose, so that the barge was set adrift, dragging its anchor. The barge drifted dangerously close to the Richmond–San Rafael Bridge, then fortuitously the tide

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changed and the barge eventually went aground. Fortunately, there was no serious damage to either barge or jacket.

Moorings designed to hold structures for relatively long periods during construction should be designed on the following basis:

Exposure	Return Period of Severe Environmental Loading
Less than 2 weeks	10 years for season involved
2–8 weeks	20 years for season involved
Greater than 8 weeks	100 years for season involved

For very large structures, such as gravity-based platforms having long response periods, and where the moorings have resilience to absorb shock loads, the design should be based on the significant wave plus a 1-min sustained wind at the relevant heights and maximum current. Allowance may be made at sheltered inshore locations for reduced wind and waves and the changed geometry of the mooring due to excursion under load.

The maximum load in a new or used mooring chain with its associated shackles and fittings, including residual pretension, should not exceed 70% of the minimum breaking load, after allowance for corrosion and wear. Note that many manufacturers and classification societies quote an average breaking load rather than the minimum. Where wire line moorings are used, the maximum load should not exceed 60% of the guaranteed minimum breaking load.

The design holding capacity of any anchor, winch, or connection, multiplied by appropriate safety factors to account for gusts and dynamic amplification, should exceed the extreme storm loading on it. Note that the effect of long-period response motions in sway, surge, and especially yaw may impose significant increases in force. The attachment points to the structure should be designed to resist at least 1.25 times the nominal breaking load of the mooring, without damage to the structure.

For positioning a large offshore structure in the open sea, either several boats or a mooring system may be used. Experience has shown that the use of four or five boats equipped with variable-pitch propellers and bow thrusters can position an offshore structure in calm seas and wind conditions within 5 m or perhaps even somewhat less, depending on survey controls. However, in many cases the use of such a boat system will not be adequate or fully suitable. Even closer control may be achieved if the boats are anchored; then the structure's position is controlled by the towing engine.

An example is an offshore mooring system required for the mating of the deck on a concrete GBS substructure. Other examples include positioning a bottom-founded structure over a predrilled template, operations in shallow water, and those subjected to significant currents. An offshore mooring system is also required when the structure or vessel must remain on location for a significant period of time, subject to changes in the sea state and wind conditions (Figure 6.12 and Figure 6.13).

In shallow waters off the coast of Queensland, Australia, ten gravity-based caissons were required to be positioned within 0.5-m tolerance. This was achieved by use of preset mooring buoys (see Figure 6.14). The arrangement adopted, using a spring mooring buoy, gave upward spring action, with the buoy serving as a spring buoy during final setting, thus absorbing shock and dynamic surge loads. This system is also called an "inverted catenary."

For mooring of an offshore derrick barge working in coastal waters, two systems have been found necessary. On the San Francisco Southwest Ocean Outfall Project, a 4-m-diameter buried concrete pipe is being installed 7000 m into the sea in water 10–30 m deep. An

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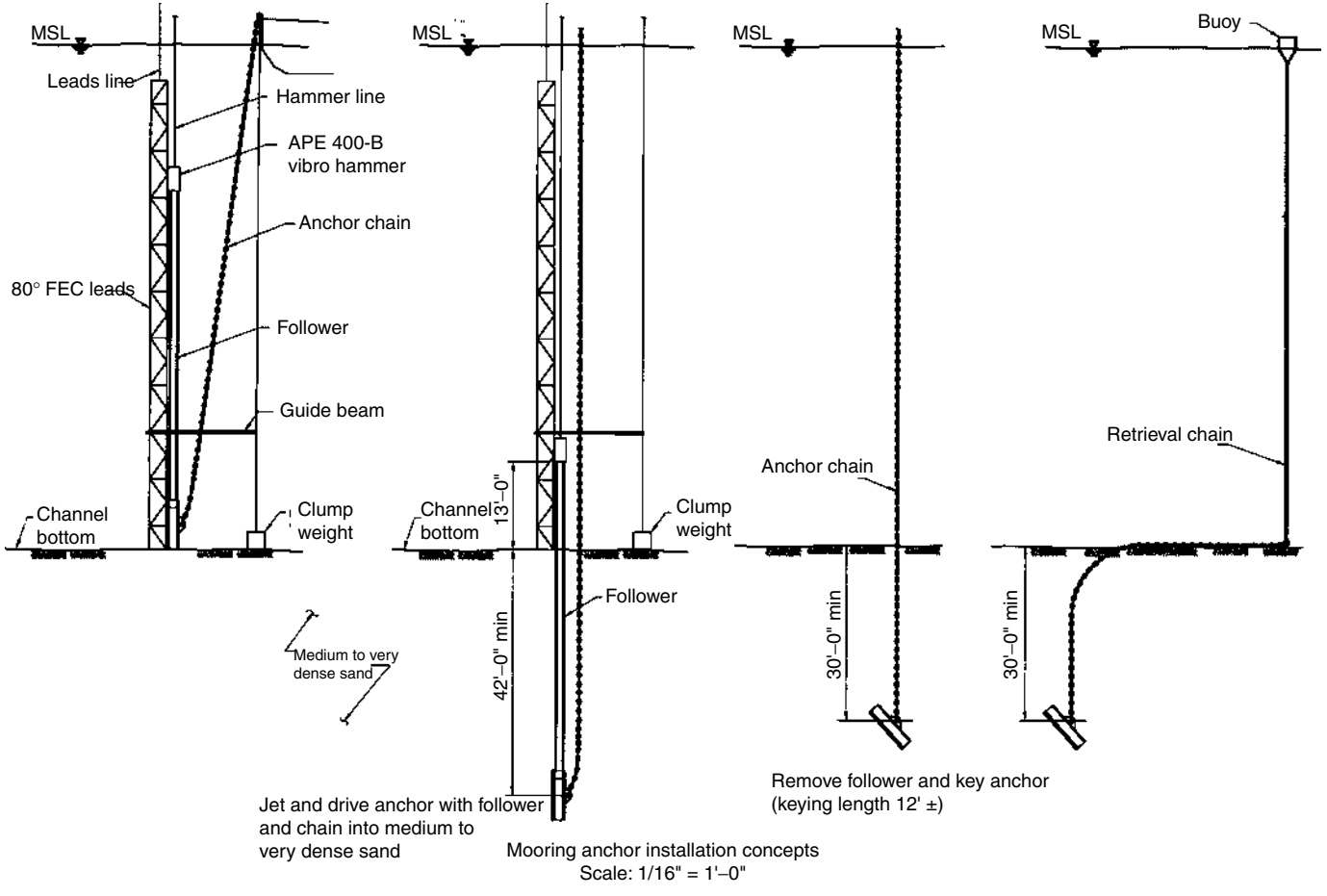

FIGURE 6.13
Installation of driven plate anchor. (Courtesy of TNC Construction.)

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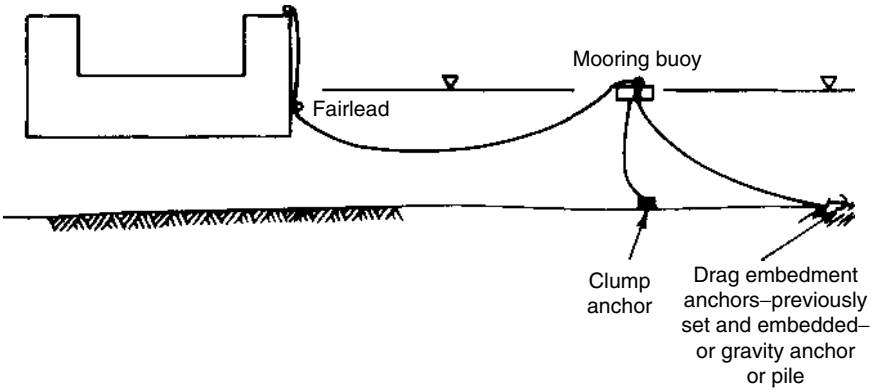

FIGURE 6.14
Mooring line arrangement for installing caisson.

offshore derrick barge was initially moored on a taut mooring in order to enable it to carry out operations requiring accuracy and control. An intense storm with unpredicted long-period waves created enormous surge forces that broke the taut mooring lines and drove the vessel onto the beach, severely damaging it. As noted earlier, long-period waves in shallow water develop elliptical particle orbits and create shorter and steeper waves with increased surge accelerations. When the offshore barge was returned to the site, the taut mooring system was again used for operations, but in addition a survival mooring system was installed (see Figure 6.14).

If the water had been deeper, the survival mooring would have been configured as a catenary, perhaps with a clump or chain in the bight to provide greater spring action (see Figure 6.15). However, in shallow water, the stretching out of the catenary permitted very little movement in surge. Hence, other means had to be taken. In this case, both a spring buoy, giving geometric travel, and a very long wire line, giving elastic stretch, were employed (see Figure 6.16 and Figure 6.17). The dynamic surge to be encountered

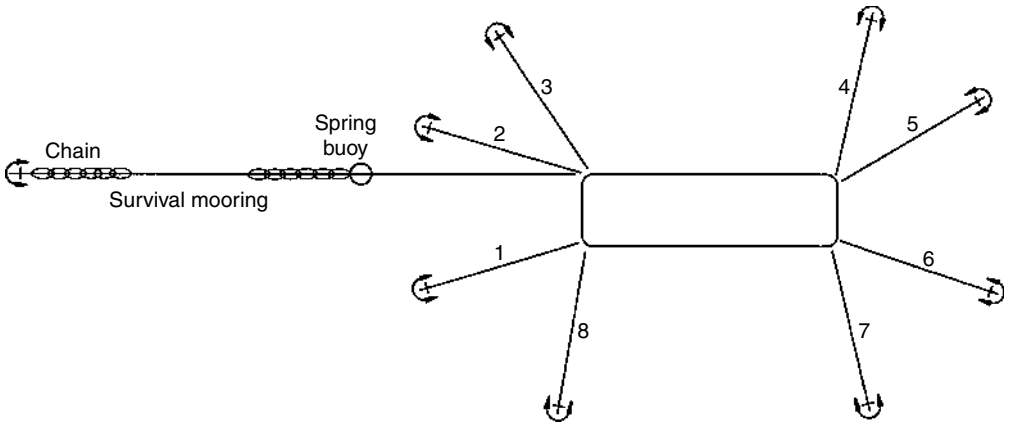

FIGURE 6.15
Taut mooring system for crane barge, with storm survival mooring.

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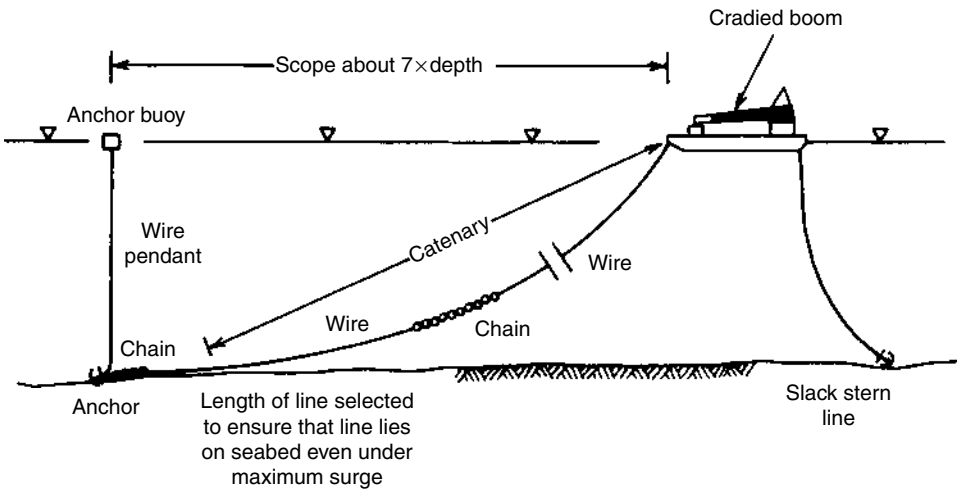

FIGURE 6.16
Storm survival mooring in relatively deep water. Note effect of catenary.

during a severe storm with an 18-s period, similar to that experienced in the previous catastrophe, was 8 m. This is single-amplitude displacement from the mean position, which means that the line will slack at the other end of the cycle. A force of 65% of the minimum guaranteed breaking strength was allowed as the maximum load under the design surge force imparted by the highest wave group in a 6-h storm.

The surge force is, of course, directly proportional to the stiffness. The lower the stiffness, the less the force. On the other hand, the surge excursion is little affected by the stiffness (see Figure 6.18). The stiffness of the system is the sum of the elastic and geometric stretch. Any attempt to restrain the dynamic surge excursion to less than its full value leads to very high forces, beyond the breaking strength of the line employed.

Consideration was given to the use of two survival lines. However, this would double the stiffness and double the force. Further, it would be extremely awkward to work with, especially since another piece of floating equipment (a dredge) would be moored only

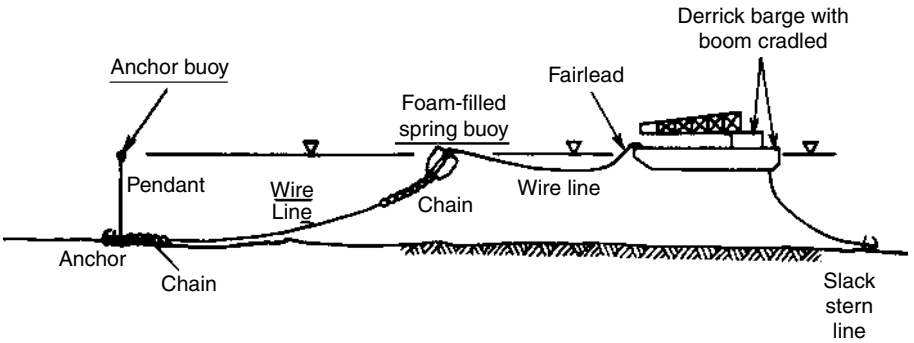

FIGURE 6.17
Storm survival mooring for shallow water.

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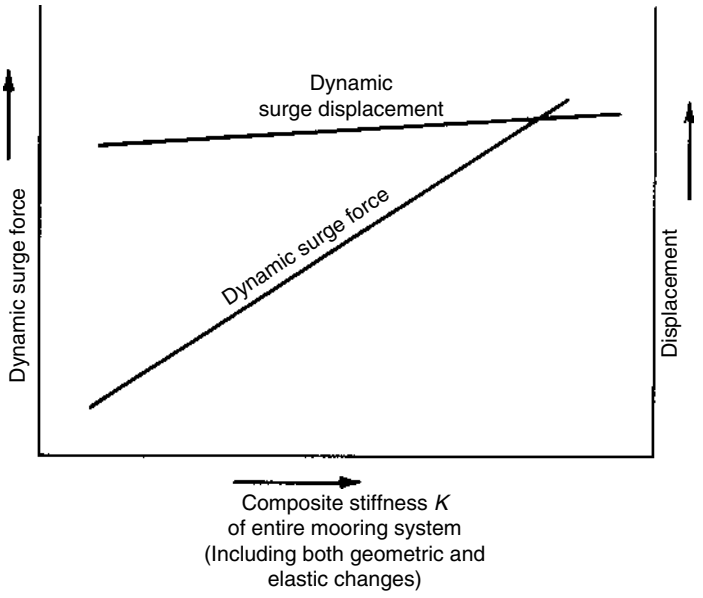

FIGURE 6.18
Effect of stiffness of mooring system on maximum forces and displacements.

1000 m to seaward. Further, due to short-crestedness, the force in one line would inherently be greater than half the total force.

Studies of the dynamic response showed that for the water depth (15–20 m) and barge length (130 m), there was little dynamic response for waves with less than 14-s period. As soon as the wave period increased, the dynamic surge jumped up dramatically. This was strictly a shallow-water effect (see Figure 6.19). Thus, the vessel must be oriented to ride into the longer-period swells, rather than approach them obliquely, due to any cross waves generated by local winds and wind-driven seas.

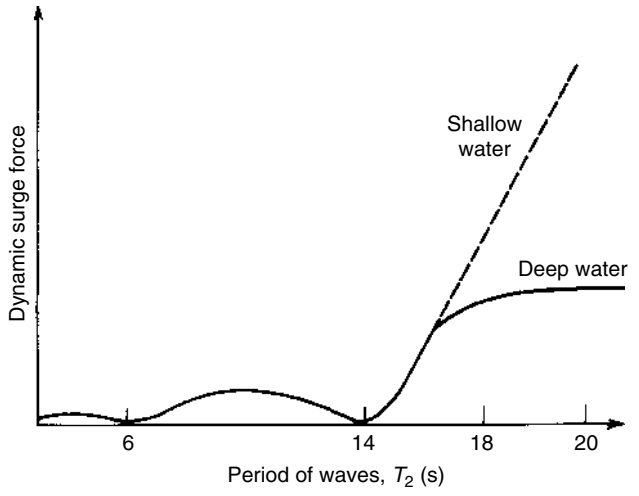

FIGURE 6.19
Dynamic surge as a function of wave period.

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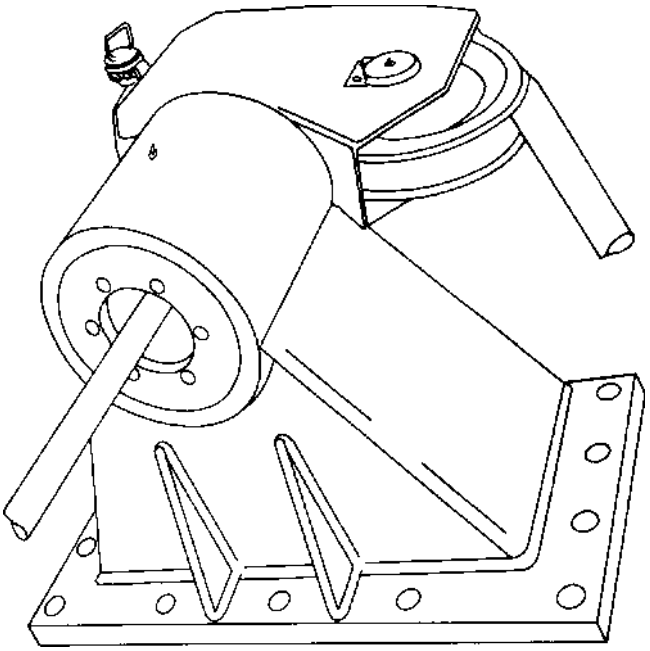

FIGURE 6.20
Fairlead.

The selection of proper lines and appurtenances for moorings is of great importance. Wire rope lines are generally employed: they are flexible, easily coiled on the drum of a winch, and can be changed in direction by means of fairleads and sheave. However, wire rope is subject to abrasion from sand and corrosion in the spray zone, this latter being a serious matter when the mooring is semipermanent. Galvanized wire rope is sometimes used, although it lowers the strength by 5%. Bending over a sheave or fairlead increases the tension in the outer wires: lines usually break in the fairlead. The ratio of sheave to rope diameter should be in excess of 20. Fairleads must be well designed and properly lubricated in operation so that they will lead fair and not chafe the wire rope (see Figure 6.20).

Wire rope tends to become stiffer with use, and this in turn may raise the dynamic loads in the line. If the line becomes flattened over a sheave or if one or more wires break, its usefulness as a high-capacity mooring line is severely degraded. Chain is relatively free from abrasion and corrosion problems and does not stretch; thus its extendability to absorb dynamic loads must come from its catenary action, which in normal water depths is usually adequate. Synthetic fiber lines made from nylon, polypropylene, polyester, or Kevlar are used extensively where their properties of light weight, flexibility, and low modulus can be utilized to advantage. They are, of course, very susceptible to abrasion and even to fish bite (including shark attack). Encasing these lines in polyurethane may eliminate these problems. Fiber lines are also subject to fatigue and to a reduction in strength when wet. Polyester and aramid (Kevlar) ropes have been found superior to other fibers in the marine environment. Fiber lines, especially nylon, tend to store energy elastically, which may give rise to resonant surge. Carbon fiber lines are very strong and stiff and extremely light in weight, but very costly. Composite lines, using chain where weight is required, wire for length, and synthetic fiber rope for stretch, are becoming more common. Longer lengths of fiber rope are employed in deep water to reduce the weight.

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The preset moorings for the construction of the Cognac Platform in the Gulf of Mexico in 320 m of water depth are illustrative of a major installation carefully engineered to ensure that the operations could proceed without difficulty. A 12-point mooring system was adopted. Each of the 12 mooring buoys was built of steel, filled with 2 lb/ft.^{3} (30 kg/m^3) closed-cell urethane foam, and equipped with controllable light signals for guidance of the workboats so they would not run up on a taut line. The buoys were then anchored by 400 m of 2I in. (68 mm) chain, which led to pile anchors. The pile anchors were 30 m long, 0.75 m diameter, with 25 mm walls. They were jetted into place by a drilling vessel. From the buoys, two parts of 50 mm (2 in.) wire rope line led to the barge. Between the two derrick barges, four 112-mm-diameter (14-in.-circumference) nylon lines were run in order to maintain relative position yet absorb dynamic loads arising from differential sway and yaw of the barges (see Figure 11.34 and Figure 11.35).

Dynamic positioning "thrusters" are being increasingly employed to maintain position of the construction vessels on some or all of the axes and thus fulfill part or all of the mooring requirements. These thrusters may be mounted on deck or within the hull. In their most-sophisticated arrangements, they are controlled by minicomputers and GPS and utilize variable-pitch propellers so that they run at constant speed. They are frequently employed in conjunction with mooring lines. The latter leads in the directions for which the greatest forces (usually longitudinal) are required, while the thrusters maintain transverse position.

6.3 Handling Heavy Loads at Sea

6.3.1 General

The installation of marine structures usually includes the lifting and setting of modules and other heavy loads on the platform. Such lifts may weigh from 2000 up to 4000 tn. and more (see Figure 6.21). Loads up to 13,000 tn. have been set by derrick barges with two

FIGURE 6.21
Hooking high capacity slings onto hook of heavy lift crane barge.

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cranes. The hammerhead crane Svanen has been successfully employed to erect piers, shafts, and girders up to 8000 tn. for very long bridges in Denmark, Sweden, and eastern Canada. For the 24,000-tn. prefabricated piers of the Oosterschelde Storm Surge Barrier, 12,000 tn. was supplied by buoyancy and 12,000 tn. by a catamaran lift barge.

Installation involves motion and hence, dynamic loading and impact effects. API RP2A, Section 2.4, "Installation Forces," recommends specific precautions to ensure safety in the handling of such loads. The DNV Rules, appendix H-1, "Lifting," specifies procedures and rules to ensure safe lifting of heavy loads at sea.

When lifting a heavy load, there are both static and dynamic forces to consider. The static forces include the actual load itself, which, if not weighed, must be computed to include the design weight, plus adequate allowances for overtolerance plate thickness, weld material, padeyes, and any supplies stored within. Static lifting loads must also include the slings, spreader beams, and shackles.

The author once investigated a critical lift that was at the limit of rated capacity of the 500-tn. crane barge. A careful physical inspection revealed that over 50 tn. additional weight, comprised of tools and supplies, had been stored aboard by the drilling crews. Worse, many of these, including an acetylene bottle, were loose, not having been properly secured.

The dynamic forces are those due to acceleration, first as the load line lifts while the load, still resting on the barge, is starting the down-heave cycle. Later, both horizontal and vertical accelerations are imposed during swing. Lifting forces on the padeyes and the structural members of the load to which they are secured have both vertical and horizontal components.

Many modules are designed to withstand the vertical quasi-static forces imposed in lifting in the fabrication yard, where bridge cranes or skids may be employed. At sea, however, the lead of the slings is usually inclined in two planes (see Figure 6.22). Although the padeyes themselves are usually adequately designed for vertical and horizontal loads, the structure to which the padeye connects must also be able to accept and transmit the total vertical and horizontal forces back into the structure. Modules fabricated in Houston

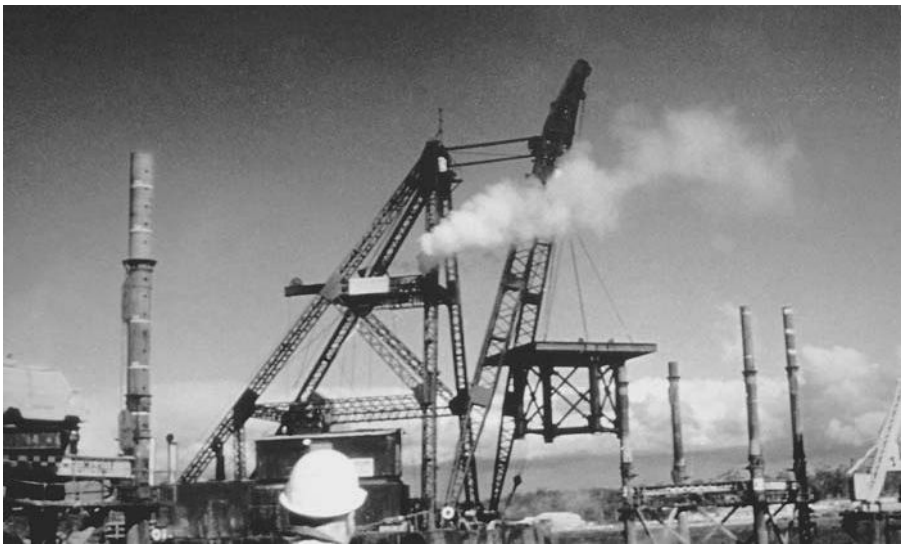

FIGURE 6.22
Lifting deck section.

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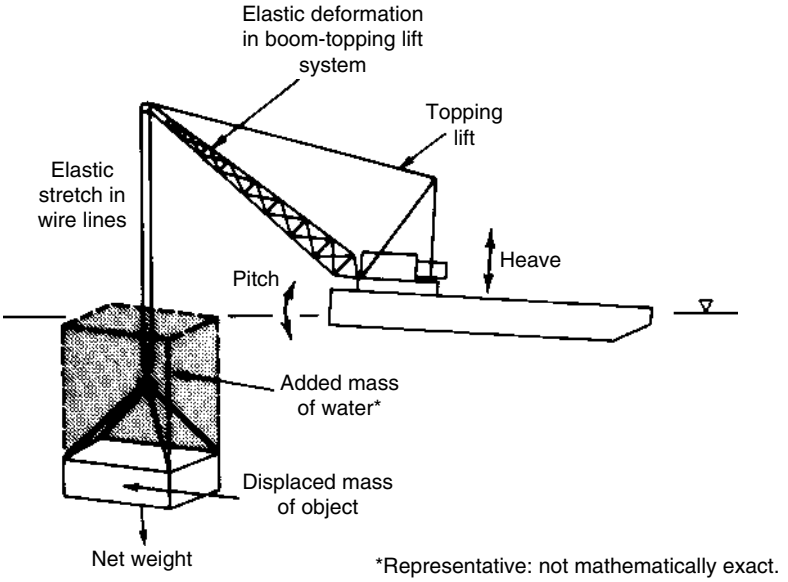

FIGURE 6.23
Dynamics of lowering load underwater.

or Singapore are initially lifted in warm weather. When later lifted at the site, which may be the Bering Sea, cold weather impact properties become important.

Vertical forces on lifting can include the favorable effects of buoyancy where applicable; however, fully or partially submerged structures may pick up an added hydrodynamic mass component. This latter may be a very high factor when the submerged surface is horizontal (see Figure 6.23). For example, a proposal was made by one offshore contractor to lower a 50,000-tn. boxlike unit to the seafloor of the North Sea. He proposed to utilize the buoyancy of the box to reduce the net weight to a few hundred tons, well within the rated capacity of the derrick barge. What he failed to recognize was that due to added mass effects and inertia, the rectangular boxlike unit would react almost not at all to a 6- to 7-s wave, whereas the derrick barge would have significant pitch and heave during that same period. While the stretch of the lines would have accommodated some of the dynamic effects, a detailed analysis showed that the derrick booms would have been seriously overloaded.

Instrumentation is now available to enable control of the dynamic aspects of lifting. These consist of sensors on the crane barge, on the crane boom, and on the barge or boat from which the module or other lift is being lifted. Typically, mini- or microcomputers then give readouts of load on hook, out-reach (radius), hook height, wave height, wave period, derating of crane capacity for sea state, hook speed, net load on deck of crane barge and effect on stability, crane hook height, off-lead (distance between load and fixed structure), automatic level luffing, and warning as to turns remaining on winch drum. Other programs are available to determine optimum heading of crane barge to minimize boom tip motion and hence, the dynamic increment of load during the operation.

An example of a heavy lift was the 5400-tn. deck of the Esmond topsides. It was carefully engineered for setting with Heerema's derrick barge Balder, with its two cranes rated at 2700 and 3000 tn. Positioning was achieved using conventional taut mooring lines, along with a computerized ballasting system for the crane barge. The entire operation took only 1 h.

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Another example was the 10,700-tn. integrated deck for the Britannia platform in the North Sea. This was set by the twin-crane barge Thialf, which has a nominal capacity of 12,000 tn. The barge is 200×88 m, and is moored by twelve 22.5 ton-anchors, using 80-mm high-strength steel lines, plus six dynamic azimuth thrusters.

A very heavy lift crane vessel, Rambo 12, was built up by joining two heavy-duty crane barges into a catamaran. The two barges were linked by a third barge arranged transversely between the bows of the two barges, and this enhanced both transversal and longitudinal stability. The two cranes, each of 2000 tn. capacity, were linked to make 3000-tn. lifts for the Tagus River Bridge in Portugal.

For the Second Severn Bridge in England, two large skid-mounted cranes were installed on a large jack-up barge. They lifted the 1500-tn. pier shells for the foundations.

The Svanen is a giant hammerhead crane that was specially built to place the pier shells, pier shafts, and girders of the Great Belt Western Bridge. It lifted 330 major precast segments weighing 7000 tn. each. Then, it was modified and transported by submersible ship to eastern Canada where it lifted and placed piers with weights up to 8000 tn. each, as well as pier shafts and girders of the Confederation Bridge linking New Brunswick and Prince Edward Island. Then, it came back across the Atlantic to participate in the Øresund Bridge joining Denmark and Sweden.

Catamaran lifting barges have been used for the transport and control of many submerged tubes, with net weights up to 1200 tn. Recently they have been proposed for lifts of 3500 tn. on lock and dam construction projects.

Lifting eyes are designed to transmit the load to the slings in the plane of the sling. As the structure swings, however, or the barge from which it is being picked sways, a side loading may be imposed. API RP2A recommends that a horizontal force equal to 5% of the static swing load be applied simultaneously with the static swing load. It is to be applied perpendicular to the padeye at the center of the pin hole.

When suspended, the lift will assume a position such that the center of gravity of the load and the centroid of all upward-acting forces are in static equilibrium. These relative positions should be taken into account in determining the inclination of the slings. The force in the sling is the resultant of the horizontal and vertical forces at the padeye, as computed for the most severe inclination of the sling. Due to swinging of the load while in the air, the load will not be uniformly distributed on all four slings. This non-uniform distribution must be considered in sizing of the slings and their fittings.

As the load is picked, and again as it is set, the position may vary from the above, due to the horizontal and vertical reactions from the deck of barge or platform, as well as those from tag lines and guides. The change in horizontal and vertical forces so occasioned must be considered in determining the forces and angles of application on padeyes and hooks. Figure 6.24 illustrates safe and unsafe use of hooks and shackles.

API RP2A recommends that for lifts to be made in the open sea, a minimum load factor of 2.0 should be applied to the calculated static loads. This must then be multiplied by the material factor of 2.0. Thus it is in general accord with wire rope rigging design, for which a factor of 4–5 is normally applied to the minimum guaranteed breaking strength to determine the safe calculated static load (without dynamic effects).

The above factors should also be applied to the padeyes and other internal members connecting directly to the padeyes. All other structural members transmitting lifting forces within the structure should be designed using a minimum load factor of 1.35. For lifting in cold weather (below +5°C), adequate Charpy impact valves should be verified.

OSHA requires a factor of safety of 5 on ultimate strength on all lifting gear (wire rope shackles and padeyes), whereas ANSI requires a factor of 3 on yield strength. The two codes are roughly equivalent. During load-out in sheltered harbors, where there are no

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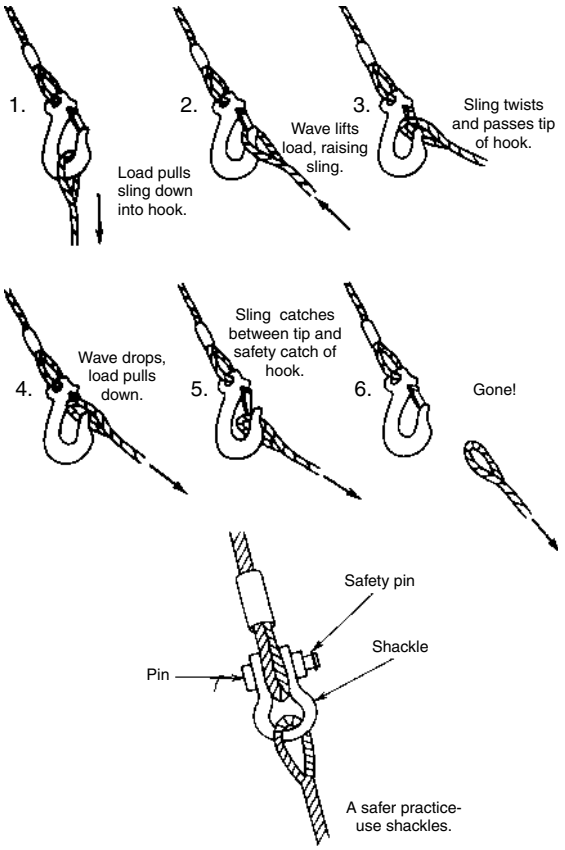

FIGURE 6.24
Potential hazard when lowering load into water. (Adapted from "The Principles of Safe Diving Practice," CIRIA, London, 1984.)

waves, the load factors can be reduced, but not less than 1.5 and 1.15, respectively. Note that such reduced factors may be technically in violation of local rules and thus require waivers.

Structural members, padeyes, and other attachments for lifting of lighter members in harbor and bridge construction are usually designed on the basis of allowable (elastic) stresses, using an approximate factor for impact (usually 2.0), with no increase in stress allowed because of short-term loading. The allowance for impact by a factor of 2.0 is also applicable to lifting inserts in concrete. In addition, all critical structural connections and primary members should be designed to have adequate ductility to ensure structural integrity during lifting even if temporary or local overloads occur. Special attention must be given to ensuring weld ductility and prevention of undercutting and adverse heat affects on the surrounding metal (HAZ). Low-hydrogen electrodes should be used. The design of padeyes requires special attention and detailing. Given the forces, including dynamic, and range of angles in both planes over which the forces may act, the padeye must transfer the load from the pin of the shackle into the structural frame (Figure 6.25).

Transverse welds perpendicular to the principal tension, where the member is subjected to impact, are prohibited by some national codes. If they are used, the details, welding procedures, and nondestructive testing (NDT) used to verify them must be such as to ensure full development of ultimate strength and ductility. Fillet welds are especially dangerous under impact tension, whereas properly made full-penetration welds may be safely employed. Cheek plates have been often used in the past, but they inherently require a transverse fillet weld.

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FIGURE 6.25
Setting Mooring Camel, Naval Base Bangor, Washington. (Courtesy of General Construction.)

Figure 6.26 shows an example of an incorrect (dangerous) design and fabrication of a padeye. In the figure, note the following potential failure areas:

1. The fillet weld at A is unable to develop the full tensile strength of the plate D.
2. The fillet welds at A and B are both transverse to the principal tensile stress under the impact of lifting.
3. The holes in the cheek plates do not line up with those in the parent plate.
4. The distance A–B is so short that any swing of the load produces sharp bending in the plate.
5. There is potential for weld undercut at A or B.

Wherever possible, the load from the padeye to the girder should be transmitted in shear. The use of cheek plates should be avoided whenever practical. The distance from girder to pin should be at least six plate thicknesses. Where cheek plates are necessary, they should be welded to the main load-carrying plates with bevel welds, sufficient for an even transfer of stresses, on the inside and with fillet welds on the outside. The bore of the cheek plates should be flush with the bore of the padeye, to achieve uniform bearing on the

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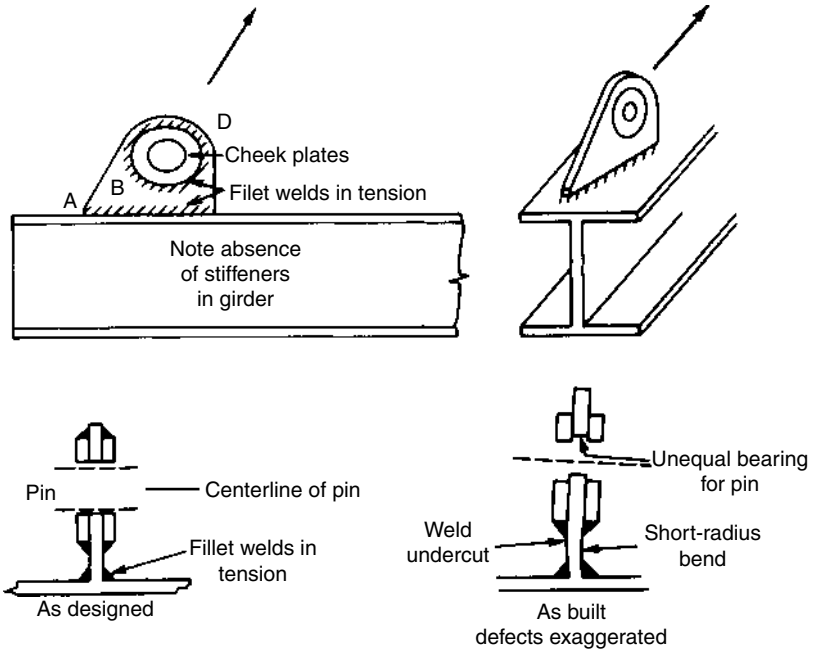

FIGURE 6.26
Incorrect and dangerous padeye details.

shackle pin. This may require reaming after welding. The direction of rolling of the parent steel plate should be determined, and this direction should then correspond with the direction of the sling.

If the connection between padeyes and structure cannot fully be transmitted by shear, then full-penetration welds can be employed, using the welding procedures prescribed for the primary structural members, with full NDT inspection. The structural plate to which the padeye is attached must have adequate through-thickness toughness to prevent laminar tearing. Figure 6.27 shows satisfactory and safe details.

API RP2A calls attention to the fact that fabrication tolerances and variations in sling lengths can redistribute the actual forces and cause significantly increased stresses in some members. The variation in sling length should not exceed $\pm 0.25\%$ of nominal sling length or 37 mm. The total variation from the longest to the shortest sling should not be greater than 0.5% of the sling length or 75 mm.

Where unusual deflections or particularly stiff structural systems are involved in the lifted load, a detailed analysis should be made to determine the redistribution of forces of both slings and structural members.

The horizontal force in the structural members connecting the padeyes can produce both high compression and bending due to minor eccentricities. It must be checked to ensure that buckling cannot occur. Similarly, spreader beams, if used, must be checked against buckling under compression on both axes, as well as for combined bending stresses between sling attachment points. Shackles and pins should be selected so that the manufacturer's rated working load, provided it includes a factor of safety of at least 3, is greater than the static sling load.

Lifted loads must be controlled against swinging by use of tag lines (see Figure 6.28). When lowering objects below the sea surface, special rigging is used to provide the

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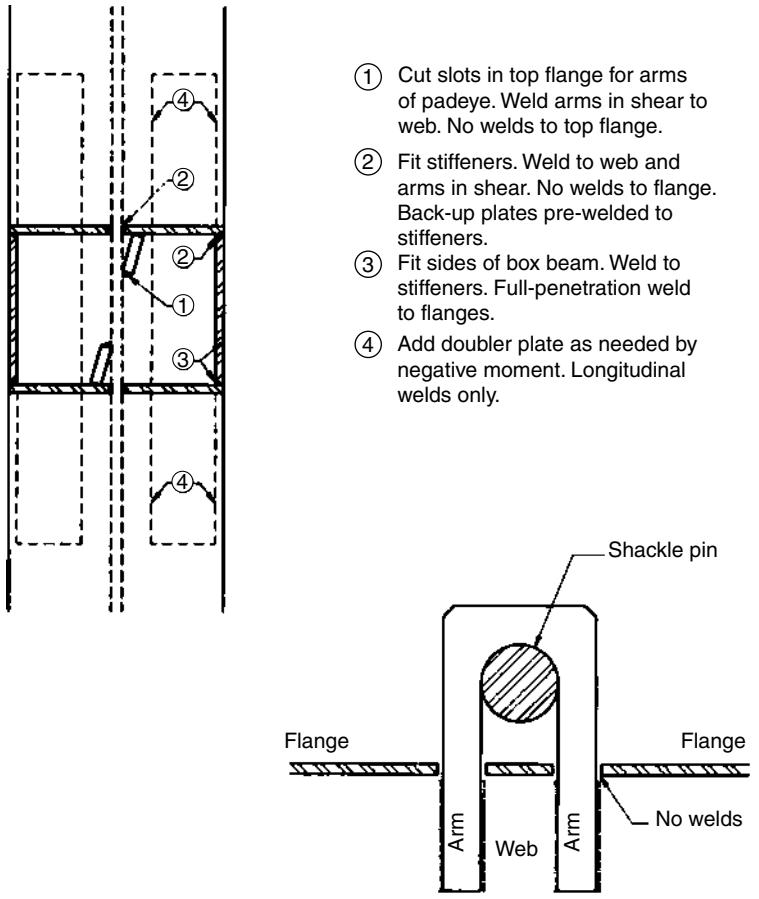

FIGURE 6.27
Suggested pad-eye details.

required length of line and to release after setting. Alternatively, hydraulically activated release hooks can be used.

Where a load is to be temporarily set on a platform deck for later skidding to its final location, the first site must be checked to ensure structural adequacy for the load, including an allowance for impact. Jacking points should be designed into the structural frame of the load to ensure that it can be safely supported at these points, which often are offset from the permanent supports. The seat should be especially checked against local buckling due to a combination of direct load and compression.

Figure 6.29 and Figure 6.30 illustrate the use of a catamaran barge arrangement and strand jacks to augment the buoyancy of the pier for transport and installation of the 20,000-tn. main pylon piers for the Øresund Bridge.

6.4 Personnel Transfer at Sea

The transfer of personnel from crew boat to offshore derrick barge or onto a fixed platform is a critical operation from the point of view of both safety and efficiency. In fact, the ability to move personnel on and off can become the limiting criterion for continued operations in

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FIGURE 6.28
Lifting jacket in Gulf of Mexico. Note inclination of slings in two planes, also the tag line to block to prevent it from swinging.

a rough sea. All too often this operation is overlooked in the planning phase. In some bridge work, shallow water near the bridge ends may prevent the use of crew boats, especially at low tide.

The boat in which the personnel are traveling to the offshore rig is responding to the wave action in all modes, heave, pitch, and roll being the most critical for the transfer operation.

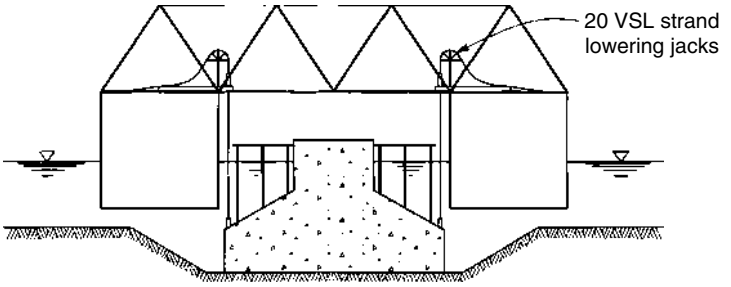

FIGURE 6.29
Self-buoyancy of large prefabricated pier is augmented by catamaran lifting barges. Øresund Bridge, Sweden.

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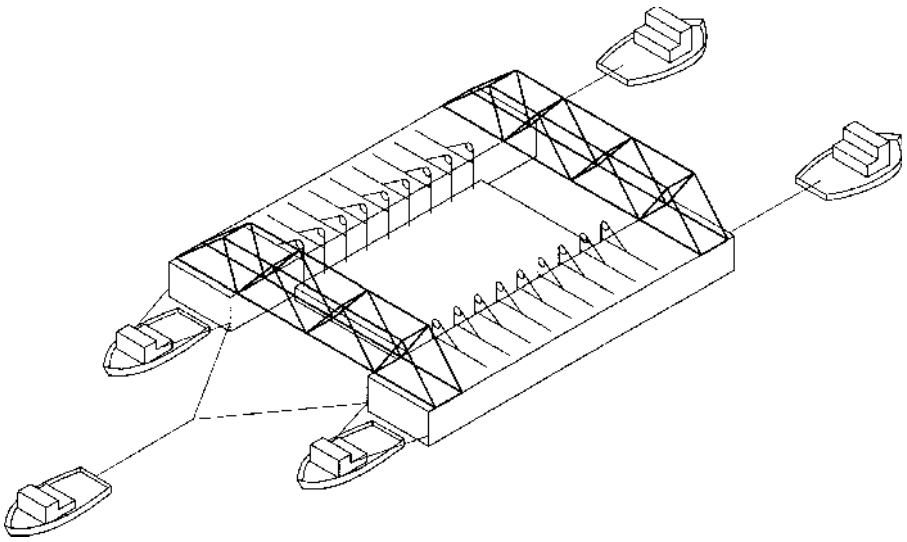

FIGURE 6.30
Arrangement of barges and tugs for deployment of 20,000-tn. main pylon piers. Øresund Bridge, Sweden. For clarity, the pier is not shown.

The use of fixed inclined "ladders" is not safe; they have been aptly called "widow makers." Articulated ladders have been developed, which essentially eliminate the roll-pitch motions; they utilize compensating pressure cylinders to maintain a steady attitude, subject only to differential heave. These are used on such major operations as the transfer from semisubmersible floaters to fixed platforms in the North Sea. These articulated ladders or gangways are available for smaller boats and operations; however, they are expensive and take time to rig.

More common, therefore, are other means of transfer. It must be also considered that there is usually a substantial height differential between boat deck and barge or platform. Properly fendered boats can come alongside a large derrick barge under favorable sea conditions, using the barge as a breakwater. In the case of head seas, they may come up to the stern. A notch should be fabricated so that the boat can push tightly against the barge and thus minimize differential pitch response.

The cargo net concept has been adopted from the World War II troop transport experiences. It is hung from a boom, so that the lower end is at sea level; when the boat moors, the net can be hauled into the boat. It is a relatively simple and safe operation for people to catch the top of the heave-pitch cycle and climb up the net. When they reach the boom, however, they face a dilemma. Somehow they are expected to scramble onto the boom and walk in to the deck. Even if the net is hung directly over the platform side, it is very difficult to scramble onto the deck unless handholds are provided. Hence, lifelines (handhold lines) need to be fitted above the boom (Figure 6.31).

Transfer back from platform to boat is more difficult. Assuming lifelines make it easy to get onto the net and climb down, below is a boat moving up and down in a 5- to 7-s period. There is an instant when the person must step completely off, at the top or just before the top of the boat's heave cycle. If a foot catches or the man tries to hang onto the net, the person may be jerked clear of the boat as it descends. For these reasons, the net should be placed in the well of the boat, about midships, rather than at the bow. Then relative motions will be minimized (see Figure 6.32).

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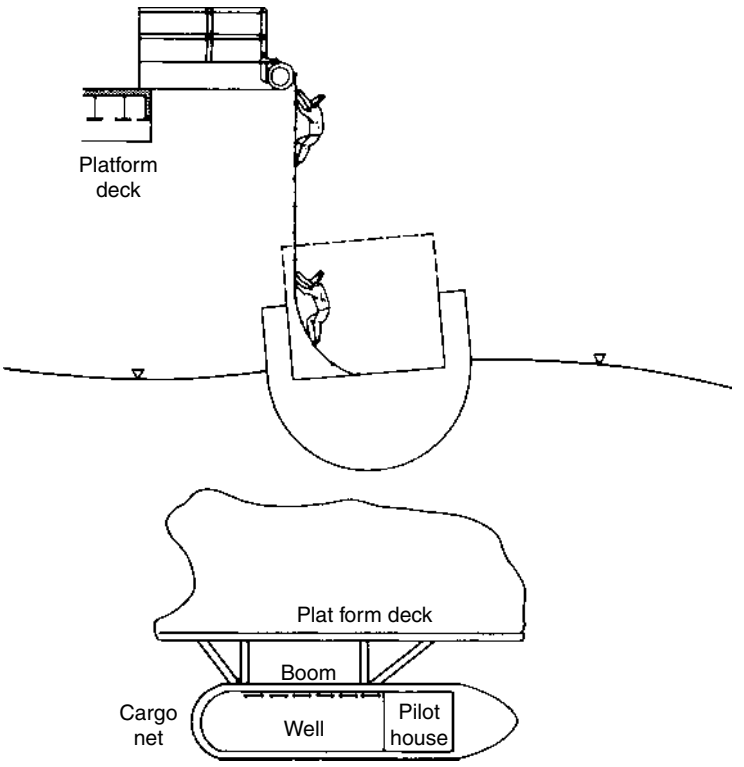

FIGURE 6.31
Arrangement of boom and cargo net for personnel transfer.

For more severe sea states, the Billy Pugh net is employed. This is a conical net hung by a rubber sling, with a safety wire rope attached. This can be lowered by a single line from a boom on the platform or derrick barge into the boat well and the line slacked to leave the net in the boat well despite heave displacements. The people climb on, placing their tools in the box in the middle. Catching the top of a heave cycle, the net is hoisted swiftly so that the net and the people are well clear by the time of the next heave. For transfer from platform to boat, the net is lowered down just clear of the heave peak; the operator waits for a relatively moderate amplitude phase. Then as the boat is at the top of the heave, the operator lowers rapidly. The net contacts the boat as it comes up on the next heave cycle; the operator slacks the hoist line so it runs free, and the net stays in the boat. This requires that the net be lowered on the brake, not in "power down." Many archaic safety laws contain a general prohibition against lowering on the brake; these are regulations pertinent to land operations but not offshore. In the open sea, attempting to lower under power may result in people being landed in the boat, and then jerked out on the next descending cycle about three seconds later.

Helicopters are today used for long-distance personnel transfer, especially where rough seas are frequently encountered. They are also employed increasingly for coastal operations such as offshore terminals and outfall sewers where substantial time is required for crew transport and tidal, wave, or surf conditions make the transfer difficult and dangerous.

Strict discipline is necessary when approaching and entering or exiting the helicopter while the blades are running. It is the tail rotor which is most lethal, although in a high

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FIGURE 6.32
Personnel transfer in seas like this can be done safely only with a Bill Pugh net. (Courtesy of Aker Maritime.)

wind, the main rotor can tip. All bystanders, personnel awaiting boarding, and guests must be kept well clear.

6.5 Underwater Intervention, Diving, Underwater Work Systems, Remote-Operated Vehicles (ROVs), and Manipulators

6.5.1 Diving

Manned intervention in underwater construction is one of the oldest forms of offshore activities, dating back at least as far as the ancient Romans, Phoenicians, and Indians, and probably to even older civilizations. As practiced in the offshore today, diving and the use of highly skilled technicians in the underwater environment are a very advanced technology, supported by extensive research and development in such disciplines as physiology, psychology, communications and control, power systems, and mechanical devices. Underwater tools and electronic-acoustic systems have greatly enhanced the effectiveness of divers (Table 6.2).

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TABLE 6.2

Manned Diving Systems Employed in Offshore Construction

Type of System	Working Depth (m)	Endurance
Scuba (air)	0–40	Very short; interrupted ascent
Scuba (air)	40–70	Very short; decompression required
Helmet (air)	0–70	Limited by diver's physical endurance
Helmet (helium/oxygen gas)	50–100	Limited by diver's physical endurance
Bounce (2 divers)	70–100	Few days
Bounce (4 divers)	70–100	Over 10 days
Saturation (4 divers)	70–300+	Unlimited
Manned submersible (without diver lock-out)	600–1500	Moderate
Manned submersible (with diver lock-out)	70–200	Unlimited
I-Atmosphere gear (e.g., JIM, WASP)	600	Limited by diver's physical endurance
I-Atmosphere diving bell	1000	Unlimited

Among the equipment available to the diver, the *Handbook of Underwater Tools* lists the following. These give an indication of the many tasks that divers may be called upon to perform.

1. Inspection and NDT
   a. Magnetic particle inspection equipment
   b. Ultrasonic equipment
   c. Eddy current/electromagnetic equipment
   d. Radiation monitors, trace leak detectors
   e. Cathodic protection monitoring equipment
   f. Range-level measuring and positioning equipment
   g. Metal detectors
   h. Thermometers
2. Photographic equipment
   a. Still cameras
   b. Cine (movie) cameras
   c. Video systems (TV cameras)
3. Underwater cleaning equipment
   a. Water jetting and grit blasting
   b. Portable brush-cleaning machines
   c. Self-propelled cleaning machines
4. Torquing and tensioning equipment
   a. Manual and hydraulic torque wrenches
   b. Torque multipliers
   c. Stud tensioners

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   d. Extensometers
   e. Flange pulling–splitting tools
5. Lifting equipment and holdfasts
   a. Lifting-inflatable bags
   b. Gas generators
   c. Lifting–pulling machines
   d. Magnetic handles and suction pads
6. General underwater equipment
   a. Wet welding habitats and equipment
   b. Underwater machining tools
   c. Chipping hammers
   d. Cutoff saws
   e. Grinders
   f. Drills
   g. Impact wrenches
   h. Hydraulic wire cutters, cable crimpers, spreaders
   i. Hydraulic fracture-initiators and breakers
   j. Power-actuated fasteners, cutters
   k. Pressure intensifiers
   l. Grouting and resin injectors and dispensers
   m. Underwater painting machines
   n. Jet pump dredges, airlifts, and ejectors
   o. Subsea marking systems
   p. Abrasive and mechanical cutting equipment
7. Subsea power packs
8. Diver-held location devices
   a. Cable tracking system
9. Explosive devices
   a. Pipe, chain and, casing cutters
   b. Perforators
   c. Shaped charges
   d. Underwater rock drills
10. Underwater lighting
11. Chain blocks
12. Jet burning equipment—thermic lancers
13. Diver-operated geotechnical tools
   a. Impact corer
   b. Miniature standard penetration test tool
   c. Vane shear
   d. Rock classifier
   e. Jet probe
   f. Vacuum corer

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The properties of the underwater physical environment that affect a diver's ability to perform work include the following:

1. *Pressure*. The increase of pressure with depth affects human sensory and reasoning powers and causes gases to be dissolved into the bloodstream.
2. *Temperature*. Low temperatures cause serious loss of body heat. This is especially critical in deep diving and when diving in Arctic or sub-Arctic areas.
3. *Turbidity*. Especially near the bottom and around structures, turbidity impairs vision. The operations of the diver and the diver's equipment may stir up the sediments and cause a turbidity "cloud."
4. *Currents*. Currents tend to sweep the diver away from location and to make the diver's position control more difficult.
5. *Refraction phenomena*. Underwater refraction of light and acoustic waves is different from those in air.
6. *Waves*. Waves endanger the descent and ascent of the diver through the sea–air interface.
7. *Marine growth*. These shield surfaces and joints from inspection and can rip a diver's suit.
8. *Buoyancy*. Since the diver's underwater weight is only marginally negative, the diver cannot exert a significant thrust from the body.

Physiological and psychological effects of importance are the following:

1. Disorientation occurs due to inability to differentiate the direction of a sound source and the loss of reference planes.
2. Hearing modes are changed.
3. Sight capability is reduced under the stress environment.
4. Speaking intelligibility is greatly reduced; the familiar ducklike sounds of a diver working on a helium/oxygen mixture are well known. Greater depth produces greater distortion.
5. Physical fatigue occurs. The work effort at depths is greatly increased.
6. The pressure causes greater absorption of gases by the bloodstream.
7. Miscellaneous: Diver safety can be affected adversely by noise, electrical shock, debris and fishing lines, and explosive concussion, as well as accidents with lifting and rigging, water jetting at high pressure, and underwater cutting and welding. Pulses and vibrations, as from nearby operating equipment, are alleged to cause distress.

Diving for inspection purposes may employ a self-contained oxygen supply. For construction purposes, an umbilical cord is generally employed. If an air mixture, oxygen plus nitrogen, is used, the blood will absorb nitrogen under pressurization. Under subsequent decompression, if carried out too rapidly, the nitrogen will form bubbles in the bloodstream, leading to serious injury and even death. This is the well-known nitrogen narcosis or "bends." The use of carefully developed gas mixtures such as helium/oxygen has enabled safe diving to be carried out to depths of 150 m, and even to 300 m. Other gas mixtures have been developed and used to permit operations in even deeper waters. Too rapid a rate of ascent, even from a shallow dive, will lead to the bends.

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Decompression rate schedules have therefore been developed to ensure that the blood's natural balance of gases can be restored without the formation of bubbles. Foremost among these and widely adopted by regulatory bodies throughout the world are the U.S. Navy's Standard Tables and "Decompression Tables for Standard, Exceptional, and Extreme Exposure" and "Single and Repetitive Dives, Helium/Oxygen Tables." A decompression tank must always be available on or near the site, depending on the depth of dive.

Saturation diving systems and "bounce techniques" have been developed that take advantage of the physiological fact that after the blood becomes largely saturated with an inert gas such as helium, then further absorption proceeds very slowly. This enables a person to stay at depths for long periods of time, working short periods, resting without decompression in a habitat or chamber, then returning for another stint of work. Using deck decompression chambers, personnel transfer capsules, and a habitat, the saturation diving system can support several teams of divers for long periods of time. With bounce techniques, the divers periodically descend to work and then return to partial decompression only.

The time required to perform work underwater increases with depth, although not as rapidly as one might anticipate. For example, a number of tests have shown that specific items of work required 20% longer when at shallow depths and 50% longer at deep depths, as compared with the time required for performance in the air. The greatest components of time are the times for descent, ascent, and decompression. The rate of descent is usually important only for deeper dives. Decompression tanks are carried on deck of the diving support vessel to enable saturation diving to be employed and to repressurize a diver who develops symptoms of the bends.

A major limitation when using divers for underwater work is that of developing a reactive force, "getting a foothold," so that the diver may exert a force. Because the diver is in a state of near-neutral buoyancy, the diver is like a person in space; the diver shoving against a pipe merely moves the diver away. When a diver can plant the feet firmly on the bottom or against a structure, the diver can exert 100–300 N (22–66 lb) force in push or pull. Various means of attachment and hydraulic tools have therefore been developed to assist the diver in exerting a force.

There are many forms of diving, based on the equipment used. These vary from the scuba gear to the new diving suits that protect the diver from injury and lightweight helmets with improved vision capability. Hard hats and full suits are required for underwater construction work, where the diver must be protected from abrasion and puncture as well as from debris. Wet suits are most commonly used. For deep diving and diving in the Arctic or sub-Arctic, the suits are heated, usually by warm water circulation. Free-swimming (scuba) divers can perform inspection tasks; they are limited in communication and, of course, have no power supply for tools. The concern is for potential injury to the lightly protected scuba diver from jagged protrusions. Conversely, the scuba diver is very mobile and can quickly report on conditions, especially in areas of good visibility. Scuba diving is severely limited in a current greater than 0.4 knots (0.2 m/s).

Tethered divers can have warm water circulation for their suits, hard wire or fiber optic links for communication, and hydraulic power. To enhance the diver's capabilities for work, diving chambers or bells may be used. These give the diver more freedom from encumbrances but, of course, limit mobility. Diving bells, operating at atmospheric pressure, enable inspection and work by engineers who are not qualified divers. A complete pressure-resistant diving suit has been developed, which enables the diver to stay at atmospheric pressure. A refined version enables a worker to descend to 600 m depth. These are very bulky which may cause problems of entry and maneuverability in confined spaces (see Figure 6.33).

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FIGURE 6.33
One atmosphere diving suit.

The current trend in manned diving systems appears to be toward diving bells and similar systems to enable better and safer control. However, there are many tasks requiring entry into congested spaces and among congested bracing, which can only be done by a diver.

Diver communications is an area in which there have been major advances in recent years. In addition to hard-wired and fiber optic systems, modulated sonar-frequency carrier systems give ranges of 150–500 m and single-sideband communications can give a 1000–1500 m range.

One of the most serious limitations of diver work is inability to determine one's position. This is due to lack of visibility, to disorientation, and to the lack of reference points. Consider an underwater concrete caisson that to the diver presents an endless wall 60 m high. Markings are required. Large orange epoxy numerals have been painted at about 10-m spacings to assist divers in determining their location. Wire guidelines have been stretched to serve as guides for divers and to hold their position in a current. Such wire guidelines are especially important if a diver must enter under or through a structure—for example, into the middle of a braced jacket—or underneath a gravity-based structure while it is still in the floating mode during construction, or into an outfall sewer. One problem of divers is that of marking locations so they can return. In addition to a wire line, acoustic pinger locators are often used. The diver may use a handheld sonar to enable a search for a pipeline or dropped object.

To clean off marine growth for inspection, high-pressure water jets as well as hydraulically operated rotary brushes have been developed. Both sonic and tethered guideline

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systems are employed to guide divers working under ice, so that they may safely return to the entry/exit hole.

Hard hat divers require extra weight and a taut wire line tether to descend in a current greater than 1.5 knots (0.8 m/s). Taut wire line tethers increase both safety and efficiency, enabling the diver to descend to a specific location, with both hands free to work. Rings and/or raised markers preaffixed to the structure enable the diver to move progressively and with proper orientation, even when turbidity completely prevents vision.

When bolts or turnbuckles or other heavy objects are to be connected underwater, they should be pre-attached to one structural element. Then the diver can raise them (or direct their raising by a line from the surface) and make the connection. Raising is safer than lowering, since lowering may accidentally hit the diver. For entry into pipelines or under structures or through bracing, a second diver should be used to tend the first diver's lines and the umbilical cord.

For especially complex tasks, a full-scale mock-up can be constructed on shore, so that divers may not only practice but, by blanking out their vision, learn the feel of every element. This was carried out very successfully for installing the complex temperature control fixtures on the upstream face of Shasta Dam at a depth of 100 m.

A major problem in diver communications is that of transmitting the information to the surface in a form that is fully understood. The distorted "ducklike" talk of a diver on helium/oxygen gas has already been mentioned. In addition to transmission of voice communications and data transmission by fiber optics, video has become a major means. The ability for an observer on deck to see what the diver sees in real time represents a tremendous advance. Underwater photography can, of course, be used for recording more clearly specific objects such as a welded joint. Both video and photography require a powerful light source; much recent development has been directed toward appropriate frequencies to reduce scattering and incident angle refraction distortion. In areas of limited visibility, diver-held videos have proved more successful than ROVs.

Many tools and procedures have been developed to enable divers to work effectively underwater. Among these are the following:

1. Wet welding techniques, using a high-velocity jet of inert gas to create a water-free zone. Wet welding can be used on low-carbon steels to as deep as 70 m. Although its qualities are strongly influenced by depth, hence pressure, satisfactory welds have recently been completed at 110 m.
2. Dry welding, using a habitat and employing gas metal and gas–tungsten arc techniques. Hyperbaric welding has been carried out to depths of over 1000 m.
3. Underwater cutting using the electric arc method. With a skilled diver, steel can be cut almost as rapidly as above water. Arc-flame methods can be used to depths up to 2000 m. At greater depths, potential problems exist as the density of water and gas tend to equalize due to the high pressure. Arc-Air has developed an electrode which will cut both concrete and steel.
4. Mechanical casing cutters and abrasive jet cutters.
5. A wide variety of hydraulically driven velocity power (explosively driven) tools have been developed in recent years, many of them by the Naval Civil Engineering Laboratory at Port Hueneme, California. These include actuators, impact wrenches, rotary brushes, rock drills, thermic lancers capable of cutting steel and concrete and even rock, explosive (power-actuated) pin-driving tools, grout dispensers (for epoxy injection), and NDT inspection devices. In addition to the use of conventional hydraulic fluids, seawater power supply systems have been developed.

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Diving and divers are really a transportation system to enable work to be carried out in an otherwise inaccessible environment. Because of the inherent limitations that still exist and because of the high costs, experienced constructors make every effort to eliminate or reduce diving requirements. For those still required, extensive planning is devoted to the diver's support, transfer, and work conditions to maximize his safety and efficiency.

An example of an advanced diving system for use in servicing the subsea well completion template of a floating production system is the Balmoral Field development. This system consists of the following:

1. A diving bell, running on tensioned guide wires
2. Guide rails and traveling cage to support the bell through the sea–air interface
3. Main wire winch for lowering and hoisting
4. Umbilical winch
5. Gas control panels
6. Two decompression chambers
7. Hyberbaric rescue vessel

6.5.2 Remote-Operated Vehicles (ROVs)

To supplement divers and extend underwater inspection, monitoring, and surveying capabilities, a number of underwater vehicles have been developed. These include the work submarine or manned submersible, which can survey seafloor sites with remarkable accuracy, using gyrocompasses, acoustic Doppler navigators, inertial guidance, side-scan sonar, and acoustic monitoring by surface vessels. Work submarines have been used to map boulders on the seafloor of the North Sea. They have been able to locate and identify these far more accurately than has been possible with acoustic means from the surface alone. A bottom-crawling vehicle was used to verify the seafloor grading for the Oosterschelde Storm Surge Barrier.

For underwater inspection and construction ROVs have been developed to an advanced state. These vehicles are tethered with an umbilical, which not only powers them but directs and controls them. The umbilical then transmits information back to the surface. This can include data from sensors and video images. ROVs can be equipped with fiber optics and advanced electronic sensors and data collection storage and transmission systems. They also can be fitted with manipulators, jet cleaning devices, and sampling devices. The development of ever more advanced ROVs is continuing at a rapid pace, spurred by the deep-water operations of the offshore petroleum industry well beyond diver capabilities. While the military concentrates on autonomous free-swimming vehicles controlled by built-in computer "memories" and directions, industry and especially constructors have generally used tethered systems with umbilicals, enabling much more diversified and specialized activities as well as more accurate maneuverability in close proximity to structures (see Figure 6.34). They can be run into and through pipelines to make surveys of conditions and to inspect joints. They can also determine loss of section due to corrosion–erosion. Recently, autonomous ROVs have been increasingly used in ultra-deep water, especially for pipeline surveys.

An ROV may embody the following equipment and capabilities:

Strobe light;

High-resolution TV video;

Low-light-level black-and-white photography;

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FIGURE 6.34
Remote-operated vehicle (ROV) ready for deployment for bridge pier survey. (Courtesy of Kongberg–Simrad–Mesotech.)

Stereophotogrammetry;
Multibeam SWATH and side-scan sonar, acoustic imaging;
Manipulators for turning bolts and nuts and for grasping;
Inertial guidance, acoustic navigation;
Corrosion potential probes;
Cleaning and grinding tools;
Pinger dropper;
Installation of fittings;
Buoyancy modules;
Attachment of lines and object retrieval;
Wire rope cutter;
Hydraulically operated tools such as cutters, drills, and jacks;
Thrusters.

ROVs are invaluable for deep-water installations since they are not depth limited. They are capable of connecting flexible pipelines, bundles, and umbilicals, in water depths up to 2500 m and deeper.

In one system, Flexconnect, the ROV and its permanently mounted skid are joined in a cassette bay, shaped like a funnel. The ROV locks to the cassette bay, inserting a rope anchor. The ROV and skid then retract and latch onto the flow line, which has been pre-installed on the adjacent seafloor. The ROV now takes in the rope anchor, pulling the flow line into the cassette where it is locked by pins.

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ROVs are especially well suited for survey and inspection of structures during installation and while in service. By a combination of inertial guidance and acoustic position-finding devices, their position in three-dimensional space may be accurately determined. Through onboard sensors, they may then determine their relative position to the structural element.

As indicated above, the ROV is a vehicle which can be specially fitted to a wide range of specific tasks involving the use of tools. Where the tool or equipment is too large and heavy to be carried on board, the ROV may place and connect guidelines which can then be tensioned to control the descent and placement of the large device.

Because of the efficiency of ROVs for underwater intervention, their use is growing rapidly. They are gradually taking over many of the functions, such as inspection of pipelines, formerly performed by divers.

6.5.3 Manipulators

Another approach to carrying out operations underwater is utilization of special-purpose devices, which are guided to a specific location by wire lines or rails. For example, a pair of tensioned guidelines can be used to guide a manipulator in both horizontal position and orientation. Some devices are very simple, such as the hydraulically actuated cutters, which slide down a wire line; others are complex such as a set of drills, which ride down a rail. At pre-set locations, they drill into concrete, retract, place a rock bolt, and inject grout. In another application, a buoyed line may be released by acoustic signal to rise up from an underwater template. A manipulator is then guided down the taut line to seat on horizontal rails. It then runs on the rails to predetermined locations, extends a hydraulic socket wrench to engage and turn a bolt. The torque is fed back to the surface control vessel.

Many manipulators have integral computers to adjust their operations to the actual requirements encountered—for example, to limit the power applied, or to adjust a valve opening. Manipulators and instrumentation can also be employed in pipelines to monitor wall thicknesses, to expand a section hydraulically, to trip valves open and close ports, and to cut the pipe. Because of their successful operation in deep water, manipulators are now being utilized in shallow water, i.e., rivers and harbors, to replace divers. The driving motivation has been safety, but additional benefits have been reduction in exposure time for operations and savings in overall costs.

6.6 Underwater Concreting and Grouting

6.6.1 General

Underwater concrete and grout play an important role in the construction of offshore structures. Historically, underwater concrete has been used to seal the bottom of cofferdams so as to prevent water and soil piping. Underwater concrete may be placed in forms to serve as the structure itself; most often it becomes the footing block for the structures. It may be used to tie together various elements in composite action—for example, to tie the piling to the footing and thence to the upper portions of the structure. Underwater concrete may also be used to fill pre-excavated holes in the seafloor and to act as a leveling mat. Underwater concrete may also be used as solid ballast to add weight and lower the center of gravity. It may be used to fill under the base of a gravity platform to insure uniform bearing and provide shear transfer. Underwater concrete may be placed in piles or caissons to give added structural strength and to prevent buckling, or it may be placed

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in belled footings which have been drilled at the tip of the piles in order to increase axial compression and tension capacity.

Underwater grout is also used for many of the above purposes; hence, no rigorous distinctions should be made in classification. Grout is also used to bond piling to jacket legs, to cement well casings, to fill small spaces between elements to provide structural continuity, and to fill the voids in preplaced rock and aggregate.

6.6.2 Underwater Concrete Mixes

Underwater concrete should be proportioned to develop a plastic, highly workable, and cohesive mix, not subject to segregation.

For many structural purposes, the following mix is suitable:

Coarse aggregate: Gravel of 20 mm (3/4 in.) maximum size. Use 50%–55% of the total aggregate by weight. For congested areas, use 10 mm maximum size aggregate (pca gravel).

Fine aggregate: Sand, 45%–50% of the total aggregate by weight.

Cement: Type II ASTM, 350 kg/m^3 (600 Ib/yd^3).

Flyash: ASTM 616 Type N, F, or C: 60 kg/m^3 (100 Ib/yd^3).

Total cementitious materials: 350–475 Kq/m^3 (600#/cy–800#/cy)

Water: (w/cm), 0.37–0.42

Water-reducing admixture: WRA or HRWRA (super-plasticizer):

Retarding admixture: as required to give desired initial and final set.

Slump: about 200 mm.

Admixtures to reduce bleed and to provide viscosity.

Cementitious materials are typically a mixture of cement type II ASTM and fly ash, the latter being 20%–30%. Fly ash retards set.

Alternatively, blast furnace slag–cement can be employed in the proportions of 60%–70% BFS, the remainder type II ASTM cement. Blast furnace slag should be coarse ground with no added gypsum.

Viscosity and bleed may be controlled by the proportioning of the mix. Admixtures may be employed to give better control, especially for structural elements.

Silica fume in proportions of 4%–6% may be added. Mixing procedures and adequate time of mixing are essential to give adequate dispersal of densified silica fume. Fly ash should always be used with silica fume. Silica fume increases both early and long-term strength.

Anti-washout admixture (AWA) prevents cement washout, thus eliminating almost all laitance while being also self-leveling to a high degree. Both silica fume and AWA require the use of high-range water-reducing admixture (HRWRA).

Air entrainment was used in the past to aid workability but is no longer widely employed due to its variable effects under pressure.

Limestone powder has been used to reduce the cement requirements and also to provide a controllable low strength underwater concrete, for use, for example, in constructing a plug in a tunnel which must later be penetrated or removed.

HRWA must either contain a retarder or one must be added to prevent sudden loss of slump. This can be especially serious for underwater concrete.

This mix will develop compressive strengths in the range of 40 MPa at 28 days (5600–7000 psi cylinder strength). It will generally flow out on a slope of 6:1 to 8:1 and,

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if properly placed, should give minimal segregation and laitance. It is suitable for placement in voids as small as 100 mm in diameter and can be used for large caissons and bridge piers.

For higher strength and greater cohesiveness, silica fume may be added in the proportion of 5%–6% by weight of cement. However, in this case, it is necessary to use superplasticizer, either alone or in combination with conventional water-reducing admixture as well as fly ash. Adequate retarder must be added to ensure that the time of initial set is long enough (typically 6 h) so that premature stiffening will not occur. Where it is important to obtain a level surface and minimal or no laitance, antiwashout admixture (AWA) can be added.

The basic mix recommended above, however, will develop a fairly high heat of hydration (about 35°C above ambient) depending on the size of the placement, leading to thermal expansion and possible cracking during the subsequent cooling. Various methods of reducing the temperature rise are available. Their use is justified in special cases. The following is a list of individual means which have been employed (they should not necessarily be combined):

1. Select aggregates with a high thermal coefficient, requiring more heat per degree of temperature rise.
2. Use blast furnace slag/cement in the proportion 70:30 to reduce heat generation. Slag should be coarse-grind (<3800 cm^2/g), with no added gypsum.
3. Increase the percentage of pozzolan (fly ash), replacing a comparable portion of cement. Recent tests indicate the percentage may be increased to as much as 50% in fully submerged concrete and unreinforced concrete, 30% in reinforced concrete.
4. Use limestone powder to replace part of the cement.
5. Precool the aggregates by water spray-evaporation and use ice as the mixing water, or by liquid nitrogen.
6. Cool the mix by injection of liquid nitrogen.
7. Insulate or cover the transport conveyance.
8. Precool pump lines and tremie pipe with chilled water.
9. Subdivide the pour to reduce the size of individual blocks.

Since various admixtures, such as pozzolans, behave differently with different cements and admixtures, trial batches of several cubic meters should be made to ensure that the resultant mix is workable and possesses a high degree of cohesiveness, that is, does not tend to segregate (see Figure 6.35).

Rate of cooling may be minimized by insulation of the sides and top surface after placement, thus preventing cracking.

6.6.3 Placement of Tremie Concrete

Tremie concrete can best be carried out by gravity flow through a tremie pipe of 200–300 mm diameter. With gravity flow, the concrete will balance out at about mid-depth of water and the flow will be smooth. The other method of placement is by pump, all the way to the discharge point. Typically, those contractors use an initial plug of Styrofoam to start a pour but this collapses under hydrostatic pressure and is useless. It is also unsuitable for re-starting a pour.

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FIGURE 6.35
Trial batch of a concrete mix of proper consistency for underwater placement.

However, the main objection to use of a pump as a tremie is that it delivers the concrete in surges under pressure. These cause disturbance and wash out of cement. The discharge of the stinger often whips back and forth. Thus defects in the concrete and laitance are typically much greater.

If placing all the way by pump line stinger, there should be a valve at the top of the vertical stinger in order to prevent a vacuum from forming which will cause segregation and jamming.

However, pumping is an excellent way to deliver concrete horizontally to feed a hopper at the top of the tremie pipe. In very hot environment, the pump line should be insulated.

The placement of tremie concrete is best carried out through a tube using gravity flow. A hopper is mounted at top. Usually, the tube must be at least 8–10 times the maximum size of coarse aggregate. For typical placements, 200–300 mm diameter tubes are usual. The pipe may be sectional, but joints should be flanged and bolted, with a soft rubber gasket or screwed joints to prevent any in-leakage of water. When the mix is poured down the pipe, if there is a gap in the joint, there will be a venturi effect which will suck in seawater and mix it into the concrete thus grossly increasing segregation and washout. Where practical, placing the tremie pipe on a slant of 5°–10° from the vertical will allow entrapped air to escape.

The preferred way to start a pour of any depth up to 50 m is to install a plywood or 1/8″ steel plate on the bottom end with a soft rubber gasket. The plate is tied with twine to the pipe (see Figure 6.36). The tremie pipe must have sufficient wall thickness so that it is negatively buoyant when empty. The pour is started by placing the sealed pipe on the bottom and then partially filling it with the, tremie concrete mix. While there will be some segregation during the fall down the empty pipe, there will normally be adequate remixing at the bottom. To ensure this, first place 1 m^3 of the mix with the coarse aggregate omitted.

When the tremie has been filled to a reasonable distance above the balancing head of fresh concrete vs. seawater (about 50%), the pipe is raised about 150 mm, allowing the concrete to flow out. “Reasonable distance” is that required to overcome the friction head, which may be only a meter or less. The lower end of the pipe is kept embedded in fresh

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FIGURE 6.36
Placing a gasketed end plate on a tremie pipe.

concrete, but always above the level where the concrete has taken initial set. With a retarder to prevent initial set, the depth of embedment becomes less sensitive. The tip of the tremie pipe should be immersed about 1–2 m as a minimum to prevent water inflow into the pipe. The flow of concrete should be smooth, consistent with the rate at which concrete can be delivered into the hopper at the top. Similarly, the method of delivery should provide relatively even feed into the hopper rather than large batches being suddenly dumped. If for any reason the seal is lost, the concrete having flowed completely out of the pipe, then the pipe should be raised and sealed and the pour restarted as in the original program.

When large areas are to be covered, multiple tremie pipes should be used or the tremie pipes reset in new locations within the slope of the fresh pour. The distance tremie concrete can flow without excessive segregation is between 6 and 20 m: the larger distances are obtainable with a flowing but very cohesive mix, which prevents excessive segregation and washing of the cement (see Figure 6.37). The slope of an extensive pour will allow the laitance and silt to flow down so that it tends to collect in a far corner, where it might be trapped under good concrete. An airlift may be operated in the corner of an enclosure to remove any laitance.

The major problems with tremie concrete have to do with segregation into sand, gravel, and a mixture of cement and water known as laitance, from the French word for "milk."

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FIGURE 6.37
Long tremie tube for placing underwater concrete in Verrazano Narrows Bridge Pier is made up in two sections with bolted and gasketed joint.

Actually laitance may be a very plastic, claylike substance and will eventually harden into a chalklike material. It is very porous and constitutes a weak layer in the structure unless properly removed. Segregation occurs primarily when concrete is allowed to flow through seawater or where seawater is mixed into the concrete, as by mechanical disturbance, divers walking in it, attempts to vibrate it, or use of a nonplastic mix that tends to build up and then break through in an overflow. Churning of the tremie pipe to promote flow and moving the pipe horizontally through fresh concrete are very bad practices. Leaky joints will act as a venturi mixer and wash the concrete during its downward flow. When filling under an upper portion of the structure or where high-quality concrete is needed right at the top, tremie concrete should be overflowed to displace any laitance until good concrete appears.

A very widespread practice has been carried forward from earlier years; this is the use of a "go-devil" or traveling plug to start the pour. For example, in the earlier days, a plug of hay or burlap was placed in the pipe, and the concrete poured on top of it, forcing it down, and forcing the water out of the pipe ahead of the plug. This practice, while crude, was a moderately effective means for the initial start. It should never be used when restarting a pour after loss of seal or when resetting a pipe into fresh concrete. The water forced down ahead of the plug will be forced to flow through the fresh concrete, washing out the

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cement. Even in starting a pour, if the bottom is very soft or sandy, this flow of water may cause erosion.

In recent years the plug of hay has often been replaced by a ball, usually a volleyball. Such a ball fits nicely in the pipe and is readily forced down by the concrete. A ball is usually inflated to only 11 psi (80 kPa). This corresponds to a depth of about 8 m (25 ft.). Beyond that depth, the ball will collapse. If it later comes back to the surface, it may have reinflated, thus disguising the problem. A ball does not provide a suitable answer except in very shallow water.

A traveling plug or pipeline "pig" is "state-of-the-art"; it is an effective and safe method by which to start a placement. It should be of constant dimensions, gasketed so that it loosely wipes the side, and self-buoyant. A wooden cylinder, or a polyurethane cylinder (of suitable strength to resist the hydrostatic head) are both acceptable. Rubber wiping gaskets can be installed on the pig. Such a pig may not rise back to the surface—that is, it may become trapped—but in the usual case that is of no concern except in the case of an end-bearing Cast-in-Drilled Shaft pile. The traveling plug is thus similar to a pipeline pig. It is necessary to use such a device when very deep pours are undertaken, since over 30 m depth the use of the plate seal becomes unsuitable.

Foot valves, mechanically and hydraulically operated, have been tried for many years. They have almost always been a failure due to jamming of the coarse aggregate in the valve or setting of the initial cement grout in the valve mechanism. However, recently developed hydraulic valves which open to give a smooth bore have reportedly been used successfully in Sweden and Japan. These have the advantage that the placement can be shut off at the tip of the pipe as the pipe is raised, thus eliminating the typical mound that forms when a conventional tremie pipe is raised at the conclusion of the pour. Their use should be restricted to special cases and only where proven suitable by tests.

A pig should not be used to restart an existing pour since it will push the column of water through the fresh concretes causing significant wash-out of cement, with resultant rock pockets and laitance.

Figure 6.38 shows the placement of tremie concrete in the pier foundation of the Dame Point, Florida, cable-stayed bridge (see Figure 6.38 through Figure 6.42). Figure 6.39 shows how a properly proportioned and placed tremie concrete will appear (Second Delaware Memorial Bridge), whereas Figure 6.40 shows the disastrous results of an improper mix and placement procedure on another pier of the same bridge. Figure 6.41 shows the excellent results obtained on the structural tremie seal course of the Columbia River Bridge, I-205, between Oregon and Washington. Figure 6.42 illustrates the placement of a very large quantity of high-performance tremie concrete in the main pier of the Akashi Strait Bridge, Inland Sea, Japan.

6.6.4 Special Admixtures for Concreting Underwater

For placement in shallow lifts exposed to water and for placement in slowly flowing water, i.e., current or wave action, one of two special solutions can be adopted. Silica fume, in an amount 6%$\pm$ of the weight of cement, can be added, or alternatively, an antiwashout admixture can be added.

In both cases it is necessary to add a superplasticizer (high-range water reducer), plus a retarder (unless the superplasticizer already contains an adequate retarder). The resultant mix, having a W/CM of about 0.45 and a slump of 250 mm, will be almost self-leveling. Either mix, silica fume or AWA, reduces the bleed. Both silica fume and AWA are added, as the mix will become very sticky. Always make a trial batch of 1–3 m^3 to verify compatibility of components and to ensure workability. The placement procedure should be the

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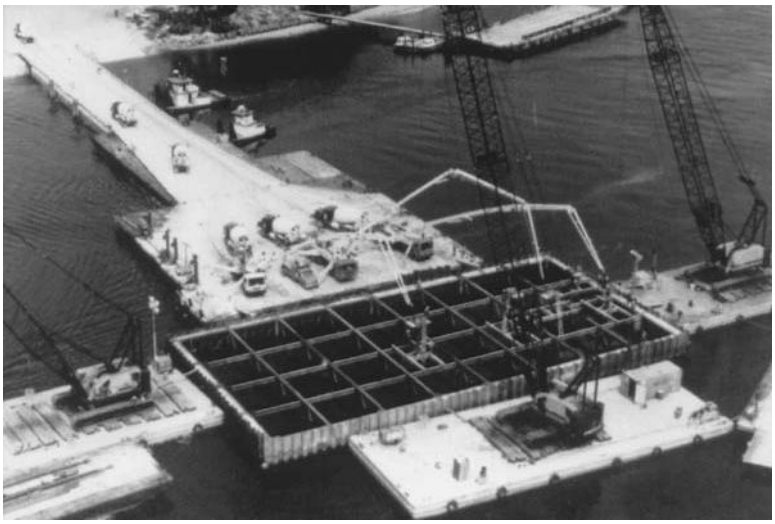

FIGURE 6.38
Delivery of concrete by pump to hoppers at top of tremie tubes so that concrete flows by gravity feed only. Dame Point Bridge, Jacksonville, Florida.

FIGURE 6.39
Surface of a properly mixed and placed tremie concrete seal, Second Delaware Memorial Bridge.

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FIGURE 6.40
Failure of tremie concrete placement due to improper mix and placement.

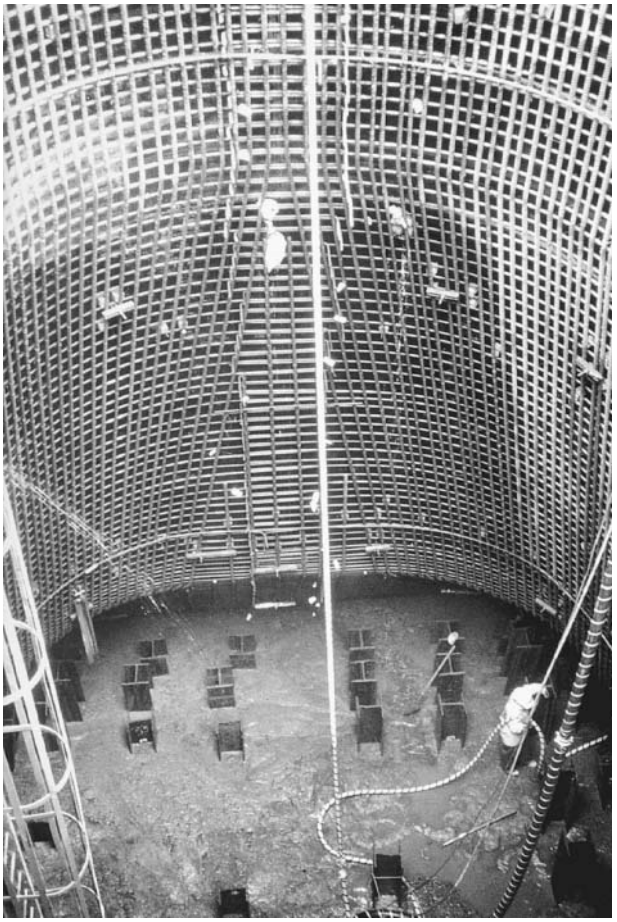

FIGURE 6.41
Excellent results with underwater structural concrete, I-205 Columbia River Bridge.

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FIGURE 6.42
Placing tremie concrete in main pier of Akashi Strait Bridge, Japan. (Courtesy of Kajima Engineering and Construction.)

same as that prescribed in this section. In no case should the resultant mix be allowed to intentionally fall through open water, despite the claims of the manufacturers.

These same two additives, i.e., AWA admixture or silica fume, can be employed for placement of mass concrete on the seafloor, using a large bucket with a closed top. AWA admixture has been used in very shallow water for filling the voids in riprap and in cyclopean construction, where alternate courses of large rock and tremie concrete are successively placed.

When using AWA, grout or concrete with small aggregate (e.g., 8 mm) can be placed through a hose, which is guided by a diver.

In a notable case, a mixture containing blast furnace slag cement (ratio 70 to 30) plus antiwashout admixture, was placed down a 3 in. (75 mm) pipe slanting at 7° from vertical, to a depth of 250 m. Maximum size of coarse aggregate was 10 mm. Tests showed no segregation and the resultant concrete had a compressive strength of 45 MPa and a tensile strength of 7 MPa. The water temperature at the site (northwest shelf of Australia) was 38°C, so the mix was precooled with liquid nitrogen. The pipe was precooled with cold water before starting the placement.

6.6.5 Grout-Intruded Aggregate

In the method of grout-intruded aggregate, coarse aggregate devoid of fines smaller than 15 mm is placed within confining walls. Grout pipes are embedded at regular intervals, horizontally and vertically. The exposed surface is covered with a mat or with an extra thickness of rock. A special grout is then pumped through the pipes to fill the interstices between the aggregate. This grout must have excellent flow characteristics and minimal bleed yet retain its general cohesiveness. Various proprietary admixtures have been developed as well as special methods of colloidal mixing. AWA may be a solution to the elimination of bleed. When the grout from any one level of pipes has reached above the level of the second set, the grout injection points are moved up. Slotted inspection

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pipes are often employed to verify the level of grout. Electrical resistivity probes may similarly be used.

In selecting the aggregate to be placed, preference should be given to cubical particles as opposed to flat particles, since bleed water tends to be trapped under the flat particles. The major concern with grout-intruded aggregate concrete is to keep the aggregate clean. Silt, organic growth, and sand particles must be kept out of the placed aggregate. When aggregate is delivered by barge or vessel, fines tend to accumulate on the barge deck at the bottom of the pile due to abrasion and chipping. The lower layer of such rock should not be placed but should be wasted or used elsewhere. Grout tends to follow the path of least resistance. Fine particles increase the friction head and thus prevent the full flow of grout around all the particles.

Growth of algae can be minimized by covering the cofferdam to keep out sunlight and by addition of an inhibitor.

Grout-intruded aggregate is an excellent solution for concreting around embedments and instrumentation where these must be kept within very close tolerances. The fluidity of the grout, an essential quality, unfortunately also makes it flow out of any gaps in the forms and up through the exposed surface into the seawater. Hence, the forms should be as tightly closed as possible, while, of course, allowing the displaced water to escape from the top.

Conversely, large placements of grout-intruded aggregate have not been uniformly satisfactory due to lack of uniform permeation and hence are unsuitable.

Admixtures must always be used with grout-intruded aggregate in order to adequately promote permeation and to minimize bleed.

6.6.6 Pumped Concrete and Mortar

Concrete made with sand only or with sand and fine aggregate (8 mm maximum) has been successfully pumped from a mixing barge down the legs of platforms to form bells at depths of 150 m and more for the offshore platforms of the Ekofisk Field in the North Sea and the offshore industrial terminal at Jubail, Saudi Arabia in the Arabian Gulf. Pipe sizes of 50–75 m were selected to maintain a substantial friction head as the concrete was pumped "downhill." A vacuum release valve at the top of the pipe prevented a vacuum forming due to too rapid descent of the concrete.

However, the widespread practice of pumping the tremie concrete to fill drilled shafts, cylinder piles, and cofferdams is not sound. For such placements, with pipes larger than 75 mm, the loss of head due to friction is seriously diminished. The result is that the concrete is discharged in cyclic surges under essentially the fluid concrete head of a full pipe. Contrast this with gravity flow from a tremie pipe open to the atmosphere and hence automatically adjusting to near balance with the external water head, which allows the concrete to flow out slowly and smoothly.

Experience shows that such pumping of mass concrete produces an unacceptable amount of voids, mud inclusions, and laitance. Its use is generally not recommended. Conversely, pumping of grout is a sound and effective method.

6.6.7 Underbase Grout

The development of gravity platforms has led to the need for suitable mixes and methods for filling underneath a large, usually flat base with grout of special properties. Generally speaking, the desired properties are homogeneity and completeness of filling, low heat development, cohesiveness, low bleed, and long-term stability. The strength and modulus of elasticity required are usually very low; properties equivalent to the natural seafloor soil

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at a depth equal to the skirt penetration are all that are necessary. High-modulus grout is undesirable as it may form "hard points," (concentrated loads), on the underside off the base.

The quantities involved on offshore platforms have been up to 10,000 m^3, so the logistics of supply, mixing, and placement in the middle of the North Sea are obviously severe. Therefore, saltwater mixes have been selected, there being no embedded reinforcement for which to be concerned about corrosion. One mix developed used cement, retarder, and a finely ground filler of limestone. Other mixes replace 50% or more of the cement with bentonite, or fly ash. Of particular interest is a stable foamed mix which uses only cement and seawater plus the foaming agent and a stabilizer. Limestone powder may also be used.

The materials are usually delivered to the platform by pump and then mixed and fed into a gravity-flow hopper located well down in the utility shaft. From there, the mix flows through pipes to the ejection nozzles. One clever scheme developed by Norwegian contrators is to suspend a short length of hose under the base slab, held in a horizontal attitude by a chain. The hose then tends to ride on the surface of the soil, ensuring that the grout will be injected at the bottom of the space under the platform.

Proper venting must be provided to allow the trapped water to escape. Usually overflow ports are provided at a few meters height above the base to allow visual verification that filling is complete. Electrical resistivity gauges or nuclear methods in the standpipe or overflow pipe can serve a similar function.

Underbase grout must be contained and sealed against washout into the water under the action of currents and waves. On both the main pier of the Akashi Strait Bridge in Japan and the Øresund Bridge in Sweden, substantial quantities were eroded.

Because of the importance of underbase grouting, a model test should be made, using the full thickness and flow length but, of course, reducing the width. Measurements should include temperature rise and bleed. Examination should be made for weak layers, large trapped inclusions, large bleed voids at the top, and laitance; 100% fill is not required, as long as any voids are small and discontinuous.

Excessive pressure can cause piping out under the skirts or even raise one portion of the platform. It could lead to local overpressure damage to the base structure. Measurement of volume is a secondary guide to the completeness of filling and possible losses due to piping under the skirts. Therefore, pressures and volumes should be monitored carefully.

A special case arises when the structure is founded on a preplaced stone fill. The decision has to be made whether it is preferred to intrude grout into the stones or to have a grout that will not penetrate. Too fluid a grout may escape into the sea through passages in the rock. A suitable grout was developed for filling under the bases of the offshore caissons of the Hay Point Terminal, Queensland, Australia, using sand, cement, and a methocel admixture, which gave significant thixotropic properties. Tests indicated little tendency to segregate and little penetration of the rock. If the underside of the base is essentially flat, it may be desirable to form inverted channels in the base slab, which will ensure that the grout will flow under the entire base. The use of a grout with AWA appears to be applicable.

Sandfill under the base is not strictly grout but is used for similar purposes. In the past, with subaqueous vehicular tunnels (tubes), sand has been placed in a semifluid state on one side and allowed to flow down, under, and part way up the other side. The scope of each operation has to be limited in order to prevent the fluid sand from raising the tube. For wide, flat-bottomed structures, Danish and Dutch engineers have developed sand-flow systems in which fluidized sand is pumped down under low head, to exit at distributed points under the base slab. As the sand spreads out in a circular pattern, the velocity drops and the sand is deposited. Any low spots become channels through which sand continues to flow until they are all filled. Thus the system tends to automatic complete

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filling. This system was employed to intrude sand fill under an exploratory drilling platform, the SSDC, in the Canadian Beaufort Sea.

A somewhat similar flow phenomenon is believed to occur with the thixotropic grout mixes, assuring relatively complete filling under the base.

6.6.8 Grout for Transfer of Forces from Piles to Sleeves and Jacket Legs

Grout is extensively used to "cement" the annulus between pile leg and jacket sleeve. An annular gap of 50–100 mm is usually selected. The grout should flow from the bottom up. The mix is generally cement plus water. Fly ash may be used to replace part of the cement in order to reduce heat of hydration. Silica fume may be added to promote thixotropic behavior, increase strength, and reduce bleed. Admixtures may be used to provide water reduction, retardation, and expansion characteristics. AWA is an also appropriate admixture. It is important that trial batches be made to ensure that the grout has the proper flow characteristics as well as strength. Flow rate should be kept low to avoid entrapment of voids. Grout should be overflowed to ensure that the initial mixture of cement and seawater is cleared. Pressures should be carefully controlled to prevent forcing the grout out from under the jacket sleeve; usually this exit is restricted by a grout retainer, but many times the grout retainer will have been damaged during pile driving. Therefore, a second entry grouting pipe is often provided, to permit the first grout to set and form a plug; then the main grouting is carried out through the upper entry port. Grouting in connection with piling is addressed in more detail in chapter 8.

6.6.9 Low-Strength Underwater Concrete

Mass underwater placements of low-strength concrete are occasionally required, for example, to temporarily plug a tunnel opening or to create a plug at the bottom of a drilled shaft riser into which a TBM will later bore. Thus the criteria will set both a minimum strength, e.g., 7 MPa at 28 days, and a maximum strength of 15 MPa at 1 year. Obviously, the cement content will be kept low. The practice of just increasing the W/CM ratio to 0.85 or more is not satisfactory: it leads to segregation and uncontrollable results. The required workability can be obtained by high proportions of fines and by use of mineral admixtures of low cementing activity. Fly ash and bentonite have been used—however, these tend to continue to gain strength with age.

Limestone powder is very weakly cementitious and has been found to be the optimum material for enhancing workability and flow while limiting strength gain. W/CM can be kept around 0.50 (where the limestone powder is included as part of the CM and still give a flowable mix without segregation.

6.6.10 Summary

There are so many variations in the size, shape, conditions, and properties desired of underwater concreting that it is important that recommendations such as those given earlier be treated as a guide only. Test mixes and trial runs are always advised to ensure that the best possible selection of mixes and methods has been made and that the personnel actually carrying out the work are cognizant of the precautions pertinent to that particular project. Both the mix design and placement procedure are all-important.

Most underwater concrete for marine operations is placed in a substantial mass. Therefore, it develops a high temperature which dissipates slowly (see Figure 6.43). The external surfaces, sides and top, lose heat most rapidly, while the core still remains hot. When the temperature differential reaches 20°C over a thickness of 300 mm, the concrete will crack.

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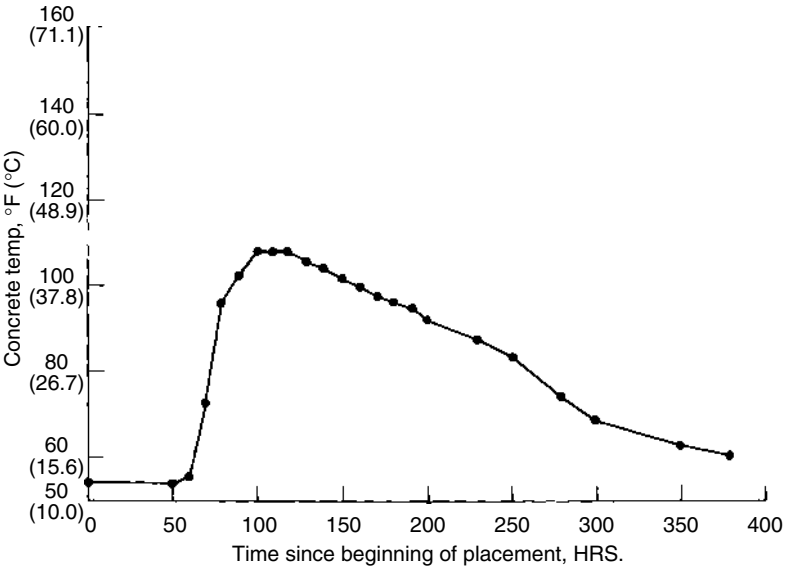

FIGURE 6.43
Typical curve of heat development in mass underwater concrete. Note long duration of elevated temperature due to heat of hydration. Mix had 20% replacement of cement by fly ash.

Insulation of the sides by soil and of the top by a thermal mattress will reduce the temperature differential. Reinforcing steel should be provided, especially in large footings and those which are supported on rock on piles, since they resist overall contraction of the newly hardened concrete. The steel area should be equal at yield to the cracking strength of concrete over the tributary concrete width times a depth of 200–300 mm from the exposed edge (see Figure 6.43).

6.7 Offshore Surveying, Navigation, and Seafloor Surveys

Navigation systems used for control of position during tow and emplacement at the site include both Global Positioning System (GPS) satellite fixes and radio navigation positioning systems. Accuracies when underway near shore or structures can usually be kept to ± 1 m, with even greater accuracy when stationary. When approaching a site in which structures already exist when leaving harbor and for final positioning, theodololite and electronic distance (range) systems are often utilized because of greater accuracy in the available time.

Many long-range electronic systems suffer from night effects, losing accuracy. They also can be misinterpreted by increments of range steps, giving errors of 50 m. Thus the use of more than one system is desirable, in order to provide a check.

In the open sea, a system such as Decca Hi-Fix can be utilized for close control; for long-distance tows, Loran C is adequate. Other systems are Sydelis, Artemis, Motorola, Argo, Racal Hyper-Fix, and Omega. Satellite fixes can give instantaneous accuracies from 1 to 10 m depending on how many satellites are interrogated. The GPS, which became commercially available in 1986, now gives positioning accuracy to 1 m worldwide. Differential GPS, along with progressive declassification, continues to increase the

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accuracy. Short-range positioning systems include Motorola Mini-Ranger, Honeywell Micro-Automatic Station Keeping System, and Simrad.

Underwater acoustic transponders can be preplaced on the seafloor and used to control the final installation of a structure in the open sea. Usually six are preset to ensure that at least three will be working when needed. The use of these for final bathymetric surveys and for borings ensures that when the structure is finally installed, it will be at the correct relative position. They can also be utilized for the underwater assembly of elements and for guidance of an ROV.

Acoustic transponders have proven very satisfactory with steel jackets, but less so with massive concrete structures, which along with the many boats in the area, create excessive noise. A major acoustic transponder system has been developed by Ocean Instruments to enable placing of a jacket near existing pipelines (see Figure 6.44). The acoustic positioning system is calibrated with satellite receivers to process Doppler information and thus provide a surface position fix every one to two hours.

Accuracy in the placement of offshore structures has been steadily improving with the advances in equipment and with experience. In the early and mid-1970s, distances off target averaged 25 m but by 1995 this tolerance had been reduced to less than 5 m. Positioning of important structures should always be backed up by more than one system. During the installation of the Tarsiut Island, a survey tower was installed to control the screeding of the seats for the caissons, which was carried out with considerable accuracy. In order to gain access to screed the last base, the tower had to be removed and replaced. Due to a combination of human and electronic error, a gross error occurred, resulting in the caissons being installed 20 m off from their intended locations, on an unscreeded bed. In hindsight, use of a secondary backup system of spar buoys, or even taut-moored buoys, would have prevented such gross error.

Today it is becoming common to pre-drill and cap a number of wells in order to get early production and return-on-investment. The structure then will be set over the pre-drilled wells.

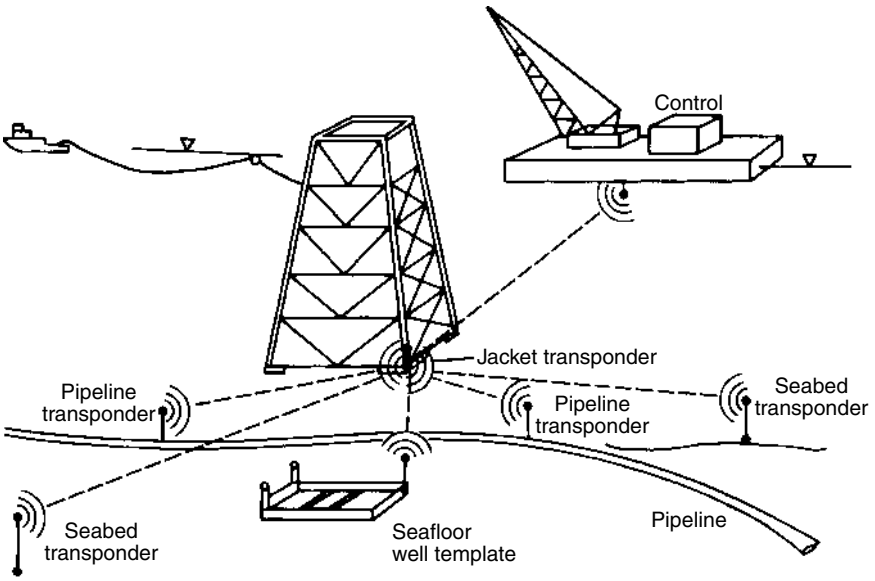

FIGURE 6.44
Acoustic system for installing jacket over seafloor template.

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To do this, a template is first set on the seafloor, with guide holes for the wells and bumper piles. The wells are drilled and the bumper piles installed. The latter are grouted and then freed from the template.

The jacket or GBS is later set by using transponders and guided to exact position by the bumper piles. This requires taut mooring lines in addition to the general sea mooring through the center of rotation.

Seafloor surveys should be carried out in the vicinity of all marine and offshore structures, as well as along the route of submarine pipelines and cables. While the continental shelf is relatively flat and level, both the deep seabed and the coastal areas are subject to abrupt changes and anomalies. Similarly, many harbors are characterized by relatively flat bottom sediments but riverine bottoms are often highly irregular. Seafloor survey assessments should be carried out to disclose slumps, scarps, irregular topography, rock outcrops, and the character of the seafloor material.

Especially difficult are those sites where a thin layer of soft sediments overlies cemented material, or where coral heads rise above the sandy seafloor. In the deep sea, the assessment should additionally address the possible presence of mud volcanoes, mud lumps, collapse features, sand waves, slides, faults, diapers, erosional surfaces, gas bubbles, gas seeps, buried channels, lateral variations in strata thickness, and subsea permafrost. For paleoglacial seafloors such as the North Sea, the presence of surface and subsurface boulders is important. Sand waves similar to above-water sand dunes are an important feature for rivers, harbors, and estuaries, and even occur far offshore where there are strong bottom currents. The fact that they are transient and therefore can alternately bury or uncover a pipeline or structure base makes their disclosure critically important.

Buried channels exist in many harbors. During the glacial age, the sea was about 100 m deeper than now, so rivers cut deep channels out to the lowered sea. Windblown sand and volcanic ash were often deposited on the bare surfaces. Subsequently, they have been flooded by the rise in sea level and the sharp topography filled by unconsolidated and weak sediments. Similar phenomena have created a series of caprock strata off coasts such as the North West Shelf of Australia. Of special concern to both engineers and contractors are soils containing greater than 15%–20% carbonates and soils containing mica.

Calcareous sands are composed of skeletons of minute organisms. Prior to fracture, these have relatively high friction angles and bearing strength; they behave as sand. Upon fracture, however, they behave as weak clay. Skin friction drops close to zero. This behavior enables piling to be driven with relatively low resistance. It also greatly reduces the pullout capacity. Direct bearing strength remains satisfactory. The presence of substantial quantities of mica not only reduces skin friction but also leads to instability of slopes and cuts. Finely divided mica is very difficult to ascertain, since minute particles do not show up visually. They can only be determined by physical-chemical testing.

Sands are very difficult to sample properly, and even the best samplers do not show the full density and consolidation, since the act of sampling and the reduction in hydrostatic pressure as they are raised to the surface invariably reduce their strength and apparent consolidation.

Siltstone and mudstone are subject to water softening. What appears as soft rock may slack and disintegrate in the presence of water. Hence, borings, especially wash borings, may be reported as "silt and mud" but may actually be consolidated firm soils with the characteristics of soft rock. They often are resistant and abrasive in drilling or driving but are easily penetrated by a jet.

Conversely, weak rock may have been determined by coring, but disintegrate when exposed to seawater. Limestone strata may have been eroded to form solution cavities at times when the sea level and consequently the tributary levels of the rivers were up to 100 m lower. They may later have been partially filled with loose sand and silty sediments,

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and then covered over with sands and even subsequent strata of cemented material. Finding these and defining their outline is a very difficult task, since borings may either miss them completely or conversely encounter a small but deep cavity, both giving an erroneous picture of actual conditions.

In many coastal structures, such as offshore terminals and outfall sewers, and in inland waters, shore-mounted lasers or even concentrated lights may be set up to provide a range that is directly visible to the barge superintendent and operator. This range, combined with electronic distance systems (EDS), may be used as the primary control, especially if frequent moving is involved. The advantage of these is that they enable the barge captain to judge rate of change as well as verify final position, and thus avoid excessive "hunt." Theodolites and electronic positioning were used to position the pylon pier caissons for the Great Belt Bridge in Denmark, with a final check made by repeated GPS observations. These caissons were set to an accuracy of about 50 mm.

Bathymetric surveys are carried out by both depth-finder sonic equipment and side-scan sonar. These must be corrected for roll, pitch, and heave and integrated with positioning systems. Multi-beam sonar enables a swaft to be covered. An integrated instrumentation system, called a "profiler," can give a plot of contours within a 200- to 400-m-diameter area. Such a system can be used for a wide variety of river, harbor, estuary, coastal, and offshore surveys. It can also develop images of large pipe, e.g., outfall pipe segments and other objects on the seafloor (see Figure 6.45).

Depth-finding sonar should be run at two frequencies, high and low, to detect the presence of soft, semifluid sediments overlying a firmer bottom. In a deep fjord in Norway, for example, use of the standard low-frequency sonar depth finder gave a depth 25 m greater than actual, since the acoustic waves penetrated the very soft soil without reflection. In the deep sea, sub-bottom profilers (tuned transmitters) can be used to determine the near-surface features. A boom-type shallow seismic system can display anomalies and density changes in strata up to 50 m depth.

FIGURE 6.45
Acoustic image by 3-D Mesotech Profiler. (Courtesy of Kongsberg–Simrad–Mesotech.)

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In areas of strong relief, with steep or near-vertical bluffs and underwater canyon sides, the sonar echoes may come back from the side walls, indicating less depth than the true value. This is because of the conical spread of the beam. Narrow beams can be used to minimize this problem. An ROV with side-scan sonar was used very successfully to survey the bathymetry immediately behind Shasta Dam at a depth of over 100 m (see Figure 6.46). Multibeam SWATH systems have been specifically developed in order to accurately depict the irregular seafloor in the deep sea.

Side-scan sonar can produce an excellent two-dimensional portrayal of the seafloor, along with any man-made objects such as pipelines, dropped objects, anchors, even anchor drag marks. Advanced acoustic imaging can now give a map of the seafloor with definition of 1–2 m.

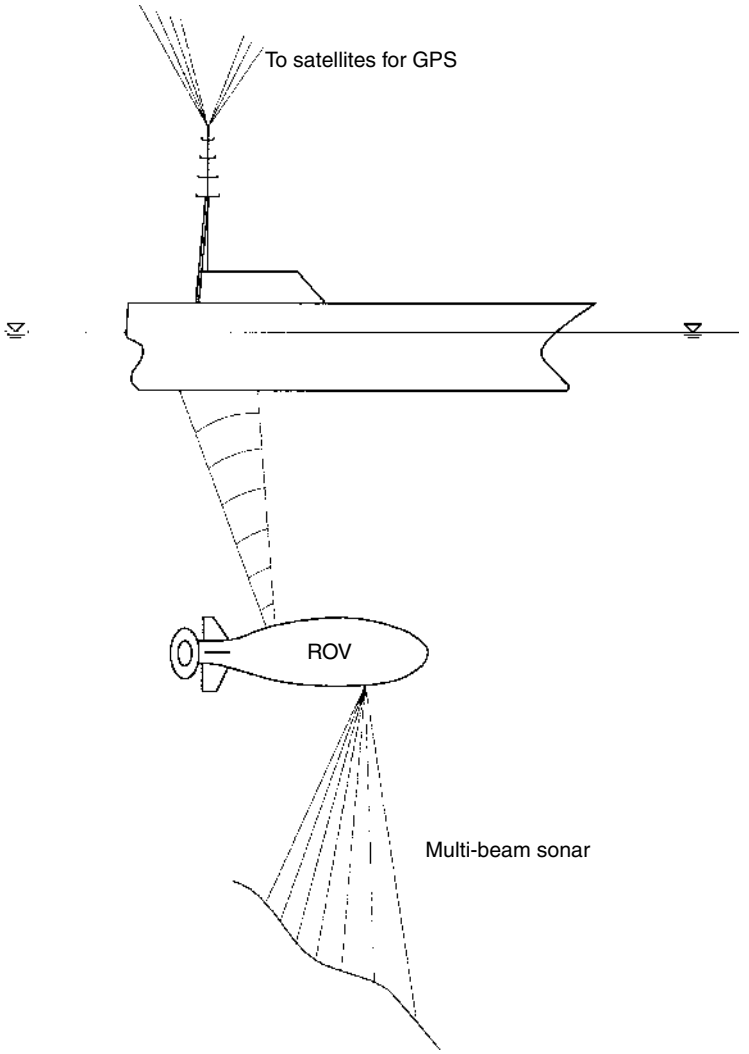

FIGURE 6.46
Use of captive ROV and multibeam sonar to profile sloping seafloor.

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Advanced photogrammetric techniques, using multiple photos, enable satellites to map a small area of seafloor to an accuracy of 25 mm in relief. Recent development by NASA of extremely sensitive film (ASA 2,000,000), combined with the use of strobe lights, has revolutionized optical seafloor search and survey. Autonomous ROVs are being increasingly employed in seafloor surveys for pipeline routes and deep sea sites.

Many new underwater acoustic systems and magnetometers are now available. Many of these can be fitted to an ROV and the data transmitted by telemetry back to the tending vessel. Others are used by divers. They enable the detection of buried cables and pipes, leak detection, high-resolution acoustic imaging of the seafloor, range measurement for short-distance ranging, and guidance for entry of mating cones and piles.

Position-sensing devices and systems have been developed to enable a vessel equipped with bow and stern thrusters to maintain station over a fixed position on the seafloor (see Figure 6.47).

Sparker surveys can be run to determine the surface of subsurface hard layers and bedrock up to 100 m below the surface. Boomers can simultaneously be used for definition of surfaces to a depth of 100 m below the surface. Air gun, water gun, sleeve exploder, and similar advanced geophysical devices can detect anomalous profiles at deeper penetrations (above 100 m).

Aerial photography can be used to carry out in progress and completion surveys of the above-water profile of breakwaters. Underwater profile may be surveyed by side-scan

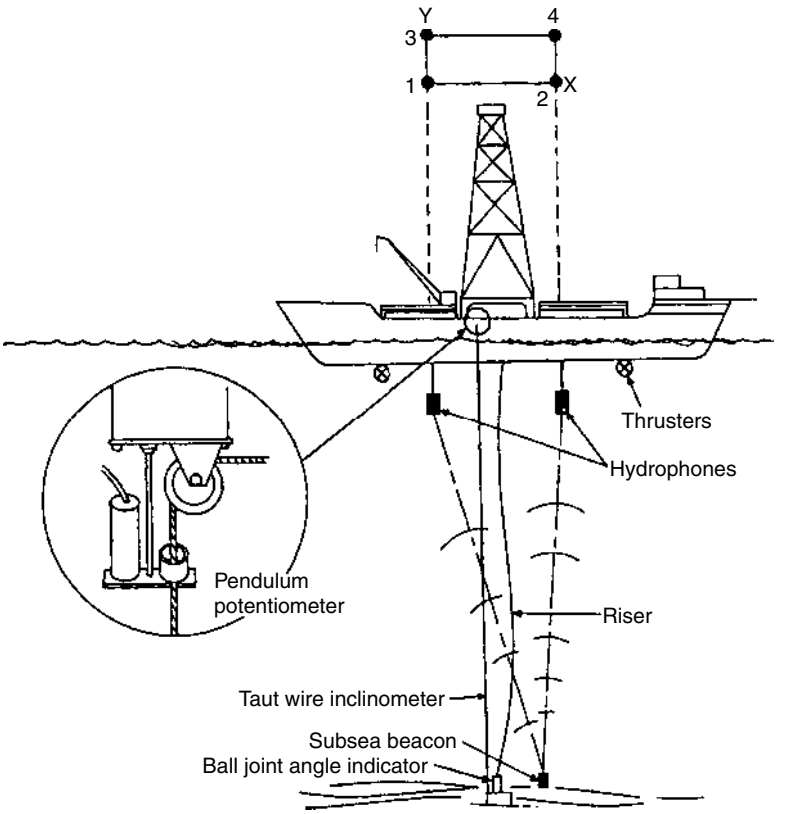

FIGURE 6.47
Position-sensing system for maintaining station.

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sonar from boats, electronically keyed in to roll, pitch, and heave of the boat as monitored by shore stations.

Another use for advanced surveying equipment by satellite is for detection of sea ice in the Arctic and sub-Arctic. Not only is mapping required but also indications of size and thickness of floes and the character of the ice, whether first-year or multiyear. These can be given by visual systems or by side-looking airborne radar (SLAR). The most advanced version of this is synthetic aperture radar (SAR), which permits five times better resolution than SLAR and can "see" through the cloud cover, which so frequently obscures the ice. Satellite coverage is now able to give all-weather, through-cloud coverage, through the use of SAR, identifying ice concentrations, presence of multiyear ice or ice floes, and presence of ice islands. SLAR and SAR flown by aircraft are available today, but coverage is limited.

For underwater mating—for example, assembly of underwater structures, positioning of a jacket over a pre-installed template or of an articulated tower over a pre-piled base—a number of systems have been employed. Generally, the control barge, usually a large offshore derrick barge, is preplaced on a taut mooring. Side-scan sonar, electronic positioning, and acoustic transducers are used to verify its position relative to the underwater element. As the structure itself is lowered, short-range, narrow-beam sonars on the structure are used to interrogate acoustic transponders on the submerged element. A video camera with high-intensity lights, mounted on the legs of a jacket, may be used to verify position at close range. Acoustic Doppler current profilers can measure currents in three dimensions (Figure 6.48).

Finally, ROVs may be used to verify conformance visually; their value is in preventing gross error due to some electronic reflections or misunderstanding on topside.

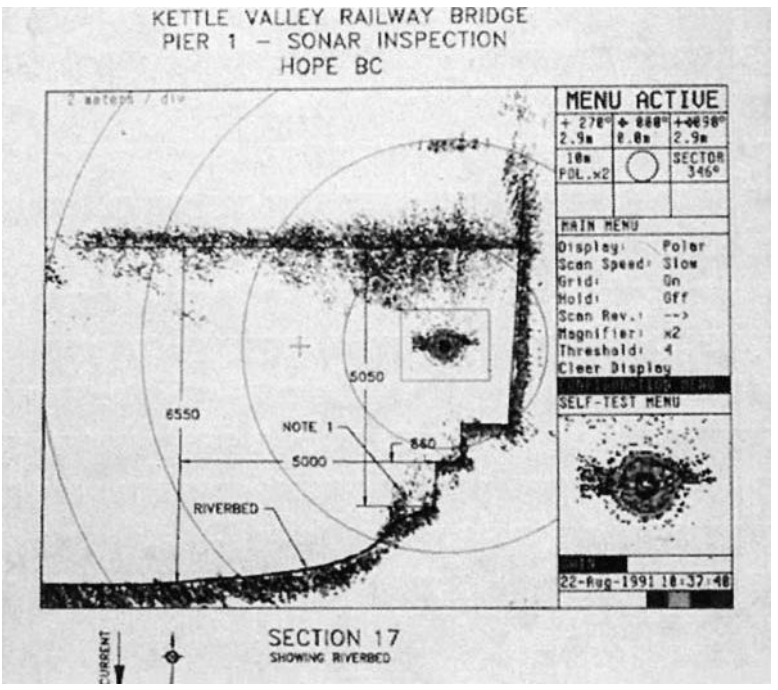

FIGURE 6.48
Sonar profile of bridge pier. (Courtesy of Kongsberg–Simrad–Mesotech.)

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All of these means were employed in mating the three sections of the Cognac jacket in over 300 m of water. (see chapter 9).

6.8 Temporary Buoyancy Augmentation

Additional buoyancy may be required at many stages of offshore operations, such as:

1. Reducing draft during floatout from a construction basin and during tow
2. Giving flotation to a pipeline or reducing its net weight in water
3. Reducing weights of structures or elements during installation or salvage
4. Changing the lead of towlines which have been attached below water
5. Providing stability to a structure during deck mating or installation
6. Providing control of draft and attitude during floatout, tow, launching, installation, and/or removal.

Temporary buoyancy tanks are usually of steel, although large concrete tanks and hybrid (steel-concrete sandwich) tanks have been proposed. They may be pressurized internally for added safety against implosion. Internal subdivision or foam-filling may be provided to ensure against in-leakage (see Figure 6.49).

The tanks must be designed for the maximum hydrostatic head to which they can be subjected, with an allowance for overdepth submergence in event of credible accident. The tanks must be connected to the structure in such a way as to ensure against structural overstress globally or locally. The structure being supported must be checked under the upward forces imparted. Prestressing, welding, and high-strength bolting are means of attachment, since the tanks will be subjected to cyclic dynamic wave loads, wave slam, and possible vortex shedding. Corrosion-accelerated fatigue may lead to failure at the connections, especially if there are stress concentrations. Buoyancy tanks attached to a

FIGURE 6.49
Temporary buoyancy tanks provide stability during immersion of concrete gravity base mooring dolphin, Queensland, Australia.

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jacket may see high-impact and high-acceleration forces during launching. The connections must also be designed to permit disconnecting after they have served their purpose.

In removal of a large buoyancy tank, the procedure must be so planned that it cannot possibly impact the structure or surface vessel due to uncontrolled rise. Adequate allowance must be made for wave and current action.

Polyurethane and polystyrene foam blocks have been used to give buoyancy to risers and pipelines. Their attachments must be adequate to withstand abrasion during installation. On one bottom-pull pipeline on the coast of Libya, the temporary floats were broken loose when the steel straps wore through from sand abrasion and the pipeline had to be abandoned.

As noted in Section 6.1, inflatable rubber bags were installed under the base of the ANDOC Dunlin A Platform to reduce draft during floatout. Such bags might well be used for reducing draft during passage of an Arctic caisson around Point Barrow on its way to the Beaufort Sea. Smaller inflatable rubber bags have been developed by the Japanese for use in handling moderate weights underwater and for salvage. Many such balloons were attached to the ill-fated Frigg steel jacket in an attempt to salvage it; unfortunately, their attachment lines fouled and were in some cases torn off by wave action, so the attempt had to be abandoned.

Collapsible rubber bags are employed in the French "S-curve" system for deep-water pipe laying; they are designed to gradually lose volume and hence buoyancy as they are pulled underwater.

Temporary buoyancy tanks can be installed beneath a structure to give flotation in shallow draft, but, of course, to raise $\overline{KG}$ while lowering $\overline{KB}$, vertical tanks may be installed alongside a member. In some cases, where draft is not a problem, tanks can be installed on top of a base, to increase the $\overline{KB}$. Obviously, their buoyancy only comes into play when they become submerged.

Where a heavy structural segment can be placed underwater near shore or in a construction basin, then a barge may float in over the top and lift it by jack rods or cable jacks, for transport in a submerged condition to the site. Thus, only the buoyant weight needs to be lifted.

It is important that the same degree of engineering expertise and construction skill be brought to bear on temporary buoyancy tanks and their connections as for permanent structures, since failure can have disastrous consequences.

They that go down to the sea in ships, that do business
on the great waters, these see the works of the Lord and
His wonders of the deep.

Psalms 107:23–24

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7

Seafloor Modifications and Improvements

7.1 General

Although in many cases the seafloor has fortuitously been covered and leveled with sediments which have subsequently been consolidated by the action of storm waves, there are many instances, of course, where the site on which the structure is to be placed is not so favorable. There may be unconsolidated, unstable, and weak sediments at the seafloor surface. Rock outcroppings may occur, with highly irregular features.

Subsurface strata of sands may be subject to liquefaction under prolonged storms or earthquakes. Unstable deposits at or near the site may give rise to slumping, mudslides, or turbidity currents or may be subject to slow and continued creep. Boulders have been deposited by glacial action on many northern seafloors. Weathering of fractured zones as sea levels dropped during glacial ages may have produced soft layers between hard rock. Solution cavities may have formed in limestone, which is now submerged and filled with loose silt. Calcareous deposits may have formed on windblown sand as it settled through the water. Recent organic silting or volcanic ash deposits may lie almost undetected on top of competent strata.

Any one or several of these or other anomalous situations may exist at a specific site. There are two possible solutions; either (1) design the structure to be stable on the actual seafloor soils as they exist or (2) take various steps to improve or modify the seafloor soil properties. The first solution has been the one employed to date in most cases of offshore construction. The second solution is frequently employed for major land structures and is increasingly employed for harbor and coastal (shallow water) structures. The second solution, seafloor preparation, presents some very significant potential advantages for deep water as well. It is being increasingly recognized that there is normally time available in the schedule to do this because of the lead time required for procurement and fabrication of the structure prior to installation. The foundation soils of the new Rion-Antirion Bridge in Greece were strengthened by driving closely spaced steel piles, thus "pinning" the soils against shear failure as well as transferring the loads to deeper, more competent soils. Similar stabilization, using concrete piles, will be used on the Venice storm surge barriers.

In some cases, there may also be time after structure installation in which to carry out soil improvement operations; this would be the case, for example, where the structure was installed at the beginning of the good weather season, leaving several months for subsequent soil improvement operations such as grouting.

It is important always to keep in mind the interactive effects among soil, structure, and the environment. Each acts on and reacts to the others. The environment imposes cyclic loadings on the soil, which sometimes leads to physical scour or erosion. The structure

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imposes forces on the soil, and the soil in turn imposes reactions on the structure. The structure and the waves interact dynamically, as do the soil and the structure, so that as dynamic effects are created in the soils, the soils in turn have a dominant effect on the dynamic behavior of the structure. This process is known as kinetic interaction or soil-structure interaction (SSI).

The adoption and implementation of seafloor preparation measures have been determined largely on the twin criteria of need and practicality. Large overwater bridge piers, for example, have required a high degree of stability and minimal displacements, or tilting. At the same time, such bridge piers have until now been almost exclusively located in water depths less than 100 m and in semiprotected waters. Conversely, with the typical large offshore gravity structure, the critical failure modes have been sliding and rocking. Both of these can be significantly improved by seafloor soil strengthening. Storm-surge barriers also depend on soil improvement to reduce underflow and to provide support. With pile-founded structures, lateral stability (the P/y effect) of the piles can be substantially improved, as can the axial capacity. The latter is dealt with under steel pile installation in Section 8.14.

Seafloor foundation modifications are designed to provide a stable base of adequate strength to support the structure and to resist failure and progressive degradation under both a single extreme event and the repetition of cyclic dynamic loads. The foundation must be graded or leveled as necessary to receive the structure and all obstructions removed. In some cases, protective underwater berms will be placed to protect the structure from ice pressure ridges and ice island fragments or from ship collision. Proper controls must be provided to ensure location and grade and to monitor the performance and adequacy of the measures taken.

These operations can be arranged in outline form as follows:

- Seafloor dredging, obstruction removal and leveling (see Section 7.3)
- Dredging and removal of hard material and rock (see Section 7.4)
- Placement of underwater fills (see Section 7.5)
- Consolidation and strengthening of weak soils (see Section 7.6)
- Prevention of liquefaction (see Section 7.7)
- Scour protection (see Section 7.8).

7.2 Controls for Grade and Position

7.2.1 Determination of Existing Conditions

Until recently, one of the more difficult tasks was to properly correlate the relative positions of operations on the seafloor with the structure's final location. Electronic navigation and even satellite positioning have not generally been sufficiently simple and accurate enough to permit accurate relative positions to be determined and repeated. Today, Global Positioning System (GPS) is accurate at sea level within about 0.2 m and it can be extended to deep water electronically. Increased accuracy can be attained by successive iterations of electronic or satellite positioning.

In shallow water, relative locations can be adequately marked with spar buoys. In deeper water, the articulated buoyant staff buoy provides a permanent marker that is little affected by the waves but strongly affected by the currents. In some cases, it may

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be possible to use inclinometers with appropriate telemetry in conjunction with these articulated buoys. Electronic means can be used to record bucket and dredge head positions at the seafloor.

Acoustic transponders have now been greatly improved in life and reliability. They can be placed on the seafloor surrounding the site; then their true position can be determined by successive iterations of electronic or satellite position-fixing of the surface control ship. For important structures, where operations will continue for a substantial period of time, enough seafloor acoustic transponders are usually placed to assure adequate redundancy in case of malfunction or destruction of one or more transponders.

Bathymetry can be determined by sonar with due consideration to the relief, contour interval, and motion of the vessel. Corrections must be made for change in water level due to tides, barometric pressures, and storm surges. Corrections must also be made for roll, pitch, and heave of the vessel. On the Oosterschelde Storm Surge Barrier, a remote-controlled bottom-crawling tractor tended by a control vessel was able to map seafloor bathymetry at the site of each pier with an accuracy of $+20$ mm.

The character of the seafloor can often be determined by video means, using work submarines or an Remote Operated Vehicle (ROV). Side-scan sonar is extremely effective in revealing obstructions and sharp breaks in the surface level. The "profiler," which combines side-scan sonar, automatic roll, pitch, and head compensation, and a 360° revolving directional acoustic beam, is very effective in providing a continuous mapping of the seafloor, especially where there are significant changes.

Existing seafloor soil conditions can be determined from grab samples (for surface classification), by cone penetrometer tests (CPTs), by in-place vane shear tests, by plate-bearing tests, and by borehole sampling. These borehole samples can be obtained by core drilling from a vessel or a work platform, by seafloor-jacked sample tubes, or by vibratory corers. When deep boreholes are run from drilling vessels, geophysical methods may be used to determine density, resistivity, and permeability.

Experimental work has been carried out with free-fall or explosively driven penetrometers, which send back their changing rates of penetration (deceleration) by telemetry, enabling a determination of relative density at various depths.

Geophysical seismic and near-surface acoustic surveys are very effective in distinguishing anomalies in subsurface geotechnical properties. When correlated with borehole sampling, they serve to portray the area situation much more effectively than linear interpolation between the boreholes by itself.

7.3 Seafloor Dredging, Obstruction Removal, and Leveling

Boulders are scattered over much of the floor of the North Sea as well as many other regions. In general, it has been felt that those less than 0.5 m in diameter were sufficiently small that they would be displaced sidewise or pushed down into the underlying clays by the piling or the structure. Boulders larger than this and clusters have been removed. There are two methods of removal. The most effective one has been to drag the boulders off the site using two tugs and trawl cables and boards, guided by the preset acoustic transponders to the location of the boulder or boulders as previously determined by visual observation from the work submarine. The second method has involved the placement of shaped charges by divers to shatter the boulders; this is obviously limited to those depths and sea conditions in which a diver can effectively work. Thermic lances can be used to cut large boulders into smaller fragments. Ultra-high-pressure water jets with pressures in the range of 15,000 psi can be similarly employed.

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Other obstructions can be removed in similar fashion—that is, by dragging or by individual hooking-on by a diver or ROV to an object previously located visually or by side-scan sonar. In shallow water, (less than 30-m depth), a large grab or clamshell bucket may be used.

Leveling of the seafloor is dependent on having a stable work platform, maintained at a relatively constant grade, from which the drag or screed can be effectively employed. Thus, a jack-up rig or tensioned buoyant platform are especially well suited for such operations.

If the seafloor is generally level but with local ridges and depressions, then dragging of the area with a heavy steel girder can help to smooth out these differences in level. The girder is suspended from a barge by two lines of equal length so that the girder hangs horizontally. As the barge moves across the area, the girder tends to knock down the ridges and fill the valleys between. This method was employed to level the shallow seafloor for the Prudhoe Bay Waterflood Facility, a 200-m-long barge-mounted plant. The difficulties arise with swell acting on the screeding barge, causing the lines suspending the girder to alternately slacken and then become taut. This can lead to the creation of low and high spots rather than their elimination. If the screed girder is suspended by heave compensators on both lines and if the barge or vessel is always headed normal to the swells, then this method can produce satisfactory results during selected periods of low sea states. Alternatively, spars (buoyant columns) can be rigged astern of the vessel to act as crude heave compensators.

Another method that has been proposed and engineered is to use a moored drill ship or semisubmersible with heave compensator from which to suspend the drag. The drag is then pulled along the seafloor by two lines that have been run to preset anchors. A major improvement, now practicable with modern technology, will be to equip the drag head with thrusters controlled from the vessel (see Figure 7.1).

Screeding frames have been developed for use in preparing a level base on which to seat a caisson or an underwater tunnel segment. Some are bottom supported, as shown in Figure 7.2. Such a concept was developed by Christiani and Nielsen for use in leveling the base for breakwater caissons in Cape Town, South Africa, and was later modified for use in seating the caissons for an offshore terminal project off Queensland, Australia, located in the open sea at a depth of 25 m. The concept has been continuously improved for preparing the beds for submerged prefabricated tunnel segments. Horizontal screws have recently been employed for screeding for an outfall sewer pipeline in 100 m of water off San Diego.

For underwater tunnel segments ("tubes"), screed barges have been developed, based on a similar screeding arrangement. However, the supporting platform has been a 100-m-long catamaran semisubmersible barge, which lowers heavy concrete block weights down to the seafloor and then pulls down against these to stabilize the barge against waves and the effect of tides. Accuracies of ± 50 mm have been achieved. For the Øresund submerged tunnel in Denmark, a standard hydraulic dredge has been fitted with a rotary screed. This is swept back and forth, with its true position determinable by means of electronic positioning and GPS.

In the Beaufort Sea, a hydraulic dredge has been used to roughly level preplaced sand embankments at a depth of 10–20 m, using a heave compensator to offset the effect of waves. The Dutch similarly used a hydraulic dustpan dredge to level the seafloor for the mattress placement on the Oosterschelde Storm Surge Barrier. In Japan, for screeding the surface of a trench in Tokyo Bay, a horizontal screw (much like a snowplow) was suspended so that it leveled off high spots by moving the ridge material sideways off the site. Attempts have been made to screed the seafloor level by diver-manipulated screeds. These have generally been excessively time-consuming and unsatisfactory. A

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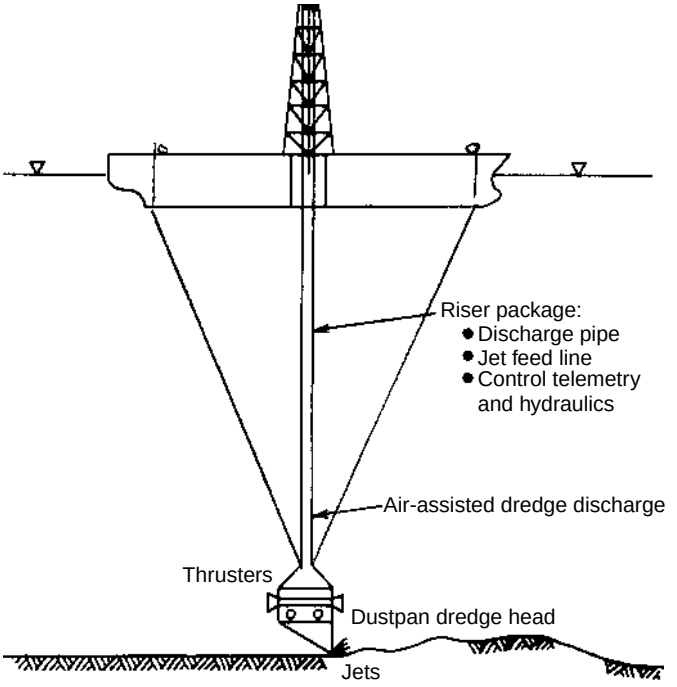

FIGURE 7.1
Deep-sea leveling device (proposed).

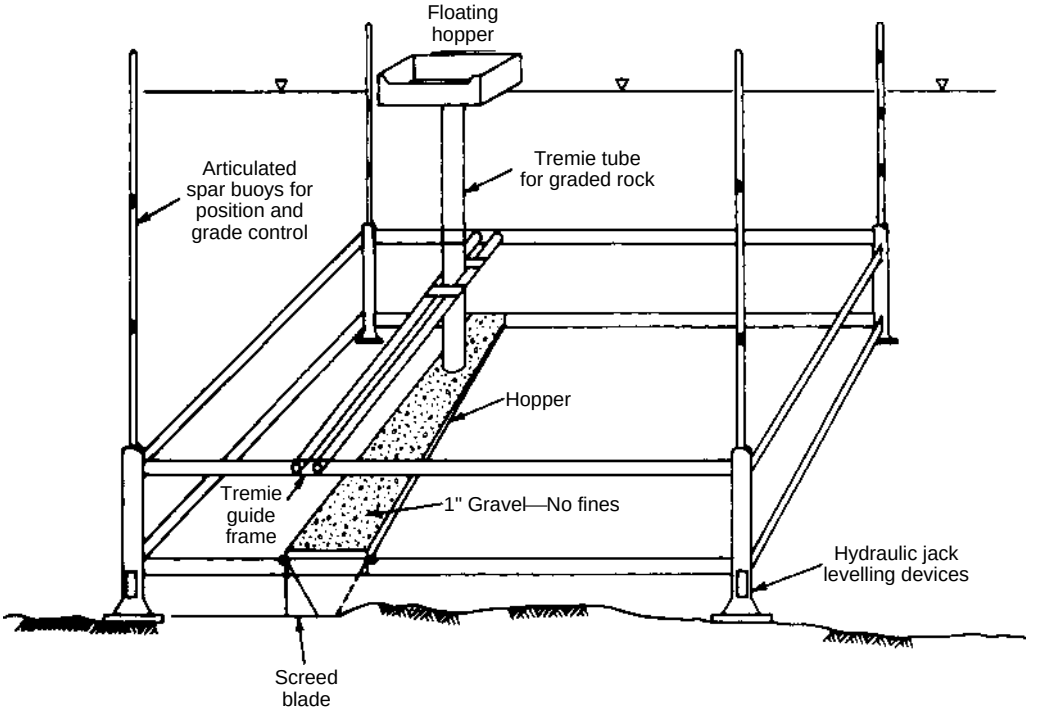

FIGURE 7.2
Screeding frame for underwater fill.

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notorious example was the Royal Sovereign Lighthouse in the southern English Channel, where strong tides and storm waves disrupted the work almost as fast as it was done.

For some bridge piers, notably those of the Honshu-Shikoku bridges in Japan, the design has required accurate grinding of bedrock in order to seat a caisson. In such cases, the rock is dredged by the means described in Section 7.4 and then thoroughly cleaned off by jets and airlift. Grinding has then been employed, using large-diameter grinding bits similar to those used on tunnel-boring machines. Horizontally rotating bits tend to "crawl" over the bottom and hence must be rigidly held by a structural frame just above the seafloor. Counter-rotating bits can be used to offset the net lateral forces.

Grinding with wheels rotating in the vertical plane about a horizontal axis is more efficient. These could be extensions of the rock-trenching wheels developed for the English Channel cable crossing and the proposed Straits of Belle Isle crossing between Labrador and Newfoundland. A mining tool was used for grinding shallow underwater concrete on a lock reconstruction project and for Pier 7A of the Bisan-Seto Bridge (see Figure 7.3 and Figure 7.4).

When weak and unsuitable sediments overlie the seafloor foundation soils, they must be removed or displaced or strengthened. In this section, removal and displacement will be discussed.

For large-scale operations in the ocean, one of the most effective tools is the trailer suction dredge. This vessel, usually self-propelled, uses its speed and momentum, operating through a suspended drag, to excavate the material, which is then sucked up the ladder to the pump and discharged, usually into a hopper for later dumping off site. The trailer suction dredge can take long runs at a site, lower its ladder as it reaches the near edge, and cut a swath across the site in one run.

The drag may be a steel plow or it may have ripper teeth or jets or even mechanical screw cutters to help cut the soil. This is an extremely economical means of removing

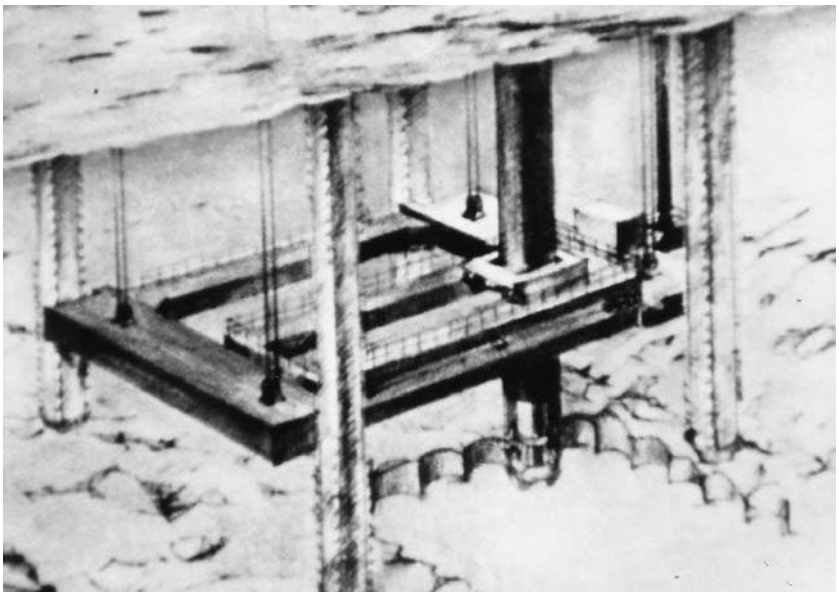

FIGURE 7.3
Leveling rock for base for caisson. Bisan-Seto Bridge, Japan. (Courtesy of Kajima Engineering and Construction Co.)

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FIGURE 7.4
Grinding rock seat for caisson at 50-m depth, Pier 7A of Bisan-Seto Bridge, Japan. (Courtesy of Kajima Engineering and Construction.)

seafloor material. It is only limited in depth to the practicable length of a ladder, in the range of 50–60 m. It can remove both soft material and partially cemented materials. Its limitation is that it is difficult to control the depth of excavation. Heave compensators have been installed in some cases to keep the drag at a reasonably constant elevation. The hydraulic cutter-head suction dredge is another tool with a long history of successful large-scale application in inland marine construction. This dredge operates most effectively when it is cutting a swath against a face of 1–2 m in height, depending on the behavior of the soil. The intent is to have it progressively cave to the cutter and suction but not to bury it. It has been found that cutting the sides of the trench or toe of an embankment "downhill" produces a more stable slope than cutting "up-hill." Dustpan hydraulic dredges have a flat plate that extends beyond the suction. Thus, there is minimal disturbance to the soils below grade. In the open sea, the hydraulic cutter-head suction dredge is very sensitive to the swell, which aggravates the movement of the extended ladder and cutter head. Use of a heave compensator to suspend the ladder is one positive step. Another is to hinge the ladder from the center of rotation of the hull rather than from its stern.

IHC of The Netherlands has developed an interesting adaptation by mounting a hydraulic cutter-head suction dredge arm (or ladder) on a jack-up rig, enabling positive elevation and position control of the cutter head. With this type of dredge, regardless of whether it is supported from a fixed or floating platform, the lateral thrust must be resisted by both the mooring lines and the legs in order to provide the necessary translation and advance of the cutter head. A monstrous walking jack-up was built in The Netherlands to permit progressive advance of the dredge, but proved too cumbersome for efficient operations. The above schemes are limited in depth to perhaps 50–60 m. By use of jet eductor and pumps incorporated in the ladder, the dredge may work to much deeper levels, but it must still be held in position by moorings (Figure 7.5).

Disposal of hydraulically dredged material in the sea may create a turbidity plume, which is environmentally objectionable. A cyclone may be used to separate out the coarser

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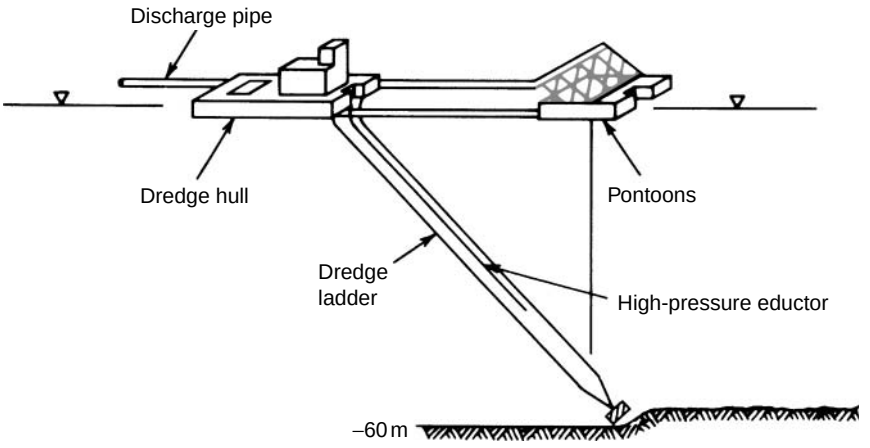

FIGURE 7.5
Deep-dredging cutter-head dredge, Skookum.

sediments for disposal by direct dumping or barge. Other systems have employed coagulants (thickeners) to precipitate suspended and colloidal materials. Another approach is to discharge down through a suspended pipe so that the discharge is at the seafloor. In the open sea, this discharge should preferably be located below the thermocline to prevent mixing with the surface waters. Deep excavations may also be performed at sea by the use of large clamshell dredges. On the Honshu-Shikoku bridges in Japan, for example, clamshell (grab) dredges have been used to excavate to depths of 50 m or more. These have very large buckets, up to 99 tn. In deep water, the cycle time for such large and heavy buckets is very long. The hoisting time may be reduced by using especially large winches for maximum line pull to reduce the number of parts in the hoisting line by increasing the line speed.

The swinging time is again long, due to the inertia of boom and bucket. In some cases, the bucket has been so arranged that it is discharged to a hopper barge moored at the stern of the dredge so that there is no swing of the boom, only a short translation of the bucket along the centerline of the dredge. A 20 m^3 orange-peel grab has been used to dig "glory holes" in the seafloor for subsea pump stations but was only moderately successful. Specially modified hydraulic dredges were then used to construct the "glory holes" up to 100 m deep to prevent damage to the well-head by deeply keeled icebergs. These proved more successful. Continuous-bucket-ladder dredges have been used to depths of 60 m in the calm seas off Thailand, digging placer sediments for tin.

The airlift, suspended from a barge or vessel, becomes increasingly efficient with increasing depth. The air pressure must be sufficient to overcome the hydrostatic head. It need not discharge at the surface; discharge above the seafloor may be sufficient. The airlift head may be augmented with jets, so arranged as to feed material to the airlift suction. However, this system is effective only over relatively limited areas and for small quantities. The airlift is especially effective when removing material from within enclosures, such as cylinder piles, provided water is continually fed in at the top to maintain the external pressure head. Airlifts are also employed to remove sand and silt from congested zones, such as around pile heads in cofferdams. They can be incorporated as air-assists in drill strings and in dredge ladders (see Figure 7.6 and Figure 7.7).

Other systems for dredging of sediments are the jet eductor (see Figure 7.8), the Marconaflo "Dynajet," the Pneuma Pump (see Figure 7.9) and the Toyo Pump (see also Section

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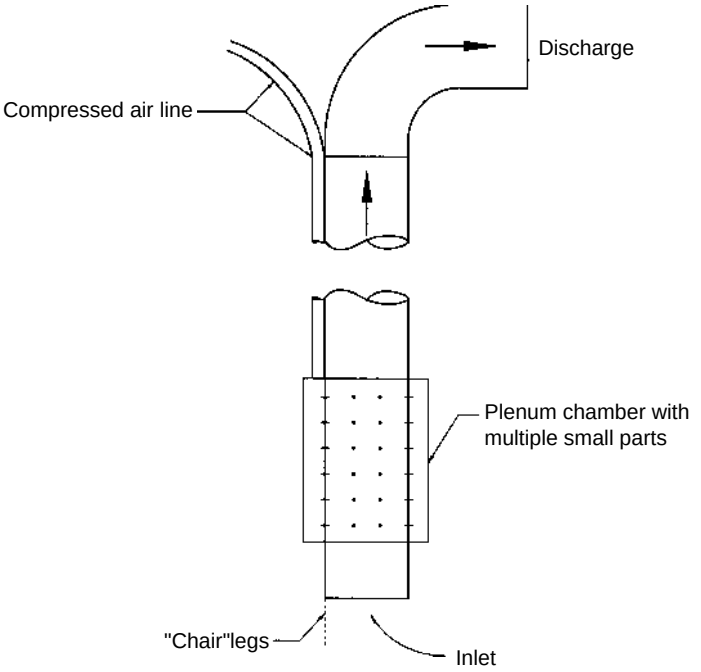

FIGURE 7.6
Airlift pump (schematic).

5.12). New submersible pumps equipped with agitators are able to move large quantities of sand at shallow and moderate depths and have proved especially effective in excavation within cofferdams and for cleaning underwater trenches (see Figure 7.10). All are very efficient in removal of material if it is loose and free-flowing. Hence, most such systems include jet systems to break down the soil structure and place the sediments in suspension.

The jet burial sled, used in pipeline burial operations, is a very efficient method of soil removal, especially in a trench or limited area. It employs the principles of high-pressure jet cutters combined with either airlift or eductor suction and discharge.

FIGURE 7.7
Large airlift pump excavating at a depth of 23 m.

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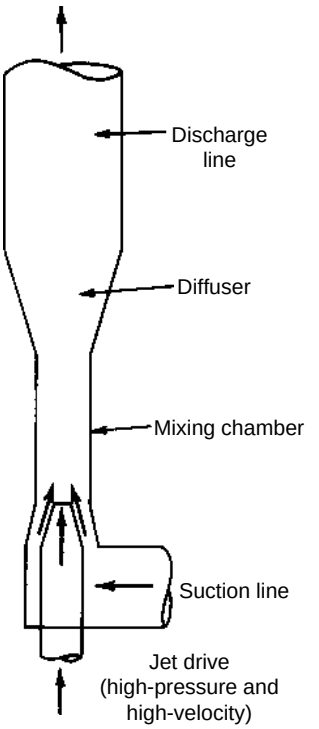

FIGURE 7.8
Jet-drive (eductor) pump (schematic).

To construct "glory holes" in very compact materials for subsurface well templates in areas subject to scouring by the heels of icebergs, an enlarged drill has been used, fitted with a rotating head of 5 m diameter. Reverse circulation methods are employed, with airlift assist, to discharge on the adjacent seafloor. The soil is partially cemented sand and gravel, and extremely dense due to compaction under storm wave action. Successive holes are overlapped to form a crater approximately 10 m deep, with a bottom dimension of 10×20 m. In the dense sands of the North Sea, a heavy orange-peel grab has been effective. Recently, hydraulic suction dredges have been modified to suspend the ladder almost vertically, with airlifts or eductors built into the ladder. These have enabled hydraulic dredging in very dense sands at depths of 100 m.

For the deep-sea manganese nodule mining operations, several types of dredge have been developed, which could have applicability to seafloor leveling. One of these employs the suspended drag principle, where a large and heavy base is dragged across the seafloor, cutting by means of jet cutters and sucking up the nodules by airlift or hydraulic transport. Another system employs the continuous-belt dragline or ladder dredge principle (slack-line dredge) whereby a slack line returns the buckets to the seafloor. This system is effective but depth of cutting is difficult to control. It tends to cut trenches.

The Amrod subsea dredge is a remotely operated seafloor hydraulic dredge, capable of operating at depths up to 300 m. The operations are controlled by a surface operator using remote electrohydraulic controls and monitoring operations by video and acoustics. One version, having a 10-in. suction, has a capacity of 500 tn./h, and is mounted on a crawler tractor.

With any type of dredge that cuts a swath or a trench, there is a tendency for the next cut or run to follow into the previously cut trench, thus producing deep trenches at the site rather than a uniform removal of soils. One means to overcome this at a specific site is to

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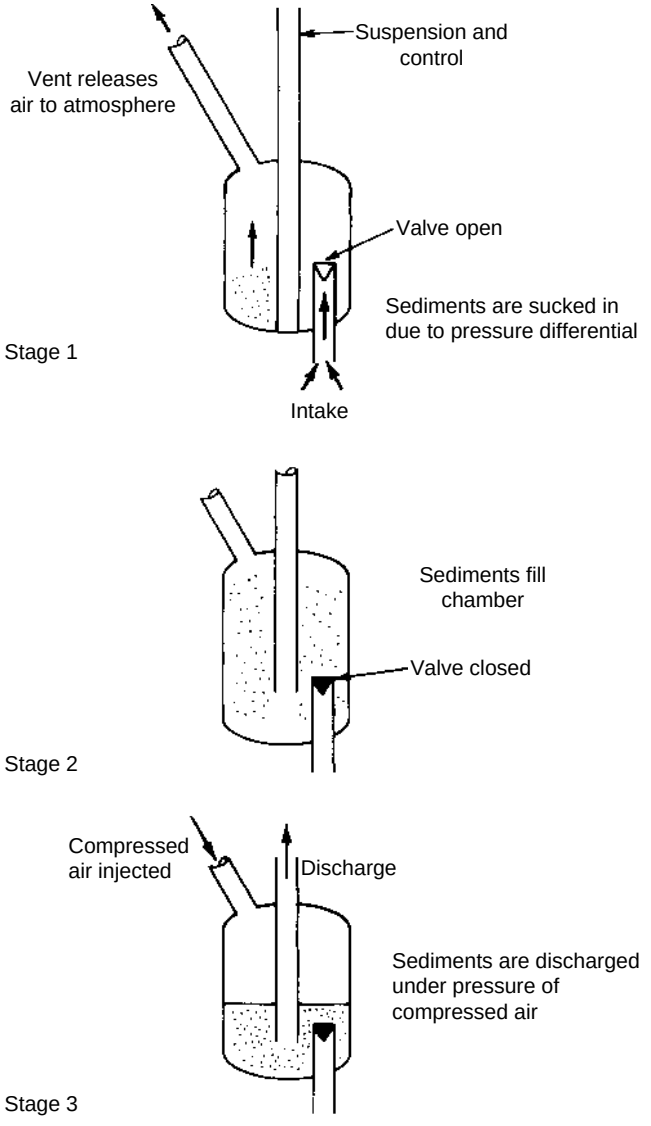

FIGURE 7.9
Pneuma's pump cycle.

make a number of runs in one direction to cover the site and then make a second series at right angles.

7.4 Dredging and Removal of Hard Material and Rock

The removal of hard material inherently requires the consideration of site-specific data such as stratification, fracture, and bedding planes. Near-surface geological information is necessary to plan such work properly.

Stratified rock, having near-horizontal bedding planes, requires particularly careful evaluation. Any dredging system that works from the top surface may involve an excessive amount of effort, whereas a method that breaks upward from underneath can be very

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FIGURE 7.10
Submersible pump with agitator removing sand from underwater trench. (Courtesy of Toyo Pump Co.)

effective. In past years, in channel and harbor dredging work, the dipper dredge was particularly efficient. Recently, large hydraulic backhoes have been adapted for barge mounting and are able to work to depths of 15 m and even 20 m. At greater depths, the slack-line dragline bucket should be able to break slabs upward.

Drilled-in explosives are generally inefficient for horizontally stratified and layered rock, as most of the explosive energy dissipates through the cracks. In this case, all that may be accomplished is to fracture the rock into very large slabs, which may make them even more difficult to remove.

In some regions, coraline caprock or limestone layers often overlie extremely soft silt. This condition occurs, for example, in the Arabian–Persian Gulf and offshore Hawaii. Explosives placed on top, such as shaped charges, can shatter the hard layer downward, but are much attenuated and hence, inefficient.

Shaped charges may be used effectively to fracture boulders and break down ledges. They have also been used to trench through rock. They must be accurately placed and held in the proper attitude so that the blast creates a cavity in the rock. Divers can place a sandbag over the shaped charge to hold it in proper position. This also improves the effectiveness of the blast. Shaped charges can also be assembled on an expendable steel frame, which is then lowered to the seafloor. Shaped charges have proved especially effective in breaking up surface caprock in order to bury pipelines. Over 300,000 m^3 of calcarenite and limestone were broken up with shaped charges in order to bury the submarine pipeline from North Rankin A on the Northwest Shelf of Australia. Shaped charges have been used to shatter massive limestone sufficiently to enable steel bearing and sheet piles to gain a toe-hold.

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Another type of hard material that must sometimes be removed in preparing a foundation is heavy boulder clay. Very large clamshell buckets with teeth can penetrate the clay and engage the boulders with the bucket (see Figure 7.11). One of the disadvantages of the conventional clamshell bucket is that the action of the bucket closing line tends to reduce the effective weight on the teeth. Therefore, hydraulic bucket-closing cylinders have been developed capable of operating at significant depths. These enable the full weight of the bucket to work downward and the full force of the hydraulic cylinders to work sideways, to close the bucket.

Another development of use with clamshell buckets in sandstones and conglomerates is the mounting of heavy vibrators on the bowls of the bucket. When the bucket is on the bottom, with teeth pointed downward, the vibrator is turned on, causing the teeth to penetrate. The proper selection of bucket size and weight, line pull hoisting capacity, and vibrator energy and duration is extremely important. There have been cases, for example, where the vibrator was so effective in penetrating that the bucket could not be raised; it had anchored itself into the rock.

Cutter-head suction dredges have been developed with tremendous power on the cutters so that they can cut soft to moderate strength rock. They have been used to deepen the Suez Canal, removing sandstone, and to mine rock salt in the Arabian Gulf. However, the action of the typical cutter, rotating around the axis of the ladder, is a grinding action that may involve excessive expenditure of energy. Another new development is the cutter that rotates around a horizontal axis (the wheel cutter head), designed to break the material upward, where it can be picked up by the suction.

In many cases the material may be prebroken to facilitate removal. For example, in boulder clay, high-pressure water jets may be used to erode the clay binder, enabling dredges to work more effectively. Water jets may be used to drill in mudstone and siltstone. Surface explosives may break the cementitious bonding of conglomerate formations. Drilling and blasting with light charges will greatly increase the productivity of hydraulic dredges in weakly cemented or overconsolidated materials.

FIGURE 7.11
Large clamshell dredge digging firm clay. Dump scow alongside. (Courtesy of Kajima Engineering and Construction Co.)

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For an intake pipeline in a deep mountain glacial lake in Alaska, the overconsolidated silt was drilled using a jet pipe and high-pressure water. These holes were on a spacing of 3 m. They were then loaded and blasted, using delays to cause the soil to break toward an open face. A hydraulic cutter-head suction dredge was then able to remove the soil efficiently, provided it was activated within one or two days after the blasting of a particular cut.

A recent development for quarrying operations on land, which may someday have application at great depth, is the use of hydraulic fracturing techniques. Short bursts of extremely high-pressure water (up to 15,000 psi) are used to propagate fractures in the rock.

The use of underwater chisels is a method of rock breaking that avoids the use of explosives. In relatively shallow water (15–20 m), the chisel may be a heavy piece of shafting, extending up above water. It may be repeatedly raised and lowered to fracture a hard layer; this rather crude but effective process has recently been employed on a large scale in Arabian Gulf port projects. In some cases, a high-pressure water jet has been incorporated into the chisel to wash away loose and broken material.

A more-controlled operation is to use an impact or vibratory hammer on top of the chisel, thus driving it into the rock. After penetrating a meter or two, the long chisel is pulled sidewise at the top, breaking off a piece of rock, just like a gigantic clay spade. On some large projects in the Arabian Gulf and the Mediterranean, a bank of such chisels is assembled along the side of the rock-breaker barge to methodically break up a hard rock layer for subsequent removal by a hydraulic dredge.

Rock-breaking chisels, driven by pneumatic, hydraulic, or vibratory hammers, can also be operated underwater. Their location must be carefully controlled. They use their weight plus impact or vibration to penetrate. Incorporation of a high-pressure jet may help to dislodge the broken rock and prevent "self-anchoring." After the rock is broken, it may be removed by clamshell.

Large hydraulic backhoes, mounted on a barge equipped with spuds, are very effective in digging soft and layered rock to a depth of 20 m.

Cutting trenches in rock for installation of power cables and small-diameter pipelines may employ rock trenchers. There are typically multiple rock saws and they are mounted on a sled that rides on the seafloor. With such a trencher, soft and even hard rock has been trenched to a depth of 30″ and more.

Plowing is a very effective method for digging a trench for pipeline burial. The plows may be very heavy, perhaps several hundred tons, and are pulled by a surface barge with heavy duty winches, such as a pipeline lay barge. The trench may be ripped before laying the pipe or the plow may be guided by a pre-laid pipeline on the seafloor. Support for the plow is by wheels or sleds that straddle the trench, controlled hydraulically to allow the full weight of the plow to function.

Drilled-in explosive fracturing has a long history in underwater rock dredging. The hole must be cased from above water down into firm material, usually to the top of rock. The casing is either driven in through the overburden or drilled in. This latter is known as the "OD Method." After the hole is drilled and cleaned, the waterproofed charges of powder are lowered with either waterproof leads or primacord attached. Sand packing is placed on top of the powder (stemming), and the leads or primacord are led all the way out of the casing at the top, with a float attached. The casing is now raised, and the leads are picked up and connected on the barge. After a series of holes have been so charged and connected, the barge pulls away 60–100 m, and the round is fired.

The effectiveness in shattering rock is greatly increased by the amount of overburden. Also, the presence of overburden makes it easy to seat and seal the casing, whereas the

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absence of overburden makes it extremely difficult. Hence, all or some of the overburden should be left in place during the drilling and shooting operation.

Typically, holes are drilled on a 2- to 3-m spacing. The spacing of holes and overdepth (below design final grade) of drilling must be carefully determined to obtain optimum results. From the dredging point of view, aimed at facilitating rock removal, it is generally best to drill below grade a depth somewhat more than half the spacing of the holes to ensure against high points (pinnacles) being left above grade. For example, a crude and conservative rule of thumb is "half the spacing plus half a meter." The reason for this is that it is almost prohibitively expensive and difficult to attempt secondary shooting on high pinnacle remnants. Staggered rows of holes appear to give better fragmentation than a rectangular grid.

For the same reason it is considered good practice to use a conservatively high powder factor (e.g., 1.2–1.8 kg/m^3) and a relatively fast powder (e.g., 60%). However, such a procedure may cause excessive fracturing of the rock below grade, which may or may not be acceptable from the foundation point of view. If not acceptable, the rock may require subsequent mechanical grinding or pressure grouting, this latter being usually done after the structure is installed. On the Tsing Ma Bridge in Hong Kong, plastic tubes were embedded in the concrete footing block. Later, they were used to drill and grout the fractured rock beneath.

If it is necessary to minimize the subsurface fracturing, then holes should be drilled on a closer spacing and to a correspondingly lesser depth below grade. Smaller charges should be employed—e.g., less than 1 kg/m^3.

Current safety regulations prohibit drilling within 15 m of a loaded hole, unless a waiver is granted. Such a waiver has sometimes been granted if a template is employed to control the spacing and verticality of the casing. However, the rules can be complied with by using a large template, to enable a full line and one or two rows of casings to be left in place until all are drilled, and then loading them. If a waiver is not granted, then several casings in a group can be used; loading is delayed until all are drilled.

Blasting of rock is always most effective if there is a face toward which the rock can break. Therefore, in some cases it may be desirable to drill and partially excavate a trench, and then progressively drill, blast, and excavate, so that there is always an open face. If there is no open face, then there is a tendency for the rock to fracture, raise, and settle back in the same compact mass. This can be particularly adverse if the powder factor is low and a slow charge is used; the effect will be to fracture large pieces without displacement. Subsequent drilling and shooting may then be very difficult, since the blast will dissipate along the fracture zones without breaking new rock.

When adjoining structures must be protected, line drilling and cushion blasting techniques are employed. Presplitting along a boundary row prevents extension of the fractures. Use of delays can ensure that the blasted material moves in the desired direction. Air bubbling reduces the water shock effects. By placing a double row of air bubblers at 3 m spacing, and 3 m outside the structure to be protected, the peak water pressure can be reduced by a factor of 10.

Particle velocity can be limited, e.g., to 12 mm/s, and water overpressure should also be limited, e.g., to 0.5 N/mm^2 at the face of a structure. Stemming the top of the hole with a depth in the top rock equal to half the hole spacing is effective in reducing damage and turbidity. Test blasts can be utilized to establish an effective value for powder factor.

The blasting of solid rock produces fractured material having typically 40%–50% greater volume, thus raising the level of the seafloor in the blasted area. If the material is to be removed by bucket dredging, the bulk quantity will be increased accordingly. This also applies to the rock, which has been blasted below grade. Thus, extensive areas of shallow

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rock removal may involve a 100% or more increase of the dredged quantity as compared to the neat solid rock volume.

For deep offshore work, either rotary or percussion drills are employed. Rotary drills are best for deep drilling in competent rock, whereas pneumatic drills are most useful in irregular, erratic material and shallow drilling depths. Pneumatic down-the-hole hammer drills are proving effective and economical in underwater shaft construction and the construction of rock sockets for piling. Figure 7.3 illustrates the use of a rotary drill, supported by a large jack-up, to level the rock bottom for the anchorage pier of the Bison Seto Bridge between Honshu and Shikoku.

When a deep trench is to be excavated underwater, drilling and blasting to a full face (full depth) has proven better than performing the work in stages. It is difficult to seat a casing and maintain circulation in rock which has been pre-fractured. Delays may be used to reduce external damage.

Intake shafts have been constructed in the deep water of existing lakes, while outfall risers have been constructed in the subsurface rock of coastal waters. These may be drilled with full-face rotary drills up to several meters in diameter. An alternative means is to construct a seafloor template for guidance and then drill numerous 300- to 500-mm-diameter holes by a down-the-hole drill. These are very closely spaced, to create a "Swiss cheese effect." The interstices are then broken down by a spud chisel. In all cases, the hole has been cased and reverse circulation methods employed (see Section 10.2.)

Drilling and blasting can also be carried out by divers and/or submersible work vehicles. This has been effective only for relatively small, isolated features and shallow depths of work, such as isolated boulders on the seafloor. However, divers are generally not able to carry out major operations over an extended period of time. Their effectiveness is also limited by their inherent buoyancy. Underwater tracked vehicles have been experimentally developed to carry out the drilling from the seafloor. For any major project, working from the surface is currently the most efficient and economical.

Considerable work has been carried out with acoustic underwater blasting devices to eliminate the need of bringing leads or primacord up through the water for collection on the barge. These devices appear to be reasonably reliable, provided they do not become silted over. They were effectively used on the rock excavation for the piers of some of the Honshu-Shikoku bridges in Japan.

Subsequent to blasting and excavating rock, it is generally necessary to clean up the foundation by removal of silt, sand, and small rock fragments. This is best done by a straight suction dredge or airlift, aided by high-pressure jets.

Turbidity has recently become a major concern for underwater blasting. Tests in Japan have shown rapid sedimentation of coarser silts, but progressively lower rates for fine silt fractions and clay particles. For these, silt curtains are employed. The most effective curtains have been supported on a large structural frame, through which the bucket is raised and held above to drain the water before swinging. When fine material is discharged underwater, it should be led down a tremie to discharge just above the bottom.

7.5 Placement of Underwater Fills

Underwater fills of granular material such as crushed rock, gravel, and even sand can be used to provide a reasonably level and uniform support for structures at a practicable and economical level. For example, they can be placed over irregularities and outcrops or used to fill depressions from which unsuitable materials, such as mud, have been removed. In deep water, they can be used to raise the base of the structure to a more favorable

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elevation, from a standpoint of economy, while still staying well below the elevation at which the design wave will have destructive effect. Such fills can also provide a foundation of known static and dynamic properties from the points of view of shear strength, pore pressure buildup, and resistance to liquefaction.

Protective islands are built surrounding bridge piers, to prevent ship impact. Fill is placed around and over submerged tunnel segments and outfall sewers. Embankments are constructed to channel rivers, for approaches to piers, and behind wharves as the storage yard for containers as well as a dike on which to construct the wharf. Fills are also placed underwater to form the core of a breakwater.

Underwater fills of crushed rock or gravel may be used to blanket an area to contain unstable sands and allow pore pressure relief without sand dispersion. They may also be used to laterally confine unstable materials such as clays, acting as counterbalancing surcharges external to the structure, thus preventing local shear failures. Underwater rock blankets may be used to cover over irregular rock outcrops to permit a structure to be founded with uniform bearing. A clay blanket may be used to blanket contaminated soils.

Underwater rock dikes, placed prior to or during dredging, may be used to stabilize side slopes against shear and erosive failures. In such cases, the underwater rock dike migrates downslope and into the sand as the dredging takes place, serving to give steeper and more stable slopes. These rock dikes are known as "falling aprons." They are most effective when no filter is used beneath; thus, the sand progressively migrates through the rock, allowing the apron to fall on a regular pattern. Clay dikes, using stiff glacial clays, have also been used to retain underwater sand fills. The use of underwater fills to prevent scour and erosion around structures is dealt with in Section 7.8.

The materials for underwater fills have to be selected with regard to their suitability to the needed objectives, their density and size gradation, and their ability to be placed at the depths and locations desired. Obviously, availability and cost are also factors.

Underwater fills are often placed by discharge from a hydraulic or hopper dredge (see Figure 7.12) or by dropping from a bucket, to fall through the water. When low-relative-

FIGURE 7.12
Discharging sand from hopper dredge to construct underwater sand island for caissons, Beaufort Sea, Canada.

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density materials are placed through water, they tend to disperse laterally and to fall through the water at differential speeds. The result is to segregate in layers of different size. In addition, the in-place density of such material is heavily dependent on the permeability and relative gradation of the particles. Relative densities of cohesionless materials (sands and gravels) placed through a substantial depth of water may vary from 40 to 60%, with 50% being most common. Lateral spreading is dependent on specific gravity, gradation, particle shape, depth, and currents, but, in general, slopes are very flat. As fine sands impact, they temporarily liquefy, allowing them to flow locally as a dense fluid. Silt or mica content is very critical: lenses formed during deposition can lead to slope failures at a later stage. If these fills are later cut with a bucket or cutter head, they will stand at a temporarily steeper slope but will be very sensitive to shock and vibration.

Air content at the time of placement has a very significant effect on segregation, spreading, and density of in-place underwater fills. The air bubbles attach to fine particles and give them added buoyancy. The tendency for such segregation can be reduced by thorough saturation of such materials prior to placement.

Sand may be discharged down a tremie pipe whose end is fitted with a special device to force the sand and water to separate and hence enable a steeper slope to be attained. Among the special devices employed are screens of "fabric" mesh, whose openings allow the water to flow out freely while tending to restrict sand passage, and a wide bell-mouth fitting which reduces the exit velocity. At the Tarsiut Island in the Canadian Beaufort Sea, for example, use of such a device resulted in a side slope of 5.5:1 as compared to slopes of 10:1 and even 15:1 when the sand was discharged at the surface.

If an underwater fill is to contain sands and prevent sand dispersion through the fill, then the material should be graded as a filter. In practice, this is extremely difficult to accomplish. One approach is to select a well-graded material similar to a concrete mix that will act over the complete range both as a filter and as a stabilization and erosion protection; some fines will be lost, but the remainder will stabilize. Another solution is first to cover the area with filter fabric mats with articulated concrete blocks attached. Then, stone can be dumped over the mat. Mats can also be held in place by sandbags or steel pins set by divers. In very deep water, filter fabric mats may be pre-attached to the structure prior to seating it or may be attached to steel or concrete frames or panels.

One of the best methods of placing underwater rock fills in the ocean is by bottom-dump or side-dump barge. By pre-saturating the material prior to dumping, segregation is minimized. The mass of the rock hangs together as it falls through the water, thus attaining the terminal speed of the mass, which is considerably faster than that of the individual particles. The impact of the mass on previously placed material helps to consolidate it.

Tests in The Netherlands by ACZ Marine Contractors have shown that a mixture of stones with a maximum diameter of 0.2 m (200 mm) will reach a maximum terminal velocity of 2.0–2.5 m/s. However, when dumped as a mass from a split hopper or similar barge, the entire mass hangs together, developing a fall velocity about twice that of the individual stones, that is, 4–5 m/s. These terminal velocities are reached within a relatively short fall distance (e.g., 5–10 m) regardless of the initial velocity of the rock falling through air or through a pipe.

Other methods that have been employed involve placement by clamshell bucket or skip lowered to the bottom before opening (a very tedious process) and placement through tremie pipes. These latter are usually suspended from a barge or from a pontoon-supported hopper, laterally restrained by lines to the barge. The pipe must have a large enough diameter to avoid any possibility of plugging: a value of three to five times the diameter of the largest rock pieces is often used. Gravel is less likely to plug than crushed rock. Elongated particles are unsatisfactory. Another method of placing rock at depths is

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through an inclined chute such as the modified ladder of a trailer suction hopper dredge. The discharge end is suspended from the vessel by a heave compensator and can be directed to the proper location.

Dutch dredging contractors have developed a number of rock-placing vessels, which are equipped to place rock either by direct dumping or through a controlled tremie pipe (see Section 15.14).

The selection of proper particle size for long-term stability requires consideration of bottom current velocities due to combined tidal and general currents and storm-wave-induced currents. The effects of the structure itself in generating vertical and eddy currents must be considered. During a storm, the pore pressures in the soil will fluctuate and make it easier for fill particles to be temporarily placed in suspension.

Another consideration, of course, is the packing factor, which is determined by particle shape, gradation, and degree of consolidation of the fill. Relative density is extremely important, since we are dealing with the submerged density. Use of a rock consisting of iron ore mineral compounds and having a specific gravity of 3.5 or higher is much more effective and stable than the typical silica rock with a specific gravity of 2.6, since stability varies approximately as the cube of the underwater net density. Larger sizes are also more stable and can serve to lock together a fill of varying gradation.

When fills are placed around a structure—for example, when sand is discharged down a tube as backfill for a pipeline—the fill is temporarily a heavy fluid and has the flow and displacement properties of a fluid. Thus, it can run under the pipeline and lift it up or displace it sideways, just as if it were a fluid having a specific gravity of 1.5. A number of major pipeline installations have been seriously dislocated or even ruptured during backfill in this manner. Of course, the material quickly returns to a solid once the excess pore pressure has dissipated, which usually occurs in a few minutes, but the damage may already have been done. This same problem, with even greater consequences, can occur when underfilling and backfilling prefabricated submerged tunnel segments causing the tunnel section to float upward or sidewise.

The proper use of underwater fills would appear to be a major opportunity in the extension of structures to deep water, to less competent soils, and to more exposed locations. Properly employed, the fills can enable construction to be carried out at a more favorable elevation with materials of known and controllable properties.

Underwater rock dikes placed around a structure can be used to prevent collision from ships, to cause icebergs or deep pressure ridges to ground, and to divert mudslides and turbidity currents. They can also, as mentioned earlier, confine soft soils such as clays, in order to lengthen the shear path and to resist local shear failure due to high bearing pressures. Particularly attractive is the fact that in many cases the underwater rock fills can generally be placed during the period in which the structure itself is being fabricated; thus, the work is not on the critical path.

In-place sands and gravel may be so loosely consolidated that they will be subject to liquefaction under prolonged storm or earthquake. Densification procedures are therefore employed to raise their relative density above the critical value (about 70% R.D.) Fills such as those described in the preceding section may also require consolidation in order to reduce settlements and to ensure stability. There are a number of methods and techniques by which soils and fills may be consolidated. One of these, which applies to granular, cohesionless materials, is vibratory consolidation. A large mandrel—up to 1 m in diameter, for example—is inserted into the material by either jetting or vibratory driving. The mandrel has horizontally oriented vibrators mounted in its tip, which are now activated, imparting high-frequency energy into the adjacent soil. This causes the sand particles to reorient themselves. Pore pressures are increased and then relieved by

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drainage through the adjacent fill. Stone columns may also be installed to prevent liquefaction (see Section 7.6).

Several brands of internal vibratory compactors are currently manufactured, some of which are able to work entirely underwater. These are able to penetrate and consolidate material ranging from 75 mm diameter down to fine sands. Piezometers should be installed and the pore pressures monitored.

It is essential that the pore water be able to drain. Layers of silt or clay will prevent drainage and seriously limit the efficacy of the vibration. If these blanketing layers exist, vertical drain paths such as gravel drains must be provided. These can be installed as drilled wells or even jetted into place. There must also be a horizontal escape path for the water that is expelled; this is usually a sand or sand and gravel layer preplaced on the seafloor, under the base of the structure. Alternatively, drainage into the structure may be provided from where it is pumped out.

Internal vibration does not compact the near-surface layer. For this layer, a vibratory plate compactor must be placed on the surface. A large vibratory plate compactor was employed on the Oosterschelde Storm Surge Barrier, where it successfully compacted stone of 350-mm maximum size in layers up to 2 m thick (see Figure 7.13a). A similar plate vibratory compactor was used to densify the soils on which the Great Belt main pylons were founded.

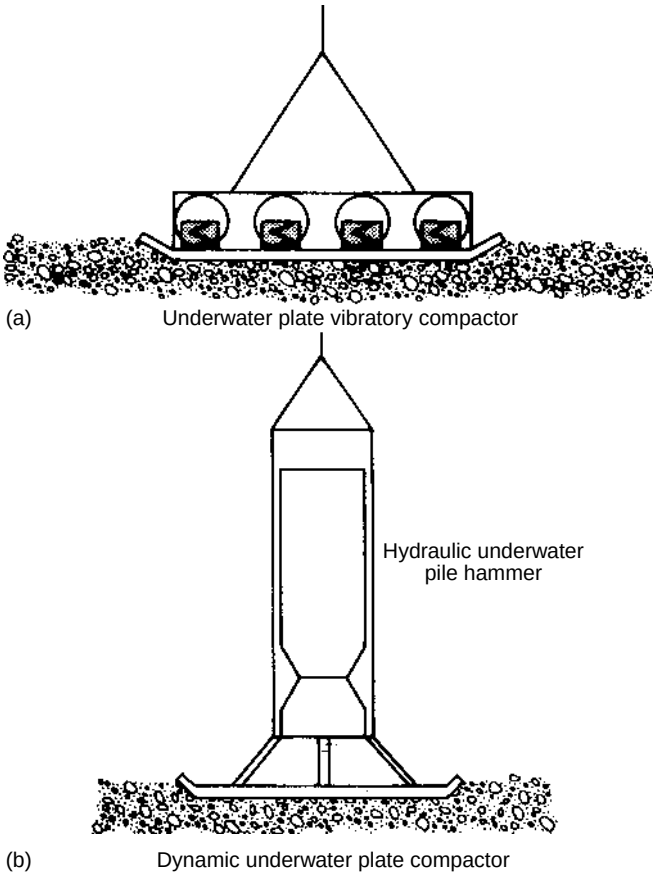

FIGURE 7.13
Compaction equipment for underwater gravel and stone fill.

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Other methods of surface compaction utilize adaptations from above-water landfill practice; these include a roller compactor and a remote-controlled underwater bulldozer. These, however, have so far been limited to relatively shallow water. Another crude but effective tool consists of a long shaft or pipe with a plate on the bottom. A pile hammer or vibration hammer is attached, enabling effective control (see Figure 7.13b). Another method, adaptable to a wide range of material sizes and gradations, is dynamic compaction (the "Menard system") which involves the repeated raising and dropping of a heavy weight. Depending on its mass, density, and distance of fall, this dynamic compaction can effectively consolidate up to as much as 10 m of underwater soils or fill. This has been used successfully on both sand and rock underwater embankments. However, the effect of the shock on the adjoining sediments must be considered, so as not to cause large-scale liquefaction and flow slides.

Air guns, as used in geophysical seismic surveys, may be used to densify loose sands; some experimental work has been carried out. It is important to recognize that this shock causes high pore pressures to build up instantly. These are then relieved by drainage through permeable materials or through shear fractures in relatively impermeable materials such as silts. This latter is, of course, normally undesirable and may result in slope failures.

Similar "shock" or dynamic compaction can be achieved by use of explosives. These are installed by jetting a casing into the fill to about two thirds of the depth of the stratum to be densified. The size of charge is limited so that craters will not form. Typical spacings of holes are 3–8 m. Delays should be used to separate the times of successive impact.

A number of sub-aqueous tunnels ("tubes") have been constructed of precast concrete segments sunk in a seafloor trench and backfilled by sand and gravel. This granular backfill was placed through the water column by tremie, skip, or direct dumping and not densified, so it was in a loosely compacted state. In earthquake, this backfill could liquefy, allowing the tube to be raised or shifted. Temporary lack of shear resistance would allow longitudinal (axial) movement, rupturing joints between segment. Therefore, in seismic areas, the backfills are being densified.

Stone columns are being installed, staggering their installation so as to avoid unbalanced loads on the tube. Pore pressures need to be monitored by piezometers. In-place densities can best be determined by standard penetrometer tests, a corrected reading of 30 blows per 300 mm (12″) being generally considered free from liquefaction.

To prevent uplift of tube segments, communication of pore pressure from the sides must be limited, since there is no way to densify below the tube. Thorough densification of the lower side backfill is generally considered to be the most practicable but requires careful positioning of stone columns and vibrates alongside the tube. The advanced electronic positioning devices described in Section 6.7 appear applicable.

7.6 Consolidation and Strengthening of Weak Soils

Sand piles or stone columns are installed by drilling or driving into clays and silts in order to strengthen them by increasing both bearing and shear resistance and to prevent liquefaction. Such piles are usually installed by a driven pipe mandrel into which the granular material is fed. This material is then forced out with air pressure as the mandrel is withdrawn. Since only limited depth of stone column penetration is usually required—that is, only the weak soils are involved—the process is very rapid. Typically, 1-m-diameter "piles" are placed on 2- to 3-m centers. These are variously referred to as "sand drains," "sand piles," and "stone columns." A similar process uses a jet to assist in sinking the pipe

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mandrel, and then feeds gravel into the column as it is withdrawn, while vibrating intensely.

Vibratory compaction, as described in the previous section, is extremely effective in consolidating loose sand deposits. Perhaps the most extensive use to date was on the Oosterschelde Storm Surge Barrier, where four such compactors, which were mounted on a barge, compacted the loose sands to a depth of 50 m below sea level. For the piers of the Great Belt Bridges, heavy vibratory compactors were used to compact the stone-fill base, working in 1-m layers to achieve 80%–90% relative density.

An effective means of consolidating weak soils is by surcharging, with subsequent removal or redistribution. An underwater fill of sand or rock can be placed by bottom-dump barge and allowed to exert its excess pressure on the soil for a period of six months to a year or more. The consolidation will be even more effective if the pore water has an opportunity to drain through natural or artificial permeable drains.

Silts and clays may be consolidated by means of drainage using vertical sand drains or wicks. This method is especially applicable if the soils are anisotropic with good natural horizontal permeable stratification, as exists in many locations where the sediments have been deposited in successive layers. This system, much used in land operations, can also be used underwater if accompanied by a surcharge.

Sand drains can be drilled in or jetted. Wicks are installed by jetting or by attachment to a dropped shaft, which penetrates dynamically. Of course, after the structure is in place, drainage wells can be drilled in and even pumped to remove excess water during consolidation. The water that is driven out of the soil moves upward through the drains and must then have a means of escaping laterally. Thus, a blanket of coarse sand and gravel should first be installed over the seafloor area. This blanket can also act as part of the surcharge.

The vacuum process has been used to accelerate the consolidation by drainage of above-water fills. After the wick drains have been installed, an impervious plastic membrane is laid over the area and sealed at the edge by fill. A vacuum is then drawn and maintained. This process has the advantage that it can be used over very weak soils for which the more conventional surcharge might cause shear failures. It was employed on the new terminal at Neva Shiva in Bombay.

With or without artificial drainage the surcharge effect of a fill over a period of six to twelve months may serve to stabilize silts and possibly even clays to an acceptable strength to receive and support the structure. Then, just prior to the installation of the structure, the surcharge fill can be spread laterally to a peripheral location, where it will lengthen the shear path and act as a counterbalancing force against bearing failure.

Cementation of underwater granular soils can be carried out by injection of cementitious material, following land-based grouting procedures. The cementing pressures must be regulated to displace the pore water yet not cause "fracturing of the formation" through channelization. Cementitious particles must be small enough (fine-enough grind) to penetrate the interstices. Addition of a wetting agent to the grout, which reduces the viscosity, and colloidal mixing both facilitate permeation.

If the cement can be mixed with the soils, even cohesive soils can be stabilized. Various techniques have been developed to accomplish this, usually based on use of an auger-type drill that is jetted and drilled into the clay. The jet water is then turned off and a thin cement slurry injected, to be distributed by the mechanical action of the auger drill. The Japanese have developed several such methods and have applied them to shallow seafloor soils. Jet grouting and deep cement mixing appear to be the most efficient systems for stabilizing weak clays and silty sands. Jet grouting replaces the soil column with a cement grout while deep cement mixing mixes cement into the soil and thus, minimizes the volume of spoil.

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Weak columns of cement mixed with soil (DCM) may be constructed as secant walls or to form cells about 4 ×4 m in plan. These can stabilize seafloor soils and slopes due to the shear developed between the columns. This system has proven very economical at the Port of Oakland, California. However, the strengths are very low. The behavior in earthquake has not been clearly established.

Jet grouting was used to prepare the foundations for the bridge across the Mississippi at Cape Girardeau, Illinois. Deep karstic fissures in the limestone, filled with silt, were discovered after the foundation was excavated. By replacing the silt with grout, in an extensive operation, the massive pier was successfully constructed.

For the underwater vehicular tunnel between Busan and Geoje in Korea, jet grouted columns will be installed to provide support and prevent excessive settlement.

Another method is based on the injection of cement slurry under relatively high pressure into clays and silts. The soil is first fractured by high-pressure water, allowing multiple lenses of cemented material to become interbedded, thus increasing the soils shear resistance.

A similar method of injection used quicklime (CaO) which is placed in weak clay or organic soils. It is placed in an augured hole in a polyethylene, or similar, sheath. The quicklime draws water from the surrounding soil and is converted to calcium hydroxide, with significant liberation of heat. Stable calcium compounds such as calcium carbonate are formed among the clay particles.

Use of chemicals other than limes and cements is also practicable in permeable and slightly permeable soils. For many years, injection of sodium silicate followed by injections of calcium carbonate has been used for stabilization. In some calcareous soils, if they are sufficiently permeable, a single-stage injection of sodium silicate may be adequate, being fixed by the free lime in the soil.

Organic polymers may also be injected, either as polymers or as monomers, which will later convert. These monomers are usually highly toxic, and hence their effect on adjacent marine life must be considered. Some of these have high penetration qualities, which make them very effective in low permeability clays and silts. Shell Oil International has developed a penetrating polymer known as Eposand. The soil is first flushed with fresh water, then with an organic solvent, following which the Eposand is injected. This process was used to provide temporary strengthening of calcareous sands under the tips of the piles of the North Rankin A platform, located in Australia's Northwest shelf.

Freezing has been employed for strengthening the soils and embankments in the Arctic (see Section 23.8) and has more recently been studied in detail for application at the North Rankin A Platform referred to in the preceding paragraph. In seafloor soils, special consideration must be given to the progressive concentration of brine lenses and pockets as the freeze front advances. The effect of the low temperatures on steel piles must also be considered.

The various methods indicated in this section for consolidating and stabilizing seafloor soils have the potential of increasing both the bearing strength and the shear strength by a factor of 2–5. Thus, even very weak soils may show significant increases in their capability to provide support for a structure and to reduce its movements during storms. Underwater stabilization of clays against potential mudslides or slumping has been accomplished by driving multiple piles on very close spacing. These tie the layers together, resulting in a reinforced earth mass. This method has been used by the U.S. Corps of Engineers to stabilize a clay embankment on the upstream face of a dam, by the Japanese to prevent shear failures under petroleum storage tanks, and is proposed to stabilize the varied sand and clay lenses under the Venice, Italy, storm-surge barriers.

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This system was utilized most successfully to stabilize the foundation for the Rion-Antirion Bridge in Greece, driving 20-m long steel piles on 3-m centers at a depth of 70 m below sea level. Then a 3-m-thick gravel blanket was placed and the gravity-base structure set on it.

7.7 Prevention of Liquefaction

Liquefaction occurs primarily in loose sands, when internal pore pressures build up and cause the soil to momentarily behave as a heavy fluid. This phenomenon often occurs in earthquakes. It also occurs with seawalls and gravity-based structures under storm waves: the rocking of the structure transmits the energy into the soils below. Outfalls and buried pipelines may be raised.

Densification is one means of preventing liquefaction. This is discussed in the previous section. The other means, which will be discussed here, is provision for free drainage, thus allowing the excess pressure to freely escape. In hydraulic fills of sand, for example, provision of vertical gravel-filled drains may be adequate. Wick drains are often installed. Both of these will aid consolidation as well as provide drainage of excess pore pressure.

Some of the large gravity-based structures in the North Sea are placed on foundations containing strata of fine sands, either at the surface or interbedded between clay layers at shallow depths. To prevent pore pressures building up under the rocking action of the structure due to waves, drainage pads and wells have been installed underneath the base of the structure. The drainage into the structure is controlled so that the ambient pore pressure in these sand strata is less than the hydrostatic head; thus, even under storm conditions, excess pore pressure is prevented.

A very effective method to prevent liquefaction of soils under the edge of a caisson or other seafloor-supported structure is by the provision of a peripheral apron of graded rock, which allows the pore pressures to dissipate without inducing flow in the sands under the rocking of the structure due to waves. The use of a filter fabric covered by an articulated mat or by rock, as described in the next section, is also highly effective in preventing liquefaction under the edge of a seafloor-supported structure.

Liquefaction of silts and sands has recently been recognized in the construction of steel sheet pile cofferdams, especially when powerful vibratory hammers have been used. Pre-installation of drainage or pre-drainage of ground water may be necessary (see Section 9.4.5).

7.8 Scour Protection

To prevent erosion due to currents, steady and transient, around structures and pipelines, scour protection in one form or another is required. At one end of the spectrum, sacrificial material such as sand is added around the structure or on the berm of an embankment. This is especially applicable to temporary structures such as islands for use in exploratory drilling, designed to last one or two years, in the shallow waters of the Beaufort Sea. Permanent sand islands have been constructed around anchorages for suspension bridges to protect them from scour; those around the Great Belt Bridge in Denmark were carefully shaped for optimum hydraulic flow characteristics.

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The most common form of scour protection is by the placement of rock of a size suitable to withstand the currents without dislodgment. This may vary in size from gravel on a seafloor at a depth of 20–30 m to 10-tn. rock in the surf zone. The stability of rock varies as the cube of the buoyant density, that is, specific gravity -1. Hence, trap rock with a unit weight of 190 lb/ft.^{3} was specified for the breakwaters of the Atlantic Generating Station, planned to be built off the coast of New Jersey, in lieu of somewhat less expensive silica rock at 165 lb/ft.^{3}. A similarly sized piece of trap rock has a 50% greater stability factor than that of silica rock. Similarly, iron ore has been considered for protection of a river bottom against scour, where the size of rock was limited by other considerations.

In general, the larger the individual pieces, the greater their stability. However, interlocking of pieces is also very important, with blasted polygonal rock being much more stable as a mass than similarly sized cobbles and boulders. Hence, chinking of the crevices and even filling them with concrete can add to the stability as long as there is sufficient permeability (porosity) to allow excess pore water pressures to dissipate.

Wave impact creates hydraulic ram effects that temporarily raise the internal pore pressure. A breaking storm wave can create a hydraulic ram effect that can (and has) hurled a hundred-ton block over the breakwater! Breakwater armor typically fails outward, at least initially, due to these effects. Significant wave forces can extend to depths as great as 100 m, where they can create internal pore pressures of 3–4 tn./m^2 (30–40 kPa). Under wave action, the sand immediately under rock fragments liquefies. The sand particles then migrate through the rock and are washed away, allowing the rock to work its way down into the sand. To prevent this, filter courses or a mat of filter fabric are placed, graded so that the sand will not work its way through the rock.

The *Shore Protection Manual* of the U.S. Army Corps of Engineers gives specific guidelines for the sizing and gradation of rock to serve as filters. Placement of several layers of rock of different gradations is difficult enough in the calm water of harbors but much more so in the open sea. It becomes necessary to increase the thickness of each layer to compensate for the tolerances in placing. The most practicable way, albeit inefficient in the use of material, is to mix all the materials of different sizes to grade from fine to coarse and then place as one combined layer. A mix of aggregate suitable for concrete (but without the cement) has been frequently used to stabilize the bottom of a shallow cofferdam.

Filter fabrics have today been widely adopted. These fabrics have specified sizes of pores, which will allow water, but not sand, to bleed through. To give adequate strength to such fabrics to accommodate differential movements, wave and current forces, and the strains induced during installation, the finer fabrics should be backed up by heavier-mesh polypropylene. The heaviest such material even has embedded stainless steel strands. Fabrics of the two types may be sewn together and laid as a unit.

Most of the filter fabrics are lighter than water and hence, difficult to place underwater. Concrete "dobe" blocks may be attached with stainless steel staples. The Dutch automated the manufacture of such articulated mats for the Oosterschelde Storm Surge Barrier. In Canada, special filter fabrics are manufactured, which are denser than water and hence, more easily installed. These mats may be laid as rolls, unreeled off a large drum, and spread onto the seafloor (see Figure 7.14). Alternatively, they may be assembled on steel pipe frames, say 20 $\times$20 m in plan, and set on the seafloor. Mat segments such as these were installed around the offshore terminal caissons at Hay Point, Queensland, Australia, and have successfully withstood the scouring attack generated by cyclones, as well as high tidal currents. To protect the underwater embankments adjoining the Jamuna River Bridge, Bangladesh, fascine mattresses of wood staves, filter fabric, and stone were placed on the slopes (Figure 7.15).

To protect underwater slopes that cannot be dredged during construction, the "falling apron" concept can be applied. Under this scheme, coarse aggregate and riprap, mixed,

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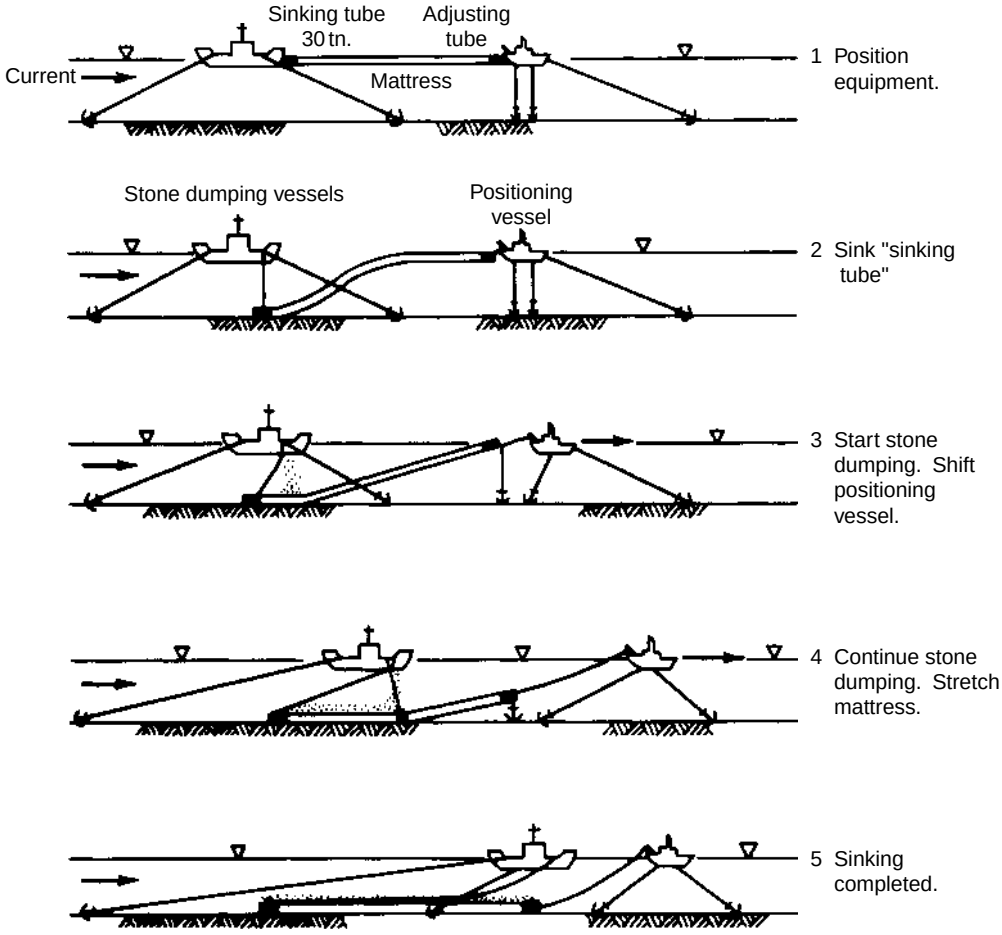

FIGURE 7.14
Sinking protective mattress with stone cover. (Courtesy of Van Oord-ACZ.)

are placed in a relatively thick layer (1–3 m) up to 10 m wide on the seafloor at the toe of the slope that can be excavated and protected by conventional procedures. As the current undermines the falling apron, the stones fall and provide protection.

On caisson-type structures, the filter fabric may be pre-attached to the caisson near its base. After seating of the structure, the mats are cut loose and laid out onto the adjacent seafloor. This was successfully carried out on the Ekofisk Oil Storage Caisson. Filter fabric mats must adequately overlap at joints; this presents some difficulties when trying to lay around the corner of a structure. Rock riprap can subsequently be dumped over the mattress.

Other ingenious forms of scour protection have been developed. Holes left in an external wall of the structure near the seafloor may develop countercurrents that tend to cause deposition of sand rather than erosion. Steel pipe frames, cantilevered out from the structure just above the seafloor and hung with multiple strips of plastic, can act as artificial seaweed, slowing the local currents, causing deposition.

Much experimentation has been carried out on the laying of a mat of concrete or asphalt over the seafloor in order to stabilize it. The Dutch have applied asphalt-sand and asphalt-stone mixes on many of their coastal dikes. These are usually designed to have

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FIGURE 7.15
Fabricating fabric mattresses of staves, filter fabric, and stone, which will be dragged out over underwater slope for scour protection, Jamuna River Bridge Project, Bangladesh.

both flexibility to accommodate local movements and porosity to relieve excess pore pressures. German and Japanese engineers have developed special viscous concrete mixes and anti-washout admixtures to enable a permeable concrete mat to flow out over the seafloor without segregation.

Where rock riprap of the required size is unavailable or excessively costly, concrete armor units have been used. There are at least forty different shapes that have been developed, of which the Tetrapod, Tribar, Stabit, and Dolos are among the best known. The Dolos has the best hydraulic characteristics and least material quantities but is weak structurally in the larger sizes. Breakage of the 60-tn. Dolos units has been assessed as the principal cause of the failure of the Sines offshore breakwater in Portugal. The Waterways Experiment Station of the U.S. Corps of Engineers at Vicksburg, Mississippi, has developed a new breakwater armor unit (corelok) that appears to optimize hydraulic and structural performance. Reinforcement has been used for some armor units, and experimental application of steel fibers in the concrete mix has been tried on the Crescent City breakwater in California, with apparent good results. However reinforcement must have adequate cover to provide protection against corrosion. Incipient cracking of the massive concrete units may be minimized by insulation during the cooling down period in manufacture.

Burlap bags filled with grout are often used to seal underwater crevices such as those produced by scour. The cement paste oozing from burlap bags knits adjacent bags together and allows them to be placed to form a low barrier wall.

For offshore sand and gravel islands in the Arctic, filter fabric has been laid over the slopes near the waterline and then 2- and 4-m^3 sandbags of polypropylene have been laid in one or two layers. The polypropylene is pigmented to prevent ultraviolet (UV) disintegration. Significant damage has occurred, however, due to sea ice attack on the sandbags, especially while they are still frozen. Articulated concrete blocks over heavy filter fabric are believed to give more permanent protection. These have been used on the

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Endicott Production Island in the Alaskan Beaufort Sea with generally good performance (see Section 23.8). Articulated concrete mats have prevented scour around caissons on the coast of Queensland, Australia and are now routinely applied to protect the banks of the Lower Mississippi River.

Another form of slope and scour protection is formed by tubular sacks of plastic. These are laid out on the surface and pumped full of grout. They are joined by integral webs of plastic, having small holes to allow excess pore pressure to escape. For the Oosterschelde Storm Surge Barrier, mattresses were fabricated, having three layers sewn together. These layers were filled with sand and stones to form a graded filter. From the constructor's point of view, the use of the fabric mat with articulated concrete blocks attached seems to be the easiest and most positive method of providing scour protection for offshore structures.

Permanent scour protection is generally the province of the designer. However, it will often be found necessary for the constructor to place temporary scour protection. An example is the landing of a caisson on a sand seafloor where the local currents are quite high; these could then accelerate under the structure's base and cause serious scour just prior to touchdown. For example, on the main pier of the Columbia River Bridge at Astoria, Oregon, the combined currents of the river and tide eroded the seafloor around and under the cofferdam as it was being constructed.

The main pylon pier of the Akashi Strait Bridge was undermined by the high currents, possibly augmented by rocking and partial liquefaction during the Kobe earthquake. Grout-filled bags were constructed to form a perimeter wall and grout injected underneath.

On the box caisson pier for the Øresund Bridge between Denmark and Sweden, storm-induced currents and waves undermined the pier shortly after it was installed. It required an extensive grouting program to fill under the base. A filter course of stone was then placed around the pier and covered with heavy riprap. Many similar cases of undermining of bridge piers have occurred on rivers during floods, since the scour increases exponentially with the velocity of the current.

7.9 Concluding Remarks

As structures are being extended into ever more difficult environments, greater depths, and less suitable soils, it is believed that increasing attention will be given to methods of modifying or improving the existing seafloor and the foundation soils, by densification, consolidation, drainage, and reinforcement.

Application of seafloor modification and preparation concepts and methods for accomplishment are summarized in Table 7.1. Further specialized methods for improvement of the seafloor are discussed in the chapters on gravity-based structures, deep ocean structures, and Arctic structures (see Chapter 12, Chapter 22, and Chapter 23).

During construction of a bridge across the lower Colorado River, the increased blockage of the river by the cofferdams for the piers caused an increase in velocity, even though there was no flood. Since the sand at the site was very loose, it scoured away, in some places all the way to bedrock. During construction of a bridge across the Skagit River in Washington, the blockage was caused not only by the cofferdams but also by the contractor's fleet of barges moored alongside. During a flood, the cofferdams were undermined and destroyed. A similar disaster occurred during construction of the Columbia River Bridge at its mouth. The ebb tide was amplified by the river flow, while its channel had been reduced by the cofferdams and moored equipment. For many years on the

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TABLE 7.1

Seafloor Modification and Preparation

	Task	Methods
A.	Survey, investigation, and control	GPS, DGPS, lasers, electronic navigation systems, ranges spar buoys, acoustic transponders, coring and sampling (CPT, grab samples, sparker survey, side-scan sonar, acoustic imaging, foundation penetrometers, video, submersible ROVs, and diver inspection)
B.	Platform	Crane barge, drill ship, barge, semisubmersible, jack-up, TLP, heave compensators, guyed tower, gravity-base
C.	Seafloor obstruction removal	Drag off with trawlers, shaped charges, underwater burning, thermic lancers, video-guided clamshell buckets
D.	Dredging removal of sediments	Trailer suction hopper dredge, cutter-head hydraulic dredge, clamshell (grab dredge), dragline, continuous bucket ladder dredge, slack-line bucket dredge, plow, jetting, pipeline burial sled, deep-sea mining drag excavator, airlift, eductors, remote-controlled seafloor dredge
E.	Dredging and removal of hard material and rock	Hydraulic backhoes, dipper dredges, power-activated clamshell buckets, plows, shaped charges, blasting in drilled holes (O.D. method), chisels, hydraulic and pneumatic rock breakers, driven spuds, rock-cutting cutter head dredges, high-pressure ($\sim$ 15,000 psi) jets, down-the-hole drills
F.	Placement of underwater fills	Dikes of rock or clay bunds to contain sand, controlled underwater deposition, dump en masse from hopper barges, tremie, bucket, skip, chute, or ladder
G.	Densification, consolidation, and strengthening of fills	Deep vibration, surface vibration, dynamic compaction with dropped weights, explosives, or air gun, deposition in mass, pre-saturation, cement grouting, chemical grouting
H.	Consolidation and strengthening of weak soils	Sand piles, vibration, freezing, surcharging with membrane and drainage, surcharging with structure and ballast, wick and sand drains, drainage wells, peripheral surcharging, cement injection, chemical grouting, lime injection, deep cement mixing, electro-osmosis, jet grouting, stone columns, closely spaced piles (soil pinning)
I.	Prevention of liquefaction	Densification as per H (above), drainage wells, peripheral apron of graded rock, stone columns, wick drains
J.	Leveling of seafloor or embankment	Hydraulic "dustpan" dredge with heave compensator suspension of dredge head, drags, bottom-supported screed frame, screed frame from TLP or heave-compensated platform, horizontal screw augur
K.	Provision of uniform support under base of structure	Preleveling as per J (above), underbase grouting, underbase sand injection or sand flow, tremie concrete, grout-intruded aggregate ("Prepakt"), mud jacking, grouting

(continued)

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TABLE 7.1 *Continued*

	Task	Methods
L.	Excavation beneath structure	Articulated dredge arms, airlift, jets, eductors, drills
M.	Scour and erosion protection	Sacrificial fill, riprap, graded stone with filter course, filter fabric, articulated mattresses, sandbags, grout-filled porous bags, skirts or structures, aprons and flow-control devices at base of structures, artificial seaweed, sand-asphalt and stone-asphalt blankets, underwater concrete slabs
N.	Turbidity suppression	Betonite-cement slurries, discharge of fine sand, blanket, bucket deposition of clay, flocculants

Source: From B.C. Gerwick. 1974. *Preparation of Foundations for Concrete Caisson Sea Structures*, Dallas, OTC 1946, Offshore Technology Conference.

Mississippi River, large willow mats have been fabricated and weighted down by stone, then covered by riprap, before starting to sink a caisson.

There are some days the happy ocean lies
Like an unfingered harp, below the land.
Afternoon gilds all the silent wires
Into a burning music for the eyes.

Stephen Spender, "Seascape"

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8

Installation of Piles in Marine and Offshore Structure

8.1 General

Piling for marine and offshore structures must be installed to develop the required capacities in bearing, uplift, and lateral resistance. For offshore bridge piers, minimization of settlements may also be criteria. Stiffness under lateral loads, as well as strength, and the ability to accept overloads in a ductile mode are also important characteristics.

For harbor structures, such as wharves and quays, and for trestle-type bridges, prestressed concrete piles are commonly employed. Once in place, their strength and rigidity, resistance to buckling, and durability often render them the most suitable. However, stresses and strains occur during driving, for which they must be adequately prestressed to resist tension rebound cracks and must have adequate confining spiral or hoops to resist lateral bursting stresses due to Poisson's effect. The rebound tension is at a maximum in soft driving at the initiation of the installation. This is contrary to intuition. Typical piles for harbor work are larger and more heavily reinforced than those employed in land foundations.

Steel H piles have been used as the bearing piles in bridge piers, especially when the footing block is at or below the seafloor or river bottom level and difficult driving is anticipated. Pile shoes are valuable in resisting local buckling at the pile tip.

Steel H piles and small-diameter pipe piles are often used as tension piles. To increase their resistance to pull-out, they may be fitted with steel plate fins or spiraled plates. Alternatively, grout pipes may be attached and grout injected under pressure, thus enlarging the zone of anchorage. In effect, they are becoming high-capacity ground anchors.

Micropiles are small diameter pipe piles, 300- to 500-mm steel piles, drilled into rock or similar competent material, often employed to resist both compression and tension (uplift). A high strength steel bar or cluster of bars is inserted and the pile grouted in the socketed annulus. The micropile tubular is then filled with grout, both for reasons of durability and to enable the micropile to carry compressive load. Sometimes it is filled with non-corrosive grease in order to permit elongation under its design load.

Deep water, long, unsupported column lengths, large cyclic bending forces, and large lateral and axial forces all combine to make the typical offshore pile large in diameter and long in length. Piles in most offshore practice are steel pipe piles ranging from 1 m up to 3 m (and even 4 m) in diameter and in lengths from 40 to 300 m or more (see Figure 8.1). Pile capacities for marine and offshore structures have design ultimate values of as much as 10,000 tn., far above those of conventional onshore piles.

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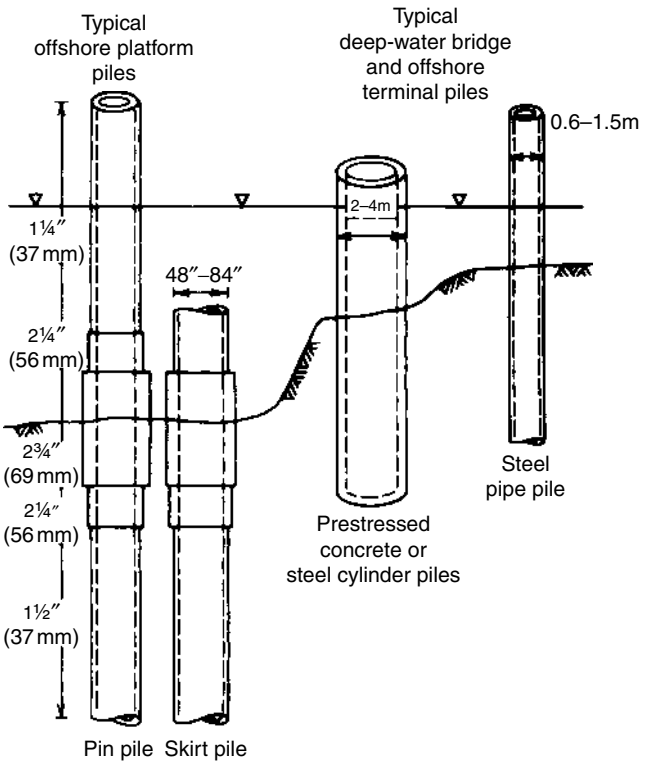

FIGURE 8.1
Typical piles for offshore construction.

For resisting axial compression, the pile transfers its load by skin friction along its outside perimeter and by end bearing on its tip, provided that the tip is either closed or plugged in such a way as not to yield in relation to the pile. Thus, for a natural plug of sandy clay, the internal skin friction must be adequate to develop the full end-bearing resistance of the plug. Large-diameter tubular piles may not plug, however, and thus the end bearing is lost. However, the interior surface will develop skin friction.

End bearing and skin friction do not develop their resistances simultaneously and hence are not usually directly additive at serviceability (elastic) levels of load. They may, however, partially augment each other at ultimate load. For this reason, deep-pile foundations are usually designed primarily as friction piles. Internal skin friction generally develops to its maximum within a one-diameter length of the tip.

For resisting axial tension, the deadweight of the pile, plus that of the internal plug of soil, plus the skin friction are available.

Cast-in-drilled hole (CIDH) piles are used for many bridge pier foundations, even in deep water. They require casing through the water, with a drilled shaft through the soils, which is heavily reinforced and filled with concrete, placed by the tremie process. The drilled holes are kept open temporarily by a slurry.

For resisting lateral loads, most offshore structures in deep water (over 30–40 m) depend on the bending resistance of the pile interacting with the passive resistance of the soil in the near-surface stratum. Since the soil resistance is a function of its deformation, the analysis is based on the interaction of the lateral load P with the displacement y at each incremental level below the seafloor. This is called the P/y effect. The pile must have sufficient strength to resist the resultant moments and shears at these levels and to prevent biaxial buckling. The capacity to resist lateral loads can be improved by increasing

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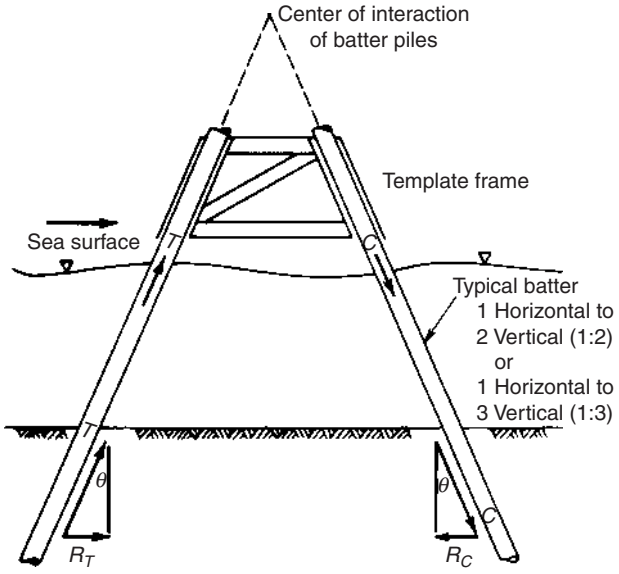
the stiffness and moment capacity of the pile in the critical zone near and just below the mudline by grouting in an insert pile, by increasing the wall thickness of the steel pile through this zone, or by filling the pile with concrete in this region.

In stiff clay soils and calcareous soils, cyclic lateral loadings may create a gap around the pile just below the mudline, which increases the lateral deformations of the piles and structure as a whole and increases the moment in the piles. Piling a loose mound of pea gravel, or even high-density rock of small size, around the pile can effectively minimize this effect by filling any gap that does form and thus minimize the amplitude of deformations.

An alternative method of resisting lateral loads, used in harbor structures and some offshore terminals, is the use of batter (raker) piles, sufficiently inclined to develop a substantial horizontal component of their axial capacities. Batter piles must have a reaction in order to be effective; this is usually provided by a mating pile battered in the opposite direction, although the deadweight of the platform may also be mobilized as a reaction force (see Figure 8.2). Under lateral load, these connections must develop high shear and moment capacity. Their performance in earthquakes and ship collision has often been unsatisfactory due to local buckling near the pile head.

For offshore piling and for bridge piers in deep water and/or very soft mud, it is customary to use either all vertical piles or to have a moderate batter, up to 1 on 6. Under lateral forces and imposed deformations such as those of currents, storm waves, and earthquakes, the increased bending moments and axial forces are generally within the capacity of the piles. The batter helps to eliminate any significant residual displacement of the structure.

Fin piles and helical piles have been used in bulkhead and quay wall construction to increase the tension capacity against pull-out.



$R_T = T \sin \theta$
$R_C = C \sin \theta$
$\Sigma R = T \sin \theta + C \sin \theta = (T + C) \sin \theta$

FIGURE 8.2
Offshore terminal dolphin.

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Special methods and equipment have had to be developed to install the large piles required for offshore structures. Driving with very large hammers is still the preferred method for most cases because it is fast. However, where soil conditions do not permit driven installations and in other special cases, drilling may be employed, with the pile being grouted into the drilled hole. Special foundations such as belled footings have been utilized in the North Sea, Arabian Gulf, and Australian Northwest Shelf. Jetting, airlift removal of plugs, and even internal dredging of soils have been used on large-diameter piling for offshore terminals, bridge piers, and a few offshore platforms.

The effect of all such installation operations on the supporting soil must be considered. In some cases, it may be beneficial but in most cases, the results may degrade the performance unless special precautions are taken. API Standard RP2A warns that piles drilled and grouted may have resisting values significantly different from those of driven piles. Large-diameter piles (over 1.5 m) may not develop their full internal skin friction.

For piles driven in undersized drilled or jetted holes in clays, the skin friction will depend on the amount of soil disturbance, including the relief of stress, which is occasioned by the installation. The strength of dry, compacted shale or serpentine may be greatly reduced when exposed to water from jetting or drilling. The sidewall of the hole may develop a layer of slaked mud or clay, which will never regain the initial strength of the parent soil or rock.

In overconsolidated clays, drilled and grouted piles may develop increased skin friction. If excess drilling slurry is present in clays or in soft rock, the coefficient of friction may be significantly reduced. In calcareous sands and some micaecous silts, driven piles

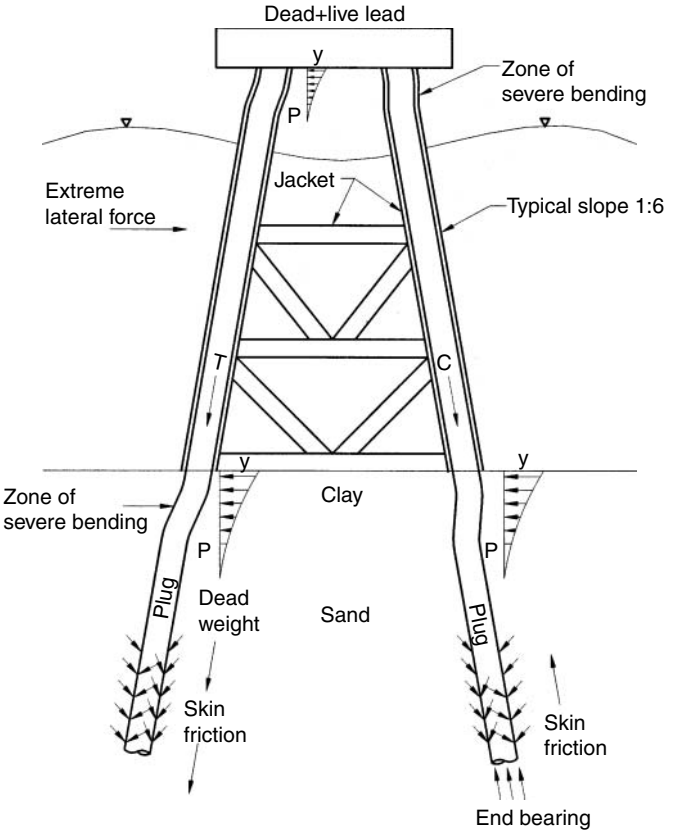

FIGURE 8.3
Typical pile-structure-soil interaction.

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may have very low values of friction compared to those attainable by drilling and grouting.

API Standard RP2A further warns that the lateral resistance of the soil near the surface is significant to pile design (see Figure 8.3), and consideration must be given to the effects of soil disturbance during installation as well as scour in service.

Great strides have been made in recent years in developing hammers, drills, and methods for installing marine and offshore piling. As a result, constructors now have a wide range of effective tools from which to choose in order to meet their particular needs.

8.2 Fabrication of Tubular Steel Piles

Tubular steel piles are usually made up of "cans," rolled plate with a longitudinal seam. Individual segments (cans) should be 1.5 m (5 ft.) or longer in length. The longitudinal seams of two adjacent segments should be rotated at least 90° apart.

A taut wire located at three 120° azimuths should be used to verify straightness of the made-up pile. API RP2A specifies a maximum allowable deviation from straightness in any 3-m (10-ft.) length of 3 mm (J in.). For lengths over 12 m (40 ft.), the maximum deviation should be 13 mm (H in.) (see Figure 8.4).

API RP2A and API RP2B give allowable tolerances for outside circumference and out-of-roundness as well as for beveled ends. Pile wall thicknesses should vary between adjoining segments by not more than 3 mm (J in.). Where a greater difference is employed, the thicker section should be beveled on a 4:1 vertical-to-horizontal level (see Figure 4.1).

To ensure proper fit-up of splices and to minimize weld time, thick-walled pile segments are best pre-mated or pre-checked. Sections can frequently be best matched by rotating one relative to the other. One method for ensuring accurate match for splicing during installation is to cut one "can" in half, welding each half to the ends of the long pile segments. The surfaces of piles that are to be connected by grout bond to steel tubulars or to the soil should be free of mill glaze and varnish.

FIGURE 8.4
Fabricated piling for Lena Platform, Gulf of Mexico. (Courtesy of J. Ray McDermott SA and Exxon.)

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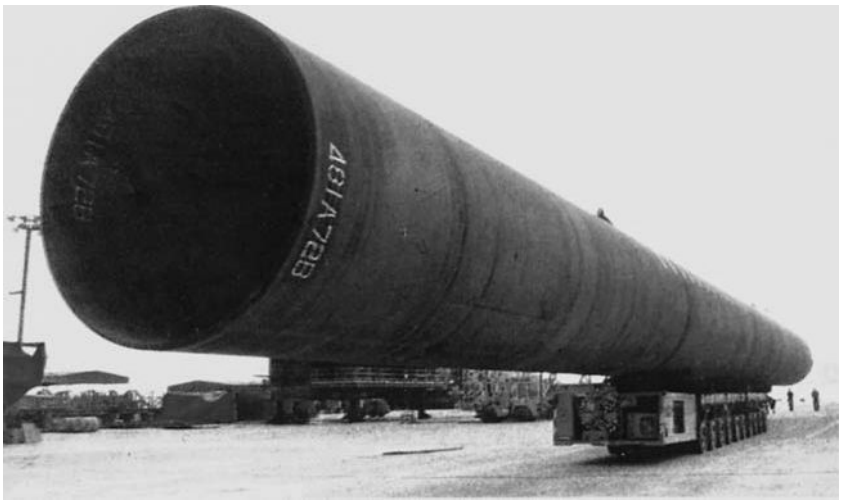

FIGURE 8.5
Steel cylinder pile for Jamuna River Bridge, Bangladesh.

In recent years, spiral-welded piles have been developed. A long strip of steel is rolled into a spiral, with full penetration welds along the seams. The thickness of plate that can be fabricated is a function of diameter and thickness. The shear stresses along the spiral seams are very high and in the past have led to failure under hard driving and in bending service. However, improvements in technology and quality control now make feasible piles one meter in diameter and 28 mm wall thickness; however, there are currently (in 2006) only a few such machines existent.

API RP 2A currently does not recommend spiral weld piles for offshore structures, due to the shear on the diagonal seams during driving. However, with high quality welding and good inspection, they are now being employed on near-shore and inshore structures.

Steel piles are usually protected below water by sacrificial anodes. Impressed-current cathodic protection is also often employed but may present problems of reliability, not only due to technical adjustment requirements and possible adverse effects on reinforced concrete members in the vicinity, but also due to the human factor, namely, that the impressed current interferes with television reception and hence, is often turned off.

Steel piles are normally given some additional corrosion allowance—for example, 3–6 mm (0.125–0.50 in.)—and protected by epoxy, polyurethane, or metallized coatings or by cladding with Monel or titanium; 10-mm corrosion allowance has been provided for some bridge piles due to their design life of one hundred years or more.

Corrosion of exposed steel in fresh water is similar but at a lower rate than in seawater. The corrosion rate is much influenced by the velocity of the current, by any abrasive material (e.g., sand or silt) suspended in the water, and by turbulence. The splash zone suffers the highest corrosion rate, whereas permanently below water the rate is reduced by 50% due to lack of oxygen, and below the mud line, by another 50% (Figure 8.5).

8.3 Transportation of Piling

Large-diameter tubular pile sections of steel or prestressed concrete are usually lifted (or rolled) onto a barge, which is then towed to the site. The pile sections must be well chocked

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FIGURE 8.6
Barge shipment of piling, Seattle to Alaska. Note structural tie-downs and chain lashing.

and chained to prevent any tendency to shift and roll in a seaway. Usually the piles have thick-enough walls so that the stack will not locally deform the piles underneath; however, this should be checked and suitable blocking or supports provided as necessary to prevent damage (see Figure 8.6).

Sometimes, it becomes practicable and efficient to transport the piles in a self-floating mode, either singly or in a well-secured (chained) raft (see Figure 8.7). This becomes especially attractive when the piles can be subsequently lifted and placed in long sections—for example, skirt piles—that can be entered well below the surface of the sea. The ends of the piles can be closed with steel closure plates or rubber diaphragms. These

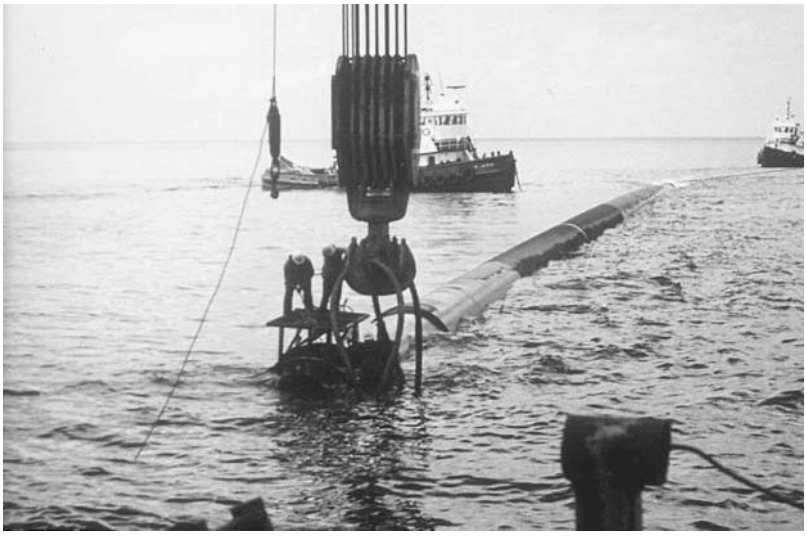

FIGURE 8.7
Transportation of piles in self-floating mode. (Courtesy of J. Ray McDermott and Exxon.)

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need to be strong enough to take wave slap during tow to the site. Upon arrival, one end is usually lifted clear of the water by a derrick line to permit cutting out the closure, and then that end is allowed to rotate down to the vertical. In some cases, the trapped air in the other end (sometimes augmented by compressed air) is used to limit the draft when the pile reaches vertical. In such a case that closure must be adequate to resist the internal air pressure. In any event, closures should be provided with a valve so that air or water can be vented and/or allowed to flood in, permitting ballasting and removal of the end plate in a controllable manner.

Removal of a closure plate below water can be a dangerous operation. In one case off Australia, the diver who was cutting out the closure plate was sucked into the pile by the rush of water and drowned. Provision of a valve and prior equalization of pressure on both sides of the closure plate would have prevented this accident.

In shallow waters, such as those at an offshore terminal, piles have been temporarily stored on the seafloor with recovery pendants and buoys attached.

When a long pile is upended in the water, very large bending moments may be introduced at certain stages of rotation. While these are usually less than those that occur when upending in the air, they are not negligible and must be checked.

For many recently constructed offshore platforms, some piles have been transported with the jacket, either set in the main legs or in the skirt pile sleeves and guides, where they provide additional buoyancy (if closed) as well as additional weight. The purpose of the preinstallation is to enable several piles to be driven down immediately after seating of the jacket on location so that the jacket will be stable against the action of waves and current.

Typically, as soon as the jacket is seated and leveled (insofar as practicable on the seafloor while resting on the mud mats) the piles are cut loose so that they can penetrate the soils under their own weight. They are then extended as necessary, and four (or more) are driven down a short distance, where they may be temporarily welded or clamped to the jacket top. Final leveling of the jacket may then take place. One by one, these initial piles may be raised as necessary to eliminate any bending stresses, relowered, and driven down to required penetration.

As noted above, the piles, when transported in the jacket, must be adequately secured against the forces of launching and upending. During the upending of the Magnus platform jacket in the North Sea, a number of piles broke loose and ran down, hit the seafloor, and were badly buckled. For a while it was feared that the entire project would be set back one year. As it was, a mammoth effort, plus favorable weather, permitted replacement of the piles and installation late that same season.

8.4 Installing Piles

The piles for the typical offshore jacket are delivered on barges, with the first section of each pile being as long as can be handled and placed by the derrick barge. Pin piles are centered inside the jacket legs and typically extend the full height of the jacket. Skirt piles are encased in sleeves bracketed out from the lower end of the jacket. Many jackets incorporate both pin piles and skirt piles (Figure 8.8).

Skirt piles must be driven either with a follower or an underwater hammer. The piles are typically clustered around the corner legs of the jacket and are aligned parallel to them, so that the piles must be driven on a batter of from 1 to 6. Guides are attached at intervals along the jacket legs to aid in setting the piles through the sleeves.

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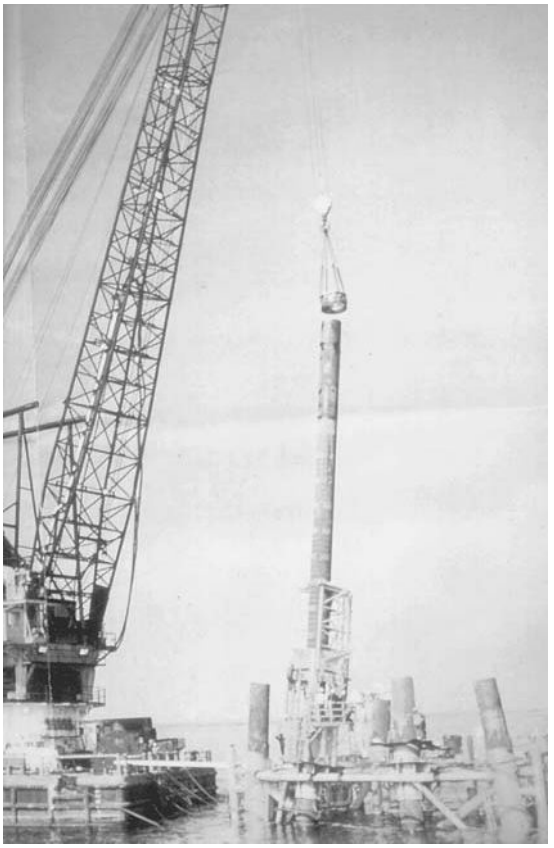

FIGURE 8.8
Releasing pile lifting ring after setting pile into jacket leg. Note pile positioning frame at deck level. (Courtesy of J. Ray McDermott SA and Exxon.)

Some recent jackets have been constructed with vertical sleeves, thus eliminating the guides and enabling a very long initial pile segment to be stabbed into the underwater sleeve. Guidance of the pile may be by means of a tensioned line or, in deep water, by the use of short-range sonar and video, and ROV. "Add-ons" (additional lengths of pile) are "stabbed" and welded onto the top of the previous pile section as it is driven down near to the top of the jacket (Figure 8.9).

American Petroleum Institute Standard APIRP2A suggests that reasonable assurance against failure of the pile will be provided if static stresses are calculated for each stage as follows:

1. The projecting section of the pile is considered as a freestanding column with a minimum effective length factor k of 2.
2. Bending moments and axial loads are calculated on the basis of the full weight of the hammer, cap, and leads, acting through the center of gravity of their combined masses, plus the weight of the pile add-on section, all with due consideration of the eccentricities due to pile batter. The bending moment so determined should not be less than that due to a load equal to 10% of the combined weight of hammer, cap, and leads applied at the pile head perpendicular to the pile centerline.
3. The pile resistance is to be based on normal (elastic) stresses, with no increase for the temporary nature of the load.

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FIGURE 8.9
Internal lift tool lowers pile through external gripper. After pile is lowered to deck level, external gripper will hold pile while internal gripper lifts add-on section. (Courtesy of Oil States Rubber Co.)

One means of reducing the bending in the pile during this stage is to suspend the hammer and leads in a bridle at the proper batter (see Figure 8.10 and Figure 8.11). This is especially important in offshore terminal construction where relatively flat batters are often employed (e.g., 1 horizontal to 2 vertical). This is also very important when

FIGURE 8.10
Setting first segment of steel cylinder pile. Jamuna River Bridge, Bangladesh. (Courtesy of Rendel.)

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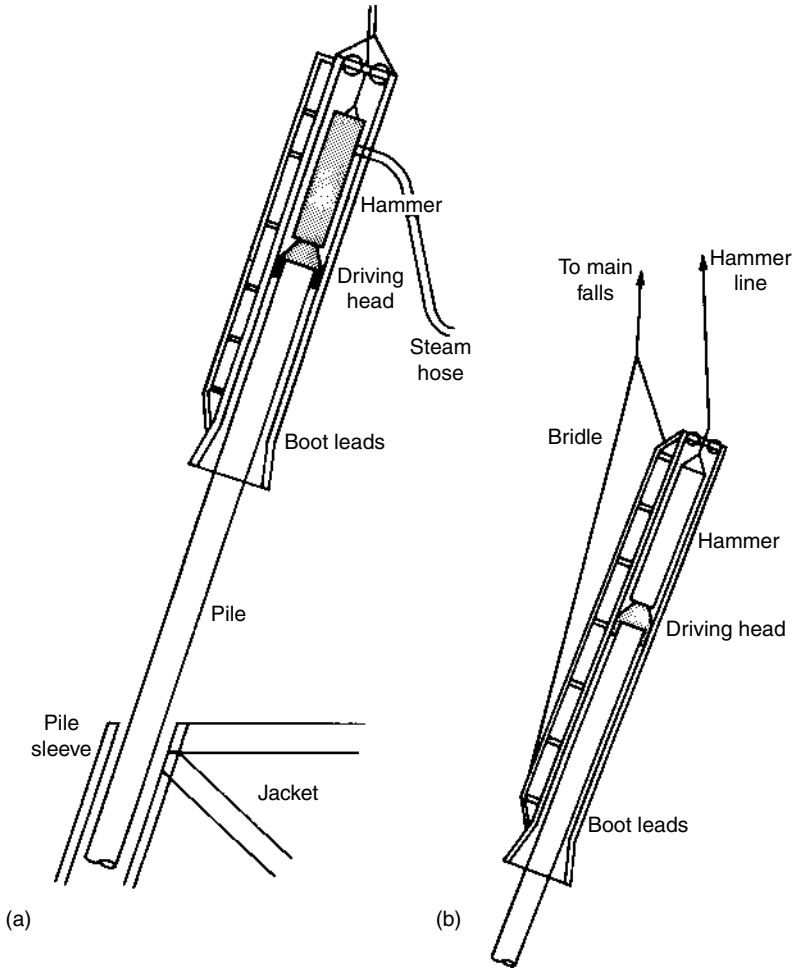

FIGURE 8.11
Driving of offshore piling. Note suspension of hammer and boot leads for batter piles.

driving piles having low bending strength—for example, prestressed concrete piles—on a batter.

Offshore pilings are typically large-diameter, thick-walled tubulars (pipe) ranging from 1 to 2 m in diameter. They are driven with high-energy impact hammers, either steam, hydraulic, or diesel. As a general rule, the hammer with attached driving head rides the pile rather than being supported by leads (see Figure 8.12). This means that the driving head (helmet) must be secured to the hammer by wire rope slings and that the driving head in turn must seat well on the pile and have a guiding bracket or ring attached in order to keep the hammer aligned with the pile. During driving, the hammer line from the crane boom is slacked to prevent transmitting impact and vibration into the boom.

For steel piles, there is usually no cushion block used between the helmet and the pile, although an internal cushion is used in certain makes of pile hammer. Because of the tremendous energy required to raise the ram, steam or hydraulics are usually used rather than compressed air. Offshore hammers are generally single-acting with rates of up to 40 blows/min. Hammer energies of current equipment range from 100 kN-m

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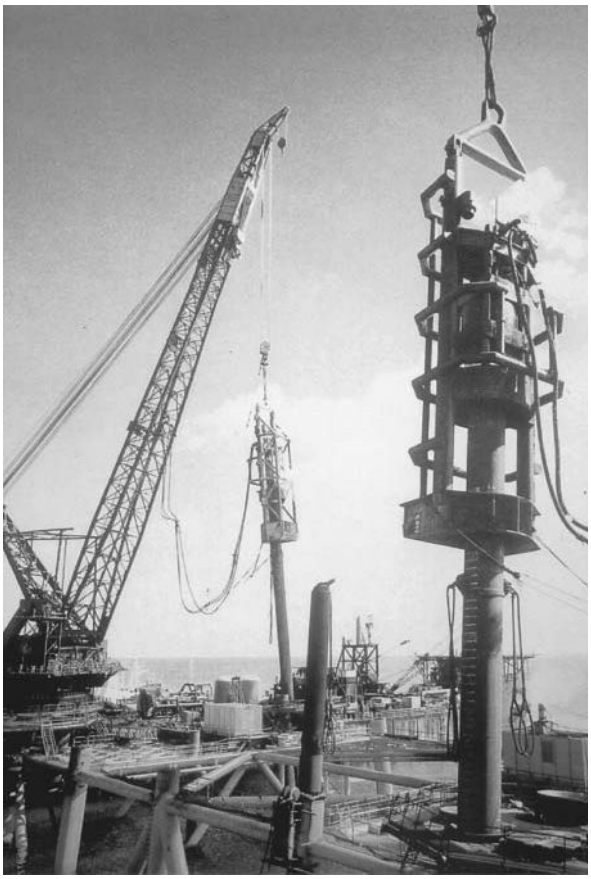

FIGURE 8.12
Installing and driving piles in jacket, Gulf of Mexico.

(67,000 ft.-lb) to 1800 kN-m (1,200,000 ft.-lb) per blow. The larger hammers represent major lifting tasks in themselves, weighing up to 300 tn.

Hydraulic hammers have been developed for offshore and especially for underwater driving. These are radically new versions of the underwater hammers formerly used in bridge pier construction. The new hammers not only have large energies, but are virtually unaffected by depth. They are thus useful for driving skirt piles whose heads may finally be located several hundred meters below sea level. Hydraulic hammers have a favorable action in that they sustain the impact over a longer number of milliseconds than a steam hammer (Figure 8.13). Slender hydraulic hammers are designed so as to pass through upper guides. Table 8.1 gives data on large steam and hydraulic pile hammers.

Large diesel hammers are much used on offshore terminals. These have nominal energies in the range 200 kN-m (130,000 ft.-lb) to 300 kN-m (200,000 ft.-lb) per blow, but in most practicable driving conditions they can be equated in effectiveness with a steam hammer of 60% of that rated energy. The diesel hammer is much lighter to handle and much more economical in fuel consumption, but its effective energy is limited. One manufacturer is developing a much larger diesel hammer, with a rated energy of 600 kN-m (400,000 ft.-lb) per blow. Table 8.2 gives data on large diesel hammers. Large vibratory hammers have been used on terminal piling: for example, a quad unit of four large vibrators was used to install the heavy steel piles on the Yanbu, Saudi Arabia, pipe offloading pier. These actually activated the piles to a resonant amplitude of about

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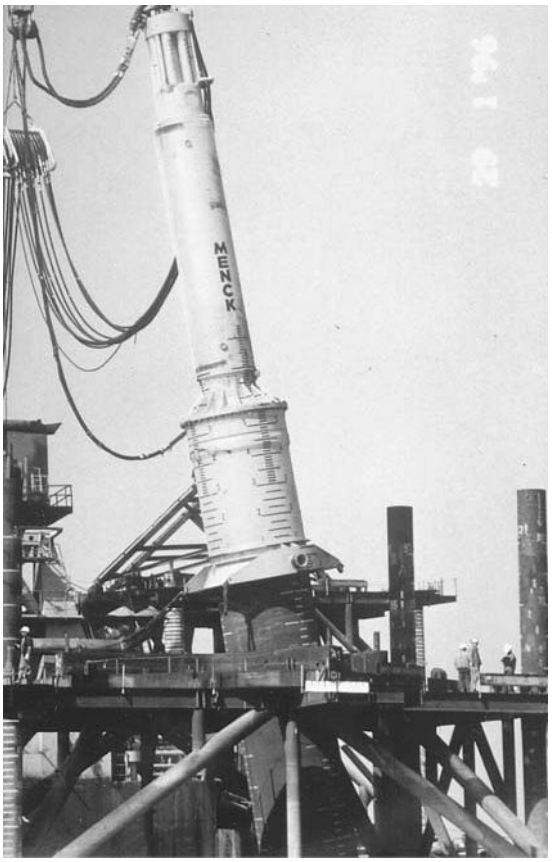

FIGURE 8.13
Large hydraulic hammer driving 3.15-m diameter steel tubular pile. Note size comparison with men at lower right hand corner.

10 mm, and the pile "drove itself" through dense sands and limestone layers. Vibratory hammers have been used to initiate the installation of piles: they are lighter and shorter than a steam or hydraulic hammer and hence allow a longer length to be initially installed. They may be assembled and synchronized in clusters of two or four when used to install a large tubular pile through firm and dense soils (see Section 9.4.9.2 for precautions against liquefaction).

Impact hammers impart an intense compressive wave to the head of the pile, which travels down the pile at the speed of sound in the pile material. This compressive wave is a dynamic stress wave which eventually reaches the tip.

The newest large steam and hydraulic offshore hammers are instrumented so that the velocity of the ram can be measured just before it strikes the anvil. A representative pile can also be instrumented to measure both strains and acceleration during the hammer blow.

The dynamic portion of the stresses induced in the pile during driving with impact hammers can best be computed by the wave equation, which is a one-dimensional elastic stress wave transmission analysis using selected parameters for the response of the hammer, cap block, cushions, pile, and soil strata. Such an analysis is useful in determining the maximum stresses at such critical points as head and tip, and also at splices and changes in section. Using advanced forms of the wave equation, ultimate resistance in tension and compression, penetration rates, and overall driving time can be computed.

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TABLE 8.1

Large Hydraulic and Steam Pile Hammers

Hammer	Type	Blows per Minute	Weight, Including Offshore Cage, If any (metric tons)	Rated Striking Energy (kNm)
Conmaco 1750	Steam	40	200	1460
Conmaco 6850	Steam	40	80	708
Conmaco 5700	Steam	40	70	500
Conmaco 5450	Steam	46	45	300
Conmaco 5300	Steam	46	25	200
MRBS 4600	Steam	36	80	700
MRBS 3000	Steam	40	45	450
Vulcan 3100	Steam	58	80	415
Vulcan 540	Steam	48	45	270
MHU 500	Hydraulic	55	80	500[a]
Vulcan 3250	Single-acting steam	60	300	1040
HBM 3000	Hydraulic underwater	50–60	175	1430
HBM 3000 A	Hydraulic underwater	40–70	190	1520
HBM 3000 P	Slender hydraulic underwater	40–70	170	1550
Menck MHU 900	Slender hydraulic underwater	45	135	850[a]
Menck MRBS 8000	Single-acting steam	32	150	1200
Vulcan 4250	Single-acting steam	53	337	1380
HBM 4000	Hydraulic underwater	40–70	222	2350
Vulcan 6300	Single-acting steam	37	380	2490
Menck MRBS 12500	Single-acting steam	38	385	2190
Menck MHU 1700	Slender hydraulic underwater[b]	45	235	1700[a]
IHC S-300	Slender hydraulic underwater	40	30	300
IHC S-800	Slender hydraulic underwater	40	80	800
IIHC S-1600	Slender hydraulic underwater	30	160	1600

[a] Underwater at 1000 m depth.
[b] Slender designation means it can ride through jacket sleeves.

Source: Courtesy of Conmaco.

Because of the transient nature of the blow and its very short duration, buckling of a pile as a column during driving has been shown usually not to be a problem and can generally be neglected.

It is not always practicable to determine all the parameters needed with accuracy, especially soil parameters, nor is it always practicable to carry out a wave equation analysis, although the widespread availability of computer programs makes this latter a fast and economical practice that is increasingly employed even on moderate-sized projects. In the absence of reliable calculations of these dynamic stresses induced during driving, an empirical rule is to limit the static portion of the stresses to one-half the yield strength of the pile.

A machined driving head is fitted between the hammer and the pile head. This ensures uniform transfer of the impact blow to the pile and prevents local distortion of the pile head.

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TABLE 8.2

Large Diesel Hammers

Hammer	Blows per Minute	Ram Weight (tn.)	Total Weight (tn.)	Energy (kN-m)
Delmag D-200-42	36–52	20	50	680-436[a]
Kobe K-150	45–60	15	36	400
Mitsubishi MB-70	38–60	8	21	200-90[a]
Delmag D-55	36–47	5	11	160-90[a]
Kobe K-60	42–60	6	17	145
Delmag D 46-02	37–53	4	8	145-60[a]
Delmag D 65	37–53	8	10	165

[a] Adjustable.

Source: Courtesy of Conmaco.

Large-diameter tubular steel piles are being increasingly utilized, not only for offshore platforms but for marine terminals and bridge piers as well. To drive these piles requires high energies per blow, hence very large and costly hammers such as those listed in Table 8.1 through Table 8.3.

The D/t ratios for piles must be limited to preclude local buckling at stresses up to the yield point of the pile steel. Where only moderate driving resistances are anticipated or where the pile will be drilled and grouted (not driven), the pile may be designed as a steel cylindrical member and checked for local buckling due to combined axial compression and bending. This latter is non-critical when D/t is less than or equal to 60. When D/t is greater than 60, then a more in-depth analysis such as that given in API RP2A should be followed.

For piles that will be subjected to sustained hard driving in excess of 800 blows/m (250 blows/ft.) the minimum wall thickness of the pile should be not less than:

$$t(\text{mm}) = \frac{6.25 + D(\text{mm})}{100} \tag{8.1}$$

TABLE 8.3

Large Vibratory Hammers

Company	Model	Eccentric Moment (in.-lbs)	Frequency (VPM)	Centripical Force (US_tn.)	Max Line Pull (US_tn.)	Power Unit Max Rating (HP)
APE	400B-Tandem	26,000	400–1500	830	500	2000
ICE	V360-Tandem	22,600	0–1500	722	450	2100
APE	600B	20,000	400–1400	543	418	1000
HPSI	2000	20,000	0–1300	480	600	1600
HPSI	1600	16,000	0–1400	445	600	1600
ICE	V125	12,500	0–1550	426	300	1320
ICE	V360	11,300	0–1500	361	225	1050
APE	400B	13,000	400–1400	360	250	1000
HPSI	1200	12,000	0–1400	334	600	1200
MKT	V-140	14,000	0–1400	tbd	tbd	1800

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Pile wall thicknesses are normally varied throughout their length in order to adjust to the in-service axial plus bending requirements. The minimum pile wall thickness should be selected as indicated above to preclude local buckling and also to maximize penetration under the hammer blows.

Maximum bending in-service normally occurs at and just below the mudline. Since designed pile penetrations often have to be modified to some degree in the field based on actual driving resistance, the length of pile with maximum wall thickness should be increased over the theoretically required length to permit some tolerance in pile penetration and hence in location of the thickened section.

In selecting pile section (add-on) lengths, the following factors should be considered:

1. The lifting and stabbing of the pile add-on segment. What is the maximum capacity of the crane and boom length to handle the segment? Check bending moment in pile during upending.
2. The capacity of the crane and geometry of the boom when seating the hammer and leads over the top of the newly added-on segment (and often over the intervening corner of the jacket).
3. The possibility that the initial pile section will "run" when it penetrates the jacket leg closure. If allowed to run free, it may drop below the level at which the next add-on may be welded. One solution, of course, is to provide a restraint, such as a preventer sling or cushioned bracket.
4. Stresses in the pile segment when lifting and when the hammer is placed (as noted earlier).
5. Wall thickness at field welds, with consideration of material properties and welding procedures required.
6. Possible interference with adjoining pile segments or structures. This is often critical in offshore terminal construction where there are batter piles that radiate in several directions and whose axes may intersect near deck level or at opposing tips.
7. Soil characteristics. It is desirable to plan the segment lengths so that the temporary location of the pile tip is in relatively soft soils, enabling driving penetrations to resume easily when the splice has been made. Conversely, if the pile tip is allowed to sit during splicing in a zone of material with high setup properties, then excessive resistance may develop when driving resumes.

The head of pile sections on which driving is carried out may be deformed during driving and hence require reheading in order to weld on the next section. Hence, API RP2A recommends an allowance of 0.5–2 m for reheading. Modern well-fitting driving heads and some hammers (e.g., Hydroblock) minimize the head damage; with thick-walled piles, reheading may be unnecessary.

When pile add-ons are placed, they are equipped with stabbing guides to facilitate entry and proper alignment. The stabbing guides should have a tight fit in order to provide a proper fit-up for the weld. The guides may be designed to support the full weight of the pile during welding so that the crane may be freed from other tasks. Further, support from a floating derrick boom during welding is often unsatisfactory due to the movement of the boom tip and transmitted vibration. However, it is usual practice for the crane to continue to hold on with a slack pile line as a safety precaution until at least one full weld pass has been made. Pile sections may also be held by temporary supports on guides that extend up from the jacket and provide a support 10–20 m above the deck. They may also be held by a hydraulically operated clamping and alignment device, which is clamped onto the

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previously set pile section or supported on a temporary work deck on the jacket. This latter allows quick-stabbing guidance and then final accurate alignment of the new section.

In addition to connecting pile sections by full penetration welds, breech-block connector sections have been developed that enable the splice to be effected rapidly by applying torque. Accurate alignment is essential, and hence a hydraulic clamping-aligning device is essential. These mechanical connectors have been used for both pile followers and permanent piles and have shown fully adequate performance during driving.

For piles to be joined by welding, the add-on section is pre-beveled, ready for a full-penetration weld. After stabbing, the bevel is inspected and, if necessary, ground or gouged to open it up to assure a full-penetration weld. Weld procedures and materials should be carefully selected with regard to the pile steel qualities and the temperature at which driving will be carried out, since these welds will certainly be under high impact. This will be especially critical for pile driving in Arctic and sub-Arctic regions, which may be carried out at low temperatures. In any event, low-hydrogen electrodes should be used. Semiautomatic welding machines are becoming available in order to speed the welding process: these have two or four welding heads.

Backup plates are usually built into the stabbing guides. However, where drilling is to be carried out, internal backup plates cannot be employed, and the stabbing guides must be external to the pile. API RP2A notes that special skills are required for single-side welding or complete-penetration welds where backup plates are not used.

The time required for splicing of the large-diameter, thick-walled piles typical of most offshore platforms is significant. On the 54 in. piles of the Hondo platform, the average times were for 1-in. walls, 3¼ h; for 1¾-in. walls, 7¼ h; for 2½-in. walls, 10½ h. On the Jamuna Bridge in Bangladesh, for tubular piles of 2.5 m and 3.15 m diameter, with 50–60 mm wall thicknesses, four teams of welders were used simultaneously. Splicing took 6–8 h; cool down, 2 h; x-ray, 1 h.

Semi-automatic welding machines reduce the requirements for highly skilled labor. They were employed on the East Bay Replacement Bridge of the San Francisco-Oakland Bay Bridge for splicing 2.5 m × 50–75 mm tubular piles during installation.

Welding machines should be properly grounded to prevent underwater corrosion damage due to stray current discharge.

When pile sections are lifted, they are usually provided with lifting eyes. The lifting eyes and their weld details are designed for the stresses developed both at initial pickup and as the pile is rotated to alignment with its final axis. Both angle of lead of the sling and the load acting on the padeye will change during this operation. An allowance must be made for impact during lifting—normally 100% for the lifting eye and 35% for the crane.

When lifting eyes or weld-on lugs are used to support the pile sections from the top of the jacket, each eye or lug should be designed to support the entire hanging weight. The same welding procedures as for permanent welds should be employed. It is especially important that a full-penetration weld (not fillet welds), with proper procedures, be used. Similarly, the load is inherently across the thicknesses of the pile plate, requiring adequate through-thickness toughness. Final removal of padeyes or lugs should be by flame-cutting to 6 mm (G in.) from the pile surface, followed by grinding smooth.

Holes have often been used in lieu of lifting eyes, especially for smaller, less critical piles such as used in offshore terminals. These holes should be burnt undersized and then reamed. They should be located near the top of the section so that they are part of the length cutoff when stabbing a new add-on. However, use of burnt and reamed holes alone is dangerous, due to stress concentrations during lifting and adverse effects during driving. With large and heavy offshore piles, internal cones or running tools are often used for lifting so that the pile section will hang vertically.

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The piles for offshore structures are typically heavily loaded in both compression and tension. There are relatively few piles employed; hence, each becomes a major structural component. The integrity of the structure therefore depends on each pile being driven to approximate design tip elevation as determined by prior geotechnical investigation and analysis (Figure 8.12 and Figure 8.13).

The driving of each pile should be carried out as nearly continuously as practicable in order to minimize the increased resistance that may occur due to "setup." When the tip of any pile has entered a zone of stiff plastic clay, for example, every effort should be made to eliminate or minimize interruptions in driving (see Figure 8.14). A backup hammer with leads should be available whenever setup is anticipated, as otherwise the breakdown of the hammer may permanently prevent the subsequent driving of a pile to prescribed tip elevation (Figure 8.14 and Figure 8.15).

Note in Figure 8.14 the cross-hatched areas. Not only were there additional blows required to (break the pile loose, but additional blows were required for all subsequent penetration. In the example shown, some 6000 additional blows were required. For a hammer striking 40 blows/min, this is 150 min extra or 2½ h/pile. Note also that this could in some cases prevent the pile from reaching its final tip elevation, regardless of the number of blows expended.

The factors affecting pile resistance during driving are many and complex. Thus, the mere attainment of a high blow count or practical refusal does not necessarily indicate that an adequate capacity has been achieved in either compression or tension. It may be necessary to continue driving either at high blow counts or to use the additional methods, such as cleaning out the pile plug, jetting, or drilling, to be described in a later section of this chapter.

API Standard RP2A states, "The definition of pile refusal is primarily in order to define the point where driving with a particular hammer should be stopped and other methods instituted such as using a larger hammer, drilling, or jetting." Continued driving at "practicable refusal" may be ineffective, may damage the hammer or the pile, and is costly. The standard further states: "The definition of refusal should also be adapted to the individual soil characteristics anticipated for the specific location." For example, when driving piles in the Arabian Gulf, piles may reach virtual refusal on limestone, coraline, or caprock layers only to break through into less dense material below after another hundred blows.

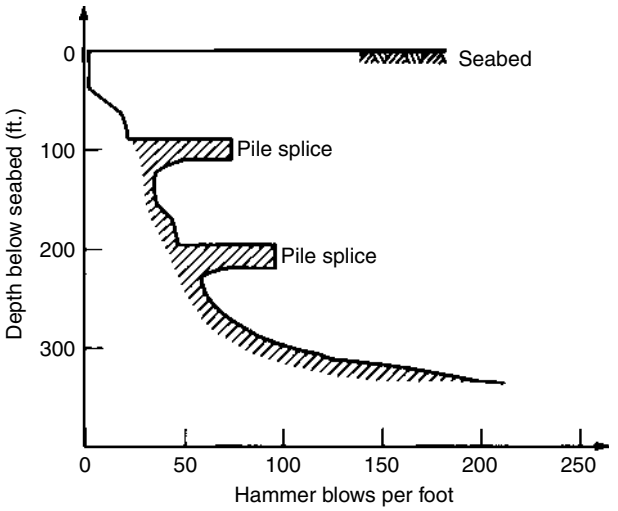

FIGURE 8.14
Typical effect of set-up in clay during suspension of driving (e.g., for splicing).

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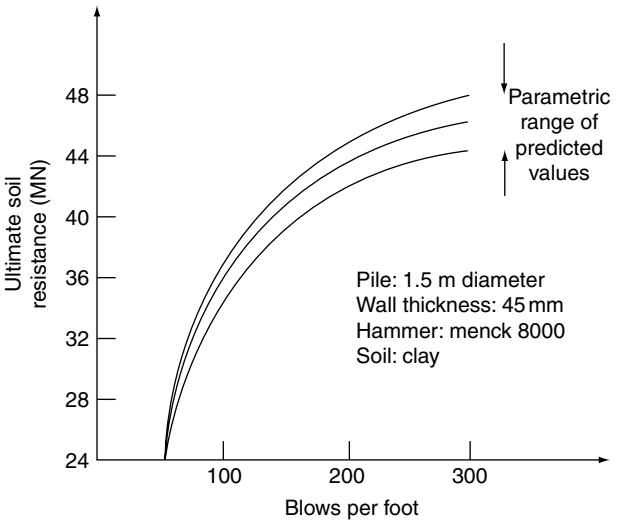

FIGURE 8.15
Typical wave equation analysis prediction of driveability.

Standard API RP2A suggests a typical definition as follows:

Pile driving refusal is defined as the point where pile driving resistance exceeds either 1000 blows/m (300 blows/foot) for 1.5 consecutive meters (5 ft.) or 800 blows for 0.3 m (one foot) of penetration. (This definition applies when the weight of the pile does not exceed four times the weight of the hammer ram. If the pile weight exceeds this, the above blow counts are increased proportionally, but in no case should they exceed 800 blows for 150 mm (6 in.) of penetration.)

If the pile driving has been stopped for 1 h or more in order to splice a pile or because of equipment malfunction, weather delays, or the like, then the pile should be driven at least 0.3 m before the above criteria are reinstated. Driving at resistances greater than 800 blows for 150 mm should not be attempted. The pile will be deforming (yielding), and no appreciable penetration will be attained.

Approximate guide lines for selecting hammer size in relation to pile diameter and wall thickness are given in API RP2A (see Table 8.4). These do not include other important parameters such as pile length and soil characteristics, but they do address the problem of preventing excessive local damage in the pile due to dynamic stresses induced by the hammer and have been determined largely from industry experience in driving offshore piling in the medium to large sizes with moderate-sized hammers.

When steel piles are driven onto rock, the rebound compressive stress at the tip may be almost twice that imparted directly to the pile by the hammer, thus often being above yield. This condition is most frequently encountered in constructing offshore terminals. The pile tip may deform, tear, or "accordion." Tip reinforcement is definitely beneficial. The wave equation is a useful tool for prediction of pile tip stresses in such cases (see Figure 8.15).

The pile-driving analyzer (PDA) attached to the pile is useful in determining actual driving forces and stresses in the pile.

The trend today is toward the use of thicker pile walls in order to increase the effectiveness of the hammer in obtaining penetration. Heavier hammers are used in order to achieve more effective penetration and driving rates. A somewhat more conservative

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TABLE 8.4

Piling Criteria and Hammer Energies

Pile Outer Diameter		Wall Thickness		Hammer Energy	
in.	**mm**	**in.**	**mm**	**ft.-lb**	**kN-m**
24	600	5/8–7/8	15–21	50,000–120,000	70–168
30	750	3/4	19	50,000–120,000	70–168
36	900	7/8–1	21–25	50,000–180,000	70–252
42	1050	1–1¼	25–32	60,000–300,000	84–420
48	1200	1⅛–1¾	28–44	90,000–500,000	126–700
60	1500	1⅛–1¾	28–44	90,000–500,000	126–700
72	1800	1¼–2	32–50	120,000–700,000	168–980
84	2100	1¼–2	32–50	180,000–1,000,000	252–1400
96	2400	1¼–2	32–50	180,000–1,000,000	252–1400
108	2700	1½–2½	37–62	300,000–1,000,000	420–1400
120	3000	1½–2½	37–62	300,000–1,000,000	420–1400

Note: (1) With the heavier hammers in the range given, the wall thicknesses must be near the upper range of those listed in order to prevent overstress (yielding) in the pile under hard driving. (2) With diesel hammers, the effective hammer energy is from one-half to two-thirds the values generally listed by the manufacturers and the above table must be adjusted accordingly. Diesel hammers would normally be used only on 36-in. or less diameter piles. (3) Hydraulic hammers have a more sustained blow, and hence the above table can be modified to fit the stress wave pattern.

approach is therefore suggested. In Table 8.3, typical values are given which, of course, can be modified based on more specific data or detailed analyses.

Large vibrating pile hammers have been joined together in pairs or even in quad (4) assemblies, synchronized to drive large-diameter steel piles in sands and silts. In this case, increased D/t ratios are suitable (e.g., 80–1). These proved adequately effective in installing tubular steel piles for unloading docks at Yanbu, Saudi Arabia, through coral sand and even thin layers of brittle calcarenite, also in deep sands of the Columbia River riverbed.

When entering an initial section of pile into a sleeve, an extra long length may be stabbed, limited only by the bending moment due to pile deadweight alone and the lifting capacity of the crane. Once entered and run down, an axial force may be applied to run the pile farther into the soil. This is done by rigging a line around the head of the pile section, down through a snatch block affixed to the jacket near the pile sleeve, and thence to a winch on the jacket or derrick barge. Exertion of a tension on the line pulls that pile down through the soil to a temporary top elevation which is safe for mounting of the hammer. Similarly, on offshore terminal construction, with prestressed concrete piles, by simultaneously jetting at the pile tip (through an internal jet) and exerting a downward tension axially aligned with the pile, the pile may be caused to penetrate through soft clays and sands to a more workable elevation.

With large-diameter cylinder piles, the pile will have to be slung from padeyes on both sides of the pile (opposite ends of a diameter) in order to hang vertically or an internal lifting device can be used.

Upending large cylinder piles presents a problem since it may be practicable to lift only from the top, yet there may be insufficient bending strength to permit such support. If the pile is on a barge, then a rail-mounted bogey car may support the lower end, yet move along the barge with the pile as it is raised to vertical. Another solution has been to cap the lower end of the pile and to turn it in the water so that buoyancy provides support along the lower half of the pile. This maneuver requires consideration of water depth, pile length, net weight of the buoyant portion of the pile, and pile bending strength. Once

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turned to vertical, the bottom cap must be removed. The pile should first be filled with water to equalize the head on each side of the cap so that the closure plate may be safely removed. Other methods of removing caps have been used, in a variety of circumstances. On the Drift River Terminal in Cook Inlet, Alaska, Lucite caps were used which shattered when the pile was driven down.

On the Hondo platform off Santa Barbara, California, a chain was welded in spiral fashion to a steel plate; the idea was that when the chain was pulled, it would rip open the plate like the cover of a sardine can. It worked in tests on land; it did not fully work in the actual installation, with the result that some plates had to be drilled out—a costly and time-consuming operation.

A similar failure occurred on an offshore terminal at Inchon, Korea. The steel plate closures were seal-welded and then a chain hinge provided at one side. The concept was that when the pile was driven down, the seal welds would break, the plate break free, and hinge around the chain link to move clear of the pile. It did not work that way. The steel pile broke the plate loose, but it then rode down as a plate cover to the pile, making it a closed-end pile and preventing full penetration until drilled out.

Similar closures are used to keep the jacket legs buoyant during launching, floating, and upending. They are designed to be ruptured by the impact of the pile. Nylon-corded and reinforced neoprene closures are now used almost universally. They are domed to resist hydrostatic pressure efficiently, yet they are designed to be easily cut out like cookies by the pile as it is driven through them. The closures are designed to resist the maximum hydrostatic head to which they will be exposed. On a few rare occasions these closures have turned out to be so strong that they could not be cut through by driving. They then had to be drilled out (see Figure 8.17).

Rock drills are not efficient in drilling rubber; as one can imagine, the teeth become fouled and the water jets cannot clear the rubber. It requires many trips of the drill stem in and out of the pile in order to finally clear the seal. One solution, then, has been to cut the tip of the pile on a scallop, like teeth, so that it slices through the seal progressively.

Reinforced rubber seals remain the state of the art despite the potential problems they can pose—for example, when drilling must be carried out through the pile and cut slabs of reinforced rubber are encountered, causing the drill to be clogged.

Diaphragm-type rubber leg closures are available in sizes from 18- to 144-in. O.D. For deep-water structures and very-large-diameter legs of jackets, sleeves, or cylinder piles, mechanically locked rubber diaphragm elements are available for pressures up to 14 MPa on 2.2 m O.D. closures and 2 MPa on 3.75 m O.D. sections. Special closure cutting tools are available (see Figure 8.16).

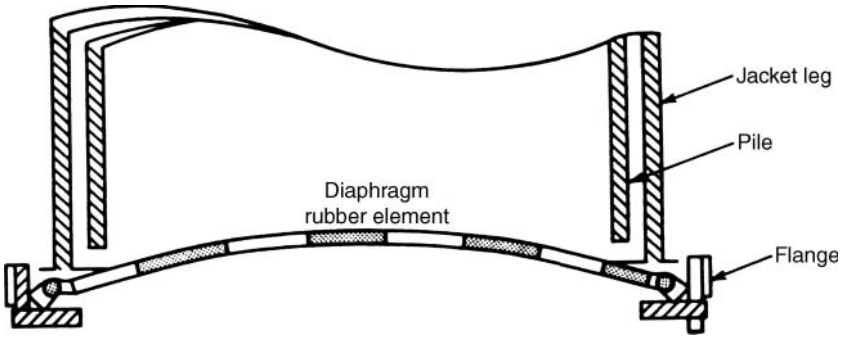

FIGURE 8.16
Pile shoe arrangements.

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Un-reinforced concrete plugs, both of lightweight and normal-weight concrete, have been employed on the concrete offshore platforms in the North Sea to seal the conductor sleeves. The plug is usually 1.5–2 m thick, with welded shear lugs on the sleeve to transfer shear. These plugs are then drilled out prior to driving the conductors.

Increasingly, skirt piles for deep-water offshore platforms are arranged in clusters around the corner jacket legs and their loading transferred to the jacket by means of sleeves bracketed out from the sides. The final top of these piles will then be underwater a distance equal to the water depth less the sleeve length. This latter is usually 20–30 m. The pile connection is made by grout.

To drive the pile so far below water requires the use of either an underwater hammer or a follower. Several types of hydraulic underwater hammers are now made, two of which can fit inside the pile guides which are bracketed out from the jacket at higher levels. These "slim" hydraulic hammers are now employed for piling in deep water since they deliver essentially full energy to the pile, without the losses inherent in the use of followers. However, for shallow water and inland marine structures, followers are usually employed. The follower is a thick-walled pile section with a machined driving head on its tip which fits snugly over the head of the pile, transmitting axial compression, while preventing local buckling.

Occasionally, due to misalignment or minor variances in the pile head, the pile becomes jammed into the driving head and the follower cannot be removed. Then the pile must be cut off, either by divers or else by a drill rig using expanding casing-cutter tools. To prevent excessive delays under such a circumstance, the corrective tools should be on board.

Experience shows that with a properly fitting driving head, a square cut on the pile, and a pile wall thickness that is not too small, that is, not less than 25 mm, there is very little loss in efficiency by use of the follower.

Where excessively hard driving is expected—as, for example, when driving though limestone or caprock strata—a driving shoe should be provided at the pile tip. API RP2A suggests that this be at least one diameter in length and have a wall thickness 1.5 times the minimum thickness of the parent pile section. Experience in driving through weak limestone containing embedded basalt cobbles has indicated that such a shoe should be two diameters in length to prevent buckling like an accordion. Steel quality should be as high yield as can be properly welded; since the weld is made in the shop, it can be properly pre-heated and post-heated as required.

Cast steel shoes are available for the small- to moderate-diameter piles, also for steel H piles. These are more easily affixed and welded (see Figure 8.17). The shoe should normally have the same internal diameter as that of the pile in case it becomes necessary to drill through the pile as a casing; otherwise, the drill may catch. The slightly larger diameter tip will relieve some of the skin frictional resistance on the main pile body. Where this is judged unacceptable by the geotechnical engineer, an internal pile shoe can be used, with its consequent restrictions on any drilling.

When grouting piles to the jacket sleeves or when grouting between an insert pile and a primary pile, it is essential that the spaces be completely filled. Experience has shown that grout can trap water and bypass it unless great care is taken. As noted earlier, the steel surfaces should be free of mill scale or varnish. Mud must be excluded from the annulus; this may require the use of wipers when working in very soft muds. The steel surfaces must also be clean of marine growth (which may form in relatively short periods of shallow submergence) and free from oil or other contamination. Bentonite drilling mud should be flushed out with water where this can be done without endangering a drilled hole. If it cannot be safely flushed out, a polymer-based mud should be used.

Both neat cements and expanding cements are used; the latter can give improved bond and shear transfer. API RP2A requires that an expansive, nonshrinking grout be used.

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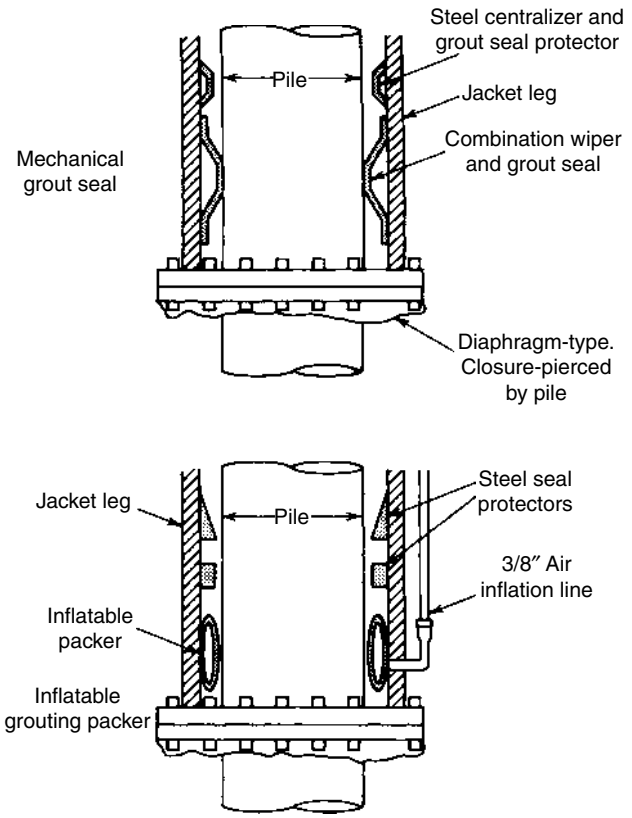

FIGURE 8.17
Wiper seals and packers for pile jacket leg closures. (Courtesy of Oil States Rubber Co.)

Admixtures are employed to promote flow, to reduce tendency to segregate or wash out, to provide controlled expansion during the curing period, and to reduce bleed. Shrinkage under water is actually not of much magnitude, but bleed is an undesirable property, which should be minimized. The bond is also affected by movements and vibrations of the structure during the setting period. Speed of set and strength gain are controlled by the type of cement and its fineness of grind as well as by the water temperature and by the use of admixtures. The grout should in any event develop at least 10 MPa within 24 h. Special grouts are available, which give improved bond strength, decreased bleed, and decreased shrinkage. Mechanical devices such as shear lugs, strips, and even weld beads, can be installed on the inside of the sleeve and on the outside of the pile to improve bond. Lugs must be designed to permit proper flow of grout and beveled so as not to trap bleed water under them. If grout is placed so that it fills not only the annulus but also the body of the pile, consideration should be given to the heat of hydration to ensure that excessive temperatures will not be developed, which may destroy the tensile strength of the concrete through internal microcracking. The width of the annulus should be limited to about 100 mm maximum. Centralizers (spacers) should be used to maintain a uniform annulus between the pile and the sleeve. A minimum 38 mm is required by API RP2A, but 50–100 mm appears optimum. Beyond 100 mm, potential shear in the grout may reduce the transfer of loads. Packers are used to confine the grout and prevent its escape around the tip of the pile (see Figure 8.17). The packers must be so installed at the bottom of the sleeve as to protect them during pile entry and driving. Experience shows that packers are often damaged; hence a double set may be a prudent precaution. Some designs of packers are passive, that is, just flexible rubber. Others are expanded by

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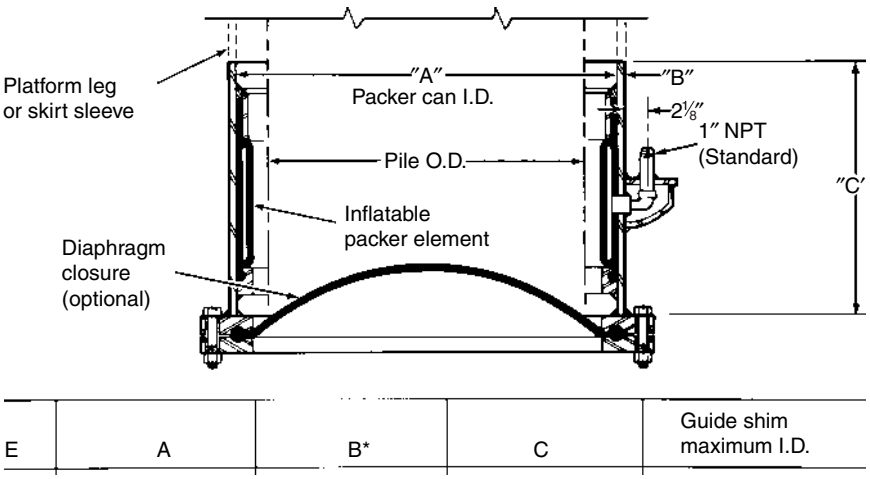

FIGURE 8.18
Diaphragm-type neoprene leg closure with inflatable grouting packers. (Courtesy of Oil States Rubber Co.)

water pressure or by the grout itself. In the latter case, the grout first fills the packer; then as the back pressure rises, the grout opens a flap valve into the annulus (See Figure 8.17 through Figure 8.20). When a packer is damaged and the grout is escaping, all that can be done is to allow the grout to set and try again. Unfortunately, it will tend to set in the grout pipe also. Flushing out slowly with water at minimum pressure can be used to keep the grout pipe open for the second injection.

For this reason, two grout pipes, with entry ports spaced 2–4 m apart vertically, are often installed. If grout escapes around the pile tip, the first grout pipe can be abandoned.

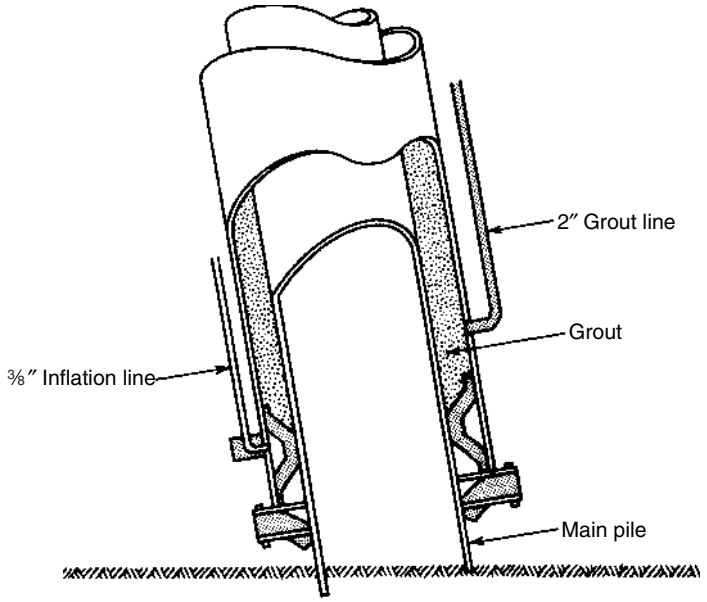

FIGURE 8.19
Typical inflatable packers and grouting arrangement. (Courtesy of Oil States Rubber Co.)

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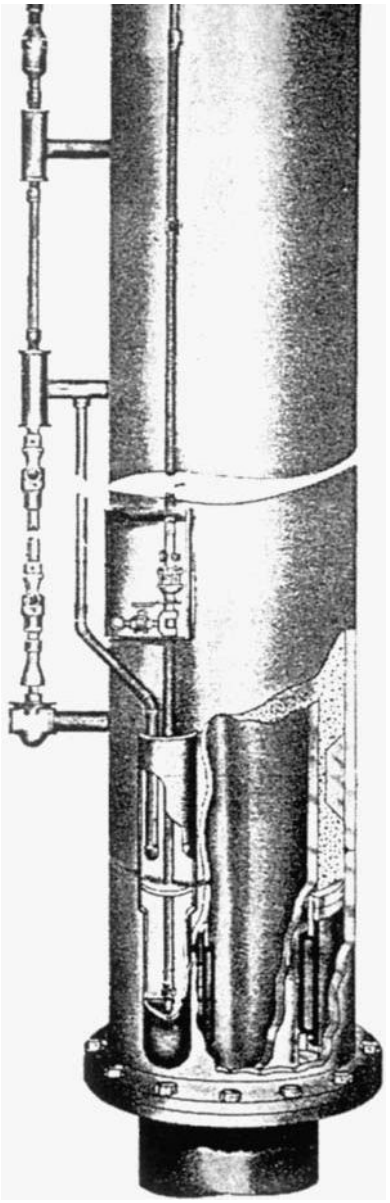

FIGURE 8.20
Reach-rod grouting system with multiple inlet ports. (Courtesy of Oil States Rubber Co.)

However, water should then be circulated slowly through the second pipe to prevent any possibility of its plugging (see Figure 8.21).

The grout equipment should maintain continuous flow until the annulus is completely filled. If the configuration and relative elevations do not permit grout to be returned to the surface to verify complete filling, then suitable means should be employed, such as electric resistivity gauges, radioactive tracers, well-logging devices, or overflow pipes, which can be verified by divers or ROVs (see Figure 8.22).

Recently, a new method of locking piles to sleeves has been developed in which the pile is "forged" into recesses in the sleeve by means of intense hydraulic pressure. This

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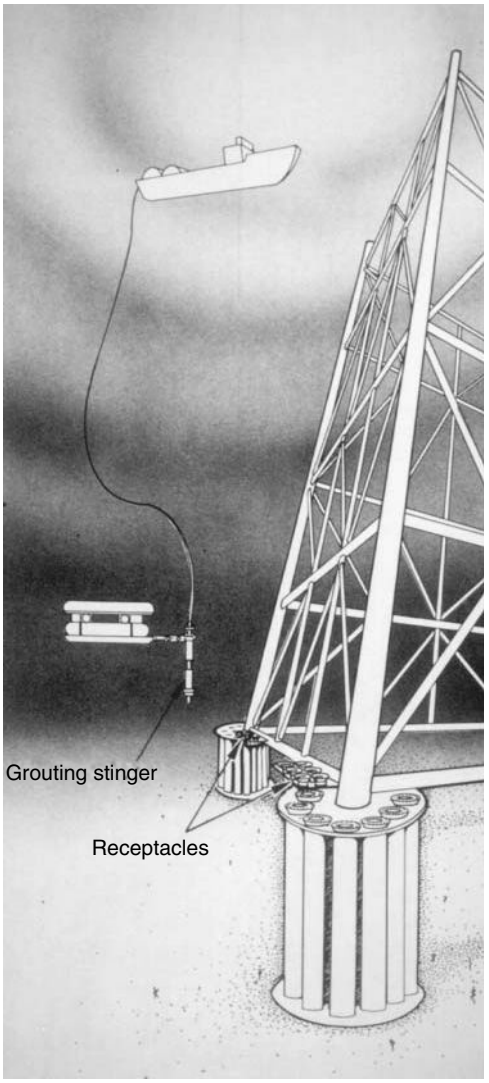

FIGURE 8.21
ROV grouting system. (Courtesy of Oil States Rubber Co.)

method, known as "HydraLok," has been successfully employed to fix pin piles to the sleeves of subsea templates of the Balmoral and Southeast Forties fields in the North Sea (see Figure 8.23). It has recently been used at depths up to 1000 m. While it forms a reliable clamping, it is usually supplemented by grout for the permanent connection.

Pile installation records must be kept to record the following data:

1. Pile identification.
2. Lengths of each segment of pile.
3. Penetration of pile under its own weight, after penetrating the pile closure.
4. Penetration of pile under weight of hammer.
5. Blow counts throughout driving.
6. Unusual behavior of hammer or pile during driving; for example, a sudden

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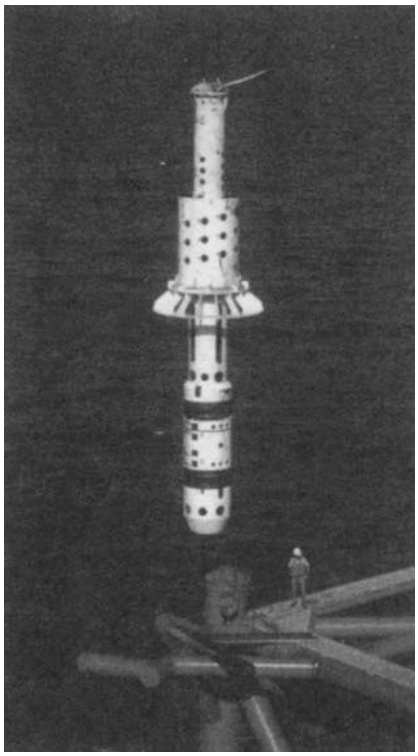

FIGURE 8.22
The Hydra-Lok pile connection can be used at any depth to lock piles to sleeves. (Courtesy of Oil States Rubber Co.)

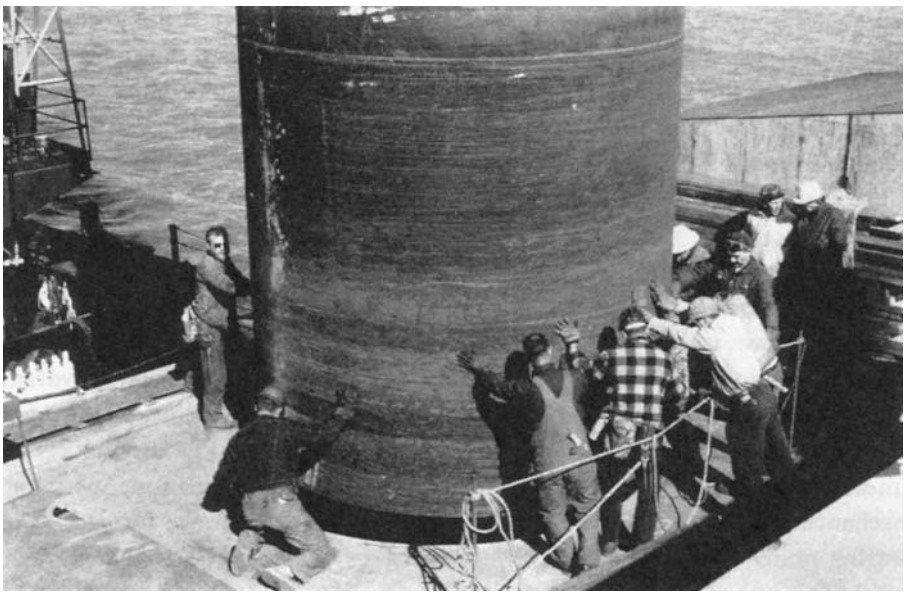

FIGURE 8.23
Seating 4 m diameter pile of offshore terminal, Cook Inlet, Alaska.

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decrease in resistance which is not explicable by a review of the soil profile may indicate a ruptured weld.

7. Interruptions during driving; "setup" time and the blows subsequently required to break the pile loose.
8. Location of welds.
9. Welding procedures and rods employed.
10. X-ray or other NDT.
11. Elapsed time of driving each pile section.
12. Elevations of soil plug and internal water surface after driving.
13. Actual length of pile section and length of pile cutoffs as each section is added.
14. Grout mix.
15. Equipment and actual procedure employed.
16. Volume of grout placed.
17. Quality of grout at intervals (tests).

Five examples of recent offshore pile driving experience follow.

The Hondo platform was built in a water depth of 264 m. The soils are primarily finely grained, normally consolidated cohesive silts. There are eight main piles, up to 382 m in length; 12 skirt piles were driven. The main piles are 48 in. O.D., the skirt piles 54 in. O.D. Breech-block connectors were used for the followers for the skirt piles. The main piles required 13 add-on sections due to their long length. Hammer size was limited by the then-current availability to the Vulcan 3100 hammer, with a Menck 4600 hammer as a backup, both developing about 400 kN-m of energy per blow. Derrick boiler capacity was also a constraint. Extensive investigations and analyses were made to predict pile-driving performance. Based on prior experience, hammer efficiencies were assumed at between 55 and 80%, with 80% as the most probable. Pile-driving logs, hammer blow counts vs. driving resistance, and pile head forces were all generated for each assumed efficiency rating. Special attention was directed to the last three add-ons.

The Thistle Platform in the United Kingdom sector of the North Sea was built in 161 m of water depth. Piles were designed for ultimate axial loads of 35 MN (3500 metric tons). These required penetrations up to 140 m below the seafloor into hard clay with multiple sand lenses. Wave equation analyses showed that only about 30-m penetration could be achieved by the available hammers. Therefore, a two-stage pile solution was selected. In the first stage, 1.37-m piles were driven to 30-m penetration using a Vulcan 560 hammer (400 kN-m energy/blow). The piles typically formed a plug at about 25-m penetration, when the driving resistance would rise from 150 to 250 to 600 blows/m. Some of the plugs could not be broken free by driving and had to be drilled out to enable further penetration. Therefore, a 1.5-m-long driving shoe of steel pipe 12 mm thicker than the normal wall was attached to the tip; this was beveled to force the soil out from the pile tip rather than wedge it inside. Piles were driven with a follower, assembled by breech-block connectors. A hydraulic clamping-aligning frame was used to hold the pile add-on. From 30 to 140 m penetration, holes 1.21 m in diameter were drilled. Salt water was used as the drilling fluid (with return to deck level at +30 m) until an 85-m penetration was achieved. Then because of interbedded sand lenses, drilling mud was employed. To prevent hydraulic fracturing in the sand lenses, the mud specific gravity had to be very carefully monitored and controlled.

Holes were drilled with a flat bit, using airlift reverse circulation in a dual-walled drill pipe. When the holes had been drilled, they were gauged and found to vary from 1.22 m in

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diameter in clay to 1.52 m in sand lenses. The insert pile, 1.06-m pipe pile, was sealed at the bottom by a plug, weighted by filling with heavy drilling mud (1.8 sp. gr.) to overcome buoyancy, and seated in the hole. Grout of 1.68 sp. gr. was injected through a valve in the insert pile plug to completely fill the annulus. The grout had a radioactive isotope admixture added; when it ran out of the grout overflow pipe at the top of the first stage pile, it was detected by a Geiger counter. The grout was allowed to set 24 h, and then the weighted mud was pumped out.

For the Heather Platform, also in the United Kingdom sector of the North Sea, large piles were driven to extremely high capacities in hard, sandy, silty clay. Pile design loads were 29.5 MN, requiring an ultimate capacity of 44.3 MN. Pile penetrations of 43 m were required. Total pile length was 96 m. Piles were of 1.52 m diameter and used 64-mm walls throughout to enhance drivability.

Connection to the sleeves was by grout. To improve bond transfer, rings of weld beads were run on the piles. The first pile section was 64 m long, the second, 32 m. To expedite add-on, a hydraulic clamping and aligning device was stabbed onto each first section. An internal driving shoe, 87 mm thick and 0.5 m in length, was attached, having the same outer diameter as the pile. Pile-driving performance was predicted by means of geotechnical investigation and use of the wave equation. Special attention was paid to the mechanism of plug formation. It was predicted that plugs, once formed, would give only partial end bearing under the hammer blows because of the different transmission velocities of the compressive wave. This was partially confirmed by the actual performance, in which blow counts built up to 500 blows/m and then stayed about constant for further penetration. A pile follower was used, made up of two sections. Gravity connectors (machined for direct bearing) were used between the follower sections and between the follower and pile.

To achieve the very high capacities, Menck 8000 and Menck 12500 hammers were employed, the latter developing 2000 kN-m of energy/blow. Initial driving, however, was started with the smaller Vulcan 560 and Menck 4600 (400 n-m/blow) to reduce pile stresses. Pile performance was carefully monitored by means of strain transducers and accelerometers in the pile head. These showed that the losses in gravity connectors were only 2%, except in one case of misalignment. Hammer efficiencies ranged from 40 to 62%.

At the site of the Maui A platform, clayey soils were interbedded with a dense layer of volcanic ash. Some piles were able to penetrate this layer; others hung up at refusal. The problem was aggravated by hammer breakdowns and weather delays, leading to high setup. Eventually, jetting was employed to break up and remove the plug of ash, which had transformed the piles into end-bearing piles. A similar problem with volcanic ash occurred on a land piling project, where long steel pipe piles were being driven. Here also it became necessary to remove the ash plug, in this case by drilling out the pile plug.

Like many sub-Arctic and Arctic areas, the recent sediments of Cook Inlet are underlain by glacial till and overconsolidated glacially derived silts. The glacial till is similar to the boulder clay of the North Sea in being very stiff and containing rock fragments of various sizes. The overconsolidated silt is extremely difficult to penetrate under hammer impact alone; the material is so dense and its structure so strong that it can neither be displaced nor consolidated. Large cylinder piles from 2 to 4 m in diameter have been installed in the several Cook Inlet terminals by taking advantage of the rapid breakdown of the dense overconsolidated silt that occurs when water is introduced by jetting. A ring of jets has been built into the pile tip, arranged so that the jets can be continuously operated while the hammer works. This jetting then breaks up the overconsolidated silt into a colloidal suspension (see Figure 8.25). Powerful free-jets can also be used to wash holes in the dense soils, which then allow further penetration of the pile. Similar benefit to pile installation has been reported from the East Dock at Prudhoe Bay, where jetting enabled

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ready penetration of 25-m-long steel sheet piles through overconsolidated silt and partially ice-bonded sands.

For the tubular steel piles of the Jamuna Bridge in Bangladesh, a Menck 12500 pile hammer was employed developing 2000 kN-m (300,000 ft.-lbs) energy per blow. The piles were 3.15 and 2.50 m diameter, wall thicknesses 45–65 mm, length 80 m. They were driven through 60 m of medium dense and dense silty sand to found in a gravel layer (see Figure 8.24). Although the contractor was equipped to jet or to clean out the piles to aid penetration, he did not do so, relying on the hammer alone to achieve penetration. About 12,000 blows were required. Because the piles were battered 1 on 6, the contractor installed the pile in three segments. Splices took 6–8 h with four teams of welders.

For driving a single 4 m diameter steel tubular pile offshore San Diego, California, to a depth of 70 m to act as a riser for an ocean outfall, the contractor elected to use a specially fabricated hydraulic drop hammer (free-fall hammer), developing 3000 kN-m

FIGURE 8.24
Driving 3.15 m×80 m diameter steel tubular pile, Jamuna River Bridge, Bangladesh. (Courtesy of High Point Rendel.)

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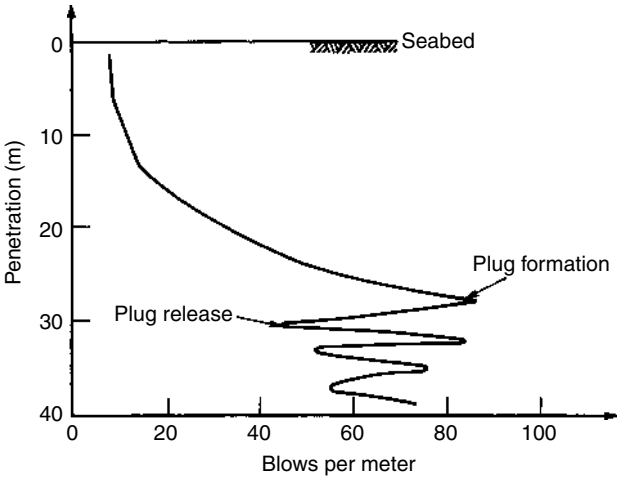

FIGURE 8.25
Typical pile penetration vs. blow count in hard clay.

of energy/blow, at a rate of about 1 blow/min. The soil was weak siltstone. Intermittent clean-out and drilling ahead was employed to aid penetration. While successful in eventually attaining penetration, the time required was excessive.

For the East Bay Replacement Segment of the San Francisco-Oakland Bay Bridge, the 2½ m diameter piles, with wall thickness ranging from 65 mm at the top to 45 mm at the tip, were driven to 100 m penetration with the Menck 1700 kJ hammer. The piles were installed in two or three sections, thus requiring splices in the field. These were carried out by semi-automatic welding machines.

Typical behavior of a large diameter pile is shown in Figure 8.25. Similar case histories on driving piles in very deep water (over 500 m) are given in Chapter 22.

8.5 Methods of Increasing Penetration

Methods of increasing the penetration include the following:

1. Use a heavier-walled pile section; that is, increase the minimum wall thickness. The wave equation indicates a substantial improvement in penetrating ability when a thicker minimum wall is employed.
2. Use a larger hammer, especially one with a heavier ram.
3. Jetting internally, in order to break up the pile plug. In many soils, a plug will form by compaction in the pile tip and transmit bearing through skin friction on the inside of the pipe pile. This plug may be broken up by jetting or drilling and thus eliminate the end-bearing resistance temporarily. However, the plug may reform in the next 5–10 m or so of penetration, requiring repeated operations. The removal of the hammer and insertion of a jet, sometimes 100 m or more in length, is a time-consuming rigging operation. Quick connectors similar to drill casing can be used to connect the jet pipes, minimizing the time required.
4. Drilling to remove the plug.
5. Drilling ahead of (below) the tip.

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As noted earlier, jets have been built into the pile, attached to the inside wall, and connected to a jet water supply by sleeves through the pile wall, below the head. These enable the continuous breaking up of the plug without interruption of driving (see Figure 8.26 and Figure 8.27). Such jets are best permanently attached to the pile and abandoned in place. Attempts to remove them are very time consuming, usually costing more than the value of the jets. More serious is the fact that loose-fitting sleeves, designed to facilitate later removal, may not adequately support the jets during the combined driving and jetting operation. Multiple jets are sometimes installed, spaced at about l m or less around the inside periphery. The nozzles should terminate 150 mm or so above the tip. If so installed, the jets must be run continuously during driving in order to prevent plugging.

Typical jet characteristics for jets built into the pile are:

- Diameter 40–50 m
- Pressure 2–2.5 MPa (290–350 psi) at pump.
- Volume 700 l/min (175 gpm) per jet pipe.

When jets prove to be inadequate, then the plug may require removal by drilling. A down-the-hole drill may be used to create a cluster of closely spaced holes, which then

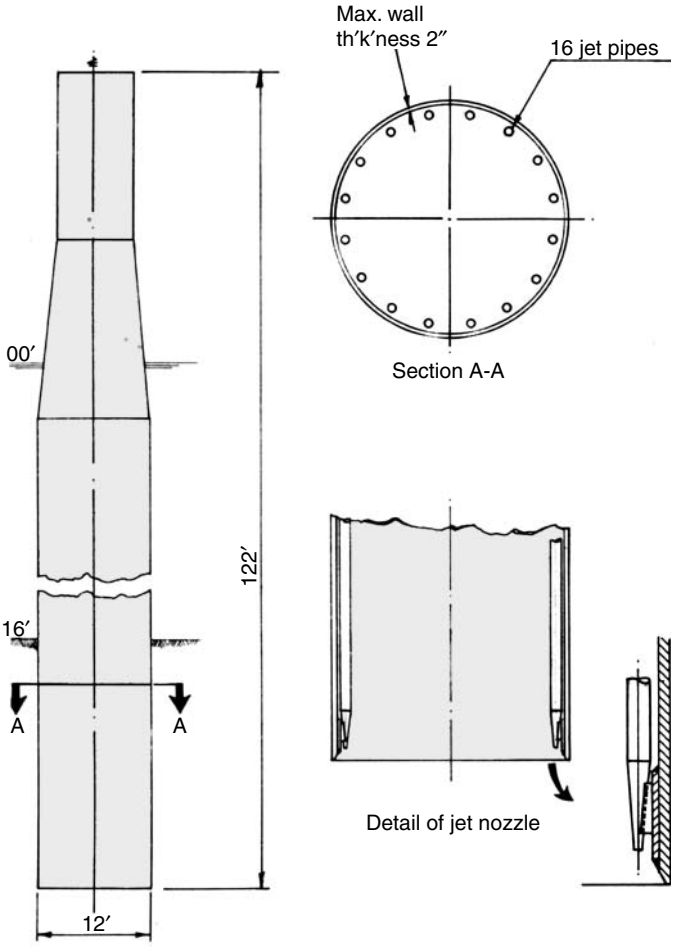

FIGURE 8.26
Jet pipe arrangement for 4-m diameter piles, Cook Inlet, Alaska.

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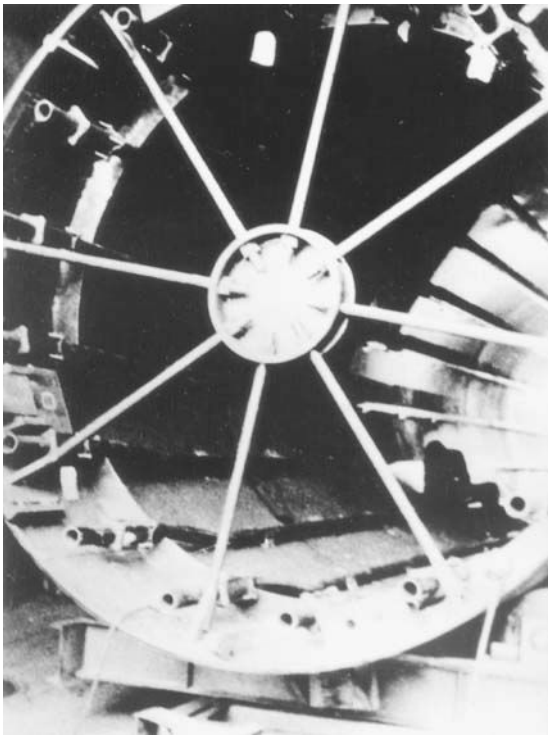

FIGURE 8.27
Jet pipes arranged in steel cylinder pile.

allow the pile to break through the remaining soil or rock under the hammer blows. Such a procedure is typically required when driving into hard clay, sandy clay, or weathered rock or through interbedded limestone or sandstone.

In inshore and harbor projects, jets are often run ahead of the pile, as free jets, or alongside the pile, in order to reduce skin friction. Such practice is generally not followed in offshore work because of the tendency to destroy the bearing and lateral capacity of the soil around the pile. Further, a free jet is extremely difficult to control as to direction and position when working at depth. However, in cohesionless material such as sands, a free jet may be used ahead of the pile, since the subsequent driving will reconsolidate the sands. Piling should be driven a specified number of blows (100–400) after all jetting has ceased in order to insure reconsolidation. Such a free jet "probe" is also useful in determining the character of the soils ahead of the pile tip and in identifying hard layers, for example, coral and limestone in the Mideast and tropical regions.

API RP2A describes the removal of the soil plug inside the pile by jetting and airlifting. The use of a drilling sub or swivel is an even more efficient way of removing compacted material, such as decomposed granite, from the plug. In the rare cases, sometimes encountered in offshore terminal construction, when the pile is plugged with gravel or cobbles, then an airlift-plus-jet combination may be used to clean out the pile. An airlift can remove cobbles of almost the same diameter as the airlift. When cobbles and small boulders must be removed, a hammer grab may be the most efficient.

When the soil plug in the pile is removed to facilitate continued driving and deeper penetration, it may be replaced by a grout or concrete plug after design elevation is reached, which is placed underwater by tremie means. The concrete or grout plug must have sufficient length to develop its full capacity by bonding to the inside of the pile. Grouting through the plug will restore the native density of the sediments.

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Excessive heat may be developed when filling large-diameter (1-m or more) piles with cement grout; this may then result in cracking and even disruption. Thus, the use of concrete, which contains a substantial portion of aggregate but lower cement content and hence, lower heat as well as higher tensile strength, may be indicated. Replacement of 50% of the cement by fly ash or use of a blast furnace slag cement will reduce the heat of hydration (Figure 8.28 through Figure 8.31).

When tubular piles encounter hard layers or boulders, it may be necessary to drill them out ahead of the tip. Usually, the internal plug is removed by jetting and airlifted down to a meter or so above the tip. Then, the pile is reseated with the hammer to prevent a run-in of sand under the tip. The water in the pile should be balanced with the water outside; flow in either direction is undesirable. Then, the drill drills out the boulder.

A churn drill will "wander" slightly and hence may drill a hole somewhat larger than the drill diameter, enabling the pile to be driven through. A rotary drill or down-the-hole drill will drill a more regular hole. In very hard rock, it may be necessary to use underreamers or hole-opening drills in order for the pile to be driven. In the more normal cases, especially where the hard layer is of limited thickness, it may be possible for the pile tip to progressively chip away the edges of the drilled hole by driving after drilling. If such layers are anticipated, the pile tip should be reinforced. Normally, where drilling through the pile is anticipated, the inside should be flush all the way (same inner diameter), which means that a heavier-walled tip or cast-steel tip protector will increase the outer diameter.

FIGURE 8.28
Prestressed concrete cylinder piles to support New Bridge pier. (Courtesy of Ben C. Gerwick, Inc.)

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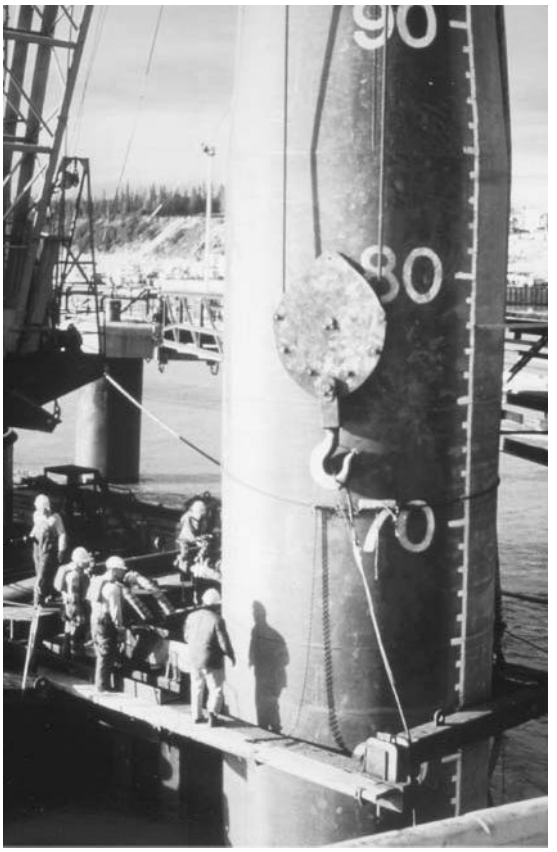

FIGURE 8.29
Positioning 3m diameter steel cylinder pile in 3m/s tidal current. Cook Inlet, Alaska.

This may be beneficial in making it easier for the rest of the pile to drive through but will, of course, reduce the permanent skin friction.

If drilling ahead is required but the drill will not bite on the gravel, cobbles, or sloping rock, one solution is to inject grout below the pile tip to form a plug in the base, which after hardening serves as a starter for the drill. Another solution is to use a hammer grab.

With large-diameter piles of relatively short length, such as are often used in offshore terminals, the hard layers may be broken up by chiseling or by drilling several small holes. A chisel can be made from a piece of shafting or from heavy-walled pipe filled with concrete. It may be most effective if driven with the hammer. Care should be taken not to drive too far ahead or the chisel may become stuck.

Drilling and blasting ahead of the pile tip to break up coral and limestone layers have been successfully employed in the Bahamas. This may damage the pile tip (splitting or curling it). In hard clays, boulder clays, and glacial till, it may be necessary to drill ahead in order to obtain the required penetration. Generally speaking, the diameter of the drilled hole should be somewhat less than the pile diameter, e.g., 50%–75%, in order to preserve the consolidated soil pressure against the side of the piles to as great an extent as possible. Drilling ahead may be done with saltwater or with drilling mud to keep the hole open. Salt water is easier and cheaper and does not coat the walls of the hole. In some cases, it may be possible to keep the water level higher inside than out and thus help to hold the sides of the drilled hole open. Polymer muds, although more

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FIGURE 8.30
Positioning pile in frame cantilevered from stern of crane barge, Cook Inlet, Alaska. (Courtesy of Ben C. Gerwick, Inc.)

expensive, help the bond of concrete and are environmentally acceptable. Usually, the length of the hole that may be drilled ahead is limited in order to prevent caving. This means that drilling and driving must be carried out in an iterative manner, repeating one after the other.

API standard RP2A notes that when a hole is formed by drilling an undersized hole ahead of the tip, the effect on pile capacity is unpredictable unless there has been previous experience in the same soils under similar conditions. Jet drilling ahead of the pile tip, while often effective in aiding penetration, is especially unpredictable regarding its effects on eventual pile capacity in cohesive soils.

8.6 Insert Piles

When it proves impossible to drive the primary pile to the required tip elevation, another solution is to drive an insert pile. The soil plug is first removed from the primary pile, and then the insert pile is placed and driven ahead. This pile, of smaller diameter, will be free from skin friction over the length of the primary pile and hence can usually be driven to a substantial additional penetration. High strength grouts are available to grout the annulus between primary and insert pile. However, due to inherent limitations on flow such as channelization, conservative values for bond strength are recommended by API (see pp. 8–17).

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FIGURE 8.31
Piles for Rio Niteroi Bridge, Rio de Janeiro, Brazil, were socketed into rock by pile-top drills. (Courtesy of H.V. Anderson Engineers.)

When insert piles are pre-planned, they usually give good performance, although representing a substantial increase in material cost. When used as an emergency measure, several problems arise. One is due to the fact that the thicker wall section of the primary pile will be up in the jacket instead of at the mudline; hence the pile's moment resisting capacity may be less than designed. Grouting in of the insert pile through this zone helps to restore the lost moment capacity. Another disadvantage is the decreased friction area per meter of pile penetration of the insert pile as compared with the primary pile. Hence, the insert pile may have to penetrate more deeply. A third problem is the carrying out of the grouting operation to fill the annulus between the insert pile and the primary pile. Since this is usually not a planned operation, it may be necessary to insert a small-diameter (20- to 40-mm) pipe between the two walls in order to grout from the bottom up. Alternatively, a grout pipe may be installed inside the insert pile, with exit nipples just above the estimated location where the insert pile projects above the tip of the primary pile.

8.7 Anchoring into Rock or Hardpan

With offshore terminals, piles frequently end up on hardpan or rock without having developed sufficient uplift capacity. Several solutions are practicable. One is to drill ahead into the rock a long socket of slightly smaller diameter than the inside diameter

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of the pile. An insert pile, either a pipe pile or H-pile or cross (+), made up of plates, is inserted in the socket hole with grout pipes attached. It is then grouted up, bonding the insert pile to the walls of the socketed hole and to the primary pile above.

A second method has been to fill the primary pile with concrete, having previously set in a small diameter tubing (say 150-mm diameter). A hole is drilled through the preformed hole beyond the pile tip, deep into the rock, and a post-tensioning tendon is inserted with grout pipe attached. The tendon is grouted to the rock. Then the bar is stressed and anchored. Finally, the tendon is grouted to lock it to the pile and to provide corrosion protection. While drilling costs are reduced, this method requires care in controlling the levels between first- and second-stage grouting; this point should be at the pile tip. The tendon is in effect a ground anchor used to tie down the pile. This method was successfully used for the Esso Terminal in Singapore.

Another solution is to drill the socket, set in a reinforcing cage, and then grout or concrete it to the socket and primary pile. The bars of the cage should be well spaced to give ample room for the concrete or grout to flow through them. Vertical bars may be bundled in groups of two, three, or four. Spiral reinforcement should be spaced out; if necessary, it can be made of heavier-gauge steel or bundled also. Spacings and clearances should be at least five times the size of the coarse aggregate. This scheme is employed on many piles for overwater bridges.

A modification of the above is to set in a precast plug that contains all the reinforcement and a grout pipe. The walls of the precast plug should be roughened to ensure good bond. Then the annular space between plug and socket and plug and pile is grouted, all in one operation. This method reduces the quantity of grout required and is a very practicable solution. As another alternative, it may be most practicable first to fill the socket with a fluid grout or concrete mix containing a retarding admixture and then to set in the insert pile, driving it if necessary. In this case, one can be sure that the grout completely fills the annular space.

Attempts to grout between the walls of a driven insert pile and the rock through holes in the pile have generally not been too successful, due to inconsistencies in filling. The grout tends to channel, trapping water on one or more sides. Use of greater pressure may fracture the formation.

On the Goodwin platform on the Northwest Shelf of Australia, thin-walled tubular steel piles were selected since the driving would be through weak calcareous sands. However, there was a hard stratum of cemented sands (cap rock) at the surface. Whether due to initial bending or other causes, a progressive longitudinal buckle developed, causing a fluting of the pile wall that ran full length. It prevented insertion of the drill to drill out the socket at the tip, so the steel buckle had to be drilled out at great cost and delay.

Precast concrete cylindrical segments were then placed and grout injected. Use of these prefabricated plugs eliminated the logistical problems of producing quality concrete 150 km offshore.

8.8 Testing High Capacity Piles

Under ever larger design loads, there have arisen methods to test and prove the capacity for both axial compression and tension.

The Osterberg Method allows this to be achieved by flat jacks placed at the tip of the pile. The hydraulic operating fluid pressure gives the skin friction developed above the jacks as well as the end bearing capacity below. Where this is less than the skin friction, in order to develop the necessary reaction, an additional plug may be constructed in an

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extension of the drilled socket that is not connected structurally to the pile length above, which is being tested.

It is essential in concreting the Osterberg device that the concrete flow in and between the flat jacks and be of high strength. Grouting with concrete containing anti-washout admixture (AWA) or silica fume may be appropriate. This also minimizes bleeds.

Dynamic tests may be also be used. These include the APPLE system in which a very heavy ram is dropped 2–3 m and the strain and accelerations are analyzed.

For testing the lateral capacity of large piles (as well as the vertical capacity), the Dynamatic System using a rocket-type explosive may be used. However this is not capable of multiple cycles representative of wave or seismic reactions. One other system that is suitable is to drive two piles a few meters apart and then employ a long-stroke jack. The "shadow" effect of each pile on the other must be considered.

8.9 Steel H Piles

These have good ability to penetrate hard strata because of their limited end bearing area. As noted earlier, when driving into cobbles or cemented soils, pile tips should be affixed so as to prevent local buckling under the high compressive stresses that develop during driving. Cast-steel tips are available commercially to be welded on.

H piles have high bending moment capacity on the *X*–*X* axis but are weak on the orthogonal *Y*–*Y* axis. They should always be rolled and lifted on their strong axis.

To splice steel H piles, full penetration welds with low hydrogen welding rods are used. Roller plates of steel or wood may be fitted to facilitate turning of the pile during fabrication before installation. Back-up plates are usually required so as to permit one-side welds. The new section must be accurately positioned as an axial extension of the first section and rigidly held by angle braces. Vibration, as from current or waves, and wave splash, will make it difficult, if not impossible, to attain satisfactory splices in the field.

To prevent local distortion of the pile head, a driving head that has grooves or similar tight-fitting restraints should be used.

Steel piles, which will penetrate the water line in service, will often be epoxy coated over the top length. This coating should be protected from scratching and abrasion, e.g., the use of pads and softeners.

A number of cases of damage to steel piling have already been noted, including collapse of the tip unexpectedly when encountering a boulder or rock. Continued driving at near refusal caused high dynamic rebound compressive stresses at the tip. The solution is to stop driving and to drill ahead, using a drilling sub or down-the-hole drill.

8.10 Enhancing Stiffness and Capacity of Piles

Concrete infill can significantly increase the axial capacity, can prevent local buckling and increase stiffness, and increase global buckling capacity. Shear between the walls of the pile and the steel tube can be prevented by a combination of the adhesive bond and by shear lugs or studs. Heat of hydration can be reduced by replacing 30% or so of the cement by fly ash or by use of blast furnace slag-cement.

Piles can have their capacity increased by grouting after installation. Grout injected along the sides, especially near the tip, can significantly increase skin friction in both

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compression and tension. End bearing can be enhanced on displacement piles by pressure grouting after installation.

On the Jamuna Bridge in Bangladesh, tremie concrete plugs were installed near the base and grout injected underneath at a pressure equal to the factored design load (limited by the uplift capacity of the pile in skin friction). On the Postsdamnplatz Project in Berlin, grout injected along the sides near the tip increased the tension capacity of the piles used to anchor the tremie concrete slab.

8.11 Prestressed Concrete Cylinder Piles

Where lateral bending capacity is required, prestressed concrete cylinder piles are employed. These piles have been extensively used for deep-water offshore terminals, such as the Ju'Aymah Terminal in Saudi Arabia; for bridges, such as the San Diego-Coronado Bridge in California and the Chesapeake Bay Bridge in Virginia; and to support fixed oil drilling platforms in Lake Maracaibo. Lengths have been up to 50 m, diameters up to nearly 2 m.

The walls of these piles, being inherently thin (125–200 mm) are highly stressed, both axially and circumferentially, and heavily reinforced. Tolerances in concrete dimensions and reinforcing steel placement must be controlled to relatively close limits.

Spirals or hoops are required to resist the splitting tensile strains developed in driving as a result of the Poisson effect, and to develop plastic hinges for control of high compression. Additional circumferential strains may be created in open-ended piles by water hammer. As a rule of thumb, the spirals at yield should develop at least the full tensile capacity of the concrete, so that any vertical cracks formed as a result of driving or shrinkage or thermal strains are closed when the splitting force ceases. Typically 1.2% steel is required. By proper spacing, the concrete mix is able to flow around the steel spirals and any longitudinal strands and reinforcement and encapsulate them. Epoxy-coated spiral or hoops should not be used because they have inadequate adhesive bond to the cover concrete, making the cover concrete subject to internal delamination and vertical cracking.

Axial prestress must be adequate to serve structural needs and to resist tensile rebound stresses in driving. The axial compression under driving plus the precompression often reaches 20–30 MPa, and even though the head of the pile is cushioned by a cushion block of wood, the tension rebound stresses in driving through soft soils can reach 10 MPa or more. Cushion blocks made of low modulus soft wood and adequate thickness reduce these high stresses somewhat. The inherent strength of the concrete in tension, perhaps 5–6 MPa, helps, but still the design prestress should be 7–11 MPa.

Another rule from experience is that the maximum size of coarse aggregate should be no greater than 1/5–1/6 the size of the clear opening in either direction nor the cover over the spiral. Since this latter is usually 50–75 mm, while the spiral and strands also limit the clear opening, the maximum size of coarse aggregate will often be 10–12 mm. One way of marginally improving the clear openings is by bundling of the bars.

The concrete mix must be carefully designed and verified by trial mix, to be sure it has the desired properties of workability, strength, and impermeability. Admixtures such as high range water reducing, along with silica fume and fly ash, the latter as a partial replacement of cement, are generally employed.

Proper curing and control of temperatures and moisture prior to, during, and after steam curing are required.

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Connections at the head can be made with a plug of concrete and dowels in the pile head. Similar closures can be effected at the tip. Driving open-ended often results in plugging the tip. This may subject the pile to increased bursting forces at the tip, requiring additional spiral.

Two schemes of prestressed concrete cylinder pile manufacture have developed. In one, segments of cylinder pile are manufactured by spinning, using a combination of vibration and centrifugal force to create the segment. Circumferential and added axial reinforcement as well as ducts for the prestressing steel must be held within tolerances against the centrifugal force, which tends to bow them out. After curing, these segments are joined end to end by epoxy glue and prestressing strands inserted in the ducts and stressed.

A modification of this system employs spinning with no vibration but has resulted in many problems, including segregation and rock pockets, as well as bulging at the ends of the segments, resulting in joint failure during driving.

In the second method, long-line pretensioning of a monolithic pile, the inner core form is rigid. After curing, the rigid form is mechanically collapsed. This method has proven to produce sound piles, in monolithic lengths up to 50 and 60 m.

Attempts have been repeatedly made to horizontally slip form the inner core of long-line pretensioned piles, but unfortunately, many cases of seriously defective piles have resulted due to slumping and delamination of the upper half and cracking due to dragging of the mandrel. This latter also has led to displacement of the spiral.

Solid prestressed concrete piles for marine structures range in cross-section from 400 × 400 mm to 600 × 600 mm and even 900 × 900 mm. The 600-mm diameter octagonal cross-section is widely employed. Lengths have been up to 40 and even 50 m. Cylinder piles of prestressed concrete range from 900 to 2200 mm in diameter and up to 60 mm in length. The walls of cylinder piles have varied from 100 to 175 mm (see Figure 8.28). For normal marine service, prestressed piles must have the ability to withstand driving stresses as well as axial plus lateral forces in service. For regions of high seismicity, additional mild steel longitudinal reinforcing is installed to provide additional bending capacity and ductility.

Prestressed concrete piles also require spiral or hoop confinement to withstand the lateral bursting stresses from driving (Poisson's effect) and to provide shear capacity as well as ensure proper formation of a ductile hinge under extreme lateral bending, such as from earthquake. Hard driving with an impact hammer will produce vertical cracks unless the confinement by spiral or hoops is adequate. Hoops have behaved better than spiral in earthquakes since they don't unravel at failure.

Concrete piles are exposed to many aggressive phenomena in the marine environment. This is especially true of the tidal and splash zones. Attention is directed to Section 4.2, where long-term durability is discussed.

For marine use, particular attention must be given to ensuring concrete durability. In arid zones such as southern Peru, Southern California, and the Mideast, alkali-aggregate reaction is a serious concern, which may be overcome by various measures such as the addition of pozzolans such as microsilica to the concrete mix. Sulfate attack is not usually a problem due the inhibiting effect of seawater. In those cases of high sulfate in the surrounding soils—e.g., gypsum—addition of pozzolans such as silica fume along with fly ash will prevent damage.

Freeze-thaw attack is of special concern in sub-Arctic environments, especially where the concrete piles are fully saturated and intermittently exposed to freezing water due to tidal rise and fall. Air entrainment is essential. Hollow-core and cylinder piles are especially susceptible to freezing climates due to thermal strains combined with moisture gradients: these can be offset by adequate spiral reinforcement.

The zone just under the wharf deck or pile cap girders is potentially subject to very high bending and shear stresses when high lateral forces are imposed on the structure, and

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hence should be well-confined by spiral. Supplementary reinforcing bars may be required in the top portion of the pile.

Driving of prestressed concrete piles in the marine soils frequently develops high rebound tensile forces, exceeding the tensile capacity of the prestressed concrete and causing horizontal cracks. Continued driving will lead to low cycle fatigue and rupture of the pile, with both failure of the concrete and breaking of some strands. It is therefore essential to control the driving stresses. The best means is by use of an adequate cushion block of softwood between the driving head and the top of the concrete pile. Cushion blocks 200–350 mm thick have been found appropriate. To prevent the softwood from early-disintegration, intermittent laminations of plywood may be used. New cushion blocks should be used for each pile, since they compress and stiffen during driving.

Prestressed concrete cylinder piles have been extensively used for deep-water offshore terminals, and for bridge piers, such as the Ju' Aymah Terminal in Saudi Arabia; the fixed oil drilling platforms in Lake Maracaibo, Venezuela; the Napa River Bridge and the San Diego-Coronado Bridge, both in California. Prestressed concrete was selected to give durability in the highly corrosive waters.

Installation through overlying caprock in such waters as the Arabian Gulf and Red Sea sometimes requires predrilling; in other cases, prolonged driving may break through. Jetting is frequently employed to aid the installation of prestressed concrete piles. The jets may be free jets, or may be incorporated within the pile walls. Due to hydraulic ram effects, the jet pipe needs to be rigid steel pipe or pultruded fiberglass-resin in order not to crack or rupture during driving, which would allow the high-pressure water to be exerted over a wider area and rupture the pile.

To prevent damage to the heads of concrete piles under the impact of the hammer, heavy spiral reinforcement should be incorporated, starting 50 mm below the head. External steel bands may be used. When the piles are driven onto rock or through rock, the tips should be similarly confined. To prevent failure under earthquake or ship collision at the critical zones of high moment and shear, especially over the 1.5 diameters below a rigid pile cap or wharf deck, heavy confining steel spiral is essential.

The book, *Construction of Prestressed Concrete Structures* (Gerwick 1993) gives detailed recommendations for the design, manufacture, and installation of prestressed concrete piles, as well as steps to take to prevent damage (see also Section 9.2.2.2 and Section 9.4.3.3).

8.12 Handling and Positioning of Piles for Offshore Terminals

The handling and positioning of offshore piles involves special problems because of the length, the lack of fixed reference points, and the continuous movement of the sea. Therefore, some form of template often becomes necessary. The typical offshore terminal employs steel tubular piles, 0.6–2.0 m in diameter, with 1.0 m diameter most common. Wall thicknesses are 20–50 mm. Lengths are of the order of 40–60 m. Use of a permanent template, frame, or jacket gives the opportunity to make splices under optimum conditions. Offshore structures are subject to cyclic dynamic loading and, hence, cumulative fatigue. Connections from pile heads to deck beams, if field welded, are not always successful, due to the vibration during welding, the difficulty in keeping the metal dry, and the sudden cooling of the weld with seawater splash. Such joints are also subjected to corrosion-accelerated fatigue.

A template can be set on the seafloor, similar to a jacket, as described in Chapter 11 on the installation of steel platforms, or it can be continued from an offshore barge or jack-up, or from the previously constructed portion of the structure. In some cases, self-floating

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templates have been used. Once one or more vertical piles have been driven, the support of the template may be transferred to these piles (see Figure 8.29).

When piles are installed on a batter through templates near the sea surface, they are, of course, in cantilever through the water column until they are finally lowered far enough to engage the seafloor and obtain support at their tip. This situation frequently occurs in the construction of offshore terminals where inclined batter piles are used to provide the lateral resistance for mooring and breasting dolphins. The deflections may be significant in deep water and may result in significant residual stresses in the piles. Various means of minimizing the deflections have been employed, including temporary evacuation of the water from the pile to provide near-neutral buoyancy. Neoprene or frangible caps such as Lucite can be installed on the pile tip, or the top of the pile may be capped and compressed air used to exclude water from entry at the tip. Removable floats can be attached to the tip, but these are awkward to install and are likely to break free. The above systems were all used on the VLCC terminal at Ise Bay, near Nagoya, Japan, because of the deep water (25–30 m).

Once driven, batter piles need to be supported at the top to prevent high moments and accompanying high stresses if the support of the top is temporarily removed. This support can be furnished by the template or by ties to previously driven piles.

Another way in which undesirable residual stresses are built into offshore piles is by re-leveling a jacket after some pin piles have been allowed to run in. The bending stresses so imposed may be significant. They led to serious problems requiring remedial action on several breasting dolphins at a terminal in Fao, Iraq. As noted earlier, good practice calls for initial setting of the jacket as level as possible, driving a few (three or four) pin piles just far enough to permit re-leveling and temporary fixing. Then, after other piles are driven, the first piles are lifted back up into the jacket, clear of the seafloor, and then once again lowered and driven. Of course, if the initial setting of the jacket was level and needed no correction later, then the initial piles can be directly driven on down. Jacket-leveling devices are commercially available.

Large-diameter cylinder piles are often used for the mooring and breasting dolphins of offshore terminals. They are often delivered by self-flotation, with diaphragm closures. Ballasting may be used to help upend them. Because of their large size, the effect of waves and currents on them may be significant. In Cook Inlet, with high-tidal-current velocities, vortex shedding caused oscillating transverse forces to act on the pile. This was in addition to the large direct force of the current. Guiding frames were required, cantilevered out from the derrick barge, with hydraulic positioning devices, in order to enable the piles to be set vertically in their correct position.

Installation methods have included driving accompanied by jetting and drilling as described in the preceding sections as well as special installation methods discussed in Section 8.13.

8.13 Drilled and Grouted Piles

Installation of drilled and grouted piles may be carried out in either one or two stages. The piles are to be placed within drilled holes, which are held open temporarily either by seawater or drilling mud. A casing must first be placed through the water and seated into the soils to prevent flow under the tip and piping due to the imbalance in fluid heads during drilling. Usually this is accomplished by just driving the casing into the overlying soils to a moderate penetration. Then a hole is drilled ahead, using either direct or reverse circulation (Figure 8.30).

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The progress of drilling depends primarily on the selection of the proper type of drill bit, suitable to the quality of the rock (hardness, toughness), and on the weight on the drill stem. The progress also depends on the ability to flush the cuttings from the teeth and to discharge them, i.e., adequate velocity and volume of the drilling fluid. If gravel is encountered, which cannot be flushed out, the particles will just roll around until they are finally ground to powder. Similar impediment to progress occurs if the fragments are not adequately flushed and discharged.

Since the piles and hence the casings for offshore piles are usually of relatively large diameter (e.g., 1–2 m), normal direct circulation will not develop sufficient return velocity of flow to transport the cuttings to the surface. The annulus between casing and drill stem is just too great in area. By building up the drill stem by means of pipe sections, the annulus can be reduced, thus increasing the return flow velocity to satisfactory velocities.

The relatively rapid flow of the fluids through the drill bits helps to clean the teeth. Conversely, the flow may erode the walls of the hole below the tip of the casing. Hence, reverse circulation is most often employed. In this operation, seawater or drilling mud is pumped into the casing to keep its level at the desired head. The fluid then moves slowly down the casing and through the teeth and then accelerates to a high velocity up through the drill stem. This high velocity can remove cuttings of high density, such as pyrites. The low velocity along the walls of the hole prevents erosion and minimizes caving (see Figure 8.31).

The head inside may be built up above the outside sea level; if carefully done to a precalculated differential head, this may help to keep the hole open. If the inside water level falls below the sea level, the hole may cave. One means of preventing this is to cut windows in the casing at the prescribed elevation above sea level. In most cases, seawater or slurry will have to be pumped into the pile, especially during airlifting. However, the inside water should be limited in its height above sea level in order to avoid fracturing of the foundation.

When holes are drilled ahead of the casing, they are in effect cantilevered beyond the tip. Hence, they must have as much guidance as possible within the casing. Centralizers or stabilizers can be used on the drill string to keep it centered. A heavy weight on the drill bit will help to align the hole vertically. These can also be used below the tip of the casing in hard and firm soils and rock. When holes are drilled on a batter, there is a natural tendency for the hole to droop. This droop can be countered to some extent by the stabilizers and by reinforcement of the drill string to increase its bending stiffness.

Drilling muds can be used to keep the holes from caving and are especially useful in sandy soils. The drilling mud has a higher specific gravity and hence overbalances any inward flow of water. It also penetrates the sands and forms a cohesive layer. Using a bentonite slurry helps to keep the hole open, but reduces the bond with grout placed later. To overcome this will require flushing of the hole prior to grouting. A thin drilling mud or seawater may be used for flushing (see Section 8.14).

Alternatively, a polymer mud may be used throughout, which overcomes the majority of difficulties but is more expensive. It actually improves bond. The polymer slurries are non-toxic. With current environmental restrictions in most coastal areas of the world, it appears definitely advisable to consider the use of a polymer slurry.

It is essential to monitor carefully the fluid weight and head of drilling mud, since a few pounds per cubic foot can amount to a significant head differential at depth. Care must be taken in all drilling operations not to "fracture the formation" by excessive head of drilling fluid. In this case, piping ensues and the drilling fluid is lost to the sea. One method to prevent this is to "spot mud" only, that is, drill with saltwater at an equal or slightly higher hydrostatic head than ambient and then "spot" drilling mud in the socket only (not the full casing) to keep it open long enough to install the pile. If excess head is developed, resulting

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in formation fracture, it may become necessary to grout the hole with a weak grout, such as a sodium silicate foamed grout. After the grout has set, redrilling in the same location must be done with care due to the tendency for the bit to wander off line as a result of differential cementation.

After the hole has been drilled, the pile is placed and grouted. It should be centered in the hole by spacers to preserve an annulus for grouting. As an alternative to inserting a pile, the drill string may be used as the pile by fitting expendable cutting tools to the tip to avoid the time required for removing the drill and insertion of the pile. This is especially effective for tension piles.

In the case of a two-stage drilled and grouted pile, the primary pile is driven to a predetermined tip elevation. This elevation is selected as one at which there is confidence that the pile can be driven and below which there is confidence that the hole can be kept open. This outer pile then becomes the casing for the drilling of the second-stage pile. When piles are placed in drilled holes (beyond the tip of the casing), the diameter of the drilled hole should be at least 150 mm larger than the outside pile diameter.

As described earlier, drilled and grouted piles using two stages were used very successfully on the Thistle Platform in the North Sea. The primary piles were driven through the overlying sands and clays having interbedded sand lenses to seal in a dense clay stratum. This work could be carried out expeditiously during the short summer weather window. Then, drill rigs were set on top to work during the winter, drilling ahead, placing secondary piles, and grouting the whole to act integrally with the jacket.

Drilled and grouted piles are especially effective in some calcareous sands. The grout penetrates the crushed shells and develops interlock with the unbroken shells behind. This appears to be the only positive method of achieving good skin friction transfer in calcareous sands. However, there are some calcareous sands, notably those on the Northwest Shelf of Australia, which are essentially impermeable and for which grouting just crushes the sand, increasing the effective pile diameter slightly but only increasing the effective friction to a limited extent.

A grouting shoe may be installed near the tip of the pile in order to permit grouting of the annulus without filling the interior of the pile. A packer, of course, serves a similar function. When piles of large diameter are plugged by concrete at the tip, it is necessary to check that the pile is not raised (floated) by the pressure of the grout as the fluid grout is placed. In soils that may soften or slack upon exposure to water, the pile should be placed and the grout injected as soon as practicable after drilling. The quality of the grout should be verified at frequent intervals during injection.

Holes for adjacent piles (for example, adjacent skirt piles under one leg) should not be open at the same time unless the soil properties are sufficiently strong and consistent to ensure that grout will not migrate during placement into the adjoining hole through fractures or seams and that the soils will not suffer relaxation.

In some cases, it becomes practicable to drill ahead of the primary pile, using a hole opener to enlarge the socket, then lower the pile to the full penetration and grout it to the soil. This saves the cost of the overlap of secondary pile into primary pile. It allows the primary pile to be used as the casing, as before. The under-reamer or hole opener employs a bit which is hydraulically expanded after it emerges below the tip of the pile and so cuts an enlarged hole. This system was used on the underwater storage tanks at Khazzan Dubai, in the Arabian Gulf, in which the drilled portion of the holes was in a limestone stratum. It has also been used for a number of major bridges with large-diameter piles of steel and prestressed concrete: for the Ohnaruto Straits and Yokohama Harbor bridges in Japan, for example.

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When large-diameter holes are to be drilled ahead, use of a two-stage bit, such as a pilot bit 12 in. (300 mm) in diameter followed by a hole opener of the desired larger diameter, affixed 2 or 3 m above the first, may be most efficient. The pilot hole can be better kept on line.

There are several types of drills available for use, depending on the size and depth of hole and the available supports for the drill. For emergency or limited use, such as arises when a single pile encounters a boulder, a churn drill may be used, suspended from the crane boom if necessary. Another type of drill, a pile-top rotary drill, is supported on the pile or casing itself, and gets its torque reaction from being clamped to the pile.

Drilling subs and drilling swivels are extremely versatile and easily handled drilling machines. They are suspended from the derrick boom. They get their torque reaction from chains attached to an arm. This arm and the chains are subjected to very severe shock loads and hence must be properly designed and secured for impact. This type of drill is very flexible in use, since from one location of the derrick, several piles can be drilled. Most recently, down-the-hole pneumatic hammers have become available and are proving both effective and flexible in their use. They are particularly useful in fractured rock whereas rotary drills may have difficulty in getting started. The Calyx system of drilling uses tricone bits mounted on the periphery of the caisson tip. These drills can drill a vertical hole to 1/2° accuracy. The drills are steerable (Figure 8.32). Coring drills cut a thin annulus

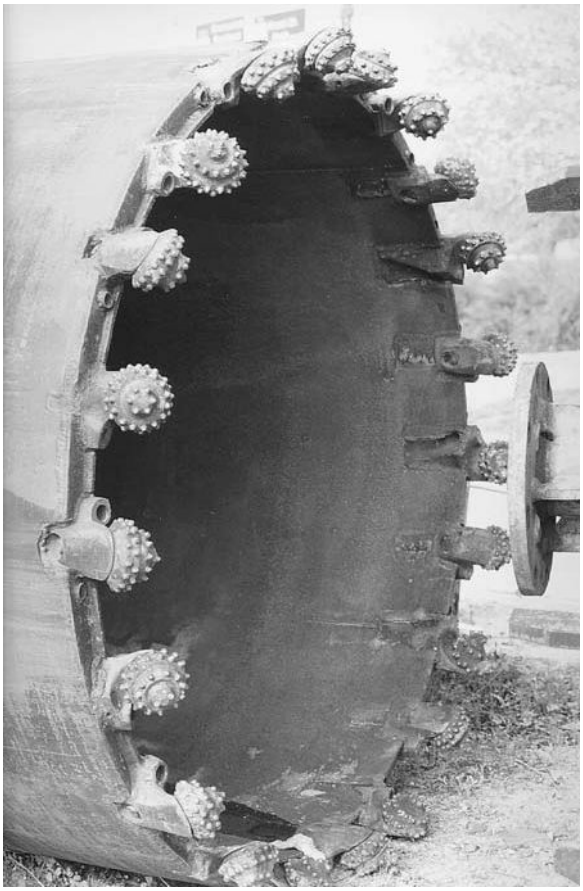

FIGURE 8.32
Casing drill with rotary cutters. (Courtesy of Flatiron Construction Co.)

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(25–50 mm) wider than the steel casing and advance the casing continuously. They drill by rotation of the casing from the surface. A counter-torque must be provided by a fixed or moored platform (see Figure 8.33).

Difficulties in drilling large diameter shafts (2.5–3 m or greater) have occurred in unstable layered rock with a high angle of dip and in squeezing rock, and where loose cohensionless material and siltstones or mud stone degrades on contact with water. The stand-up time of an open, slurry-filled hole, is inversely proportional to an exponential factor of the diameter, thus a 3-m diameter hole may have only 1/4 of or less of the stand-up time of a 1.5 m diameter hole.

One method used in the competent, very hard rock on the East River in New York was to drill a group of smaller diameter holes by a down-the-hole drill, spaced by means of a template, over the full section of the socket. Then, the interstices were broken up and removed by a hammer grab.

To start a drill on sloping hard rock, such as that which occurs in areas swept by past glaciers or fast currents, sealing of the casing is difficult. One solution, adopted successfully for a coal-loading terminal at Prince Rupert, British Columbia, was to seat the casing as best possible, without hard driving which might deform the tip. Then a 300–500-mm deep plug of viscous grout (e.g., containing silica fume AWA) was placed. When this has gained 10-MPa strength or more, drilling can usually commence without loss of circulation.

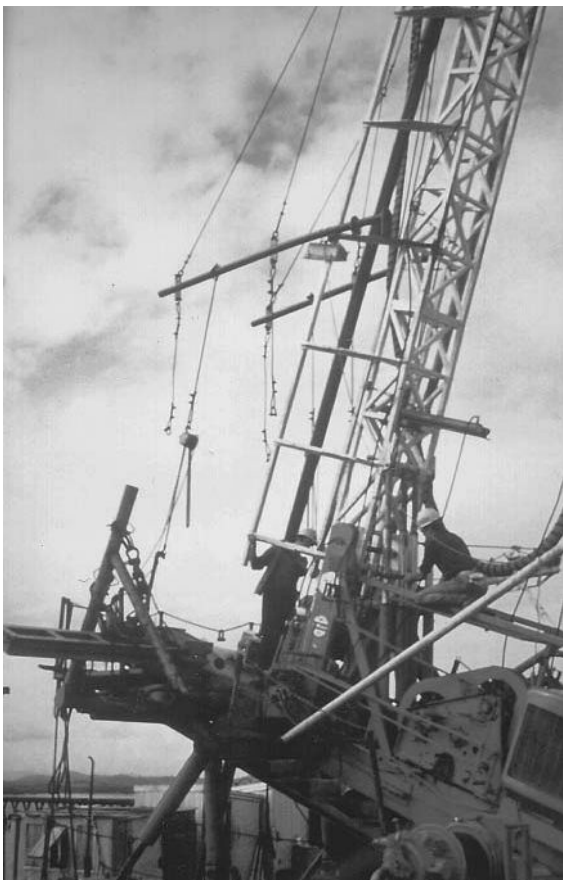

FIGURE 8.33
Drilling pile sockets for Hay Point Terminal No. 1, Queensland, Australia.

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In loose sands, the opposite situation, it is often necessary to embed the tip of the casing far enough to cut off inflow and outflow of the circulating drilling fluid. Here, a grout plug of one or two meters, placed under a positive head, may also be useful or the casing may be reseated by driving a meter or so deeper wherever run-in or lost circulation is experienced.

To drill through highly fractured rocks, coarse gravel, or cobbles is extremely difficult. While a churn drill may work, it is very slow. A hammer grab may be more effective. Down-the-hole drills are limited in diameter and may not be able to maintain continuous action. Pre-grouting of the stratum may be necessary and silica fume. Two or more small diameter holes (5–8 mm) may be drilled and pressure grouted with a flowing mix containing AWA and silica fume. Then, after hardening into a homogeneous mass, the rock may be drilled. This process may have to be repeated at deeper penetrations.

This Chapter has highlighted the benefits, especially as regards environmental aspects and permits, of polymer slurries. However, there are some instances when the polymer slurry is not as effective as bentonite. The bentonite slurry weighs about 10% more, which gives it more hydrostatic head to resist the collapse of the drilled hole. It has the ability to penetrate permeable soils, most of which are cohesionless and hence unstable, and to form a filter cake. This may reduce the bond transfer to the future concrete fill unless cleaned before concreting by clean water (salt or fresh). However, it may provide a solution, admittedly more costly, in soil conditions in which polymers are not effective.

Bentonite slurry, since it is considered a contaminant by the U.S.-EPA requires special disposal. It can be stabilized by addition of cement (about 50 kg/m^3).

Pre-grouting may be employed to stabilize cobbles, after first flushing with water. Since the grout takes the path of least resistance, it may be necessary to drill and grout several holes around the periphery to prevent fall-in.

For major pile installations, the use of a drilling rig with inclining mast appears to be most applicable, since it can attain faster drilling rates and make and unmake long drill strings more rapidly. Such a drill rig is usually skid-mounted and is set on a temporary work deck mounted on the jacket. These drills use conventional rotary drill bits. They may be operated either in direct circulation or reverse circulation, although the latter is generally most satisfactory.

An excellent description of drilling and grouting of piles for offshore structures is given in the Chapter "Grouted Piles" by Paul Richardson in the book *Planning and Design of Fixed Offshore Platforms* (McClelland and Reifel 1986).

8.14 Cast-in-Drilled-Hole Piles, Drilled Shafts

These piles are usually reinforced by a heavy cage of reinforcing steel, consisting of closely spaced large diameter bars around the perimeter, confined by spiral or hoops. Hoops have proven more reliable in confining plastic hinge zones. There must be sufficient clear space between bars and in the cover allowance to allow flow of the tremie concrete. A rule-of-thumb is that the clear space must be five or more times the diameter of the maximum coarse aggregate. This is because the driving force for the flow is the buoyant weight of the concrete in water or slurry, about half that of conventional gravity flow in air. Heavy cages for long piles must be sufficiently rigid for lifting and placing. They need heavy-duty centralizers to maintain concrete cover thickness. Flash welding of the spiral and centralizers to the main bars, when performed in a shop under controlled conditions, is now considered acceptable, since it does not affect the bars ultimate strength nor fatigue endurance.

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Large-diameter drilled shafts (CIDH piles) are increasingly being used because they are economical. They are also relatively quiet compared to pile driving. They have been employed on many of the world's great bridges. However, many difficulties can arise in construction that compromise the quality and reliability. The construction is both technical and an art and is very sensitive to the skill and competence of the contractor. When problems have arisen they often have caused extended delays and large over-runs in cost.

The concept is not only applicable to piles in soil but also to sockets drilled in rock and competent material below the tip of large-diameter tubular piles.

For marine structures, a casing must be installed through the water and weak soil. Where large bending resistance is required below the mudline, the casing may be required to be driven or cored to a significant depth in the soil. Where such bending capacity is not required, then the casing needs to be seated only far enough to seal so as to prevent water flow in and out and inflow of sands and silts.

It is important that the casing not be distorted in its body by squeezing, or at the tip by buckling, as this will prevent the drill from passing. The casing therefore needs to have an adequate d/t ratio. A ratio of 80 is probably a maximum. Where hard driving, such as into rock, is required, the guidance of API-RP2A is recommended. Typically, a d/t of 50 will be satisfactory, especially if a driving shoe is employed.

Driving shoes, if used, should be external, that is, thickened on the outside, so as not to impede the passage of the drill.

To drill the hole, a variety of drills are available. The proper selection is important in determining the rate of penetration. Churn drills and star drills have historically been used for short sockets and for penetrating boulders. Today down-the-hole drills are best and fastest in fractured rock. However, they may cause incipient fractures in the periphery of the hole, leading to fall-in by raveling and impairing the subsequent skin-friction transfer from the concrete pile to rock. Conventional rotary drills are fast and efficient. Drag bits may be best in hard clay and soft rock.

As the size of drilled shafts increases, the tendency for caving and fall-in increases exponentially. Factors involved are the cohesion or lack of cohesion in sedimentary soils. Damp sands have apparent cohesion whereas dry and saturated sands do not. Graded material, gravel to silt, has more stability than uniform sized materials. When ground water saturates cohesionless material, it causes the soil to flow into the drilled hole.

It must be kept in mind that exploratory borings, 75 mm or so in diameter, will not bring up gravel over 25 mm or so in diameter and certainly not cobbles. The small borings either hit refusal or by random chance, penetrate without hitting the larger particles head on; thus are sometimes able to displace them into the surrounding soil. In river beds where the potential for cobbles exists, 500 mm or greater exploratory holes or wells are recommended.

Some clays and rocks are "squeezing" material, which creeps in plastic flow as soon as the drill has passed.

Stability of holes can be treated in a manner similar to the stand-up time in tunnels and the drilling program planned to take full advantage of this.

In the case of the south tower pier of the Third Carquinez Strait Bridge in California, the highly fractured rock, much of it with a high angle vertical dip, had a stand-up time of only 16–20 h. Three-meter-diameter cast-in-drilled-hole piles were required to a depth of 30 m in the rock. After encountering several serious cases of fall-in, in one case trapping the drill, the following procedure was adopted:

1. Drive 30 m long $\times$ 3 m diameter casing through the water and clay mud to penetrate and seal into the rock.

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2. Drill a 2.8 m socket as far as could be drilled in 8 h (typically 6–7 m), keeping hole full of slurry.
3. Change to a hole-opening drill and enlarge the hole to a 3.4 m.
4. Place concrete by tremie methods. Concrete contained 8% silica fume so as to attain 35 MPa in two days.
5. Re-drill a 2.8 m socket through the top lift and on for another 8 h, gaining another 6 m.

And so on until the design depth was reached.

This step-by-step process was made practicable by working on several adjacent piles in sequence.

However, even this procedure did not suffice on the second Benicia-Martinez Bridge ten miles to the east. There, a stratum of loose saturated granular material ran in as soon as the hole-opening drill started enlarging the original hole. The solution adopted was to change to a coring drill.

A coring drill has teeth on the tip of the casing that create a hole about 20 mm larger in diameter, so that the casing can penetrate and prevent fall-in that will affect the pile (see Figure 8.34). Then, after the reinforcing cage has been placed, the pile is filled with tremie concrete. Vibrating the casing facilitates withdrawal of the casing. The tip of the casing is maintained 2–3 m below the level of the concrete that is rising in the casing.

Since almost all bored piles in the marine environment encounter soils that are unstable to some degree, slurries are employed. Their initial function is to offset the external hydrostatic head.

They also function to stabilize the soil and prevent fall-in. They help the drilling process by lubricating the bits and coating the drill cuttings, thus facilitating their rise to the surface.

There are several types of slurry, including fresh and salt water, mineral slurries, and polymer slurries. The most common mineral slurry is bentonite, comprised of minute flat plates of montmillironite which slide easily past each other. Various chemical and mineral additions are available, including barites to weight the slurry for greater resistance to fall-in. Mineral slurries contaminate and pollute the water and hence their discharge as well as the cuttings must be disposed of in accordance with regulations. Thus, their use in construction is gradually being phased out.

However, bentonite has certain unique properties, which help to stabilize the soils. It penetrates permeable material and forms a thin "filter cake," which is very cohesive.

FIGURE 8.34
Wirth "Hole Opener" (under-reamer) widening 3-m dia. drilled shafts on Third Carquinez Bridge, California.

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Unfortunately, this filter cake and the residue of the slurry on the walls of the hole reduce the subsequent bond of the concrete.

Bentonite slurries must be continuously circulated to keep material from settling out. Cuttings and fine particles are removed by continuously circulating the slurry through a shale shaker screen and then a filter. The alkalinity must be controlled to prevent coagulation of the bentonite. If the ground water is saline, the bentonite must be first chemically modified.

All holes, especially those for large diameter bored piles, must be covered when not being worked on. Bentonite slurry is indistinguishable from muddy water and wet soil and therefore is a trap for personnel who may inadvertently step in.

Polymer slurries are rapidly replacing mineral slurries because they are biodegradable, hence their disposal is not restricted. However, they are more expensive. They give cohesion to the soils at the periphery of the hole and coagulate the sands and fine particles. The unit weight is less and they do not form a filter cake; hence, in cohensionless soils they may not hold as well as bentonite. While limited re-use is permissible, they eventually become contaminated with colloidal clay.

Polymer slurries, in contrast to bentonite, improve the bond between the soil and the concrete. The drilled hole should be maintained full of slurry or water to above the outside water level, at least 2 m. The casing should be extended above sea level and filled with slurry to the top, so as to create an unbalanced outward pressure.

Due primarily to the increased demands imposed by seismic design, the reinforcing cages are very congested with both longitudinal and circumferential steel bars. Longitudinal bars are often bundled in pairs of large-diameter reinforcing bars, in order to keep an acceptable clear space between pairs for concrete flow. Similarly, spiral reinforcement and hoops are sometimes bundled. An empirical rule states that the clear space between bars should be at least 5 times the maximum size of coarse aggregate. The cages now become very heavy: if they are long, they must be supported for lifting at close intervals by a trussed cradle or picking beam, especially since the connections between spiral and longitudinal bars are not structural. Alternatively, the cage may be made structural by inclusion of intermittent hoops and clamps. Mechanical couplers are used, since there is not room for lap splices. Then, complete cages or sections of cages are usually fabricated horizontally, and then tipped to vertical, by means of a hinge at the tip of the cradle. The hinge is at the stern of the barge, thus allowing the cage to be lowered into the water. In the case of very long piles which are heavily reinforced, the cage has been fabricated vertically on a scaffolding above the top of the cased hole. To the cage are affixed the tremie tube, grout pipes and returns, inspection ducts, external spacers, and instrumentation.

Typical cover outside the spiral is 75 mm. To keep this spacing, plastic wheels or shoes are affixed at frequent intervals.

The concreting operation has turned out to be extremely critical in the case of deep and heavily reinforced bored piles. First the mix must be such that the concrete can flow freely between the mesh of reinforcing steel and in the cover. The driving force for the concrete is gravity but under water or under slurry, the net force is no longer the air weight of the concrete but the buoyant weight of the concrete in slurry.

To flow through the mesh and along the cover over the bundled bars, small coarse aggregate, preferably gravel, (as opposed to crushed rock) and a high proportion of sand is required. Typical values for the coarse rock are one-fifth to one-sixth the smallest clear opening between the bars in either direction, and the cover thickness. So with a 75 mm cover, 12–15 mm aggregate is maximum. Sand content should be from 45 to 50% of total aggregate.

A high cementitious material content is required, typically 400–450 kg/m^3. Of this, 25% can be fly ash and the rest, cement, or 70% can be blast-furnace slag with the remainder

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Portland cement. High fineness of grind is not suitable—a moderately coarse grind, say 200 m^2/kg, is preferable.

Also, the mix must retain its flowability through the full time of delivery and placement, while the tremie is embedded in the rising concrete. This embedment is typically 3–5 m. To accommodate contingencies, a longer period of flowability is required, typically 2 h additional. On all but the very largest piles requiring the most concrete, the most practicable requirement is to specify "the concrete shall maintain its minimum flowability for the full time of delivery and placement plus two hours."

The best performing tremie concrete is essentially "flowing concrete" to which a viscosity admixture and high range water reducer have been added, along with a set retarder as necessary to maintain flowability. Bleed must be minimized, otherwise flow channels will carry up the bleed and produce excessive laitance.

Flowability is best measured by the slump flow test, although the standard slump test and the Kelly Ball Penetration Test are also employed.

Recommended values (approximately equivalent) are as follows:	
Slump Flow Test	300–400 mm
Slump	200–300 mm
Kelly Ball Penetration	100–150 mm

These are initial values. At the end of the period prescribed earlier, the values should not be less than 250, 150, and 75 mm, respectively.

To meet these criteria, retarding admixtures are usually required. Care must be taken not to overdose, since the fly ash also has a moderately retarding effect.

Viscosity and reduction of bleed are important to prevent segregation of these high slump mixes. These can be attained by addition of microsilica (silica fume), typically 6%, or by an AWA.

Finally, in order to attain these desired properties without the use of excess water, and to attain the required concrete compressive strength, a high-range water reducing agent (HWRA) must be used. Fly ash is desirable to replace 25% of the cement.

To place the concrete, it can best be delivered horizontally by pump and discharged into a hopper on top of a tremie pipe of about 250 mm diameter. The tremie itself should have sufficient wall thickness to be negatively buoyant and to resist the inward hydrostatic pressure at full depth as well as the net outward pressure if full of concrete. The pipe must use flanged joints with gaskets or screwed joints. Drill casing is very suitable.

The tremie should initially be empty and sealed by a plate at the tip, wired to cleats near the tip. The plate should have a gasket. When the empty pipe is placed underwater, the hydrostatic pressure holds it on. Then, it is filled halfway and the pipe raised 150 mm. This breaks the wires and allows the concrete to flow out as more concrete is added. The common practice of using an inflated ball is not effective; it collapses in deep water.

While concrete is often placed directly by pump, with the stinger used as a tremie pipe, this causes the concrete to discharge in high velocity surges, often producing anomalies at the tip and excessive laitance. The common practice of temporarily plugging the end of the stinger by plastic foam is inherently unsound; the plug squeezes due to hydrostatic pressure at depth.

The only effective plugs for deep water resemble pipeline "pigs," that is, short rigid cylinders with "squeegees" to seal as they push down under the weight of concrete. In water less than 30 M deep, a plywood cap, gasketed by soft rubber, can be wired to the tip of the tremie pipe.

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To verify the completeness of the concrete fill, five gamma–gamma tubes of plastic pipe are tied to the reinforcing cage and gamma–gamma readings taken of the concrete after placement. These measure density: low readings indicate seams of laitance, or water-filled voids or rock pockets. Gamma–gamma tests indicate the thoroughness of cover.

These plastic tubes can be used to conduct cross-hole sonic tests of the hardened and cured concrete. These measure the modulus of elasticity and thus indicate the strength. Cores may also be drilled from the top of hardened piles. It is difficult to keep them vertical and to prevent running into the reinforcing steel.

Where anomalies (defects) are found, repairs must be carried out. The tubes furnish access for hydrodemolition tools, which fracture the plastic tubes and clean out soft laitance and fragments.

There must be two holes, one to inject the grout and the other for the water and water-diluted grout to be expelled. Then after flushing, cement grout, often using microfine cement and anti-washout admixture, is placed under low pressure. Pressure must be sufficient to overcome the hydrostatic head plus friction, but should not be higher than necessary. Usually gravity flow only is needed. The pressure should be maintained for a period, in order to allow the grout to penetrate. An anti-washout admixture will be found useful in preventing bleed (see also Section 9.4.8).

As stated earlier, large-diameter drilled shafts have been employed on a number of major offshore bridges such as the cable-stayed bridges across the Seine in France, the new bridge at Charleston, South Carolina, and the Nanjing and Sutong Bridges across the

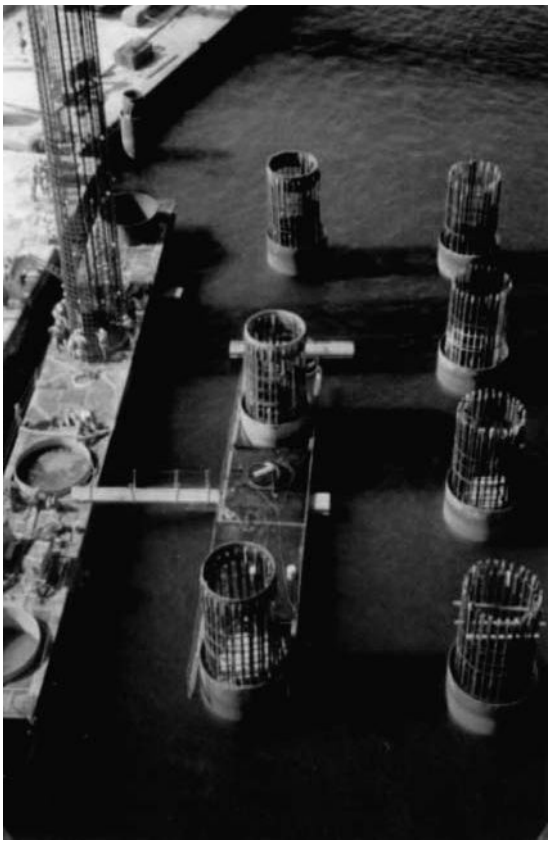

FIGURE 8.35
3-m diameter drilled shafts Cooper River Bridge, South Carolina. (Courtesy of Ben C. Gerwick, Inc.)

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Yangtze in China. This latter bridge, under construction in 2006, required 131 piles, each 2.5 m diameter and up to 117 m long. Scour of the silty sand bed, due to the high flood currents, can be almost 30 m (Figure 8.35).

The ultimate capacity per pile, as tested by Osterberg cells, was confirmed at 92 MN. Tip grouting after the pile was installed increased the capacity by 20%, and the load-deformation curve showed increased stiffness. Installation was from a steel platform. Rotary drill units performed the installation. A minimum 3 m positive head of the bentonite slurry was maintained.

On the Sutong Bridge, tremie concrete was placed through a central pipe in the reinforcing cage (Figure 8.36). Tip grouting was performed through six loop-shaped pipes preattached to the reinforcing cage and extended across the bottom, with eight ~8 mm holes. To keep the system from plugging during concrete placements, water was continually pumped through the pipes. Post grouting was performed at 9 MPa pressure in three stages spaced several hours apart.

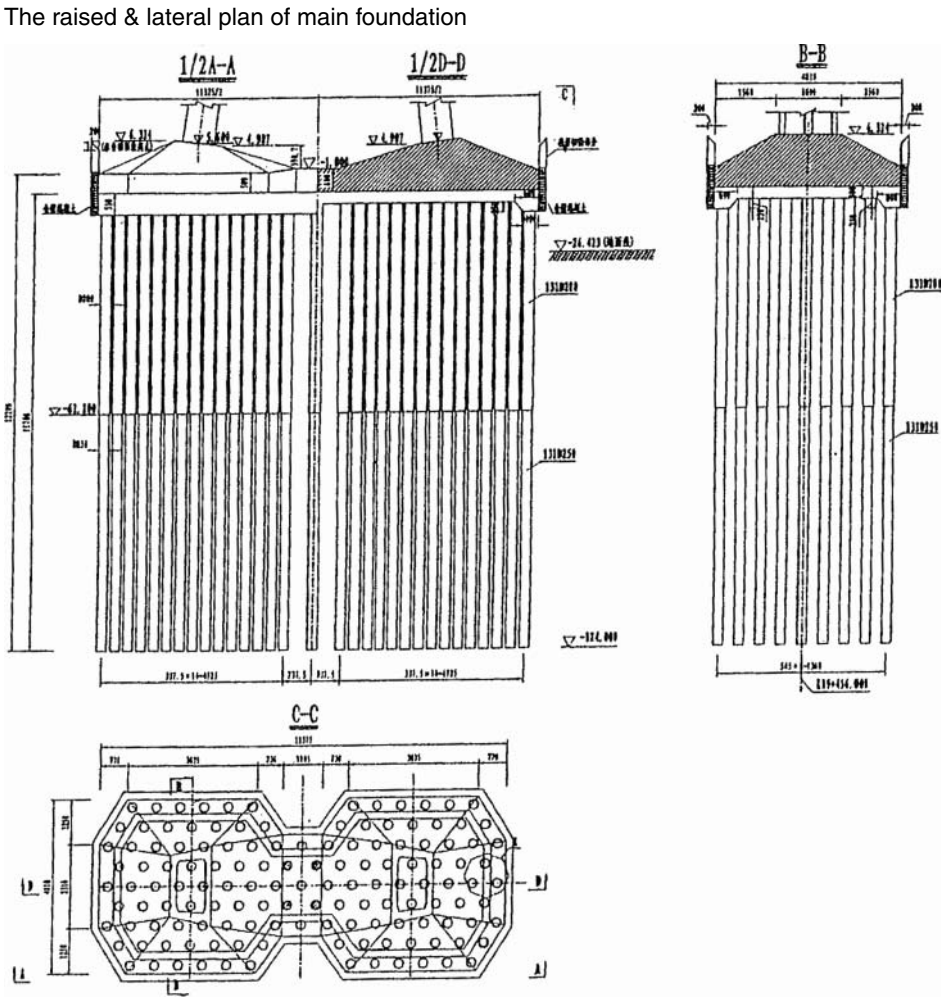

FIGURE 8.36
2.5-m diameter×117-m long drilled shafts for Sutong Bridge, Yangtze River China. (Courtesy of Sutong Bridge Design Authority.)

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One important development in pin pile installation of offshore platforms has been the belled footing at the tip of a driven pile, first used on the Ekofisk Field platforms and since extended to other structures in the Arabian Gulf, and the Northwest Shelf of Australia. In this case, the primary pile serves as a casing through which a drill rig drills a moderate-length hole ahead. Then, it employs a belling tool to enlarge the socket into a bell of 4- to 5-m diameter. Reverse circulation is employed, usually with a bentonite slurry (drilling mud) as the drilling fluid. Then, a heavy reinforcing cage or steel insert pile is set. The bell and socket and a portion of the casing are filled with underwater concrete, using "fine concrete" aggregates: for example, maximum size 9 mm. As with straight drilled shafts (sockets), saltwater may be used as the drilling fluid and the bell "spot-mudded" with polymer mud to hold it open until concreting (see Figure 8.37 and Figure 8.38).

API Standard RP2A describes belled piles as they are used to give increased bearing and uplift capacity through direct bearing on the soil. A pilot hole is first drilled below the base of the driven pile to the elevation of the bell base and slightly below to act as a sump for unrecoverable cuttings. Then the bell is drilled, using an expander tool. Reverse circulation must be employed in order to gain enough discharge velocity to remove the cuttings. Slurry is usually used to keep the bell from collapsing. The sands surrounding the bell can be stabilized by epoxy injection.

Then, an insert "pile" is run down; this may be a tubular or structural member or reinforcing steel bars assembled in a cage. The bell and pile are then filled with concrete to a height sufficient to accomplish load transfer between the bell and the pile. Shear rings on the insert pile or the deformations of the reinforcement are used to gain high shear transfer in the relatively short height of the bell. End bearing may be obtained by a closure plate on the end of the insert pile, sloped at 7° from the horizontal to prevent trapping of bleed water and laitance. Structural reinforcing was employed at Ekofisk, while a closed-end steel tubular was employed at North Rankin A.

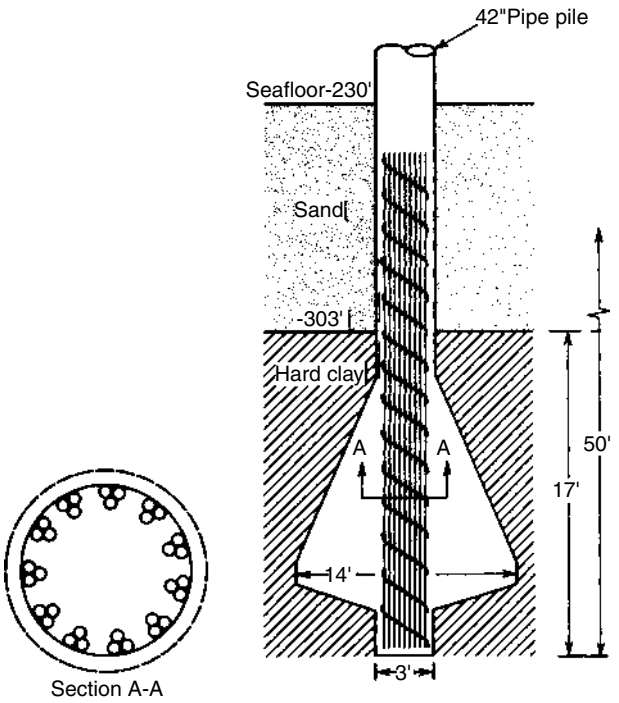

FIGURE 8.37
Under-reamed bell footing for Ekofisk Platform, North Sea.

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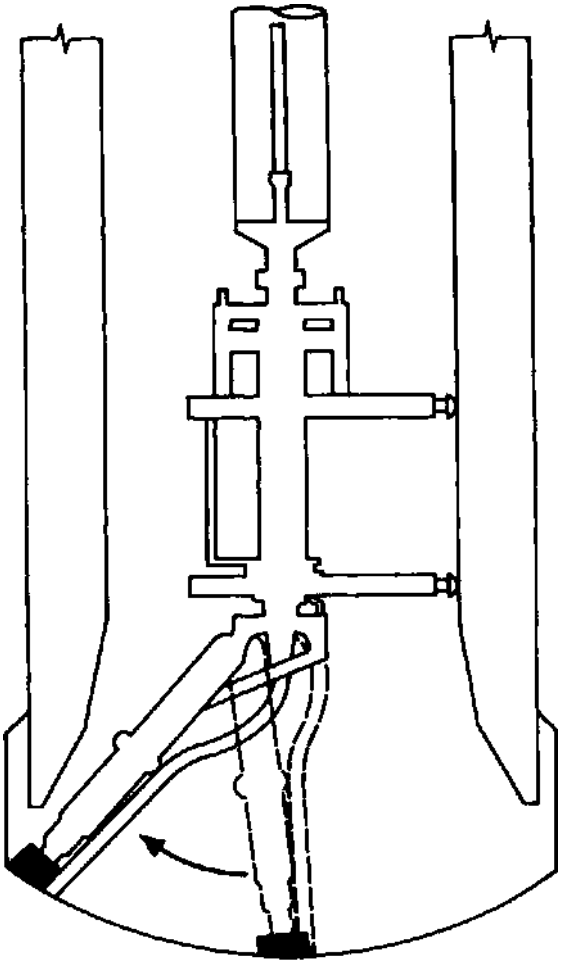

FIGURE 8.38
Mitsui "Aqua-Header" special drilling tool for large-diameter belled footing in clay.

Reinforcing steel bars are usually bundled to permit the concrete to flow between them and out into the bell. The reinforcing bars are enclosed in spiral: the spirals may also be bundled to facilitate flow. In the typical installation of offshore belled footings, the primary piles are driven down to seal in the bearing stratum. The holes are then drilled down and belled, reinforcing cages are inserted, and the whole is filled with concrete.

It is not currently possible to confine the bell concrete with hoop reinforcing. There is no way the hoop steel can be placed. If the bell has been drilled into rock, the rock may confine it; however, bells at both Ekofisk and North Rankin A platforms were used in soils of relatively low stiffness compared to the concrete. Hence, there will be flexural and shear stresses in the concrete under service and extreme loading conditions. This means that the shear and tensile strength of the concrete must be utilized. In the future, use of steel fibers in the concrete to develop shear and tensile capacity may be considered.

Such a confined mass of concrete will get very hot during the period immediately following placement, due to heat of hydration. Subsequent cooling, starting at the outside edge, may produce severe cracking. Cracking may also be caused during the expansion stage. Hence, the cement selected should be a low-heat-type cement, such as ASTM Type

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IV or Type II, with pozzolanic replacement of up to 30% or more of the cement. Alternatively, coarse ground slag-Portland cement in a 70:30 mix may be used; it has very low heat generation. The mix should be as cool as practicable at time of placement; aggregates, for example, can be sprayed with water to cool them or liquid nitrogen injected into the mix.

Drilling contractors prefer to use a cement slurry (i.e., cement plus water plus admixture, but no coarse aggregates), since they are familiar with it, it is relatively simple to handle, and it can be directly pumped. Cement contents will consequently be very high; hence heat of hydration is a very serious problem considering the mass of concrete in a drilled shaft. Temperatures exceeding boiling and disruption of the concrete have occurred. Use of blast furnace slag-cement is indicated. Moreover, the tensile strength and shear strengths of neat cement slurry (grout) are relatively low. Steel fibers have been proposed as one means of enhancing the tensile strength. However, they may tend to segregate in a cement slurry. The use of polypropylene is another solution. A mix that incorporates sand—that is, a sand-cement mortar or, even better, one incorporating small aggregates (8–10 mm) such as pea gravel—will have relatively low heat properties and good tensile and shear strengths, yet will be able to flow readily. Plasticizing admixtures should be used. If silica fume or antiwashout admixtures are added, then a superplasticizer high-range water reducer is also required as well as a retarding admixture to prevent the superplasticizer from a sudden loss of slump due to the heat generated.

Use of valves or restrictions at the tip of the tremie should be avoided. If necessary due to other aspects of the operations, the valve must be carefully designed so as not to jam with a piece of aggregate and must not abrade under the flow of concrete. Several devices have been developed for this purpose; the most successful involve hydraulically operated squeeze valves.

As described previously, to start the placement, the tip of the tremie pipe is closed with a gasketed plate, wired onto the end of the pipe. The pipe is assembled and run down to the tip. Then the pipe is charged, initially with the 1 m^3 of cement grout, and then filled to slightly above halfway. The pipe is now raised a few inches above the bottom, allowing the plate seal to break free and the concrete to flow out as slowly as it can be controlled, until it reaches a balance of heads. Then, concrete is continuously fed in at the top. The concrete in the tremie pipe will attain a hydrostatic balance at approximately half the water depth. At greater depths, a polyurethane pig may be employed, being pushed down the casing by the initial flow of concrete (see Section 6.6). Using a pig, tremie concrete was successfully placed in belled footings under the primary piles of the North Rankin A platform, at a depth of 240 m below sea level.

The annulus between insert pile (or reinforcing cage) and primary pile must be at least five times the maximum size of coarse aggregate to permit flow up around the insert pile.

Electrical resistivity or well-logging devices can be used to determine the level of the top surface as it rises up in the primary pile. Obviously, the placement should be extended several meters above the design level to account for any mixing of the initially placed grout with the drilling slurry. Bentonite drilling mud should preferably be converted to a calcium base to avoid coagulation upon contact by cement or, even better, a polymer mud should be used.

Although drilling contractors and others continue to urge placement by pumping, their experience has primarily been with the cementing of oil field casing and with grouting of annuli. For the much-larger-diameter drilled shafts and belled footings, experience conclusively shows that reliable placement can only be attained by gravity feed.

The belled footings of the North Rankin A were constructed in a weak calcarenite. An elaborate stabilization process was used, by which the surrounding matrix was impregnated with a thin epoxy. Then the bells were drilled, an insert pile placed, and the bell and

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pile filled with tremie concrete placed by gravity. Because the surrounding soils and seawater were at an ambient temperature of 38°C, the mix was precooled by liquid nitrogen to 5°C. Cold water was flushed down the tremie tube before the concreting started. Coarse aggregate was 8 mm maximum. Subsequent coring showed concrete tensile strengths of 6–7 MPa and compressive strengths about 60 MPa.

8.15 Other Installation Experience

Very-large-diameter piles, 3–4 m in diameter, have been sunk by a combination of drilling, weighting, vibration, and internal excavation. Thus, they resemble an open caisson of a single cell. The piles for the Oosterschelde Bridge, 4 m in diameter and 60 m in length, were installed in this manner. In addition to the self-weight of the pile (about 600 tn.), another 600 tn. was applied by attachment to and hoisting up of the derrick barge through a multi-part tackle. Excavation was by means of an articulated cutter suction dredge arm suspended vertically from the crane barge boom. Weighting is, of course, often used in seating the pile legs of jack-up barges by applying a major portion of the barge weight in succession to the individual piles. Drills up to 4 m diameter and more are commercially available.

8.16 Installation in Difficult Soils

Difficult geotechnical conditions have occasionally been encountered and have required ingenious solutions. Some specific examples are:

a. Calcareous sands, consisting primarily of the shells of diatoms. While these show a relatively high angle of internal friction, they crush like Styrofoam in shear, giving little, if any, skin friction resistance.
b. Mudstone and siltstone. These weaken when exposed to water or slurry. Special slurry formulation and incremental procedures have had to be developed to enable drilling of sockets.
c. Glacial outwash, consisting of uniformly sized gravels or cobbles, developing little skin friction internally or externally. Displacement piles develop good end bearing.
d. Riprap, boulders, cobbles. Displacement piles, with blunt tips, reinforced against local crushing, are best. They push the stones aside whereas tapered or pointed tips fail in local shear or buckling.

Driving piling within an excavated cofferdam or caisson has presented many problems. One is that the vibration causes liquefaction, leading to run-in of soil and loss of passive resistance at the toes of the sheet piles.

In other cases, driving of displacement piles has caused excessive heave, driving the sheets away from their bracing. When open-ended tubular piles plug, temporarily becoming displacement piles, cleaning out to within one diameter of the tip will usually relieve the plug. If the soils below the tip are cohesive—e.g., clay—then drilling ahead a smaller hole for a limited distance (e.g., 2 diameters) may help. Adjacent or nearby holes should not be open at the same time.

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Non-displacement piles such as steel H piles are generally more appropriate for this condition. Alternatively, displacement piles can be driven and followed down prior to installation of the cofferdam or caisson.

Because many of the above problems have arisen in remote areas, where information on existing geotechnical condition is inadequate, the following precautionary preparations are recommended:

1. Select a pile hammer of larger capacity than would normally be used.
2. Be equipped with a star drill or churn drill, and a down-the-hole drill.
3. Be equipped with a heavy-duty jet and adequate pump capacity.
4. Be equipped with an air lift and a hammer grab.
5. Adequately reinforce pile tips.
6. With concrete piling, have a fresh cushion block of softwood for each pile.
7. Be conservative in selecting thickness of steel cylinder piles.

8.17 Other Methods of Improving the Capacity of Driven Piles

Once piles are installed, it is necessary to evaluate their load-bearing capacity to ensure that the required capacity has been attained. Sometimes, the skin friction may be deficient; this is frequently encountered in calcareous and micaceous soils. Another situation that may arise is where an existing platform is to be upgraded to withstand greater environmental or operating loads.

One method is to clean out the internal soils to a safe distance above the tip (usually several meters) and then to construct a concrete plug inside the pile tip, of adequate length to develop bond transfer to the pile. This converts the pile to an end-bearing pile. This method was successfully employed for the previously driven piles in the Kingfish platforms A and B in Bass Straits, Australia, which were driven into calcareous sands. On the North Rankin A flarestack, also in calcareous sands, after the concrete plug was placed, grout was injected under the plug to reconsolidate the sands and minimize consolidation settlement.

When inadequate capacity is encountered in the initial driving and the pile fails to develop adequate resistance at the design penetration, then appropriate steps taken promptly may enable the construction of an adequate foundation with minimal additional cost. For example, on one offshore terminal in the Mediterranean, the 2-m-diameter open-ended steel piles failed to develop the required resistance with 60-m penetration. On the first pile, it was found that the penetration would have to be increased by more than double, at a prohibitive cost. By welding on a tip closure plate which closed 80% of the tip, leaving a small central hole for water escape and relief of soil resistance, the piles developed adequate static capacity in the calcareous sands and safely sustained a test load of 4000 tn.

When increased tension capacity is required, two methods are possible. One is to drill in an insert pile. In one case, the drill string itself was used as the insert pile and grouted into place. Another solution is to weight the pile, similar to placing weights on the legs of a table. The pile is cleaned out, a concrete plug is placed, and iron sand or barites are placed. These should be carefully selected for in-place density, durability, and freedom from corroding effects on the steel. In-place densities of 3.5 have been achieved with iron sands. Both

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systems were employed for the piles of the Kingfish A and B platforms in Bass Strait, Australia.

A third solution, applicable in stratified soils where both compression and tension capacities must be increased, is to construct a belled footing, as described above.

Insert piles may be driven through existing piling, being made up in short segments, welded as they are installed. They are driven with a follower using mechanical threaded connections for the follower segments. After driving below the original tip a sufficient distance to develop the required bearing, the insert piles are connected to the primary piles by grouting of the annulus. Alternatively, the insert piles may be installed by drilling and grouting.

Lateral resistance of existing piles may be increased by installation of an insert pile, grouted, or concreted in, to stiffen the pile in the vicinity of the seafloor. This was successfully carried out for a platform off Bombay High, India, where the pile was showing excessive deflection under cyclic loads. The soils were calcareous sands. The seafloor sediments surrounding the pile may be in some cases strengthened by vibratory densification or pressure grouting, or they may be surcharged with gravel, either normal density or high density, to consolidate the existing soil, replace any settlements, and fill any gaps that occur under cyclic wave action. Stiffness of the pile may also be increased up to 25% or even 33% by filling with concrete.

It has long been the practice with tubular piles for bridge piers to clean them out and fill them with concrete, in order to develop the design axial and lateral capacities. Reinforcing steel cages are usually installed. To prevent loosening of the soil around the tip, during clean-out, the pile should be kept full of water and the soil clean-out should stop two diameters above the tip. To prevent settlement under high loads, the pile tip should be grouted under pressure after the concrete fill has set. Composite action between steel and concrete core can be obtained by shear rings on the inside of the pile.

Freezing of the soil around the pile and under the pile tip is another method that has been proposed as a means of increasing the capacity of previously driven piles. The concept is that, after driving, the steel pile would be cleaned out by jetting, airlifting, or drilling and then used as the freeze pipe casing. Among the matters which must be considered here are the following:

The behavior of saline soils when frozen and the formation of brine lenses and pockets. While most soils show dramatic increase in strength when frozen, this is apparently not true of all carbonate soils. Some others may develop weak planes.

Required temperature to achieve solid freezing. Due to salinity, this may be -7°C to -10°C.

Adfreeze from frozen soil to pile

Load transfer from stiff pile to elastoplastic frozen soil

Frost heave

Creep of frozen soil under sustained load

Load transfer properties from strongly frozen to weakly frozen to unfrozen soil

Sensitivity of steel pile, and especially the welded joints, to low temperature. Resistance to brittle fracture under impact or cyclic tensile loads, rapidly applied

Rate of warm-up (thaw) in case of system failure, especially since the pile acts as a heat conductor

Energy requirements of maintaining the soil in a frozen condition.

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Freezing was successfully performed on the elevated pile-supported structure of the Alyeska Pipeline from Prudhoe Bay to Valdez, Alaska. Steel tubular piles were driven into the existing soil and cleaned out. Freeze pipes were then inserted. A freeze pipe consists of a fabricated tube that allows a solvent to flow down the accurately machined grooves of the inside tubular wall. When it reaches the lower portion of the pile, it extracts heat from the surrounding soil and the solvent, similar to propane, vaporizes. The vapor rises up to fins extending into the air at the top of the pile. In air temperatures below freezing, the vapor is condensed to liquid and flows back down the inside wall of the pipe. All that is required to freeze the soil around the tubular pile is that the air temperature is below freezing for lengthy periods each year. In warm periods, the cyclic process automatically shuts down (see also Section 8.10).

8.18 Slurry Walls, Secant Walls, and Tangent Walls

These have been very rapidly developed in recent years to serve both during construction and in permanent service to retain deep excavations. Among their advantages are the ability to install through very dense lenses or strata, through obstructions and cobbles, even through boulders and bed rock.

The principles are simple: for a secant wall, drill and concrete every other pile, carefully spaced and vertical. Then drill an overlapping pile in between, reinforce it with a cage of steel bars or a structural member, and concrete it, with similar technology to cast-in-drilled shaft (CIDH) piles.

Tangent walls are drilled as close together as possible. Then, a second line of tangent piles is drilled behind the first line, as close together as possible, or grout is injected behind each intersection.

For a slurry wall, the excavated slots are rectangular, typically one meter thick and 5 m long. These are formed at the ends so as to have shear transfer. These slots are typically drilled in a similar alternate pattern (i.e., every other one). They are reinforced and concrete placed. Then, the intermediate panels are excavated. Reinforcement and concreting should follow the principles recommended for CIDH piles.

For both the secant and slurry wall systems, the multiple vertical construction joints present some problems. Accurate construction, limiting tolerances, and the use of vertical shear keys, can ensure the transfer of transverse shear but neither vertical shear between panels nor moment transfer. The joints are a source of leakage.

Slurry wall contractors have developed many ingenious schemes to overcome these problems. Many of the schemes are proprietary. They include the use of steel wide flange beams to form the stop ends, guide the alignment, transfer transverse shear, and reinforce the wall. This is known as the soldier pile and tremie concrete (SPTC) system and has been widely used, primarily for underground rapid transit projects.

Other systems use removable pipe stop ends. By use of these systems, waterstops can be installed, and overlapping reinforcing bars can be installed between segments.

At the present time, tangent and secant walls can be installed underwater, using temporary casings to extend the drilled holes through the water. However, slurry walls can only be installed below water by temporarily transforming the water column locally into land by, for example, filling sand between parallel sheet pile walls, as was done for the slurry walls at the Kawasaki Ventilation Shaft in Tokyo Bay, and then constructing the wall in the conventional manner.

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8.19 Steel Sheet Piles

These are employed for quay walls and bulkheads for cofferdams and for underground cut-offs. They are available in a variety of configurations, depending on the design requirement and the installation demands. All include interlocks so as to enable construction of a continuous wall. Lengths available have in recent years been extended to about 30 m although lengths over 20 m command a premium price.

Most steel sheet piles are hot-rolled, with yield points of 300–400 MPa. For less demanding installations—i.e., reduced strength requirements and less interlock strength—cold-rolled sheets are available. Cold-rolled (cold-formed) steel sheets are thin, hence relatively lower in cost. They have high yield strength. Interlocks are crimped plate. Thus, they are restricted in use to shallow water and low surcharge, such as marina quay walls, and installation by vibration and/or jetting. The remainder of this section will address hot-rolled sheets only.

Hot-rolled steel plate thickness is typically 9.5–12.5 mm. Configurations vary from flat to deep Zs and even steel wide-flange and tubulars with interlocks attached. As a result, section moduli vary over a wide range. Sheet piles are often pre-joined in pairs. Interlocks are "thumb-and-fingers" or "ball and socket."

Recently, the use of steel sheet piles has been extended to very deep cofferdams. Where necessary, the sheet piles have been spliced. While full strength across the splice can be

TABLE 8.5

Sheet Pile Drivability

Dominant SPT "N" Value	Minimum Wall Modulus (cm^3/m) Grade S355 GP	Minimum Wall Modulus ($in.^3/ft.$) Grade $F_y = 50$ ksi
Drivability in Cohensionless Soils		
0–11		
11–20	450	8.4
21–25		
26–30	850	15.8
31–35		
36–40	1300	24.2
41–45		
46–50	2300	42.8
51–60		
61–70	3000	55.8
71–80		
81–140	4000	74.4
Clay	**Minimum Wall Modulus (cm^3/m) Grade S355 GP**	**Minimum Wall Modulus ($in.^3/ft.$) Grade $F_y = 50$ ksi**
Drivability in Cohesive Soils		
Soft to firm		
Firm	450–600	8.4–11.2
Firm to stiff	600–1300	11.2–24.2
Stiff	1300–2000	24.2–37.2
Very stiff	2000–2500	37.2–46.5
Hard	4200–5000	78.1–93.0

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achieved on the plate portions, it is impracticable to achieve full strength across the interlocks. In splicing, the interlocks of the two segments are temporarily threaded over a short companion interlock so as to keep accurate alignment. Nevertheless, this is a location where failure usually occurs in hard driving.

Sheet piles are usually driven with a vibratory pile hammer, often as a pre-assembled pair. In hard driving, impact hammers may be needed. Steel sheet piles may be driven underwater with vibratory or hydraulic hammers.

A factor not widely recognized is that when penetrating firm or hard soils, a higher section modulus is required. Further, with deeper penetrations, the cumulative skin friction resistance may require thicker sheets and higher section modulus, as well as a larger hammer. Pipe-sheet piles provide these properties and are especially appropriate when encountering hard strata and where long lengths are required.

Sheet piling need guides to ensure out-of-plane guidance. These can best be provided by driving in echelon, so that each sheet extends not more than 1.5–2 m below its neighbor, and thus is guided by it. This is known as driving in panels. Short steel sheet piles may be installed by progressive jacking in order to reduce noise and vibration.

More detailed instructions concerning the several common applications of steel sheet piles are given in Chapter 9. Minimum steel sheet pile wall thickness for both cohensionless and cohesive soils of different densities are given in Table 8.5.

8.20 Vibratory Pile Hammers

Vibratory pile hammers are almost always used when driving steel sheet piles. The rapidly rotating eccentrics impart a longitudinal force at high frequency. This liquefies the soils along the sides of the piles and at the tip. Their use is especially effective in sands but also works in soft to moderate clays. Some soils, like lenses of volcanic ash, may not be penetrated by vibration alone. For cautions regarding excessive liquefaction, see Section 9.4.9.2. Misalignment of the vibrating hammer may dissipate much of the energy laterally.

Vibrators are also effective in extracting sheet piles, as well as old timber and steel piles.

8.21 Micropiles

These small-diameter, yet relatively high-capacity piles, have been widely used for underpinning structures on land and recently were employed in the seismic strengthening of the Richmond-San Rafael Bridge (California) crossing San Francisco Bay. These piles are installed by drilling through water and soils into the founding rock or dense soils and then inserting and grouting heavy walled steel pipe casing, augmented as necessary by central bars of reinforcing steel. On the bridge project, the micropiles were subject to test loads up to 500 metric tons in both tension and compression. Four hundred micropiles in 31 piers were used to counteract longitudinal overturning forces under the safety level earthquake.

A permanent steel casing, 300 mm diameter, was installed down to seat in the weathered rock. The socket was drilled in massive greywacke and various metamorphic rocks, altered by weathering and fracturing. Then a 210 mm $\times$ 25 mm steel pipe insert was run, all the way up through the footing. The annulus in the footing was filled with grout under

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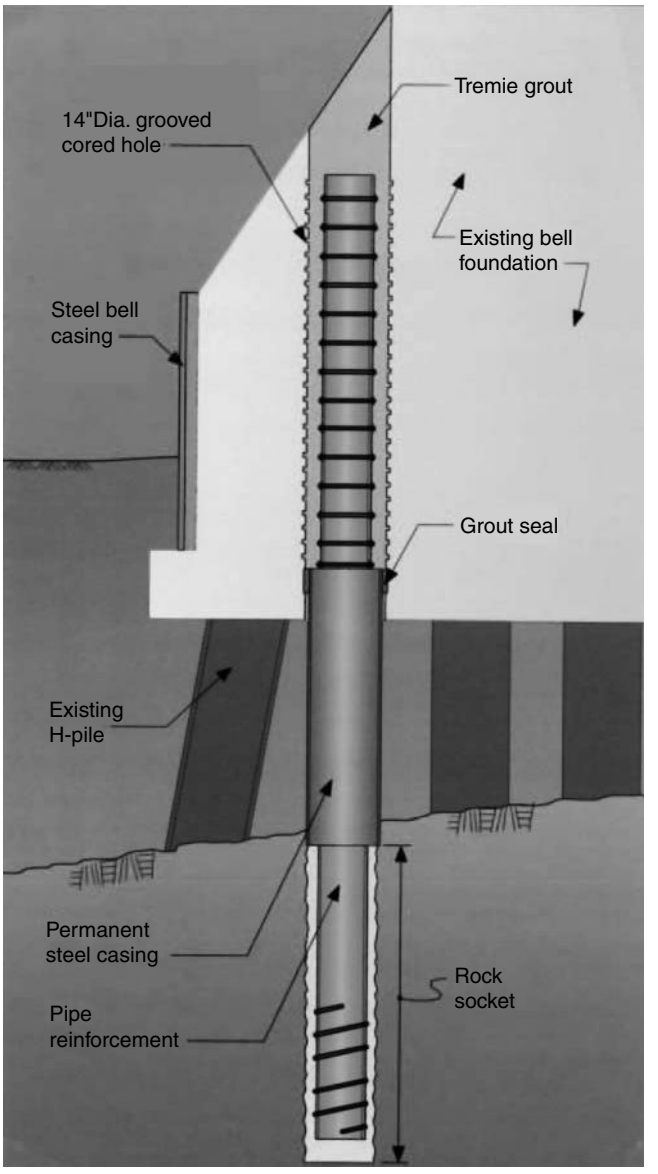

FIGURE 8.39
Micropile used to anchor belled piers of Richmond–San Rafae) Bridge, California: Seismic Retrofit. (Courtesy of Ben C. Gerwick, Inc.)

pressure. In some cases, a central #18 reinforcing steel bar was added in order to increase capacity (Figure 8.39).

And now the storm-blast came, and he
Was tyrannous and strong.
He struck with his o'ertaking wings
And chased us south along.

Samuel Taylor Coleridge, The Rime of the Ancient Mariner

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9

Harbor, River, and Estuary Structures

9.1 General

These structures have their roots in ancient civilizations. The Babylonians and Chinese built bridges; the Phoenicians built breakwaters and crude quays using timber pile cribs filled with stone. The Romans built an artificial harbor at Ostia, for which they may have used pozzolanic cement for underwater concrete. We know for certain that they did build timber cribs and cofferdams for bridge piers as well as pile-supported trestles.

Included in this chapter are harbor structures and wharves, river structures such as locks and low-level dams, piers for overwater bridges, submerged prefabricated tunnels (tubes), and storm-surge barriers.

9.2 Harbor Structures

9.2.1 Types

Currently, the dominant structural type of harbor structures is the quay or marginal wharf for the unloading and loading of containers. Finger piers, extending normal to the shore, are used for transfer of petroleum products and trestles are built to furnish access to loading platforms and wharves.

9.2.2 Pile-Supported Structures

These consist of either steel or concrete piles, driven into the soft clays and sands of the seafloor. They typically support a reinforced concrete deck. To obtain economy, the piles are generally on a spacing of 7–10 m. Thus, the design capacity per pile typically ranges from 100 to 250 tn. per pile (1.0–2.5 MN) although somewhat larger capacities are now emerging.

Both steel and prestressed concrete piles are utilized for the harbor structure addressed in this section. They are also used in larger sizes for bridge piers (see Section 9.5).

9.2.2.1 Steel Piles

These are either tubular pipes of 400–600 mm diameter, or H piles. The pipe piles present less surface area for corrosion and hence, are more readily protected. Although in many soils, steel H piles penetrate more readily; in stiff clays the soil plugs between the flanges and the pile act as a partially end-bearing pile of square cross section. The steel pipe pile

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also plugs, as does its larger diameter counterpart in the offshore. The plugging occurs when the internal skin friction exceeds the ultimate bearing value of the soil at the tip.

Since the water depths for modern container and cargo ships are about 16 m, and the design pile capacity is only of the order of 200–400 tn., in most cases piles will range from 30 to 40 m in length. The trend is now towards container ships of deeper draft and greater beam requiring 20 m depth at the berth; piles may be up to 40–50 m in length to provide greater capacity for the support of the water-side crane load and 1000 mm in diameter to provide the stiffness for lateral loads. Petroleum terminals require greater depth at the face of the wharf, usually 23 m, and hence larger and more heavily loaded piles. Steel piles can be picked (pitched) readily, and placed into the leads. At deck level, the pile may be centered by a template or a mechanical gate, while at the top a hydraulically operated arm may swing around the pile to center and align it in the leads.

9.2.2.2 Concrete Piles

Today, these piles are almost always prestressed concrete piles. The sizes up to 500 mm are typically square in cross section, those of 600 mm size are octagonal, and from 900 to 2100 mm diameter are round with a hollow core. The precompression in the pile, after losses, is 7–10 MPa. Experience indicates that values less than 7 MPa do not perform as well under driving impacts as those with the higher values.

In pitching (upending) concrete piles, because of their weight and bending capacity, multiple pickup points are usually required. Their location in the fabrication plant is usually determined on the basis of vertical lifts on a horizontal pile. In the case of pitching into the leads (up-ending), the lines (or slings) will lead at angles that will vary as the pile changes from horizontal to vertical. The vertical and horizontal components of the various lines will change with the leads of the lifting lines and the pile's inclination. Usually (but not always), the critical case is the initial lift from the horizontal. The reinforcement, conventional and prestressed, must be adequate for both transport and pitching (see Figure 9.1).

9.2.2.3 Installation

After pitching, the steel or concrete pile is then allowed to run down under its own weight. The hammer and driving head are now lowered onto the pile with the pile line slacked and the pile penetrates farther under the weight of the hammer. Driving now commences. The

FIGURE 9.1
Upending (pitching) a long prestressed concrete pile. Note six pickup points along pile.

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first blows may cause the pile to run a meter or more; hence the hammer line must be free in order to keep the hammer and driving head on the pile.

Although vibratory hammers may be used to obtain initial penetration of steel piles in cohesionless soils, final seating is normally by impact hammer. Concrete piles are generally driven by impact hammer alone. Three types of impact hammers are in common use today, the single-acting steam hammer (which can also be activated by compressed air), the hydraulic hammer, and the diesel hammer. All three types depend on the mass of the ram impacting on the driving head, which in turn transmits the impulse to the pile. In the case of the single-acting steam hammer, the propelling force is gravity alone; in the case of the hydraulic hammer, the gravitational force is amplified by hydraulic pressure. With the diesel hammer, the force is applied sequentially; first the compression of the air by the falling ram "preloads" the pile, then the ram delivers its impact, and finally the thrust of the exploding fuel reacts downward while at the same time raising the ram for the next stroke. As a result, the hammer energies are computed differently; a rough rule of thumb equates the rated energy of a steam or hydraulic hammer to 1.6 times the rated energy of a diesel hammer insofar as the effectiveness in driving. This is not only due to the differing modes of impact but also to the proportionally lesser weight of the ram in the case of the diesel hammer, which is partially compensated by a higher velocity.

The driving heads are designed to transmit the impact to the pile head. Hence, in the case of steel piles, the head is configured to match the pile section. The blow is transmitted steel to steel; no cushion material is interposed. The driving head serves to confine the pile head and prevent it from local buckling and crimping.

In the case of the concrete pile, a cushion is required to attenuate the blow and extend its duration in order to prevent cracking under rebound tension. The driving head is configured to contain the pile head cushion. The pile head cushion is best made as a 200- to 400-mm-thick laminated softwood block. Plywood layers may be affixed top and bottom and inserted in the middle to help hold the block together during driving. Experience, supported by dynamic measurements, shows that such a cushion will usually be adequate.

The hammer blow creates a compressive wave, which travels down the pile at the speed of sound in the pile material. When the wave reaches the tip, it either causes the pile to penetrate, thus producing a tensile wave in the pile or, if the pile tip is on hard material such as rock, causes a rebound compressive stress wave of twice the intensity. It is this stress concentration which causes the tip of steel piles to buckle, tear, or accordion. The high rebound tensile waves, alternating with the input compression, may lead to low-cycle, high-amplitude fatigue of any welded splices in the steel piles.

In the case of concrete piles, two special stress patterns must be considered in order to permit driving to the required penetration without damage. Unlike steel piles, which are most damaged by the high compression stresses in hard driving, concrete piles are subject to damage in soft driving, such as the initial blows, which produce tensile rebound stresses. Without the proper cushion block and hammer control, horizontal cracking may occur through the entire body of the pile, showing itself initially as a puff of dust at each hammer blow. Continued driving will lead to damage to the concrete at the crack and stresses in the steel beyond yield; the prestressing steel will eventually fracture. This phenomenon is also noted when, after driving through competent soils, which seize the pile in friction, the tip protrudes into weak material or a void below. Such a void may have been created by excessive jetting below the tip. This phenomenon is amplified underwater, where water is progressively sucked into the crack, then subject to high-impact pressure under succeeding blows.

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The second special stress pattern is due to Poisson's effect: the lateral bursting of either the head or the tip under the high axial compression in hard driving. Fortunately, concrete that is well-confined can resist very high compressive forces. Hence, closely spaced spirals will prevent damage. To prevent vertical cracks due to bursting, spiral or hoop reinforcement should be proportioned so that the steel area at yield stress is greater than the concrete tensile strength in the area of concrete in the tensile zone; this will ensure that any crack formed by driving will be pulled closed. Since the problem is greatest at the head, extra confinement is needed.

Piles being driven onto or through rock should have square or blunt ends. Tapered and pointed tips deflect and break the pile. Piles should initially be seated vertically or on the design inclination. This may require two points of lateral support, the drilling head at the top and a template near or below the waterline. After initial penetration and before the full impact of the hammer is delivered, the lower support should be freed. Otherwise, with a third point of support, the pile may be subject to bending and breakage. As the pile then penetrates, the hammer should be gradually repositioned to "follow the pile." Attempts to correct the pile location and inclination are generally ineffective and may lead to pile breakage.

The above has been demonstrated on a number of projects with slopes covered with stone riprap in sizes up to 500 mm and stone dikes up to 10 m thick. If the underlying sands consist of dense sands or silts for which jetting is required, a jet pipe can be cast into the center of the pile and operated continuously during the driving, to prevent plugging.

Steel piles, driven into or through rock and riprap, tend to buckle at their ends. Eventually the flanges may tear. Steel pile tips should be reinforced with either fabricated or cast steel tip protectors to prevent this distortion.

9.2.2.4 Batter (Raker) Piles

Batter piles are typically used to resist lateral loads (Figure 9.2). They act in conjunction with a vertical or opposing batter pile, one taking compression, the other tension. Under imposed displacement, such as that caused by earthquake or ship impact, this may result in damage to the pile heads or cap beams. The present trend in seismic regions, therefore, is to use all vertical piles and take the lateral forces through elastic or elastoplastic bending of the piles. In the usual case of a sloping bottom, the shorter piles, being stiffer, will take almost all the lateral loads. Thus they may be made larger and reinforced more heavily or be spaced more closely than those in deeper water.

FIGURE 9.2
Prestressed concrete vertical and batter piles set through templates and run down under their own weight, preparatory to driving.

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In some seismic retrofit projects where existing batter piles must be used, ductile fuses have been inserted at the connection: they are based on a connection which deforms in bending under high load and thus limits the compression in the pile which is the most highly loaded. This concept has also been used in new construction for quays in Long Beach Harbor, California.

9.2.2.5 Pile Location

Piles in finger piers, extending out from the land, are typically driven with floating equipment, although in areas of extreme tides, such as southern England, piles may be driven by rigs moving out over the top of the completed construction. In quay construction, although the outboard piles are typically driven afloat, the inboard piles may be driven with a land-based crane, rigged as a pile driver, with a hydraulically operated spotter to position the pile vertically or on a batter.

Due to the close spacing of piles in relation to the width of the typical pile driving barge, the pattern for installation has to be laid out carefully, fanning out from the shore in order to be sure to have access to every pile. This problem is compounded when there are batter piles, since the barge must be offset from the design head location. Some clusters of batter piles, radiating in different directions, may prevent direct driving, so a template must be used, supported by already driven piles and additional spud piles, in order that the new pile may be properly placed.

Batter piles are typically supported in the leads and at the deck level. When they extend down through the water, they are cantilevered and droop until they are embedded in the soil. Thus, in deep water they need underwater support, such as telescoping leads. Among the other options available to the contractor are to increase the size of the pile, the concrete strength, and especially the level of prestress. Thus on Pier 23 in San Francisco, 50-m-long batter piles in 15 m of water depth were increased in size, with approval by the designer, and were prestressed to 10 MPa, in order to facilitate setting on a 3 in 12 batter.

9.2.2.6 Jetting

Jetting is frequently used to assist penetration of a pile, especially in dense sands. The water cuts a hole below the tip, and rises up along the pile, reducing the skin friction.

Jets may be "free," that is, operated independently of the pile, or fastened to it by intermittent sleeves. Jets may be incorporated into prestressed concrete piles. However, the jet pipe must be a continuous membrane, e.g., thin-walled steel, so as not to allow direct water access to the concrete. Otherwise, if any horizontal crack opens under tensile rebound, the water pressure will be sucked in the crack and subsequent compressive stress from impact will fracture the pile. Although many contractors use plastic jet pipes for economic reasons, there is always the potential for a ruptured pile as the rigid plastic pipe mirrors the crack. Thick plastic composite, such as reinforced fiberglass pultruded pipe, has given satisfactory service.

9.2.2.7 Driving Through Obstructions or Very Hard Material

The first rule is to have a compact section pile at the tip, in order to prevent local damage and progressive buckling and damage. With steel pipe piles, the tip should be closed by a heavy plate. The wall thickness for at least the lower two diameters should be increased. The tip of the pile can be filled with concrete for two to four diameters. Points cause the pile tip to deflect, thus producing high bending stresses in the pile, so they should not be used.

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Alternatively, a cast steel tip protector of the "external" type can be fitted. Such a tip will give a constant inside diameter at the tip of a pipe pile and thus not interfere with subsequent drilling. Then, when an obstruction is encountered, a drill may be used to drill through and ahead. Churn drills and down-the-hole drills are the most frequently employed.

Steel H piles, which penetrate conglomerate and dense sands and gravel quite well, do not behave well when they encounter cobbles and boulders or hard irregular rock. The flanges tend to tear and may curl up like spaghetti. There is often no easy way to verify their continuity. Steel tip protectors are needed but may not always prove adequate. Making the tip into a blunt end may help but may lead to buckling above the tip.

The Norwegians have apparently been successful when founding concrete piles through alluvium onto glacially smoothed rock, by incorporating a short (300–500 mm) dowel of high yield steel in the concrete pile tip to get a toehold in the rock.

For the Trident submarine wharf in Puget Sound, Washington, steel H piles were embedded in the tips of the prestressed concrete piles in order to penetrate dense layers of overconsolidated silt. Jetting is also very effective in aiding penetration of these over-consolidated silts. On a number of projects, concrete piles penetrated stone fill embankments and riprap quite well, pushing the stones aside as they penetrate. With graded rock fills having maximum size stones up to 500 mm, little difficulty is encountered in driving and the pile location can usually be kept within 150 mm. Even when larger rocks must be penetrated, this can usually be accomplished, but tolerance in location will be greater, e.g., 300–400 mm. The pile tip should be square, with chamfered edges. The pile must not be restrained once it has penetrated a meter or two. The old-fashioned practice of driving a spud and pulling it before driving the permanent pile is counterproductive; when the spud is pulled, large stones are "sucked" into the hole.

Since many existing harbors have had several generations of wharf and pier construction, usually founded on closely spaced timber piles, it may be necessary to drive the new piles through the remnants. While, again, prior practice was to remove the existing piles, this is often not advisable. The old piles reinforce the slope; removal may lead to instability. In the case where a compact steel pile or prestressed concrete pile encounters an existing timber pile, it will usually deflect the timber pile out of its path or cut through it.

When driving piles, they must not be unduly restrained, as this will induce unacceptably high bending stresses. During driving, the pile is supported in the driving head and at the deck level. Once it is embedded in the soil, the gate at the deck level must be opened. Otherwise, if the tip of the pile drifts off because of encountering obstructions, there will be high bending at the deck level, which will damage either the gate or the pile. This problem becomes more severe as the driving head approaches the deck level. These warnings are especially applicable to batter piles.

9.2.2.8 Staying of Piles

In typical deep harbor situations, the piles are subject to continuous movement due to current and waves. In some cases pile heads may move up to 300 mm with the change in tidal currents. Batter piles, already stressed close to their bending capacity, may lean and crack. Therefore, a system must be implemented to stay the piles in position so they will be stable and the heads can be later incorporated into the deck.

Typically, timbers are used, bolted well below the future deck forms so that they will not interfere. They are secured by the friction developed by the bolts (Figure 9.3). When batter piles are involved, the timbers may be supplemented by wire rope ties. Batter piles should

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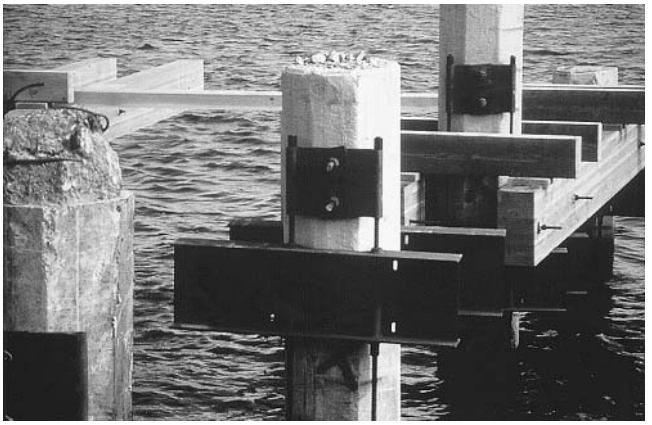

FIGURE 9.3
Concrete piles are stayed in place by gluelam timber resting on steel collars.

be stayed and tied before the pile-driving rig releases its temporary support (see Figure 9.4).

9.2.2.9 Head Connections

After driving, the pile is cut off to grade; for a prestressed concrete pile, it should first be encircled by a 50-mm-deep cut made with a diamond or carborundum saw, and then broken down from above by a small air hammer. Without the precut, the concrete,

FIGURE 9.4
Prestressed concrete piles are properly stayed. (Courtesy of General Construction Co.)

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being under stress, will spall badly. Hydraulic shears are able to cut both steel and concrete piles with limited local damage to the pile head.

Due to the limited thickness of the typical deck, it may not be possible to get full fixity, but a high degree of partial fixity may be obtained by either embedment of the pile or by the use of reinforcing steel. Embedded steel piles should have shear keys or plates. They should be heavily confined by either reinforcing steel or plates. For a steel pile, when reinforcing steel is used for the connection, it can be welded in shear to the steel pile or embedded in concrete in the pipe pile. For concrete piles, a good connection is to form 50 mm holes in the head of the pile into which bars may be inserted and grouted.

9.2.2.10 Concrete Deck

The typical concrete deck usually consists of deep transverse cap girders incorporating the pile heads, working monolithically with a one-way longitudinal slab. If cranes will be mounted, there will also be a longitudinal girder at or near the face and rear wall. This cast-in-place system is made continuous over the transverse caps.

Support for this during construction is typically obtained by transverse timber girts, accurately set to grade, locked to the piles by bolts. To increase the friction, high-friction pads may be placed on the timber-pile contact interface. Even then, the friction factors must be selected conservatively because the surfaces will be wet. Timbers are used because of their strength in bending and because they will float when later removed. Gluelam timbers may be employed for heavier loads and longer spans. The critical design condition for these girts is usually horizontal shear adjacent to the pile.

In many cases, the contractor has attempted to integrate the pile-staying system discussed in Section 9.2.2.9 with the form-supporting girders. The problem that develops is that the form supports must be set to exact grade and must be heavy timbers to carry the loads, whereas the stay timbers can be lighter and be set to any convenient location, generally below the form supports. Steel channels and even wide flange beams have also been used for longer spans.

When the loads from the fresh concrete of the deck, multiplied by an appropriate safety factor (1.5–2.0), are not able to be developed by friction alone, then hangars are used over the top of the piles. High friction pads are an alternative. The consequences of form slippage during concreting are severe, not only in relation to the lost concrete and bent reinforcement, but also possible injury to workers and pollution of the water by the wasted concrete. Holes can be cored through the top of the pile and bolts used to carry the load in shear and/or the friction of the form supports or collars, but these can reduce the durability of the pile at a critical location.

The typically cantilevered deck overhang on the face of the wharf is especially difficult to support without sag under the weight of the concrete. The contractor has to choose between an extra temporary support pile or heavy cantilevered girts. Hangars are almost always essential on the outboard pile. Forms for the pile caps and girders are usually prefabricated as boxes, and may be set with cages of reinforcing steel inside. Wave impact on a horizontal underside may generate very intense local forces. This can be minimized by slightly sloping the soffit forms, say by 7°–10°.

Many modern heavy-duty container wharves are designed with a two-way continuous flat slab, hence no deep-cap girder. Special shear-head reinforcement is required over the piles. The deck forms in this case are very simple, consisting of joists and plywood sheets. Nailing is only that necessary to prevent dislocation by wind. This system greatly facilitates the future form removal from under the deck.

All deck forming should be designed with regard to its removal, since the removal is usually more costly and is definitely more dangerous to the workers than the initial

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placement. Form removal is usually performed by workers on a float using scaffolding. Life jackets are mandatory, and there should always be more than one worker present. To minimize the exposure of workers, temporary small holes can be formed in the deck so that the forms and girts may be lowered down into the water after they have been broken loose, then floated clear for retrieval and reuse.

Innovative alternative systems for forming have been developed, aimed at reducing the labor of forming and stripping. One of these utilizes precast concrete segments. First, the lower half of the cap girder is constructed using light timber girts for temporary support. Then precast full-depth or half-depth precast slabs are set on the half-depth caps, and the whole made monolithic by cast-in-place concrete. This system requires that the cast-in-place half-depth pile cap be widened as necessary to correct for any mislocation of the piles.

In location of large tidal range, forms may become submerged and even float up. They must either be tied down or precast concrete forms employed.

9.2.2.11 Fender System

For modern container terminals, designed for the berthing, unloading, and reloading of containers only, the fender systems are usually heavy rubber fender units, mounted on the face beam of the wharf, spanned by a steel beam and timber rubbing strips. The fender is supported by chains from the deck, both for gravity load and for the longitudinal displacement by friction from the ship. These latter chains are placed at a 45°–60° angle.

More complex fender systems are required when the quay or wharf must also handle barges and smaller ships. They must not only be fendered but prevented from getting underneath the dock. The outboard bearing piles must also be shielded from impact. Creosoted-timber fender piles were much used in the past, but their use is prohibited in many countries, including the United States, for environmental reasons. The U.S. Navy has therefore sponsored the development of prestressed concrete fender piles and also fiberglass fender piles. While more costly than the timber which these replace, they are able to accept more energy of impact and hence can be spaced more widely.

Hardwood timber or laminated composite rubbing strips are usually bolted to the face of the prestressed concrete fender piles. Such piles have given excellent service for over 30 years in the Arabian Gulf (Kuwait) and in Singapore (Jurong). Steel, concrete, and timber panels, bolted to the face edge beam of the wharf, have also been used, but are very difficult to repair or replace after damage.

9.2.3 Bulkheads, Quay Walls

9.2.3.1 Description

This term is used to designate a wharf or quay that has a solid face, either sheet piles or caissons, which support and retain fill behind them. They are especially well suited to relatively shallow depths of water, although caissons have been used for berths for large vessels as well.

9.2.3.2 Sheet Pile Bulkheads

Single-wall steel sheet piles are the principal type, anchored back to deadmen at the waterline or slightly above. They develop the majority of lateral shear resistance by passive pressure on the below-ground length. Thus adequate penetration is essential, and design tip elevations must be achieved.

The Z configuration pile is the most common, giving maximum bending strength while maintaining economy. For greater depths of water and for higher surcharge

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loads, H sections or pipe-pile sections with interlocks are employed, often in combination with Z piles.

Hot-rolled steel sheet piles have "thumb and finger" interlocks, which have guaranteed tensile capacity, whereas the less costly cold-rolled sheets used for shallow depths of water and fill are somewhat unreliable. They also are not suitable for driving into heterogeneous soils containing cobbles or hard lenses.

Driving is resisted by tip bearing, by friction with the soil on both sides, and by friction in the interlocks. This latter item can be a major problem increasing with depth and resisting, or even preventing, attainment of the design penetration. Accuracy of the interlocks is controlled by the manufacturer. Binding of the interlocks is determined by the care with which the sheets are set. Hence, they should be set against a supporting wale with care to ensure verticality. The sheet piles have a designed spacing, usually giving 6–10 mm play in the interlock. Therefore, the location of each pile should be marked on the wale to prevent crowding or stretching. Z piles in particular are easily rotated, which not only increases the friction slightly but decreases the effective wall strength. Lubricants such as greases are often employed but may not be fully effective due to contamination by sand grains. Hydrophobic sealants can be applied to the interlocks to make the sheet piles essentially watertight.

Since it is possible to set many sheet piles in a workday, there is a tendency to set them rapidly, without the required care. Then, when it comes time to drive them, the friction increases and often gives the illusion of having struck a buried log or other obstruction. On occasion, it has proved necessary to extract a length of wall and reset the piles, a very costly and time-consuming operation. Individual sheet piles may be threaded into the previous one by hand, although this poses risk of injury or falling. Automatic interlocking devices are available.

The steel sheet pile wall, once set, is very exposed to the wind. Entire walls have been blown over, bent across the top wale. Since it is usually practicable to support from one side only, one out of every three or four sheets should be both tied and strutted. Steel sheet piles may be set in panels or in pairs. Driving of sheet piles should proceed in echelon, so that no tip of an individual sheet or pair is more than 1 m ahead of its neighbor. In the early phases of driving, where there is usually little resistance, there is a strong temptation to drive piling down several meters beyond the adjacent piles to avoid changing the hammer. The problem arises due to the fact that sheet piles have low stiffness transversely and hence can be easily deflected by a relatively minor obstruction. Then, when the adjoining pile is driven down, it is forced to deflect also, since the interlocking pile is firmly embedded in soil. The result is excessive driving friction and a high probability that the tips will not reach design tip elevation.

Vibratory hammers are especially effective for driving steel sheet piles. They temporarily liquefy the sands and silts along the faces of the pile, which accounts for their ability to achieve rapid penetration. These hammers work best in cohesionless soils but are not effective in hardpan, or in layers of dense ash or peat. There, impact hammers must be used.

Excessive and prolonged vibration may liquefy large masses of adjoining soils, leading to slope failures. In sensitive soils, impact hammers may pose less risk. Piezometers can be installed to monitor pore pressure. Vibratory hammers cause permanent loss of shear (skin friction) in clay soils, whereas impact hammers allow for long term set-up.

Driving sheet piles in dense Arctic silts has proved impracticable unless the soil has been loosened by jets. On the West Dock at the Arco facility on Prudhoe Bay, on the North Slope of Alaska, sheet piles met refusal at 10 m for both vibratory and impact hammers. Running a powerful jet down along the line of each sheet sufficiently loosened the silt so that the piles were readily driven by the vibratory hammer.

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A sheet pile bulkhead, being essentially a vertical wall, reflects incident waves, forming a clapotis, or standing wave, one-half wavelength in front of the wall. There is a strong upward flow of water immediately in front of the wall, which in the presence of even a moderate current, can lead to severe erosion. Use of a filter plus stone and riprap may be required. A filter fabric, properly overlapped and covered by rock, is preferable, although a properly graded stone filter may be used.

Prestressed concrete sheet piles are also used for bulkheads, especially those in shallow water, because of their superior durability. However, to achieve comparable bending moment to steel sheet piles, they must be relatively thick. They must be heavily prestressed. Interlocks are usually of the tongue-and-groove type, often with provision for grouting of the interlock after installation.

Typical dimensions of prestressed concrete sheet piles are 1000×300 mm^2. To overcome the large end-bearing area and the skin friction, jetting is usually necessary. The jets may be embedded in the pile, using thin-walled steel pipe. Plastic pipe is generally not suitable. There is always the possibility of tensile rebound stresses producing a crack in the concrete pile, which will also be reflected through the plastic, with the result that the jet water pressure now acts on part or all of the cracked area, leading to hydraulic ram effects on the crack under every subsequent blow, with washout of cement. Low-cycle fatigue will occur under as few as 20–50 blows.

Unlike steel sheet piles, prestressed concrete sheet piles are each driven to full penetration, one by one. To ensure that the subsequent pile will follow down close to the previous pile, the tip should be tapered on about a 45° angle, wedging the tip of the new pile back against the first. The top can be held in alignment by a line from a hydraulic winch, with a roller to reduce friction.

Plastic interlocks have been used to prevent sand leaching through the joint as the tide alternately runs in and out. These have limited strength and cannot be counted on for structural purposes. To provide a structural interlock, a half section of a flat steel sheet pile is embedded in each side of the prestressed concrete pile, and anchored by reinforcing bars welded to the sheet. The interface between the steel and the concrete will develop a high oxygen gradient, and hence be subject to corrosion at the line of embedment. Special attention should be paid to this corrosion susceptibility, e.g., by coating the joint with epoxy or installing zinc anodes in the joint between the concrete sheet piles.

Sheet pile walls, both steel and concrete, can support only a limited height of backfill as a cantilever, depending on the passive resistance and moment (P/y) developed by embedment. For greater depths and surcharges, tie-backs are required. These tie-backs are typically steel rods extending back to a buried concrete deadman. Mild steel rods are preferable even though of limited yield strength: they have high ductility and inherent corrosion resistance. More efficient prestressing bars have been used but have resulted in some failures due to their lack of ductility. Piles, driven on a flat batter into the soil behind, and drilled-in anchors have been used to resist lateral loads. Recently, helical and fin piles have been used as ground anchors for bulkheads.

Sheet pile quay walls rely on passive pressure below the final dredged soil line to resist part or all of the lateral load. In weak soils, such as soft clay, soil reinforcement such as jet grouting has been employed to give added shear strength to the soil. Reinforcement by stone columns is preferable. Piles, if used as reinforcement, must be terminated below the adjacent seabed so as to prevent puncturing a ship's hull at low tide. To prevent sand fill from leaching through the interlocks of steel sheet piles, a special swelling rubber compound such as "Adeka" may be brushed into the interlocks as the sheet piles are set. The fill should be drained, at least above low tide. Filter cloth sacks of geosynthetic may be sand filled and placed behind each drain hole.

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9.2.3.3 Caisson Quay Walls

Concrete box caissons are frequently used as quays in Europe, the Middle East, and Canada, especially where the soils are firm, such as dense sand or hardpan. A trench is dredged below the adjacent seabed and a leveling course of gravel placed. If necessary the soil may first be densified by vibration or stone columns to prevent liquefaction. The stone bed on which the caisson is to be landed may also be densified by a vibratory screed. Then, the bed is screeded to a tolerance of about ± 20 mm.

The concrete box caisson has usually been designed as self-floating. A typical box will have been fabricated in a casting yard, moved forward on rails by sliding or transporter jacks, and set in the water by a large sheer legs crane barge. This transfer from land to water is a critical operation, depending on tidal range, seasonal variations in water height, and relative elevation of land and low water.

The procedure described above depends on the existence of a quay wall or trestle from which to lift the concrete box caisson. Other methods that have been employed are

1. Slipway or launch way, with a cradle that can be lowered down
2. Trestle-supported gantry which lowers the unit into the water
3. Transporters or skidways, allowing the unit to be moved laterally onto the barge. Later, the unit is launched from the barge by ballasting the barge down, allowing the caisson to float off. This procedure requires special considerations concerning stability and buoyancy, which are discussed in the next paragraph
4. Constructing in a basin or graving dock, and then flooding the dock to allow the caisson to float out
5. Catamaran picks up units from a trestle
6. Construction on a barge, with subsequent submergence for float-off

Launching of a box caisson from a barge requires a ballasting process, which ensures control of draft, list, and trim. It also requires that the combined barge plus caisson be stable at all stages of ballasting down and that the barge itself be stable after the caisson has been floated off. The barge must withstand the external hydrostatic head during submergence. To control draft during ballasting down, the caisson should be at a site of limited depth and, if possible, a site with a level bottom. Stability during ballasting down can then be obtained by ballasting one end, so the barge tilts down to contact the seafloor. This then gives transverse stability to the combined caisson-barge unit while it is ballasted down to float off the caisson. The barge should be overballasted to ensure that it stays down while the caisson floats off. One end can then be deballasted to tip the end to the surface where its water plane will give it stability for full deballasting. In less favorable sites, stability control has been attained by buoyancy columns attached to the barge.

The above refer to self-floating boxes, with a flat bottom. In some cases, open-bottom caissons may be selected, especially where the bottom soils are weak. Then piles may be driven, the open bottom caisson set on leveling pads, the bottom edge sealed, and tremie concrete placed to tie the caisson to the piles. In this case, the units must be transported on barges or supported by a sheer-legs crane barge or catamaran.

The caissons must be accurately aligned and the joint between adjacent caissons must be sealed. A short arc of steel sheet piles can be threaded into interlocks embedded in the caisson walls. The joint must accommodate the differential settlement of the caissons.

Caissons typically fail outward, even under wave attack, due to erosion under the edge and liquefaction of the supporting soils.

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After seating, caissons are typically filled with sand or quarry-run rock. The caisson's walls should be designed for the temporarily high pressures created by the infill.

In some cases, the caisson is filled with mass concrete. The caisson walls are initially loaded by the fluid pressures of the concrete infill and subsequently, by the thermal expansion of the concrete during hydration. This latter will be relieved as the concrete mass cools, but since the caisson's walls and outer portions of the mass cool faster than the core, cracking may be imposed on the caisson walls. While this can be resisted by adequate quantities of reinforcement, the heat of hydration can best be minimized by replacement of much of the cement by fly ash, and use of ASTM Type IV cement or at least a coarse-ground Type II cement, or by use of blast furnace-slag and/or by pre-cooling of the mix.

Note that all these steps tend to lengthen the setting time and hence increase the initial outward fluid pressure.

Caisson quay walls require special design and construction considerations in seismic zones, due to the potential for liquefaction of the sands under the base and especially under the toe. Liquefaction of the sand fill behind can add to the outward pressure. The extensive caisson quay walls at Kobe, Japan, were badly tipped and displaced by the Great Hanshin Earthquake of 1997. Therefore, provision must be made for the rapid dispersion of pore water pressures which build up under earthquake. These high pore pressures and liquefaction can also occur due to heavy wave action which causes the caisson to rock. The stone bed underneath should have high permeability. Along the toe, a rock filter should be constructed to protect the rock from being clogged by fine sediments. The stone bed itself should be graded to prevent sand infiltration from below, or placed on filter fabric. It should be densified. An underwater steel sheet pile cut-off will prevent loss of material.

9.3 River Structures

9.3.1 Description

These include locks, low-level dams, overflow structures, and flood walls. Historically, these have all been constructed "in-the-dry," using large sheet pile cellular cofferdams to permit dewatering, with subsequent construction of the concrete structure by conventional land methods.

In recent years, the U.S. Corps of Engineers has decided to construct some of these structures "in-the-wet," using marine construction methods and equipment. This enables the key structural elements to be prefabricated in a yard, for subsequent transport and installation underwater. The key benefits of this approach are the reduced costs and time, and the reduced interference with river navigation.

Both the construction of sheet pile cellular cofferdams for "in-the-dry" construction and construction "in-the-wet" by marine and offshore methods will be described.

9.3.2 Sheet Pile Cellular Structures

Steel sheet pile cells have been extensively used for large cellular cofferdams to enable the subsequent dewatering so that a structure like a lock or a dam can be built in the dry.

By this method, a portion of the river, usually one-third or one-fourth, is closed off and surrounded by multiple sheet pile cells. The site may have been dredged beforehand to remove unsuitable material. Each cell will be circular, approximately the same diameter as the depth of water for which it is designed. Two or three ring wales are placed as a space

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frame, and then held in place by temporary spud piles. Flat steel sheet piles are then set around the ring wale, taking care that they are truly vertical in both directions. The Y piles are set first. Since they are stiffer, they should be set accurately. To ensure proper spacing, the location of each sheet should be pre-marked on the top wale. In deep water and swift water, the lower wale may need to be located underwater. The sheets are then driven down incrementally with a vibratory hammer, working progressively around the ring, driving each in turn up to 1.5 m below the previous sheets. The purpose is to keep the protruding pile held in alignment by the adjacent sheets (see Figure 9.5 and Figure 9.6).

After closure of a complete cell, the sheet piles cells are then filled with coarse sand, which is compacted by vibratory probes to a suitable density. This vibration builds up pore pressures, which can lead to local liquefaction, and high internal pressures, especially if the cell fill consists of fine sand. Vertical drains of gravel can be used to allow ready escape of the water. This also aids and speeds the process of densification. Once two cells are filled, they are connected by arcs of sheet piles which engage the Y sheets in the circular cells. This mini-cell is then filled with sand.

Once all cells of a cofferdam for a lock or dam are filled with sand, a berm is constructed on the inner side to give added lateral support to the wall. This may be of rock that is free draining. Most berms, however, are constructed of sand, with wells or well points to ensure against saturation by seepage. The slopes may be protected by a stone blanket over a filter fabric to prevent erosion by rain and overtopping.

The cofferdam is now dewatered, and the structures built by conventional land methods (see Figure 9.7 and Figure 9.8). During this period, it is essential that a monitoring program be followed to reveal any scour along the cofferdam outer walls, so that if necessary, riprap

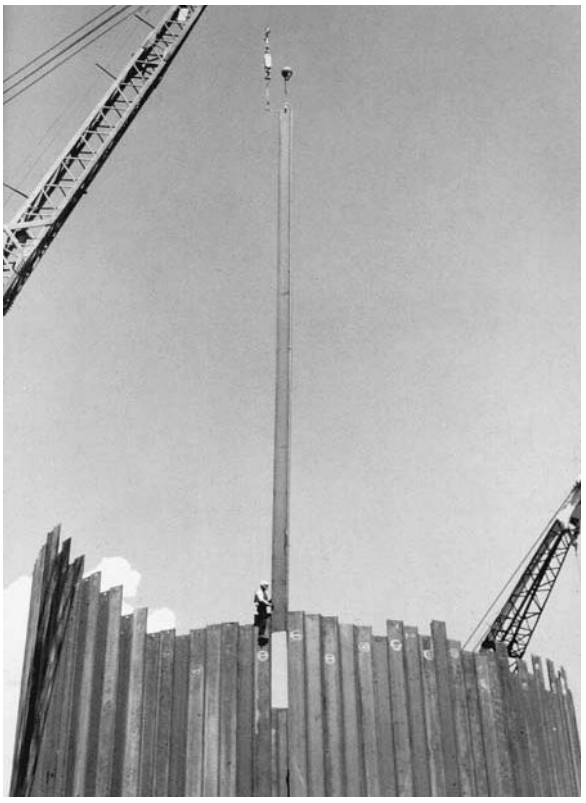

FIGURE 9.5
Setting steel sheet piles for cellular structure.

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FIGURE 9.6
Setting closure sheet in cellular structure, Columbia River Terminal, Oregon.

can be placed. Water levels and pressures within the cells and berms are also measured by piezometers or wells so that dewatering measures can be installed on a timely basis. Inclinometers check the cells for any tendency to lean.

If the river floods so that overtopping of the cofferdam is imminent, the cofferdam is intentionally flooded, taking care not to erode the berms. Arrangements are incorporated

FIGURE 9.7
Steel sheet pile cellular dry dock for Trident Submarine, Washington.

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FIGURE 9.8
Sheet pile cellular cofferdam for construction of Olmsted Lock, Ohio River. (Courtesy of Corps of Engineers.)

at one or both ends of the new structure so that the end cells of the next stage cofferdam can be constructed. After all construction is completed, the cofferdam is flooded and the sheet piles are extracted, for reuse on the next stage cofferdam. There is always a certain amount of damage to sheet piles, so that only about 75% or 80% are usable in the next stage.

The setting of the flat section sheet piles poses difficulties when the water is deep, the current strong and the sheet piles are long. For initial picking, the piles should first be rotated onto their strong axis before being lifted. While lifting, the piles must be controlled by tag lines since otherwise wind will make them uncontrollable. Threading used to be done by hand. A person straddling the previously set sheets, using a portable chair, threads the interlocks. Today two alternatives are practicable and safer. One is to have an upper ring wale on which a person may stand and from which the sheet pile can be threaded. The other is by use of an automatic threader, which rides up the interlock of the previously set sheet.

The current and wind affect both the individual sheet as it is being slid down, and the partially set wall. That is why two or more levels of ring wales are necessary. The sheets should be secured against wind and wave action from both sides. In strong currents, the use of a deflector shield may be necessary. A large barge, moored close aboard upstream, will still the surface currents but may accelerate the lower flows. A protector shield may be a fabricated shield which is secured temporarily by spud piles. It is located 5 m or so upstream. In slower currents, a deep draft barge, moored upstream, may be adequate.

The upstream arc of the sheet piles should be set first. Sheet piles in the downstream arc and those parallel to the current should be temporarily tied back to the ring wales. With long sheet piles, a strong-back may be needed to support the sheet pile while it is being lifted, upended, and entered and run down. The strong-back is usually a wide-flange beam, with clamps that lock to the sheet pile until they are released mechanically or hydraulically.

Cellular cofferdams are very vulnerable to scour, especially at the upstream corners. Vortices may also erode the downstream corners. Filter fabric tied to an articulated concrete block mat is useful in preventing serious scour. Heavier stone may be needed in addition where currents exceed 2.0 m/s.

During construction, a sheet pile cell is very vulnerable to currents, waves, and wind acting on the sheets prior to the time they have been interlocked, driven, and filled.

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Current not only exerts a direct force but vortices form on the exposed ends of the arcs. Individual sheets (or pairs) may need a stiff beam attached to support the lower end until it is embedded in the seafloor far enough to develop a shear reaction.

The sheets must be supported by a stiff template with multiple wales. As noted in a current, the upstream arc should be constructed first. Sheets need to be temporarily tied to the wales to prevent working in and out, especially in waves and swells.

When the cell is constructed in the open sea, subject to long period swells, such as occurred in building a terminal offshore Brazil and one in the Bering Sea, the only way found successful was to prefabricate the cell around a jacket-and-pin pile template, and then lift the entire cell with template into place with a large offshore crane barge.

In the more normal case, moderate depth water, low velocity of current and only local waves, driving the sheets incrementally in panels and stages and use of a multiple wale template supported by spuds will be found adequate. Most partial failures of cells have been due to driving sheet piles out of interlock due to driving adjacent pile tips too far ahead.

Other causes of failure during construction have occurred due to saturation of the fill, plugged drains, excessive vibration without drainage causing liquefaction of the cell fill, and backfilling behind partially completed cells. Rapid drawdown during dewatering the cofferdam or graving dock may lead to failure.

A new type of so-called "open cell" form has no closure on the landward side. It is really not a cell but an anchored bulkhead. Arcs of sheet piles are connected by Ys, but the anchorage is constructed of sheet piles extending normal to the face. As with closed cells, temporary falsework is required. Although an effective and economical solution for weak sediments, it is vulnerable to progressive collapse. Therefore, special anchorage has to be developed for the "tails" at the ends of the bulkhead or quay wall as the safety of the entire structure depends on them.

9.3.3 "Lift-In" Precast Concrete Shells—"In-the-Wet" Construction

This concept employs prefabricated concrete shells, which are segments of the exterior surface of the lock or dam. The segments are sized to the maximum lifting capacity of equipment that can reasonably be assembled for the project. For the Olmsted Navigable Pass Dam across the Ohio River, there will be 40 segments of over 3000 tn. each. These are reinforced concrete shells, designed to act in composite action with tremie concrete in-fill (see Figure 9.9). The shells will be precast in a yard that is above high-water level, thus enabling year-round production. Each will be slid forward onto an inclined slipway, and then launched down into the water. A catamaran barge will float in over the segment and lift it clear. The catamaran, with suspended segment, will be towed to the site and moored to preplaced moorings.

A structural steel frame will be secured to the top of the shell before launching. This frame serves many purposes, one being to distribute the lifting forces from the catamaran hoists to the shell to reduce bending moments. A second purpose of the frame is as a guide for accurate positioning of the segment as it is set underwater (see Figure 9.10). Prior to arrival of a segment, the site will have been dredged, rock scour protection placed, and screeded to grade. Piles are driven, guided by a light steel temporary template. Leveling pads will be constructed to later support the prefabricated shell. These pads will consist of an underwater concrete cap, set accurately to grade on a cluster of piles. When the catamaran and its precast concrete shell arrive, the segment will be lowered onto the leveling pads and then pulled laterally into contact with the preceding shell. Once in position, a reinforcing steel bottom mat is placed and the entire cell filled. Note precautions in Section 9.2.3.3. The shell is then filled with tremie concrete, placed through tremie pipes by gravity feed. These tremie pipes are guided and supported by the temporary frame

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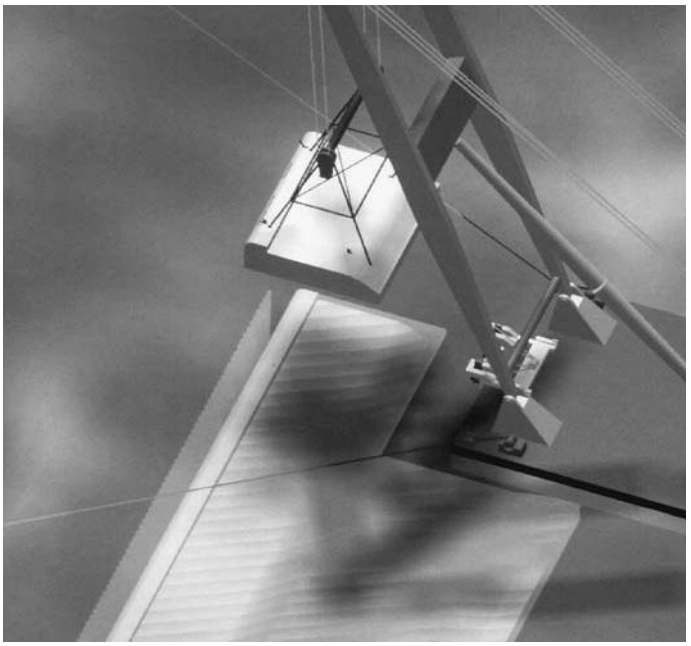

FIGURE 9.9
Lift-in precast concrete shell for River Dam.

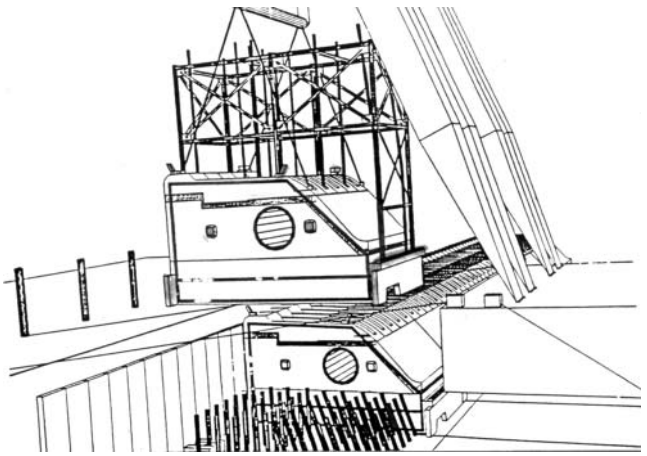

FIGURE 9.10
Temporary structural frame mounted on precast concrete shell to hold tremie pipes during concrete infill.

attached to the shell. Then, the frame is removed by releasing bolts operated from above water.

9.3.4 Float-In Concrete Structures

9.3.4.1 General

Complete sections of navigation structures, such as locks and navigable pass dams, are constructed in a construction basin or fabrication yard, then launched, towed to the site as

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self-floating structures, and installed by ballasting down. This process closely resembles that of prefabricated immersed tunnels (see Section 9.5). However, the differences are important.

First, the cross section of these structures may be irregular and require special ballasting in order to eliminate heel during the floating phase. The section is typically difficult to design and detail for resistance to hydrostatic pressures during installation. Second, unlike the tube (immersed tunnel), which is set in a trench, the navigation structure extends above the bottom, up through the water. Positive cutoff has to be provided in order to prevent underflow. Third, because of the unequal water pressures and, in many locations, seismic considerations, lateral support has to be provided in the form of piling, or other means. For example, locks see many cycles of high global shear as they are filled and emptied. River structures see unbalanced water levels. Fourth, navigation structures often have exposed appendages, such as movable gates, attached, which require special protection during tow and installation. Finally, navigation structures must usually be installed in close proximity to existing structures, which have to be kept in service.

9.3.4.2 Prefabrication

Phase one of the construction is the prefabrication. Typically, this takes place in a construction basin. A thin layer of crushed rock is laid on the floor of the basin and covered with corrugated steel sheets or plastic on sand. Plywood sheets covered with polyethylene have also been used. Note that whatever is used will stick to the underside of the caisson, held by adhesion and buoyancy. Therefore, if steel corrugated sheets or plywood are used without polyethylene, they should have ties to hold them to the bottom to prevent any possibility of their coming loose during final installation, where they might interfere with the set-down operations. If the base slab is to be later prestressed, the sheathing must slide to accommodate the shortening. More recently, the hard-finished concrete floor slab in the basin has been coated with bond breaker and the new slab cast directly on it. Through plastic pipes spaced about 5×5 m^2, water is injected at low pressures for sustained periods of several hours; then the basin is flooded and the barge or segment rises clear of the bottom.

When shear keys or a saw-toothed base is required, the effects of concrete of the base slab shortening under shrinkage and prestress must be considered as they will tend to "lock" the caisson base to the casting slab. Adhesion and suction will add to the "locking" effects. Incorporation of crushable or elastic pads are a partial solution but must be removed from the caisson base after it is afloat since otherwise they will tend to adhere to it. Inflatable flat hoses may be employed. They can be used to jack the caisson base laterally or vertically and are readily stripped from the caisson underside.

Construction in the basin then proceeds, often using precast concrete slabs, joined by cast-in-place joints. Construction joints must be properly prepared by water jet blasting. Epoxy bonding compound may be used, applied to the joint face just before the concrete is placed. Forms for the joint must be grout-tight. Waterstops are not usually used; they impede proper concrete placement and just create a potential water path.

Reinforcing steel must go across the joint. While this is a matter for design, the constructor should be aware of the labor required in interweaving the steel. Accurate placement of staggered bars reduces the problem. Consolidation of concrete in deep, heavily reinforced joints may be aided by pre-placement of a tube. The vibrator is lowered down the tube, extending just below its tip, and then both are gradually withdrawn as the concrete rises.

Roof slabs and other near-horizontal members are usually precast or else formed with stay-in-place forms. Corrugated steel flooring, plywood, and thin precast concrete slabs

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have been used. Precast concrete half-depth slabs, with protruding reinforcing ties, have also been used, designed to act compositely with the cast-in-place topping.

The final stage is bulkheading of the open ends, if any, in a manner similar to that for immersed tubes. Upon completion, flooding of the basin should be carried out, with an allowance of sufficient time for the water to fully penetrate the crushed rock base or under the slab before flotation occurs. The structure can then be winched out of the basin and towed to the site.

An alternative method is to cast the structure on a large barge or barges. The barges selected must be able to withstand the external hydrostatic pressure for their maximum submergence. Standard cargo barges may not be strong enough. If two barges are joined together, the connections must be designed to transfer the moment and shear. Tying across by beams on top and cables underneath, while suitable for moment and axial force, must be supplemented by shear plates. The connections must extend back far enough into the structure of each barge to prevent development of a weak plane. Local loads may require internal posting of the decks.

The barge hulls, if floating during construction, will deflect as loads from concreting occur. Therefore, precast segments should be used, with essentially all of them in place, before jointing. Alternatively, the barges may have been seated on a screeded sand base prior to concrete construction. This will allow concreting to proceed without special provisions for global deflection. However, even though the completed structure may provide shear strength to a barge combination during tow, it will generally not be adequate during launching; hence the previous provisions regarding joining of two barges still apply.

A recent advance has been to fabricate the structure on land behind a bulkhead. The structure is then slid onto a launching barge, using an air-slide or Hillman rollers.

9.3.4.3 Launching

Launching from the barge can take place in several ways. The safest and best is by ballasting down while maintaining a level attitude. Stability during the several stages of ballasting must be verified, since the center of gravity is usually well above the center of buoyancy. The water plane of the barge provides stability during the initial ballasting; then there is a sudden reduction as the barge deck is immersed.

Although the structure itself, still sitting on the barge, may initially provide additional stability, once the barge lists far enough to allow the structure to slide, stability is lost.

When the concrete structure has sufficient water depth to float off, the barge or barges underneath must be separated from the structure by additional ballast. After separation, the barge below has little or no stability and is uncontrollable by itself. One way to solve this problem is to launch at a location where the depth is only a meter or so greater than that required, and the bottom is level. In this case, the barge is overballasted to sit on the bottom until the concrete structure has been towed clear. Then, the barge can be deballasted by pumping from selected compartments and tipped up, using the end remaining in contact with the bottom to provide transverse stability.

A more positive system is to provide steel hollow columns at the corners of the barge, which provides sufficient moment of inertia through the water plane for stability at all stages. Alternatively, an end-O tipping down process can be used for the launching, if the water depth is less than one-third of the barge length. The critical situation here is transverse stability as the deck of the barge goes underwater and the combined barge-structure unit tends to roll sidewise. However, this inherently hazardous operation has been successfully performed if the concrete structure has sufficient width to provide water plane stability during the immersion stages.

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9.3.4.4 Installation

Installation at site requires both horizontal and vertical control. Horizontal control is usually provided by moorings, running to preplaced anchors. Winches are mounted on the unit being placed. Typically, a temporary steel frame is left on top of the preceding unit. It extends up above water and thus furnishes a convenient reference for locating the new segment relative to the preceding one. Except in unusually deep water, this is more practicable than locating the new segment by sophisticated electronic, differential GPS, and underwater sonics.

The new unit can be guided to exact position by tapered wedges or horn guides. Alternatively, it can be lowered to final elevation, but offset 100–150 mm, then, while it is still buoyant, drifted laterally to contact.

9.3.4.5 Leveling Pads

As with precast shells (see Section 9.3.5) leveling pads are pre-installed. Practice differs as to whether two of these seats should be extensions from the previous segment, with one or two independent pads located at the leading end, or three or four independent pads. The first method ensures correct relative elevation, the second reduces stresses in the shell, and eliminates overload on the previous segment and its leveling pads. The determining factors are the foundation capacity and the allowable underwater span length of the new segment. The method for construction of leveling pads has evolved through a long series of projects.

In Norway, with rock outcrops on which the segment must be seated, precast concrete slabs are lowered in exact lateral position, to rest on the seafloor at a slightly lower elevation (−150 mm) than final requirements. The slab will have flexible skirts or, in shallow water, the diver will place sandbags to seal underneath. Grout will be pumped under the slab. Once it has set, an accurate measure will be made of three points on the slab. Precast concrete "shim" slabs will be then placed to raise the top to the exact grade. In The Netherlands, similar precast slabs and shims are used, but here the slab is initially seated on a crushed rock base. For the Prince Edward Island Bridge, a three-legged jacket was set on the dredged rock seafloor. A central tube was accurately positioned and plumbed to vertical. Tremie concrete was then placed to a slightly low position. The surface was cleaned by jet and airlift. After measurement, a precast shim slab was placed. For the Øresund Bridge, the support pads were varied from 1.5 m square to 3.0 m square. They were 150 mm thick. The pads were attached to a three-legged tower (jacket). The jacket was precisely located by differential GPS (DGPS). Jacks in the legs of the tower adjusted the height of the pads. Sand-filled sacks were used to seal underneath and grout was injected. For the Olmstead Dam, leveling pads will be supported on four predriven steel piles whereas for the Braddock Dam they were supported by drilled-in shafts.

Because the leveling pads are "hard points" and furnish support during the subsequent placement of tremie concrete, they may experience greater loads than the remaining piles or shafts. Flat jacks may be fitted, either on the top of the leveling pad or on the underside of the segment, permitting adjustments of load on a Teflon or thick neoprene pad on a stainless steel plate, which will facilitate lateral adjustment by jacking. When final seating has been obtained, they are pumped full of grout. The segment will undergo higher bending during the subsequent infill ballasting and concreting. Both temporary and residual stresses and loads need to be calculated and considered. To overcome this matter of hard points, when the structure is to finally rest on a sand or crushed rock base, which is slightly compressible, the pads may include a layer of wood or

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polyurethane, which will yield during infill and transfer the load to obtain a more uniform distribution.

9.3.4.6 Underfill

Underfill may be sand, concrete, or grout. Sand underflow systems have been used in Denmark and The Netherlands, for which a liquefied sand—water slurry flows down and automatically fills underneath by channeling. Underbase grouts have been used, varying from a cement–sand grout to foamed cement grout, depending on the requirements and

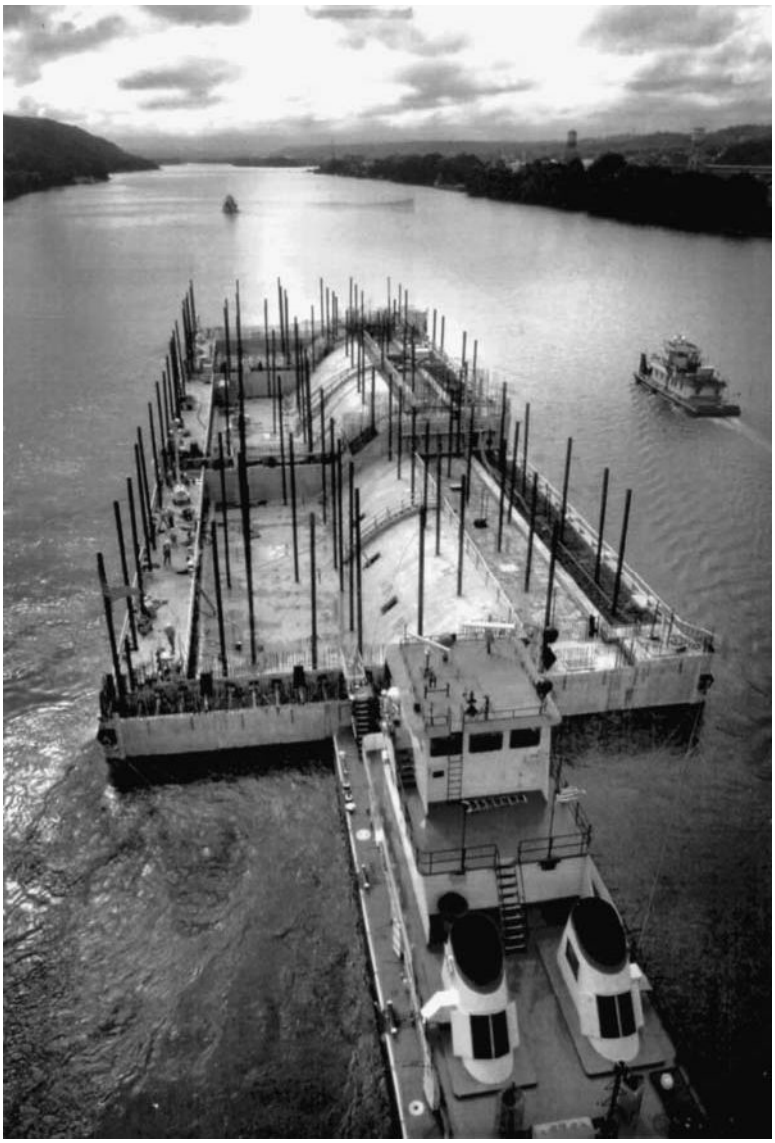

FIGURE 9.11
Segment of Braddock Navigation Dam being towed to site on Monongahela River, Pennsylvania. (Courtesy of U.S. Corps of Engineers.)

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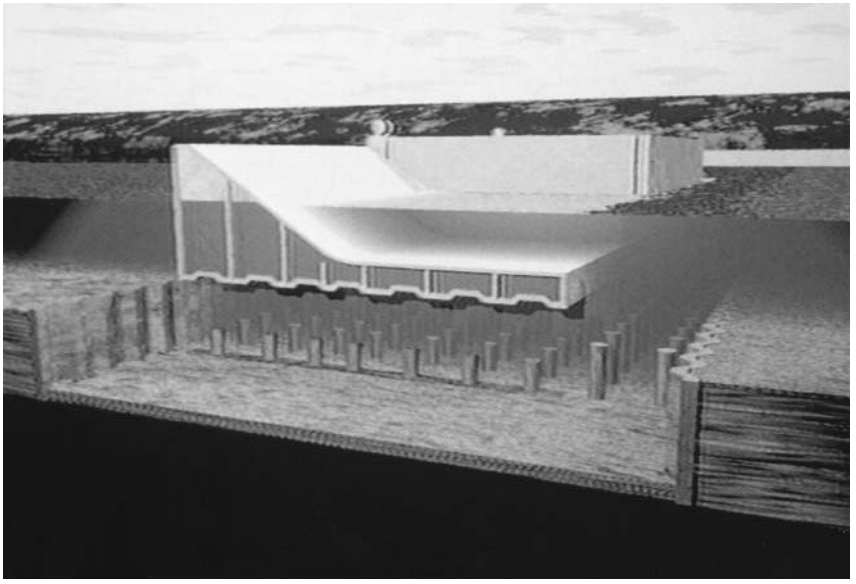

FIGURE 9.12
Ballasting down floating concrete dam segment onto pre-drilled shafts, Braddock Dam, Monongahela River.

the depth of fill required. Heat of hydration must be considered; pure cement grouts or cement-bentonite grouts can get excessively hot, even to the boiling point.

Concrete underbase fill, with aggregates, gives higher strength, high modulus, and reduced heat of hydration (Figure 9.11 through Figure 9.13). Usually small size coarse aggregate, 8–10 mm, is used. Limestone powder and fly ash are means for reducing heat and modulus, minimizing the potential for hard points under the base slab (see Figure 9.14). Underbase fill must be placed under low pressure to avoid excessive pressure, both locally and in global uplift. Gravity feed should be used in preference to pumping, since friction head loss is uncertain and the tendency is to develop too high pressures in order to ensure flow. Grout overflow pipes should be provided. The

FIGURE 9.13
Fully expanded flat jack for leveling dam segment on underwater supports. (Courtesy of Ben C. Gerwick, Inc.)

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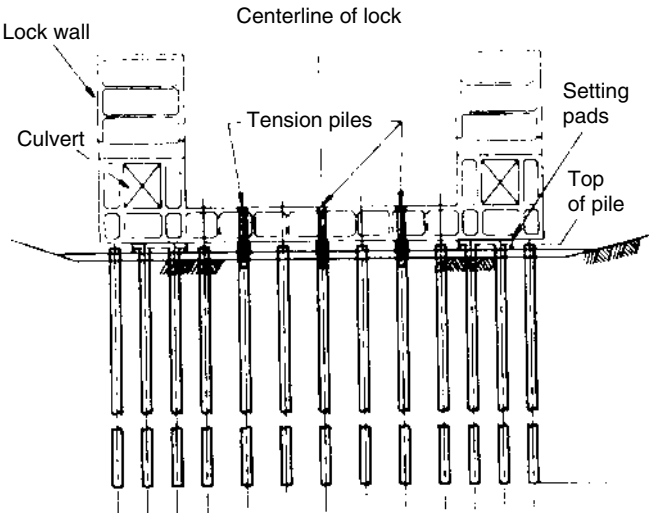

FIGURE 9.14
Planned construction for prefabricated ship canal lock in New Orleans, Louisiana. Tremie concrete underbase fill will join piles to base of float-in lock.

edges and joints must be sealed to prevent flow out to the water. Any current tends to suck the grout or concrete out. Sandbags and grout-filled bags have been used to seal, also expanded metal screens with filter fabric attached. The sides and ends need to be sealed so as to confine the underbase fill and to prevent underflow of the river. Skirts or sheet piles are utilized.

Filling completely underneath a horizontal slab requires use of a self-leveling "flowing" concrete or grout. It also requires a small positive head to overcome friction and to displace water and laitance. An antiwashout admixture is a valuable aid in obtaining the required properties and in minimizing or eliminating bleed. To ensure shear transfer, corrugations or horizontal shear keys can be provided, but the location of discharge and venting holes needs to be properly planned. Vents are required to expel the water and any laitance or foam (see Figure 9.15).

To transfer tension requires that steel bars, strands, or shapes extend down from the base slab of the segment. This, of course, presents difficulties in the prefabrication stage, requiring that the base slab be supported on a raised soffit. It also increases the draft during tow. Care must be taken at all stages to avoid crushing these or worse, pushing them through the base slab, due to accidental grounding on an obstruction. For these

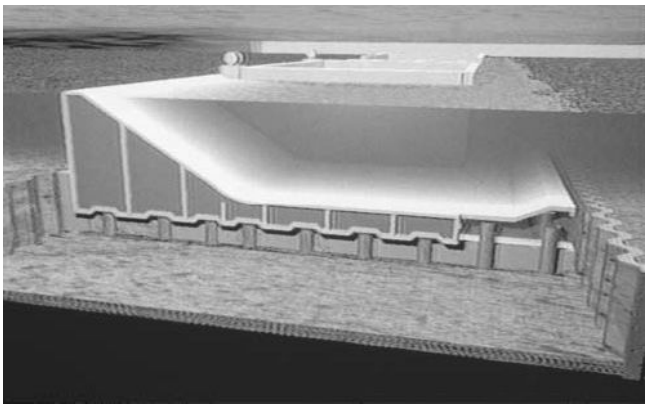

FIGURE 9.15
Grouting under base of prefabricated float-in segment of dam, Braddock Dam, Monongahela River.

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reasons, the use of semiflexible headed strands where structurally adequate should be considered. Alternatively, bars can be pushed down through formed or drilled sleeves and grouted after the segment is set. The planned construction for the Inner Harbor Ship Canal Lock in New Orleans will require the transfer of tension and shear through the tremie concrete underbase fill.

The Braddock Navigation Dam across the Monongahela River at Pittsburgh, Pennsylvania, is intended to control the flow so as to maintain a navigable water depth in the upstream pool during periods of low flow. Space between the banks and the existing lock was very limited, with a transcontinental railroad on the far side and the lock on the near side. Therefore a float-in solution was selected, using two 100 m long prefabricated concrete elements.

After dredging the site, 1.2 m diameter casings were driven and socketed onto the rock below. A template was used in order to maintain very tight tolerance on position and cut-off elevation.

Each prefabricated segment was constructed on a two-level construction basin. After the base raft was fabricated in the upper level, the basin was filled and the raft floated to the deeper level and seated. When fully complete, it was floated once again and moved out into the river, for a 15-mile upstream trip to the outfitting pier. When completely outfitted, it was towed downstream to the site, rotated into position, and ballasted down to seat on leveling pads. This engaged the socketed piles with their shear corrugations and reinforcing dowels for transfer of both tension and compression. After seating, flat jacks were used to adjust grade and the sockets were grouted. Lastly, the underbase gap was filled with tremie concrete. The sides were sealed with previously driven sheet piles. The success of this "in-the-wet" construction has led to its preference for future navigation improvement on the Mississippi and Ohio Rivers.

9.4 Foundations for Overwater Bridge Piers

9.4.1 General

Since Roman times, bridge foundations have been constructed as cribs, or in cofferdams, originally built of timber but today built of steel sheet piles braced with steel frames, or circular walls of reinforced concrete. Even prior to that time, the bridges across the seasonal rivers were founded on wells constructed on sand islands. The wells are sunk as open caissons, originally built of brick, today of reinforced concrete or steel–concrete composite structures. Cofferdam construction is described in Section 9.4.9.

Piles were used by Caesar for his bridges across the Rhine. Today, the timber piles have now evolved to steel and prestressed concrete, in some cases, tubular piles up to 4 m in diameter and over 100 m in length, and driven by very large pile hammers. In other cases, especially in difficult soils, they are drilled in. Installation of tubular piles by vibrators through clay soils can permanently reduce the shear resistance of the soil on the pile walls (skin friction).

Piles can also be constructed as drilled-in shafts, into which steel reinforcing is placed and concrete cast-in place.

In many recent cases, driven piles or cast-in-drilled-shaft (CIDH) piles have been constructed in cofferdams so as to allow the footing block to be constructed below scour level. Two areas of potential problems must be considered and steps taken to prevent their occurrence.

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One is that of liquefaction or weakening of the sediments which occur under the impact of pile hammers or where the piles are sunk by heavy vibrators. The second problem occurs when slurry is employed, using the slurry to prevent caving; the slurry should be kept at or above the external water level, which may require extension of the casing. Cofferdams are addressed under Section 9.4.4.2.

Box caissons have been a relatively recent development, especially suitable as foundations for large piers founded on rock or competent strata. These are prefabricated of steel or reinforced-prestressed concrete and floated into place, where they are submerged onto prepared bases.

Box caissons evolved into large gravity-base offshore concrete platforms in the North Sea, and the technologies developed there can be used for major bridge foundations in such potential future projects as the bridge across the Strait of Gibraltar.

The piers for the bridge to the Isle of Skye in Scotland had to be constructed in soft soils under conditions of swift currents. Figure 9.19 shows the procedure adopted for construction of the pile-supported caisson.

The most recent development has been to found box caissons or gravity-base structures on thick mats of stone ballast, in turn supported on multiple driven piles of steel or concrete but with no fixed connection. This was employed on the Rion-Antirion Bridge in Greece.

The use of prefabricated concrete shells, seated over large-diameter piling and filled with high-quality underwater concrete, is now replacing the conventional cofferdam for footing block construction, in many cases, because of speed of construction, feasibility in open-water structures, quality, and economy.

9.4.2 Open Caissons

In principle, these are an extension by modern materials and methods of the "wells" of India, first used for river piers in 1500 BC. The walls of the pier are built up, adding weight as the caisson sinks by progressive excavation inside. The inside is kept filled with water to overcome the external hydrostatic head. Internal bracing, as required, is built into the structure integrally with the walls.

When the founding level is reached, it is sealed by underwater concrete. Then it can be dewatered and the pier footing block and shaft constructed in the dry.

The first modern open caissons were constructed for bridges in New York, across the East, Harlem, and Hudson Rivers. Steel shells were prefabricated and filled progressively with concrete. A structural steel cutting edge was designed to prevent local and global distortion as boulders and obstructions were encountered. The concept has been extended to the Lower Mississippi River crossings.

To enable the caisson to be initially floated out and seated into the soft sediments, the internal cross-framing divides the caisson into cells. False bottoms of timber and more recently, steel, are fitted. As the caisson penetrates, soil-pressure adds to hydrostatic pressure. The false bottoms are then removed, one by one.

It is desirable to equalize the pressure on each false bottom before it is removed to prevent a sudden drop of the caisson and to prevent injury to the personnel involved. This can be attained by first opening small-diameter penetrating pipes or by excavation beneath the false bottom by airlift. When the caisson reaches founding levels, boulders or irregular rock are excavated underwater, often by divers, and backfilled by underwater concrete.

Unfortunately, several of the early open caissons tipped dangerously during the early stages of penetration, due to unequal soil resistance and the process of removing the false

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bottoms and subsequent dredging. One principle emerged, that correction of tipping must be carried out in small steps or it may result in worse tipping in the other direction.

On the San Francisco-Oakland Bay Bridge, instead of false bottoms, steel domes were installed on top of each cell and air pressure introduced in individual cells so as to maintain better control of the attitude of the caisson. Even this did not prevent serious tipping of the central anchorage caisson, which, however, was finally overcome by judicious use of external excavation and backfill plus the selective use of internal air. This scheme was also employed on the first Tagus River Bridge, in Portugal, this time successfully.

Double shells have been adopted for recent open caissons, such as the Mackinac Strait Bridge in Michigan and some new bridges across the Mississippi. While expensive in their use of steel, they give excellent control during installation.

Open caissons of reinforced concrete were used on the First Tacoma Narrows Bridge and for the bridge across the Mississippi in New Orleans, and then the Second Carquinez Bridge and the Second Tacoma Narrows Bridge. The walls of the caisson have been extended as the caisson has penetrated, using either precast concrete or steel sheets, in conjunction with cast-in-place reinforced concrete.

Mooring of the caissons in a swift river or tidal currents has proven to be a major consideration. "Pens," consisting of pile-supported dolphins are often used, incorporating "master-piles," carefully aligned vertically, to support the caisson as it penetrates. In deeper water and in more severe currents, anchored moorings are installed. To restrain the caisson and not introduce tipping forces, the center of mooring force at the caisson must be kept at or near the center of rotation of the caisson. Since this is constantly changing due to increasing depth and changing weight distribution, this point of reaction must be moved upward in increments.

For the early caissons, the lines were slacked one at a time, and then the connecting shackle was unbolted and re-connected to a fitting higher up. All this was done by a diver, a time-consuming and dangerous task. On the Tagus River Bridge in Portugal, multiple trip lines were connected, enabling the lower pin to be pulled from on top. Successive moves required multiple trip lines. At Tagus, these were colored so as to prevent confusion.

On the caissons for the Second Tacoma Narrows Bridge, with a minimum of six attachment points for 32 mooring lines, the scheme of trip wires obviously became impracticable. A sliding beam was used instead, so that each line need be connected only once. When it was desired to move a pair of lines up, the lines was slacked and the beam pulled upward through its guiding brackets.

At Tacoma, driven plate anchors were employed, since the dense sands and gravels of the seafloor were unsuitable for conventional drag embedment anchors, while the dense gravel seafloor, deep water and the very high tidal currents (up to 9 K) made the use of gravity anchors such as concrete blocks impracticable.

9.4.3 Pneumatic Caissons

First developed in France, pneumatic caissons were later used on the Eads Bridge across the Mississippi and subsequently on the Brooklyn Bridge.

A multi-cell caisson is constructed with a heavy concrete slab 4 or 5 m above the bottom of the external cutting edges. This chamber is filled with compressed air at a pressure to match the external hydrostatic head at that depth. Two shafts with air locks are installed, one for personnel, another for removal of excavated material (and later for concrete). The compressed air keeps the water out, although some pumping is still needed.

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Men, entering through the air lock, excavate the soils. The caisson meanwhile is loaded by concrete extensions of the walls and sinks slowly down. The process, and its accompanying toll on health and life, is told vividly by David McCullogh in his book, *The Great Bridge*, describing the construction of the Brooklyn Bridge.

Although largely abandoned today in favor of open and box caissons, the Japanese have continued to use it, replacing manpower by robotic equipment. The piers for the Rainbow Suspension Bridge in Tokyo and the Bai Chai Bridge in Vietnam are modern examples of Japanese robotic technology (Figure 9.18).

9.4.4 Gravity-Base Caissons (Box Caissons)

Box caissons are fabricated of concrete or steel, with a closed bottom, so that they may achieve full or partial flotation during transport to the site. This buoyancy also aids during seating, being progressively overcome by ballasting. Steel box caissons were used on the Honshu Shikoko Bridges. Concrete box caissons were employed on the Great Belt Western Bridge and again on the Eastern Bridge. Crane barges provided additional lift and guidance during transport to the individual pier sites. Large caissons, for which the shells exceed available lifting capacity, e.g., more than 3000–8000 tn., may be constructed in a basin or graving dock. When complete, the dock is flooded and the caisson floated. The dike or gate is opened and the structure floated to its location where it is positioned, seated down by ballasting onto its prepared base, and permanently weighted down by underwater concrete and stone or sand fill.

The concrete box caissons for the pylon piers of the Øresund Bridge in Sweden were too heavy to float by themselves. They were fabricated in a large graving dock. Then the dock was flooded and two large dewatering barges floated in and positioned, one on each side. After dewatering, multiple jacking rods were attached from the side of the barge to the side of the box caisson and stressed. The barges were then ballasted so that they would retain a balanced transverse attitude. The dock was again flooded and the combined structure plus barges floated out and towed to the site, the self-buoyancy of the box caisson being supplemented by that of the two barges. On arrival and positioning, the caisson was lowered by the jacking rods until it made contact with the prepared base.

Stability is maintained hydrodynamically by always keeping a positive metacentric height, $\overline{GM}$, supplemented if necessary by attached buoyancy tanks or by support from external vessels. Particular care has to be taken at abrupt changes in water plane, e.g., at the transition from a large footing block to a shaft. The foundation for the caisson must be level to a close tolerance, and sufficiently dense so as not to settle differentially, not to liquefy due to rocking of the caisson under wave action, and not to liquefy under earthquake.

Initially the unsuitable soils are removed by dredging. Usually a clamshell, hydraulic backhoe, or ladder dredge is used. Soft sediments remaining must be removed by suction, e.g., by use of an airlift. The surface must be sufficiently clean that a layer of weak clay does not exist across the foundation, but limited amounts of soft silts may be left in isolated pockets. If the soil is sand, densification may be undertaken using heavy vibratory probes. Sand or small stones may be fed down during the operation.

A stone bed is then placed. This bed is constructed in layers, each about 1.0 m deep, which can then be compacted by a large and powerful vibratory screed. Finally, the surface is screeded to exact elevation, usually ± 20 mm, using a 300 mm layer of 12–20 mm stone. The screed is usually supported on preplaced rail beams. A traveling hopper is fed by a tremie pipe or pipes from the surface. The stone is then distributed laterally and struck off to grade by a blade on the trailing edge of the hopper. In some recent cases, the distribution and striking off is accomplished by use of a continuous flight auger, riding horizontally on the rail beams.

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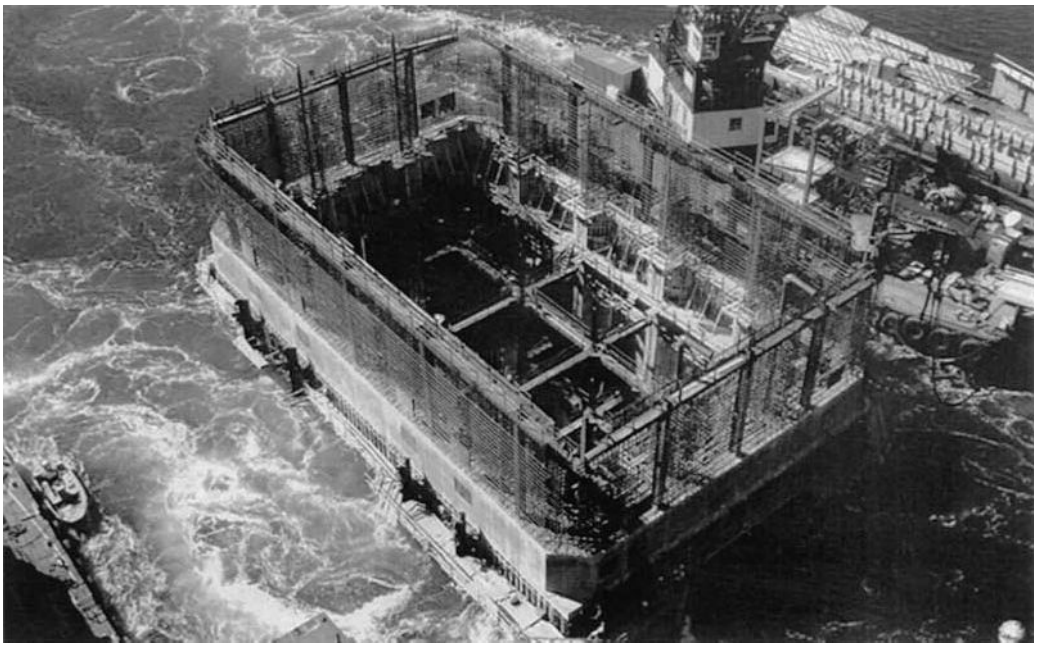

FIGURE 9.16
Second Tacoma Narrows Bridge. Caisson in nine Knot Tidal Current. (Courtesy of TNC Construction.)

For the Honshu-Shikoku Bridges, the foundation was excavated to bedrock or hardpan, and then leveled carefully by grinding to ensure proper seating of the peripheral edges of the caisson. Compressible seals were used under the outside bearing surfaces.

If there has been any appreciable delay after final screeding of gravel, then prior to positioning the caisson, a bottom survey should be made, either by acoustic depth finder from the surface or by side-scan sonar to verify that there are no obstructions or sedimentation (Figure 9.16 through Figure 9.18). Zebra mussels can grow very rapidly on the underside of a caisson and prevent seating.

The caisson is now floated into position (see Figure 9.19). Although GPS is used for general control, a laser range is very valuable for initial positioning, due to the fact that the

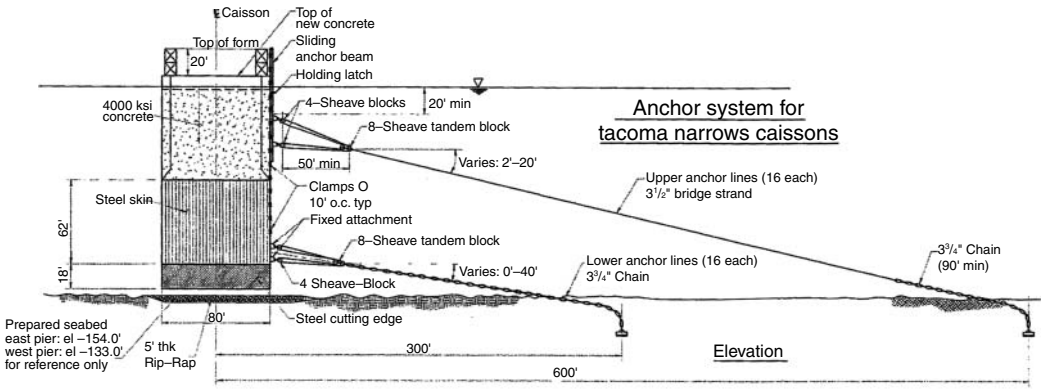

FIGURE 9.17
Sliding beam mooring system for caisson, Second Tacoma Narrows Bridge. (Courtesy of General Construction.)

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FIGURE 9.18
Robotic excavation in Rainbow Bridge pneumatic caisson, Tokyo. (Courtesy of Shiraishi Foundation Company.)

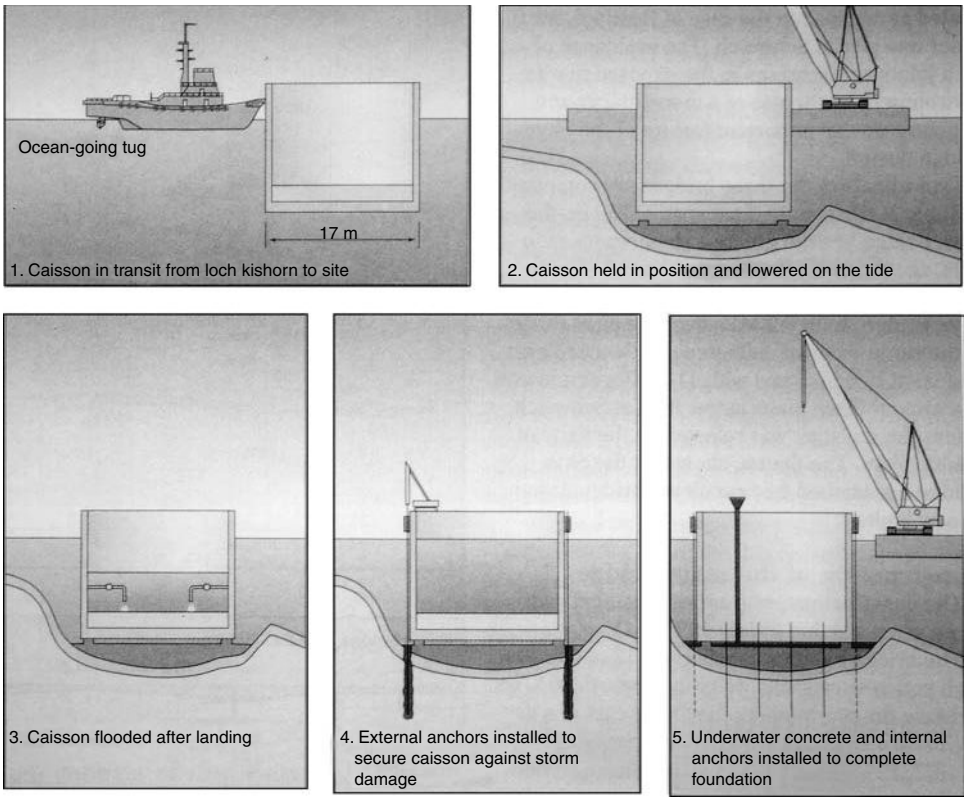

FIGURE 9.19
Installation of caisson for bridge to Isle of Skye, Scotland, UK. (Courtesy of Miller Civil Engineering Ltd and Dyckerh off-Widmann AG.)

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amount of correction needed and the rate is readily apparent to those controlling the position. Theodolites are used for precise control.

Moorings are used to effect final positioning to the required tolerances. In order to facilitate corrections, lines should be led out orthogonally, each subject to separate control by a winch. This typically requires six to eight lines. Attempts to reduce the number by running lines at 45° are usually counterproductive, since taking in on one line moves the structure on two axes and rotates it. Preset mooring buoys are installed.

Once the caisson has entered the area, lines are run from winches mounted on the caisson, initially leading down to fairleads attached to the sides at the approximate center of rotation, and then led out to the buoys. The caisson is then ballasted down onto its prepared base. The rate of ballasting down over the last 2 m should be slowed, in order to let the trapped water escape. During this final set down the current will accelerate underneath the base, tending to pull the caisson down while at the same time scouring underneath. In fact, in very loose sands, the bottom may drop progressively as the caisson is lowered. This may require prestabilization of the foundation by willow mats with stone or with their modern replacement, filter fabric plus stone.

Immediately after seating, position, orientation, elevation, and level are checked to ensure they are within tolerance. If one parameter slightly exceeds tolerance, an inquiry should be made to the design engineer, to see if the caisson can be accepted as is with perhaps minor changes to the superstructure, since the act of deballasting and repositioning often causes damage to the prepared base.

Large box caissons for bridge piers that are being floated into position and then submerged are usually of prismatic shapes and hence, have a substantial water plane through the whole seating process, which ensures stability at all stages. Conversely, those that have a radical change in water plane, e.g., where the top of the footing block is immersed and only a smaller shaft extends through the water plane, usually require some external means of providing stability. One reliable means is the attachment of vertical buoyancy columns or tanks, rigidly connected to the caissons. Post-tensioning the column from the top, through a steel pipe sleeve, and anchoring in the caisson is a useful solution because it permits subsequent release from above water. Another means is to supply stability from a large crane barge, since a lift on long slings can create a righting moment $Pl \sin q$, where P, the lift force; l, the length of the slings; and q, the angle of heel; such a lift effectively raises the $\overline{GM}$ by $P \times 1 \div$ delta, where "delta" is the displacement.

Once the caisson is in position and fully ballasted, grouting underneath should be performed promptly. It is important to prevent runout or leaching of the grout from under the edge of the caisson, so the caisson base is usually provided with short skirts to retain the grout. A grout with low heat, antibleed, and antiwashout characteristics is desirable. The grout should have low modulus, hence low strength, so that local deformations can be accommodated. Vents must be provided in the base of the caisson to relieve pressure, and the grout injection pressure carefully controlled to avoid raising the caisson. During this operation, the elevations of the corners should be continuously monitored.

The two foundation piers for the Ma Wan Tower of the Tsing Ma Bridge in Hong Kong were prefabricated box caissons, cast on a large submersible barge, of the type used for ocean transport of dredges and jack-up drilling rigs (Figure 9.20). The barge, with the two prefabricated box caissons aboard, was towed to a site where the water depth was just adequate for the launching operation. The bottom was sand and approximately level. The site was carefully surveyed, both acoustically and by underwater video, to ensure that there were no boulders or obstructions. Both caissons were securely chained to the barge in both transverse and longitudinal directions to prevent a premature launching. The barge was then ballasted down to the point that the caissons were close to being buoyant. The barge was then given a 3° trim down-by-the-stern, the rear caisson cut

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FIGURE 9.20
Tsing Ma Suspension Bridge, Hong Kong, China, under construction. Piers supported on box caissons. (Courtesy of Hong Kong Bridge Authority).

loose, so that it slid off the stern. The barge was now freed of half its load and began to float up. This was the most critical point for stability. In this case, the submersible barge performed according to calculations and remained stable.

The process was then repeated and the second caisson was launched. The two were now floated to the site. It was necessary to move into position at high tide to have adequate underkeel clearance. Each caisson was moored and ballasted down onto seating pads, which had been built to exact grade. In this case, the "prepared" foundations consisted of bedrock that had been drilled and blasted, then excavated. Although great care had been taken in the blasting operation—close spacing, minimum charges, and minimum overdepth blasting—the rock had many natural and man-made fractures. Plastic tubes were inserted in the prefabricated footing block so that when they were later concreted, drilling and grouting of the fractures could more easily be carried out. The bedrock was thoroughly cleaned by airlift. Sealing of the edges was difficult, requiring diver packing by grout-filled bags. Then tremie concrete was placed to fill the box. To prevent runout underneath, a stiff mix was used. Sandbags were placed to fill the gaps at the edges. The box had a sloping half-roof. Due to the stiff concrete, as well as bleed and laitance, a void arose under the roof. This was filled by supplemental grouting. Use of a concrete mix containing an antiwashout viscosity admixture and higher slump would probably have eliminated the void.

In the case of the earlier Honshu-Shikoku bridges across the Inland Sea, the steel box caissons had preplaced grout pipes installed. Once the box caisson had been founded, it was filled with coarse aggregate, from which all fines had been screened out. An algae inhibitor was added to the water to prevent the growth of slime on the aggregate. Then, grout was pumped through pipes, starting with those discharging at the lower levels and progressing upward. Indicator tubes were used to monitor the rise of the grout.

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FIGURE 9.21
Steel caisson for main pier of Akashi Strait Bridge, Japan. (Courtesy of Kajima Engineering and Construction Co.)

However, subsequent corings indicated significant zones that remained ungrouted. It is believed that these were due to trapping of bleed water under the coarse aggregate. On the most recent and largest of the Honshu-Shikoku bridges, that across the Akashi Strait, tremie concrete was used in lieu of grout-intruded aggregate (Figure 9.21). A special low

FIGURE 9.22
Concrete caisson for Great Belt Eastern Bridge, Denmark. Note attached cofferdam. (Courtesy of Great Belt Link Ltd.)

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heat of hydration mix was employed, incorporating limestone powder, fly ash, and blast furnace slag.

Scour protection must be placed as soon as possible to prevent erosive scour of the prepared bed. Shortly after installation, one bridge pier box caisson on the Øresund Bridge, between Denmark and Sweden, was seriously undermined by scour from the joint action of waves and current. Extensive grouting was required to restore the foundation.

The process described earlier for grouting under the base to fill completely any gap between it and the stone bed was used on the caisson piers for the Eastern Bridge across the Great Belt of Denmark. The caisson bases were configured with three or four integral bearing pads at the corners, typically 300–500 mm thick, to ensure that a proper gap was maintained for grouting (see Figure 9.22). Alternatively, bearing pads may be preset. For the forty-four caisson piers of the Confederation Bridge in eastern Canada, three bearing pads were preconstructed by placing tremie concrete to an exact grade, using a small bottom-founded jacket. Then, after the caisson was set on them, the bearing space was filled with a flowable tremie concrete containing silica fume to ensure cohesiveness (Figure 9.23). However, on the Western Bridge across the Great Belt, the 66 box caissons were set directly on the stone bed. This concept requires that the base be adequately reinforced to span any gap between high points (hard points) and that the screeding be carried out to great accuracy (Figure 9.24 and Figure 9.25).

FIGURE 9.23
Eight-thousand-tn. precast concrete piers for Confederation Bridge, Canada. (Courtesy of Stanley Engineers, Inc.)

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FIGURE 9.24
Prefabricated bridge pier for Great Belt Western Bridge. (Courtesy of Great Belt Link Ltd.)

On the Second Severn Bridge, prefabricated box caissons with open bottoms were loaded by transporters onto barges, then towed to the site at high slack water and picked by a pair of heavy-duty, high-capacity cranes supported on a large jack-up barge (see Figure 9.26). After setting, they were filled with tremie concrete. Initially, difficulty was experienced in sealing the caisson edges against the effect of the very strong tidal currents, which led to extensive washout. Eventually, a solution was found by placing closely meshed screens around the perimeter and using a thixotropic antiwashout admixture in the tremie concrete.

A similar problem had occurred many years earlier on the Columbia River Bridge at Astoria. There, after the box caisson shells had been placed, the river currents augmented by the outgoing tide scoured the sand from under the base. When the tremie concrete was placed, it flowed out and was washed away by the current. This problem was only remedied by placing a filter course and scour protection around the pier shell, placing stone above the bottom of the shell, and following with extensive underbase grouting to fill the voids that had been formed during the initial concrete placement.

The foundation piers for the Rion-Antirion Bridge in Greece had to cope with deep water, soft soils, and a major active earthquake fault in mid-channel. The piers were constructed as gravity-base caissons, founded in the soft clays, which were improved by closely spaced driven steel tubular piles to provide adequate bearing and shear

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FIGURE 9.25
Crane barge setting 7000-tn. prefabricated concrete pier, Great Belt Western Bridge, Denmark. (Courtesy of Great Belt Link Ltd.)

resistance (Figure 9.27 through Figure 9.30). The piles were steel tubular piles 2.0 m diameter with 20 mm thick walls driven 30 m into the seafloor. They were on a 7× 7 m^2 grid spacing. Approximately two hundred piles per pier were required to resist a shear force of 500 mm.

The weak seabed sediments had first been dredged, and the piles were cut off at the new mud line which in some piers was 65 m deep. Then a 2.8 m layer of crushed rock was placed and screeded. The gravity base caissons were now founded on this gravel stratum.

FIGURE 9.26
Concrete caissons for piers of Second Severn Bridge, England. (Courtesy of Halcrow-SEEE.)

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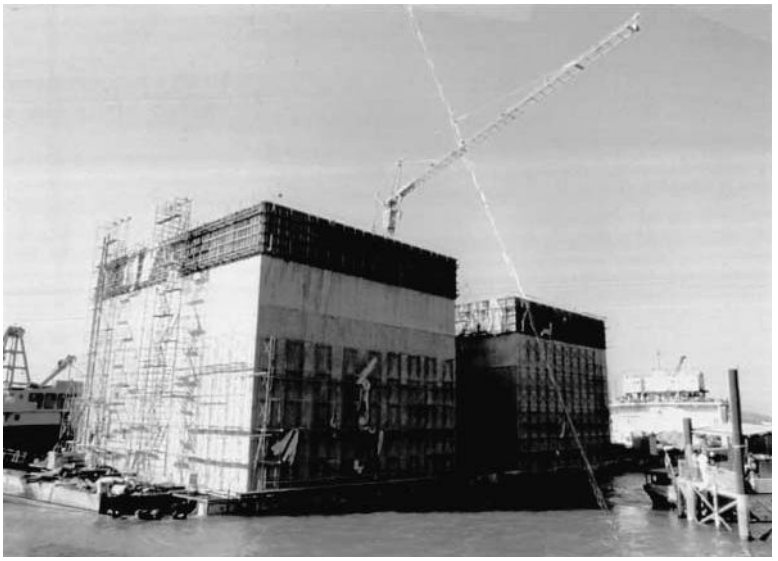

FIGURE 9.27
Fabricating caissons for the main pylon pier of the Tsing Ma Suspension Bridge, Hong Kong, China. (Courtesy of Hong Kong Bridge Authorities.)

They were designed so as to permit sliding and rocking under extreme seismic forces. Thus no moments will be transferred to the piles. The piles and gravel mat covered the footprint of the pier and a 10 m annulus beyond.

This same concept is being applied by the Italians for support of the Venice storm surge barrier. Precast concrete piles will be employed to stiffen the soil and to pin the multiple lenses of soils together.

Where heavy-lifting equipment can be made available, the complete pier is prefabricated, either in a special construction basin or on a quay (bulkhead wharf) of adequate capacity. Then, it is lifted by crane barge or catamaran and carried to the site, to be set in place on the prepared stone bed. For the Great Belt Western Bridge, the concrete piers were prefabricated on a quay, slid progressively forward by hydraulic rams on concrete tracks, and lifted by an 8000-ton capacity crane barge. A similar process was employed on the

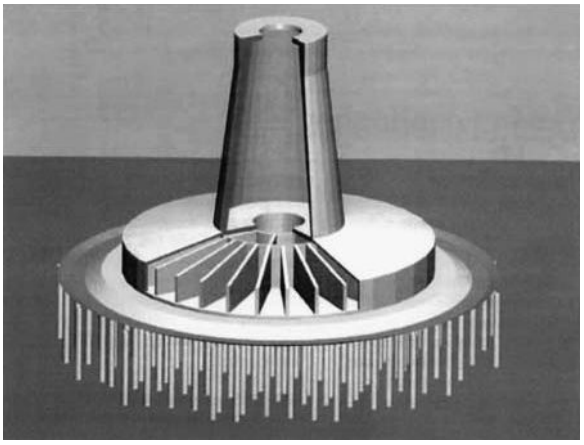

FIGURE 9.28
Foundation concept for Rion-Antirion Bridge, Greece. (Courtesy of Geodynamique & Structure.)

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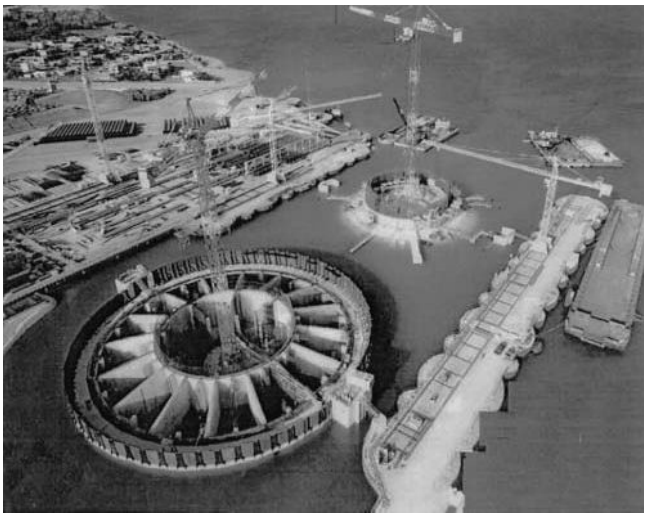

FIGURE 9.29
Caissons being fabricated for Rion-Antirion Bridge, (Courtesy of Geodynamique & Structure.)

piers for the Confederation Bridge, although in that case, the prefabricated piers were transported by barge. Heavier segments may be supported by a combination of crane lift and buoyancy. This was the scheme adopted for the approach piers of the Øresund Bridge.

As noted earlier, many box caissons are configured so as to perform as a submerged footing block, supporting the lower portion of a shaft, which then extends through the

FIGURE 9.30
Placing steel and concrete footing block overpiles. San Francisco-Oakland Bay Bridge skyway. (Courtesy of KFM.)

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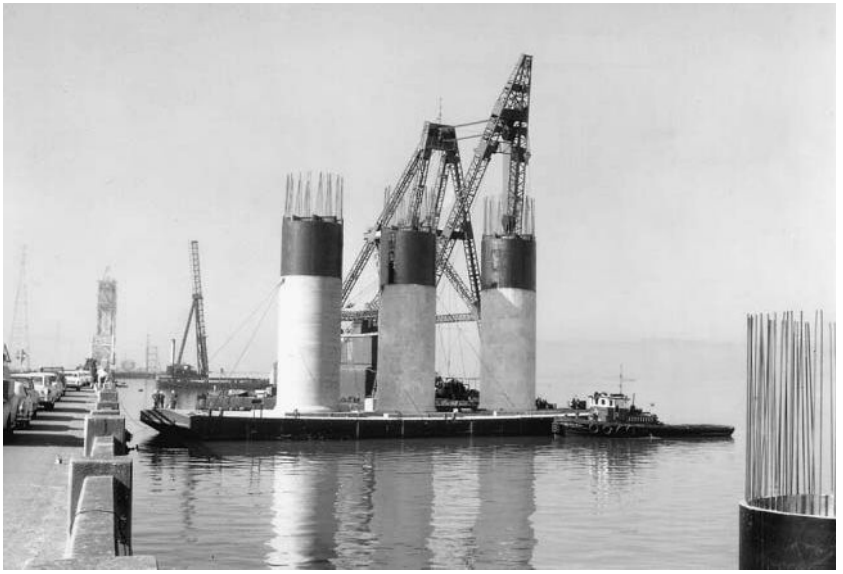

FIGURE 9.31
Prefabricated concrete shafts for bridge piers, San Mateo-Hayward Bridge, California.

water plane. If the shaft is not mounted until after the footing block has been fixed on the base, then there are two solutions. In the first, a short box cofferdam of steel sheet piles is temporarily affixed to the top of the footing block caisson, to permit dewatering and construction of the shaft in the dry. In the second solution, the shaft is precast as a hollow shell. It is then set over projecting reinforcing bars, sealed to the top of the footing block by gaskets, and filled with tremie concrete. The projecting bars may have mechanical heads in order to gain positive anchorage (Figure 9.31).

There are many variations for making a full-moment connection between a precast shaft and the footing block. For the Prince Edward Island piers, they used the mortise-and-tendon concept, with the hollow shaft fitting closely over a 1-m-high tendon. The annular space between was grouted. On the Richmond-San Rafael Bridge, and again on the San Mateo-Hayward Bridge, a ring of heavy reinforcing bars extended downward 2 m into a recess on the footing block. The void and the lower part of the shaft were then filled with tremie concrete.

The early success with the underwater precast shaft to footing connections has led to the current practice of making the above-water shafts by means of precast segments, each joined to the others by setting each upper hollow shaft segment over reinforcing bars projecting from the previous segment. The hollow in the shaft segments does not have to be of constant cross section for full height, but can be principally a pocket of sufficient depth to accommodate the projecting bars, with a smaller vertical hole for placing the tremie concrete. This concrete should use 10 mm maximum size coarse aggregate, a slag-cement, or fly ash to minimize thermal expansion, and silica fume, the latter to eliminate bleed, gain strength rapidly, and enhance bond.

9.4.5 Pile-Supported Box Caissons

In this concept, the prefabricated concrete shell is seated over pre-installed piling. The box is then filled with tremie concrete, locking the box to the piles (Figure 9.32). In many

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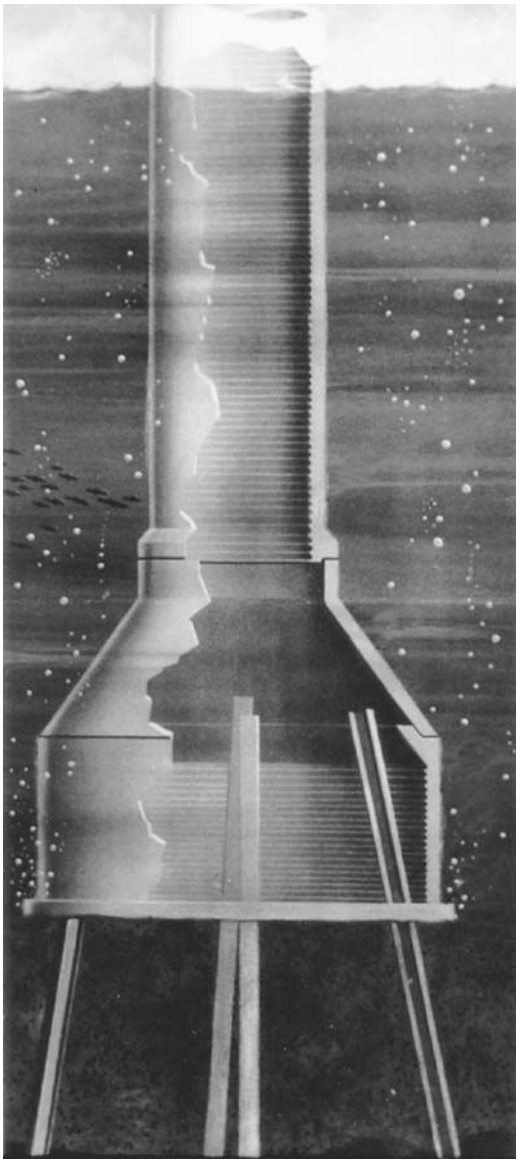

FIGURE 9.32
Precast concrete bell piers on pre-driven steel H-piles, Richmond-San Rafael Bridge, California.

applications, the box has no bottom slab, so must be transported on a barge, to be set by a crane barge, or must be transported while suspended from a crane barge or catamaran, then lowered into position. The piles are best driven through a template, such as a grid of steel beams or a precast concrete flat slab with appropriate holes, in order to ensure that they are correctly positioned and particularly to ensure that the pile heads all lie within the periphery of the box (Figure 9.33).

On the Richmond-San Rafael Bridge, the 300-mm-thick precast concrete bottom slab had multiple H-shaped slots, correctly positioned and oriented to accept vertical and battered piles. The 25 mm wide annuli were sealed by a rope grommet forced down into the annulus by a diver, although slotted neoprene pads might be used today. Grout was then placed in the annuli. This locked the slab to the piles, and thus also furnished

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FIGURE 9.33
Precast concrete base shells, San Mateo-Hayward Bridge, California. Note corrugations for shear transfer to infill concrete. Concept.

support to the initial lift of tremie concrete. On other bridges, such as the Narragansett Bay Bridge in Rhode Island, the bottom "slab" was a heavy mat of timbers and steel beams, similarly serving both as a template and temporary support.

For a recent bridge across the Kennebec River in Maine, large-diameter drilled shafts were first installed through a template. These vertical shafts have permanent steel shell casings. The drilled shaft-tubular piles are cut off about 1.5 m above the design bottom of the footing block. Then, precast boxes were floated in. Each box had four externally fitted spuds (1 m diameter steel tubulars) which were dropped to hold the box in position while it was being lowered over the piles. The base slab of the box has large circular sockets, which fit over the tubular piles. When the box was at proper elevation, it was fixed to the spuds. The annuli around the piles were sealed and grouted with grout containing silica fume. Then after the grout had gained strength, a tremie concrete seal course was placed. The footing block, while floating as a prefabricated box, had a temporary cofferdam attached on top, which not only gave stability during immersion but enabled dewatering after the tremie concrete seal course was placed to complete the connection of the caisson footing block to the drilled shafts.

On the first Benicia-Martinez Bridge, prefabricated box caissons were floated into place and moored. The box had sockets for the 2-m-diameter driven and drilled piles. These sockets were covered during the flotation mode. One by one they were uncovered and the socket used as a template for setting and drilling of the piles to bedrock. After four piles had been driven, the box was ballasted to grade and then grouted to the piles to fix the elevation of the box. Then, the remaining piles were driven. All piles were then socketed into the shale bedrock. A reinforcing steel cage was placed and whole pile and socket filled with tremie concrete. The annuli between the piles and the base slab of the box were then grouted and the pockets in the footing block were filled with concrete to transfer shear.

This project, completed in the 1960–1970 decade, has performed well. Recently, however, it was reanalyzed for a much larger earthquake using up-to-date seismic input projection and analyses. As a result, additional driven-and-drilled tubular piles have been installed, the remaining voids in the box filled with concrete, and the entire

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FIGURE 9.34
Float-in precast concrete footing for Third Carquinez Bridge, California. (Courtesy of Ben C. Gerwick, Inc.)

box will be post-tensioned in the two orthogonal directions to lock the piles to the footing block.

On the Third Carquinez Bridge, the shells for the footing blocks were precast on a barge, then launched and floated over the pre-driven steel casings (Figure 9.34). The annuli are sealed by grout and a thin tremie concrete seal placed. Heavy reinforcing steel was placed in the dewatered box and the footing block completed in the dry. A similar concept is being used on the Second Benicia-Martinez Bridge, except that the footing block will be post-tensioned on both axes. On the Hangzhou Bay Bridge, a precast concrete slab was set over the piles and sealed by grout. On this, a precast concrete ring wall was set and the footing block was completed by cast-in-place concrete.

9.4.6 Large-Diameter Tubular Piles

9.4.6.1 Steel Tubular Piles

This concept utilizes the piles not only in axial loading but also in lateral bending. Typical diameters are 2.0, 2.5, and 3.0 m, and wall thickness from 45 to 75 mm. The large diameter gives the necessary stiffness to restrain the displacement or drift under lateral loads. The axial load capacity under compression is developed by skin friction and end bearing. The axial tension capacity is developed by skin friction and the weight of the pile and its plug of concrete and soil.

The steel tubular pile is driven, utilizing offshore platform practice and technology, to a sufficient depth to meet the criteria for lateral and tension loads. The piles often need to be driven deeper than needed for compression loads alone; this requires special equipment and/or procedures in order to obtain the needed penetration to resist tension and/or lateral loads. Where the tubular piles are exposed to high currents, vortex shedding needs to be considered. External strakes and/or filling with sand may prevent this.

In the case where drilled shafts are to be installed, a steel tubular casing is first driven deeply enough to seal off the water and any loose sands or very soft clays. Experience teaches that it is essential to establish a full seal in competent material in order to prevent run-in during the drilling operations. Although most installations of large diameter tubular piles and drilled shafts are vertical, moderate batter or inclinations may be employed: the standard for offshore platforms has long been a batter of one horizontal to six vertical.

In installing offshore piles, their positioning is obtained by threading through sleeves in the jacket, whereas, for bridge piers, jackets are not normally employed for a variety of reasons including the generally limited depth of water in which bridge piers are usually

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founded. Nevertheless, pile location and verticality or inclination are even more important for bridge piers than for offshore platforms. This is particularly true for large-diameter tubular piles.

Temporary templates are usually employed for the purpose of accurate initial setting where the bottom has a slope or where inclined batter piles are to be installed and special procedures must be used, since once a pile has penetrated two or three diameters, it is impossible to correct it if it starts to drift off. Further, a pile which is drifting out of plumb or position can move the template laterally. Therefore, in such situations, it is best to partially drive one or more vertical piles in the cluster first, to secure the template location. Then, the remaining piles may be driven. Finally, the four initial piles can be spliced and driven. The sequence should be selected so that after one batter pile is driven, the opposing batter pile is driven next. The template should ideally provide support at two elevations.

Pile hammer selection and use follow closely the practice on offshore platforms. To obtain penetration in dense soils, the hammer should if practicable be sized to do this by driving alone. This may mean the provision of very large pile hammers along with the crane equipment necessary to handle them. However, it may not always be feasible to obtain the design penetration by driving alone. Jetting, clean-out, and drilling ahead are often required.

Unlike offshore platform installations, the need to develop tension and lateral capacity and to minimize settlements may lead to severe restrictions on installation methods. Thus, external jetting may be prohibited in order to maintain the effective lateral support. Cleaning out helps in obtaining penetration by reducing the internal skin friction, but this in turn reduces the effective end plugging. This then allows more soil to fill the tip until end plugging is regained, with a consequent reduction in the effective soil friction on the sides of the pile. Internal jetting to break up the plug has a similar but lesser effect. In extreme cases, the clean-out and jetting processes may allow a temporary void to form beneath the pile, with a consequential permanent loosening of the lateral soil confinement.

Soils near and below the tip which have been loosened by the process described above allow greater settlement under loads than where the penetration has been obtained by driving alone. Thus the common bridge design practice of requiring clean out and installation of a concrete plug may be counterproductive. This practice is not used for offshore platform piling. The designs for several recent bridges provide for clean out and concreting of only the upper 20–30 m.

Where piles are jetted to aid penetration in dense sands, much of the original capacity of the sands can be restored by continued driving after the jetting has stopped. Even 50–200 blows of the hammer may fail to develop any significant additional penetration but will compact and densify the sands, internally and externally. Some archaic piling specifications in bridge practice attempted to attain this effect by requiring a penetration after the jetting ceases of 1–2 m, but with large-diameter piling, this is often impracticable to achieve.

One method that has been successfully applied in order to limit settlements under high loads is to clean out the pile to within a few meters of the tip (typically one diameter) and then place a concrete plug of sufficient length to develop load transfer in bond. Beneath the plug, flat jacks or similar are installed. After the plug has gained strength, the jacks are pressure-grouted. A variant on the process is just to grout the space without flat jacks, controlling the pressure so as not to cause run-out around the tip or fracture the formation. The body of the pile is then filled with sand or concrete.

When driven tubular piles require a socket beneath their tip in order to achieve the required capacities, then obviously the pile will have to be cleaned out to its tip. Cleaning out by mechanical means such as grab will cause less disturbance of the external soils than a jet and airlift, at least over the last several diameters. In all cases of clean-out the pile

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must be kept full of water to near the external water level. If the water inside is lowered below the outside water level (e.g., by airlift), a sudden inflow under the tip may ensue; if raised more than a few meters above the outside water (e.g., by jet water), a sudden outrush may occur, causing piping along the side of the pile.

This regulation of pressures can be provided by pumping in continuously as an airlift is operated, or pumping out continuously as the jets are operated. Where design considerations permit, holes in the pile wall below water may provide equalization. Experience in bridge foundations has shown many cases where it proved difficult to maintain this head while operating an airlift. Consequently, the use of the grab bucket is now being increasingly employed or specified. It is an especially appropriate method where contaminated soils are involved or where there is restriction on discharge of sediments into the water. Even the grab bucket must be designed so that the water can equalize above and below while the bucket is being lowered and especially while being raised. For this purpose, the grab bucket should have holes or provide spaces around its periphery. Alternatively, an augur or bucket drill is employed.

If an airlift is employed, with discharge into the water alongside, turbidity can be greatly reduced by turning the discharge pipe down as by means of an elbow plus hose, so that the discharge occurs near the seafloor. Additional guidance on installation and testing drilled sockets is given in Section 8.13. Figure 9.35 and Figure 9.36 show the large-diameter steel shafts being constructed for the Ohnarutu Suspension Bridge, Japan.

Due to limitations on crane boom reach and length as well as lifting capacity, the tubular piles may have to be spliced during driving. This operation is not only very critical to the

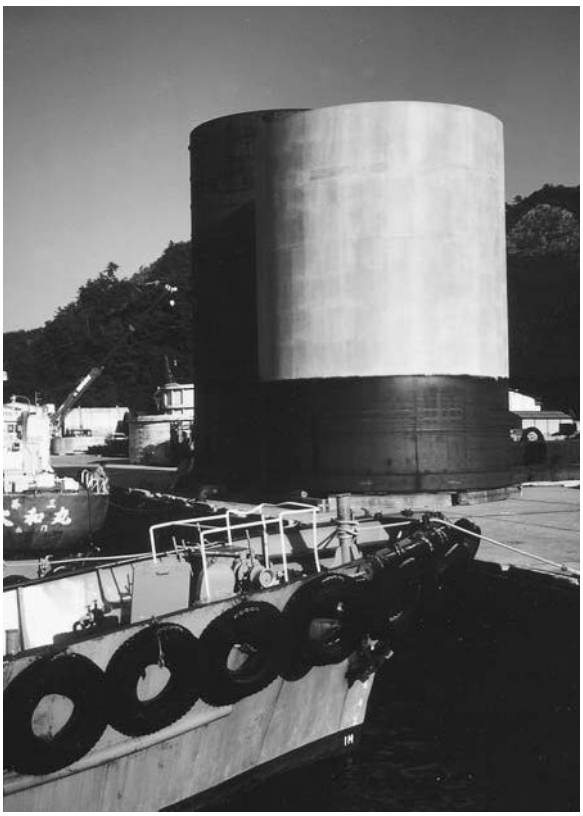

FIGURE 9.35
Large diameter steel shafts for Ohnaruto Bridge, Japan, are protected by aluminized coating in splash zone and epoxy below water.

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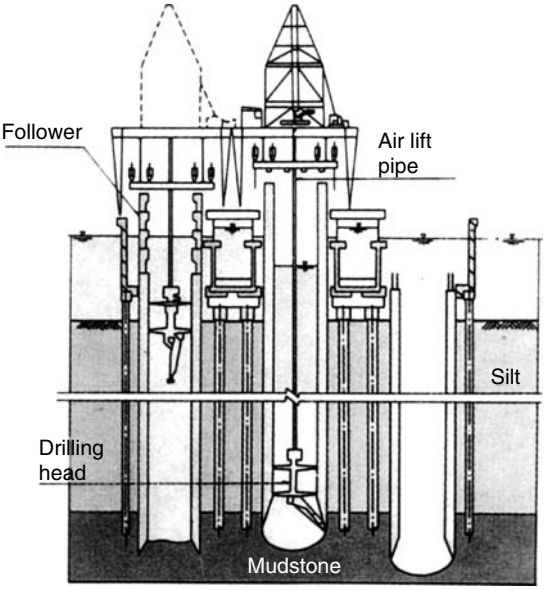

FIGURE 9.36
Installation scheme for Ohnaruto steel shafts. (Courtesy of Honshu- Shikoku Bridge Authority.)

integrity of the pile, but is also very costly in time and money. Further, the stopping of a pile section with its tip in dense material may lead to setup and difficulty in getting the pile penetration restarted. Hence, the location of pile splices needs to be carefully considered. Splicing of steel tubular piles is covered in detail in Section 8.4.

As noted, steel tubular piles for bridges may be made to act in composite action by filling all or a portion with concrete. Filling both increases the stiffness and prevents local buckling of the pile under seismic or impact. Although bond is often adequate to transfer the shear from the steel pile to the concrete core, near the head shear connectors are required. Induction-welded studs are often specified but have the problem of interference with drilling and clean-out operations if these become necessary. Weld beads are sometimes employed but the most reliable is the use of spiral or circular bars such as 10 mm flat bars, welded around the inner circumference with fillet welds. The shear connectors should be continued over a sufficient length to transfer the shear due to high moment just below the pile head, usually one to one and one half diameters. Similar shear connectors are also often used to lock a concrete tip plug in place where such a plug has been required.

For the Jamuna River Bridge, crossing the Brahmaputra River in Bangladesh, 2.5 and 3.3 m tubular steel piles were used, typically 80 m in length. They were driven on a 1:6 batter in pairs and threes, to resist both axial loads due to current, earthquake, and braking along with liquefaction of the overlying sands.

Installation was performed by splicing the pile in three sections, joined by full-penetration butt welds. Each splice took about eight hours, using four teams of welders. The time includes time for cooling and UT inspection.

Driving through the lower dense sands required a pile hammer developing 1,300,000 ft. lbs. of energy per blow. Each pile took an average of 12,000 blows.

On the Skyway portion of the SanFrancisco-Oakland Bay Bridge, 2.5 m piles are being driven with the same size hammer to a total depth of 105 m, also on a 1:6 batter

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(Figure 9.37). They encounter weak silts, then dense sands and gravels and finally hard non-plastic clay. Splicing is carried out by uses a semi-automatic machine, strapped around the pile.

On the Second Benicia-Martinez Bridge, 40 m thick×3 m dia. tubular piles are being driven as permanent casings to 15 m penetration in fractured rock. The purpose is to provide lateral support if the overlying silts and sands are scoured away by floods.

In the upper 15 m, they have encountered very hard layers of interbedded sandstone, alternating with fractured rock and loose uncemented dense silt lenses. Some pile tips

FIGURE 9.37
Driving 2.5×105 m^2 steel tubular piles. San Francisco-Oakland Bay Bridge replacement—Skyway. (Courtesy KFM Constructors.)

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were locally crimped. With a heavily reinforced tip, the piles could be driven to the required penetration. However, the drilled sockets below the tip had excessive fall-in when they encountered loose, cohesionless silts, which could not be stabilized with the polymer slurry. The drilling was done with a drag bit. It is believed that a roller-cone bit might have been more successful. So the work on the troublesome piers is being completed by coring with a large hydraulic shaft rotator. This is a slow and costly process but it is working.

On the Third Carquinez Bridge, the piles in the southern pylon pier encountered similar unstable zones during socketing with excessive fall-in. There, the procedure adopted was to drill ahead (with a roller-cone drill) a limited distance at a time (3–4 m), and then open the socket to a 600 m wider diameter and fill it with tremie concrete. After allowing time for set and initial strength gain, the original socket was re-drilled through the tremie concrete and on for a second 3–4 m increment. Again, slow, but the piles were completed and the bridge is now in service.

An interesting concept was developed by Caltrans for the main pylon support pier for the San Francisco-Oakland Bay Replacement Bridge. The piles have to be installed through 20 m of water, then 35 m of very soft mud, and then 40 m into hard rock. The rock is weathered and fractured to a depth of 10–15 m. Very high lateral seismic accelerations (in excess of 1.5 g) have to be resisted in essentially elastic action. The requisite combination of stiffness and strength was obtained by a double steel shell. The outer shell will be 4 m diameter and the inner shell 3 m. The outer shell is to be driven first and seated in the rock. Then a socket, of slightly less diameter, will be drilled through the fractured rock and on into sound rock a sufficient distance to develop fixity. An inner steel tubular will be set and

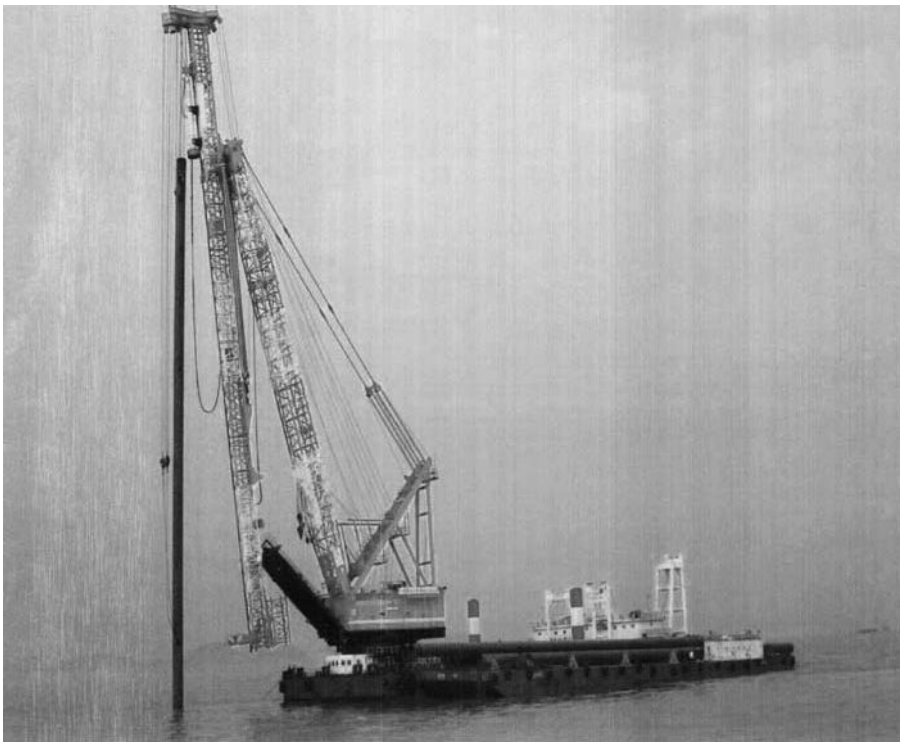

FIGURE 9.38
Hangzhou Bay Bridge. Pile-driving crane barge. (Courtesy of Hangzhou Bridge Design Authority.)

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grouted to the rock. Then the socket will be drilled and a reinforcing steel cage set. Tremie concrete will be placed in the inner pile up above the rock line. The annulus between the two tubulars will now be filled with tremie concrete.

For bridge pier piling, with long design life, such as one hundred years, corrosion protection is of increased importance. The use of an additional sacrificial wall thickness (typically 6–12 mm increase), or the provision of sacrificial anode cathodic protection below water, especially at the mudline, and epoxy, metallizing or other protective coating in the zone near the waterline are the most common solutions.

Steel tubular piles, 2.5 m diameter, have been used for the Woodrow Wilson Bridge across the Potomac River. Lengths are up to 90 m, penetrating deep bay mud and plastic clays and then dense sandy clay and hard overconsolidated clay. At the bascule spans in deep water, batters up to 1–6 have been employed, whereas at shallower sites or where upper sands are present, only vertical piles are employed, the objective being to obtain maximum ductility in earthquake. In both cases, piles were positioned by templates.

Steel tubular piles up to 3.5 m dia. have also been employed as shear piles to stiffen the response of the piers of the Richmond-San Rafael and San Mateo-Hayward Bridges under a projected strong earthquake.

The 36 km Hangzhou Bay Bridge is under construction across Hangzhou Bay, south of Shanghai. It is being built successfully despite severe tidal currents, ranging up to 6 m/s, tidal bores, and a tidal range of 6 m. Shallow methane gas deposits, winter storms, and threat of typhoons render this an extremely demanding venture (Figure 9.38 through Figure 9.40).

FIGURE 9.39
Hangzhou Bay Bridge. Note eddies due to tidal current. (Courtesy of Hangzhou Bridge Design Authority.)

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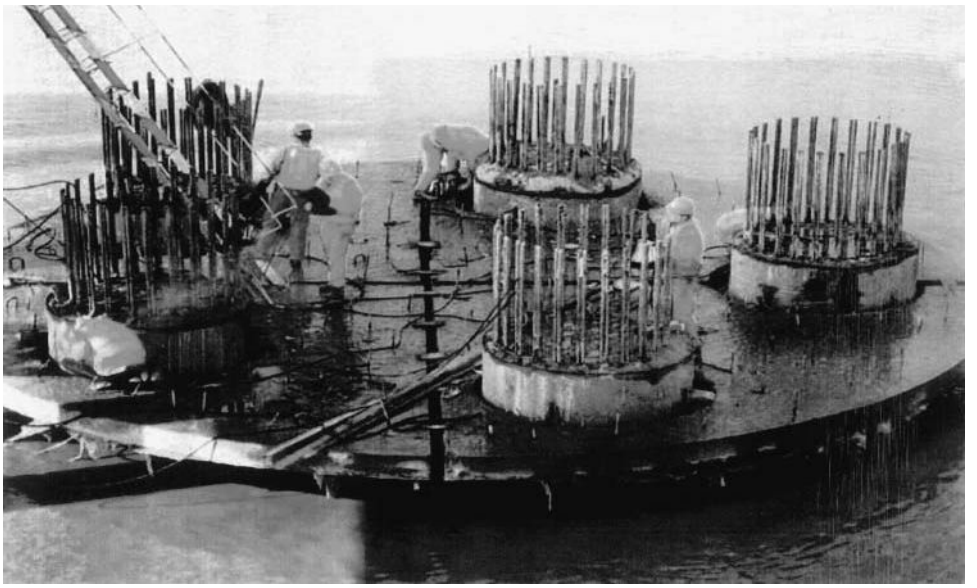

FIGURE 9.40
Hangzhou Bay Bridge precast concrete slab forms soffit and base for footing block. (Courtesy of Hangzhou Bay Bridge Design Authority.)

Excellent progress has been made due to fine engineering design and the construction of two very large offshore derricks, with powerful deck engines able to maintain their position in the current and to handle and drive the long piles as a single pile. Strict quality control has enabled successful installations of 1.6 m × 28 mm thick spiral welded pipe piles 80 m long.

9.4.6.2 Prestressed Concrete Tubular Piles

These have been employed in diameters up to 3.5 m and length of 60 m on such projects as the Oosterschelde Bridge in The Netherlands and the King Fahd Causeway Bridges between Saudi Arabia and Bahrain (see Figure 9.41 through Figure 9.43).

The largest-diameter piles (above 3 m diameter) have generally been manufactured as vertically cast segments, then tipped to the horizontal and assembled by post-tensioning. Joints have been cast-in-place concrete. The large piles are transported either on barges or suspended between two pontoons. This latter method make it less demanding on the pile when picking, since the lower half can be partially supported by its buoyancy during upending. In sandy soils, these piles have been sunk in place by a combination of their own weight, plus pull-down forces from the crane barge, plus jetting and internal excavation by a dredge head.

In the case of the King Fahd Causeway Bridges, holes were predrilled, using temporary casing, then 3 m diameter piles were seated, grouted in place, and plugged by tremie concrete. "Piles," 6–10 m in diameter, were employed for the Yokohama and Hiroshima bridges in Japan. Those for the Yokohama Bridge were sunk by progressive drilling and under-reaming while those for the Hiroshima Bridge were sunk by internal dredging and clean-out.

In the United States, many bridges are supported on 0.9–1.7 m diameter driven prestressed concrete tubular piles. These include the Chesapeake Bay Bridges, the Lake

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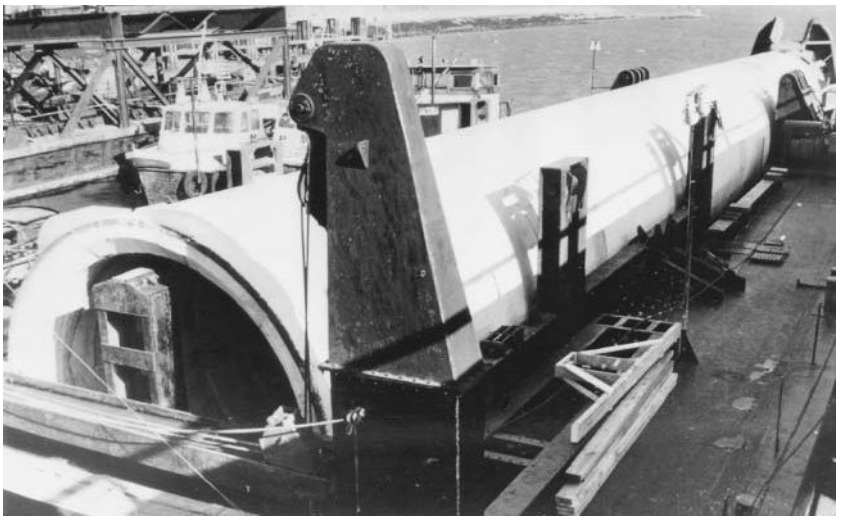

FIGURE 9.41
Transporting pile for Oosterschelde Bridge, The Netherlands.

Pontchartrain Bridge, the Napa River Bridge, and the San Diego-Coronado Bridge in California. Similar piles have been utilized on a number of deep water wharves (quays).

As for all prestressed piles that are installed by driving, enough prestress must be imparted to ensure that the piles do not crack under the tensile rebound stresses. These are especially prevalent in driving through weak sediments. The amount of longitudinal prestress required depends on many factors including the cushion block on the head of the pile, and the magnitude of the impact but with the larger hammer generally required for major projects, a residual prestress of 1200 psi (8 N/mm^2) to 1600 psi (11 N/mm^2) is usually employed.

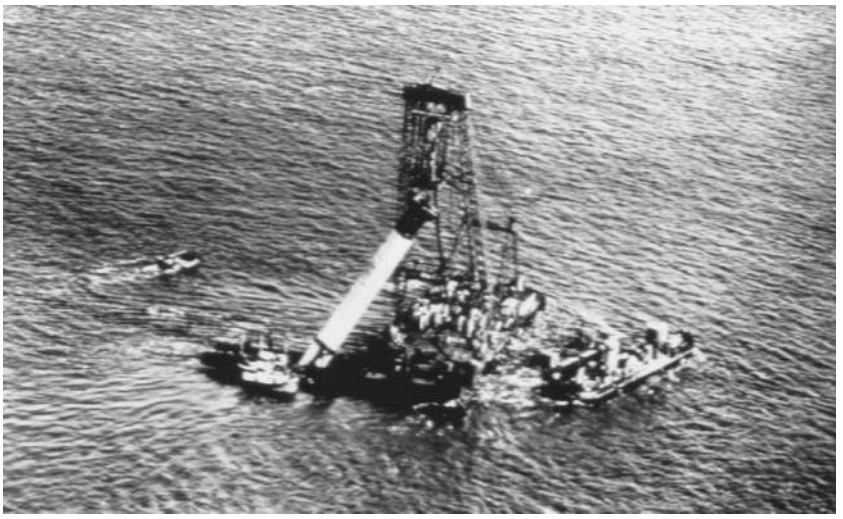

FIGURE 9.42
Upending concrete tubular pile for Oosterschelde Bridge.

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FIGURE 9.43
Installing 60 m long pile-shaft for Oosterschelde Bridge by internal excavating, jetting, and pull-down from crane barge.

Circumferential confinement must be adequate to prevent vertical cracks due to thermal action and the bursting effect of the driving impact. Due to the prestress, inclined and radial stresses all combine to produce vertically oriented cracks between the longitudinal strands. Experience has shown that the circumferential steel must be adequate to at least match the cracking capacity of the concrete tensile zone without exceeding the yield strength of the steel.

One of the major advantages of these large-diameter prestressed concrete piles is their durability and corrosion resistance. This property enables them to be extended through the water column and splash zone. However, durability can only be assured by adopting methods appropriate to the environmental exposure. Below water level, where oxygen is limited, durability can be achieved by using a dense impermeable concrete, e.g., a concrete mix including silica fume and an adequate cover over the outer steel. In this below-water zone, marine life will attach itself. Dense impermeable concrete is normally impervious to marine borers. However, in the Arabian Gulf, marine borers can eat through the cement coating into the limestone aggregate beneath and quickly bore to a depth of 50 mm or more. In locations such as these, only silica (igneous) aggregates should be employed. On the Ju'Aymah trestle, where high-grade limestone aggregates were used, the piles were quickly riddled with holes. Repair was made by removing the barnacles and other growth,

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filling the holes with epoxy paste, and coating the pile with hydrophobic epoxy, which could be placed in water. With the knowledge gained from that experience, on the King Fahd Causeway Bridges, igneous rock was used for both coarse and fine aggregates.

At the seafloor, moving sand may abrade the pile surfaces. Generally, this is not significant, except in the surf zone. Again the use of dense, impermeable concrete, obtained by the use of silica fume, is the best answer.

In the splash zone, there are many aggressive forces to be dealt with. The primary problem is that of corrosion of the reinforcing steel, since there are available oxygen from the air, chlorides from the sea, and water to provide the electrolyte. Although impermeable concrete and adequate cover are the principal protection, this zone is subject to cycle wetting and drying as well as cyclic thermal gradients, all of which combine to increase the permeability. Epoxy coating of reinforcing bars has been tried but is not generally satisfactory due to the loss of adhesive bond and delamination in seawater. The concrete cover tends to spall during driving and under high bending forces. External epoxy coating of the concrete surface of the piles in the splash zone was employed on the King Fahd Causeway Bridge piles. The coating extended up to the bridge superstructures. Three coats were applied. After ten years, detailed inspection showed no chloride penetration whatsoever. The epoxy coating extended up to $+4$ m. Examination showed considerable chloride penetration into the concrete from $+4$ to $+6$ m. It appears that the splash zone needs to be extended well above normal high water and waves. The outer epoxy coating is, of course, subject to considerable UV deterioration.

In locations subject to sustained freezing air during winter, yet where the tide periodically causes a rise and fall of the water, the tidal zone is exposed to severe freeze–thaw attack, along with thermal gradients and wetting and drying. Solid concrete piles with adequate spiral reinforcement have proved durable, but hollow core piles have suffered vertical cracks, which open wider over time due to ice-jacking. Freeze-thaw attacks the crack surfaces. The exposed circumferential steel then corrodes. The primary means of prevention is to incorporate air entrainment in the concrete and to use substantial circumferential steel, so that under the combination of thermal change and wetting and drying, the reinforcing steel is never stressed beyond yield. High bond is desirable to minimize crack width. Epoxy coating of the reinforcing bars is therefore not appropriate. In the Chesapeake Bay, the splash zones of prestressed concrete cylinder piles, saturated by high water, and frozen by winter winds, have suffered vertical thermal cracking through the tidal zone. Application of an external epoxy containing glass spheres for coating and insulating the surface of cracked concrete piles through the splash zone, has proved to be an effective remediation.

As noted in Section 9.2, pretensioned solid concrete piles, either 24″ octagonal or square, have been widely utilized for wharves in moderate to deep water. The guidance on longitudinal prestress and circumferential spiral, given in the preceding text, is also applicable to these piles. Solid piles are easier and less costly to manufacture but of course lack the stiffness, bending capacity, and the stability of the larger cylinder piles.

Prestressed concrete piles have been successfully spliced where longer lengths have prevented their being driven in one piece. Experience has shown the importance of elimination of all eccentricity in the splice or local moments will be introduced and lead to rupture under continued driving.

9.4.7 Connection of Piles to Footing Block (Pile Cap)

In most cases of bridge piers, a fixed connection between piles and pile cap (to the extent feasible) is employed in order to minimize deflections. A concrete plug of two to three

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diameters length will prevent local buckling and will slightly increase strength and stiffness in this upper zone of the pile. Reinforcing bars can be embedded in this plug, and extended on up to the top of the footing block. They can be most effectively anchored there if T-headed bars are used. Even so, it is generally not possible to get enough reinforcing bars into a plug to transfer the full moment of the steel shell, due to the smaller effective diameter and the need to provide adequate spacing for reinforcement.

Where the pile head extends above water, at least temporarily, or by placing a portable cofferdam over the pile head, bars may be welded to the steel shell at close spacings. Alternatively, the steel pile shell can be extended up into the footing block or pile cap and connected by shear lugs or strips along with horizontal steel bars top and bottom. It is often only possible to transfer a portion of the moment in a footing block of reasonable depth. A dramatic alternative employed on the Rion-Antirion Bridge foundations was to purposely isolate the piles from the footing block by a layer of compacted gravel. Thus the footing was able to slide and tilt under a strong earthquake.

9.4.8 CIDH Drilled Shafts (Piles)

When used in marine applications such as bridge piers, casings extending above water must be provided to permit drilling fluid flow to expel the cuttings and lubricate the drill. To support the casings and the subsequent drilling, a rigid falsework "template" is required, extending high enough to control the verticality or batter. Often these CIDH piles are installed from inside a shallow cofferdam which is subsequently employed for construction of the footing block. Casings must be embedded deeply enough to prevent run-in of sand or silt.

Provision must be made for disposal of the cuttings, either in a hopper or, if permitted, on the seafloor. After drilling and cleaning of the pile hole, a reinforcing steel cage is set, and the entire pile is concreted by tremie methods.

Casings that extend through water are best left in place, to serve as permanent protection. To protect the casings against corrosion, one of the following methods is used:

- Aluminized or galvanized plating
- Epoxy or polyurea coatings
- Added thickness of steel to act as sacrificial steel over the design lifetime
- Concrete encasement
- Cathodic protection

On the bridge portion of the Trans-Tokyo Bay Crossing, the piles were sheathed with a thin layer of titanium.

Drilled shafts 117 m long and 2.5 m diameter were installed for each pylon pier of the Sutong Bridge across the Yangtze River in China (see Figure 8.35).

From the Tsing Yi Bridge in Hong Kong, a galvanized corrugated pipe was inserted in the casing and the annulus filled with pea gravel. Then, the outer casing was withdrawn as the concreting took place.

For detailed discussion of the CIDH technology, see Section 8.14.

9.4.9 Cofferdams

These are temporary structures that are installed to permit dewatering so that the footing block and below-water portion of the shafts can be constructed in the dry. The cofferdam

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must not only exclude water from the sides, but also from underneath. The principal forces acting on the cofferdam are hydrostatic but may also include soils, especially when the soils are weak. Unbalanced soil loads are a special case for cofferdams built on a slope, since the cofferdam as a whole must be braced to withstand the global forces. Similar steps must be taken to resist current, wave, and ice load. The global load must then be transferred to the soils, by passive pressure or batter piles.

Structurally, the typical cofferdam consists of the peripheral wall, a tremie concrete base slab (seal), held down by dead weight and piles, and internal bracing. Circular cofferdams have been built, with circular wales or of concrete slurry walls. They are efficient in resisting hydrostatic forces but very sensitive to unbalanced and local loads, which produce circumferential bending.

9.4.9.1 Steel Sheet Pile Cofferdams

Steel sheet piles are ideally suited to rectangular cofferdam construction. Sheet piles of high bending capacity are employed, either Z-shaped or a combination of a structural wide flange shape or pipe pile with Z-shaped sheets. The sheet piles are interlocked. The sheet piles are set in panels along the wale which gives temporary support. They are then driven in echelon, so that adjacent pile tips do not differ more than 1.5 m–2 in. elevation. Vibratory hammers are very efficient in driving steel sheet piles. In some difficult soils, impact hammers may be required to attain the final penetration.

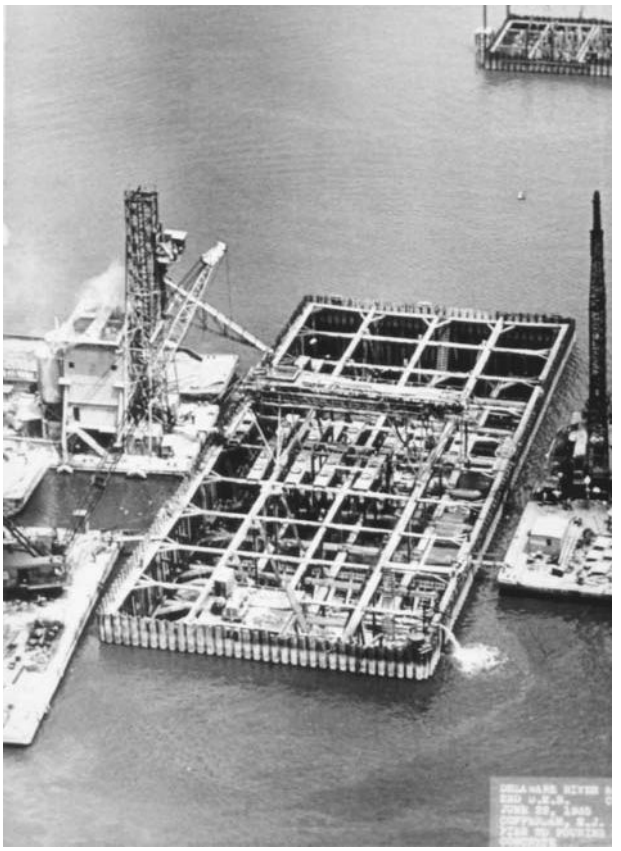

FIGURE 9.44
Steel sheet pile cofferdam, Second Delaware Memorial Bridge.

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Bracing must now be installed to permit excavation of the soils within and future dewatering. Top bracing may have already been installed to support the sheet piles. Lower bracing must either be preplaced or lowered down underwater and wedged against the sheets. Excavation now proceeds to a level deep enough to allow the tremie seal course to be placed and the footing block constructed. Actually, it should usually be carried 0.5–1.0 m deeper to allow placement of a crushed rock underbase (Figure 9.44). Piles are now driven. These will be the permanent piles, plus any additional piles required to resist the uplift when the cofferdam is dewatered. Batter (raked) pilings give special problems if they will intersect the sheet pile tips. In this case the batter piles may either have to be relocated or the sheet pile walls moved outboard to clear.

The sheet piles must be bolted or welded to the wales so that the entire cofferdam acts as a unit. Bracing members that will not roll under lateral load should be selected. The bracing frame must have diagonals and verticals to prevent it from racking, especially if an unbalanced lateral load will be applied, since the cofferdam is essentially a gravity wall with almost no weight. The completed bracing will be a space frame. Design of the bracing must consider the forces from currents, including the augmented drag due to the configurations of the sheet piles and any barges moored alongside.

When the cofferdam is dewatered it must resist the uplift forces, both locally and globally. Usually there is not enough dead weight for this, so piles must resist the net forces by their capacity in uplift, i.e., skin friction.

The tremie concrete seal is placed as previously described in Section 6.6. It is usually designed to lock to the piles and to span between piles as an unreinforced slab in order to resist hydrostatic uplift. However, on several recent cofferdams, reinforcing steel cages have been placed to resist both uplift and thermal stresses. Alternatively, steel fibers can be

FIGURE 9.45
Dewatering steel sheet pile cofferdam, Second Delaware Memorial Bridge.

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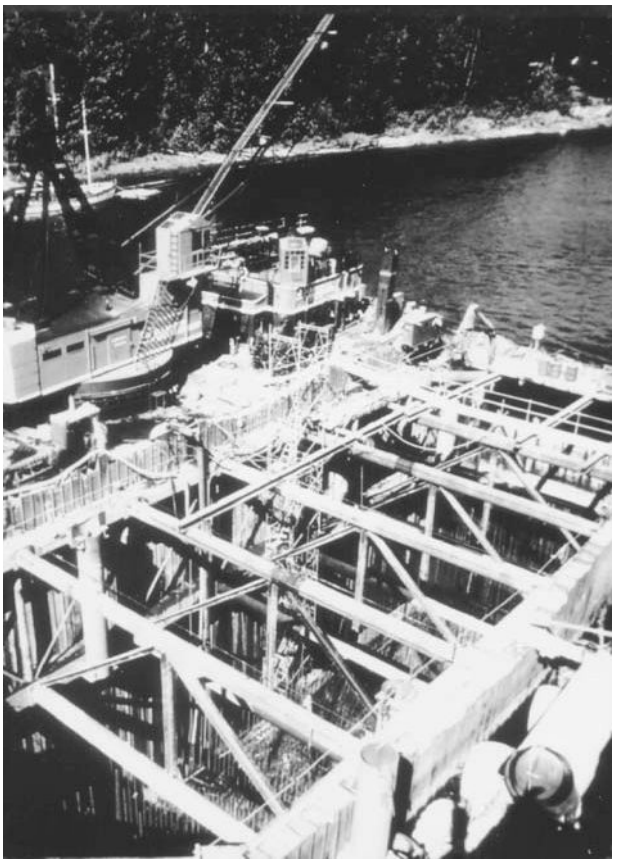

FIGURE 9.46
Bracing system for deep cofferdam, I-205 Columbia River Bridge, Oregon.

incorporated in the mix as was done for the on-land cofferdams in the reconstruction of East Berlin.

The cofferdam is now dewatered (Figure 9.45 and Figure 9.46) and the footing block and shafts constructed in the dry. The cofferdam is now flooded, the sheet piles are extracted, again by vibratory hammer, and the bracing members lifted out for reuse. Backfill is placed around the pier, followed by antiscour riprap or stone.

The above description is oversimplistic. Following is an alternative sequence, which can be taken on many cofferdams to achieve the most expeditious and economical completion.

1. Pre-excavate. This will, of course, develop side slopes, increasing quantities of excavation, but is much less costly per unit dredged.
2. Drive all required piling, using a follower or underwater hammer.
3. Construct the entire bracing system as a prefabricated space frame. Set as a unit and hang from spud piles driven through sleeves in the bracing.
4. Now proceed to drive sheet piling. Backfill around the sheets or above the surrounding seafloor, as necessary to resist the outward pressure of the tremie concrete.
5. Place crushed rock underbase, then place tremie concrete seal. Where approved by the engineer, the seal course may be combined with the footing block and the

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latter constructed underwater by a preplaced reinforcing steel cage and tremie concrete.

6. Block bracing frame against sheet piles and dewater. Construct remaining structure in the dry.
7. Pull sheet piles. Lift bracing out as a unit to permit reuse.

As noted earlier, sheet pile cofferdams depend on passive resistance below the deepest excavated level, in order to resist lateral pressure. Attempts to increase the shear resistance by jet grouting have led to several failures including one catastrophic case in Singapore. This was believed due to reduced shear resistance from the upward flow of water into the dewatered cofferdam.

Further information on cofferdam construction will be found in Chapter 7 of *Handbook of Temporary Construction,* 2nd ed. (Ratay 1996).

9.4.9.2 Liquefaction During Cofferdam Construction

The increasing use of heavy vibrators to drive steel sheet piles has occasioned a sharp rise in the occurrence of unintentional liquefaction. Strangely, this phenomenon has been recognized only recently, and that in a publication not widely read by contractors.

The Corps of Engineers Manual EM1110-2-2504 states: "The potential for liquefaction may exist any time a dynamic operation takes place upon a granular foundation or a stratified foundation which contains granular soils. The risk should be evaluated on a case-by-case basis. Limitations should be set as required on pile driving (especially the use of vibrators). A total ban on pile driving may be warranted in extreme cases."

Because the causes and consequences are not widely appreciated, space will be devoted to several examples (see also Section 2.3).

1. A bridge was being constructed across the lower Colorado River. One sheet pile cofferdam had been driven into the sands. A second cofferdam, 60 m away, was starting construction. Sheet piles were being driven with a vibratory hammer, just like the first cofferdam, which had been completed two weeks before. Suddenly, they realized that the sheet piles of the first cofferdam were disappearing below water.
2. A very deep cofferdam required sheet piles 30 m long to be driven through silts and sands into a limestone stratum. The piles were spliced and thus developed considerable friction in the interlocks. Soil friction was high. As a result, the sheet piles took extensive vibration with a powerful vibratory hammer to drive them close to grade. Complete seating was not attained. As the final excavation inside was being carried out, the liquefied sand flowed under the tips and through gaps in the interlocks, filling the lower half of the cofferdam with sand.
3. Another deep cofferdam was being constructed on the slightly sloping bank of a river. The top 15 m was loose sand. Sheets were driven in stages, extending below the bracing. The upper sands liquefied and exerted high pressure against the sheets, turning them inward. Continued efforts to penetrate just made matters worse. The bank and top of the cofferdams started to move towards the river.
4. On a major Navy graving dock, steel sheet pile cells were used as the permanent side walls. After filling, they were to be densified to 70% R.D. The fill for one cell inadvertently contained a stratum of low permeability silt. Vibration by a large penetrating vibrator was carried out for a prolonged time without

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obtaining the desired results. Suddenly, a sheet pile ripped out of the interlock. This was repaired by excavating the sand fill, pulling several sheets each side of the rupture and re-settling and driving new sheets. On subsequent cells, wells were first drilled and perforated casing placed to allow the pore water to escape. Not only did this prevent the build up of excessive internal pressure, but it greatly expedited the densification.

9.4.9.3 Cofferdams on Slope

A special condition arises when a cofferdam must be constructed on a slope, for example, on the bank of a river. Then there is an unbalanced global load created by the soils, especially if one side extends above water. The active pressure of the soil on the upside exceeds the passive resistance on the downside. Once a lateral slope failure initiates, even by a small amount, the acting unbalanced force may increase, especially in clay. The acting force may be reduced by excavating and removing the surcharge, especially that above water.

While the cofferdam can, and should be, diagonally braced to act as a unit, it then becomes a gravity retaining wall weighing almost nothing. It must be anchored in some fashion. This can be done with deadmen and ties on shore, batter piles, or very stiff vertical master piles or shafts. The lateral anchorage must be effective not only at the top but extended to the center of net lateral force. This can be done by internal diagonal bracing, making the cofferdam a vertical truss.

This matter has to be investigated at each stage of construction.

9.4.9.4 Deep Cofferdams

One especially difficult construction problem is encountered in the cofferdams for piers whose footing blocks are placed deep in existing sediments as a precaution against future scour. This situation occurs on rivers such as the Mississippi.

Conventional steel sheet piles used in the U.S. lack desirable stiffness for extreme lengths. They have an inherent sweep and hence are difficult to set and drive accurately. For these very deep cofferdams, the use of special sheets such as those embodying wide-flange beams, used in Europe or pipe piles with interlocks as used in Japan, have advantages. The pipe piles also permit drilling ahead. Both of these minimize the number of bracing frames, although the total load remains the same.

For these, it is generally not possible to pre-excavate. The top levels of bracing can be installed above ground level, and the deeper bracing, prefabricated as horizontal frames, hung beneath. Then as the excavation proceeds, the frames may be lowered down and wedged against the sheets. Vertical support and, where needed, diagonal framing, must be installed in-place, which means for most piers, underwater. Such braces may be partially prefabricated, designed for connection by conventional bolting, or, where slotted holes in crossed pairs can be pre-drilled, with high-strength or non-slip bolts.

Another scheme has been to prefabricate the entire frame above ground level, thus initially extending well up, and then lower or force the frame down as the excavation proceeds. Sheet piling may be driven ahead in steps. Obviously, this means that the sheets must slide along the frames. If they have been forced in by external pressure, the sheets will develop high friction, making driving slow and difficult. This may prevent the subsequent lowering of the frames. Initially, driving the sheets on a slight outward angle is one solution adopted. This latter scheme is inherently unsound and may result in hang-up of the bracing frame or bending distortion of the sheets.

Both of the above methods are facilitated if the external pressures are slightly overcompensated by filling with water above the external water or ground level.

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The increased need to drive steel sheet piles to greater depths in order to enable the permanent footing to be below the depth of potential scour, or to provide that scour protection in place, has led to another problem. If the interlocks must penetrate a substantial distance into fine sand, they become plugged, especially where there is some cohesive material included. As the second sheet or pair is driven down, it compacts the sand plugging the interlock to the point that refusal is reached. Further vibration or impact only makes it worse. The tip of the pile becomes heated by the concentration of energy, the interlock yields and the sheet pile jumps out of interlock. As a general rule, this can be prevented by having the ball interlock always in the lead.

Frequently, the footing block is supported on steel piles, whose cut-off elevation is below water. Although possible to drive these with an underwater hydraulic hammer, there is a serious problem of interference with the bracing. A second problem arises when the pile has not brought up to bearing since underwater splices are not practicable. Therefore, a better solution is to use piles longer than required in final position. Even better is to use steel piles long enough to stay with their tops above water. The extra length can be cut off and re-spliced on subsequent piles. The cut-off may be made underwater or after the seal has been placed and the cofferdam dewatered.

Where solid or closed-end bearing piles are used inside the cofferdam, their displacement may cause excessive heave or lateral movement of the sheet piles. Open-ended or H-piles may be preferable, especially in dense soils.

For very deep water and soft soils, the Japanese have selected a modified version of the slurry wall technology. A circular ring structure is built, one element at a time, using the standard slurry wall techniques (see Section 8.18). To provide soil all the way above the surface so as to facilitate construction, two concentric sheet pile walls are first constructed and filled with engineered soil of proper shear strengths. The individual elements of the slurry wall are then excavated and concreted. To ensure ring compression with little or no eccentricity, great efforts are made to keep each element truly vertical and in correct circular alignment. Bentonite or polymer slurry is used to keep the excavated slots open. Joints between adjacent elements are formed to give a shear key and to provide a moderate degree of moment and shear transfer through the use of overlapping reinforcing bars. Corrugated steel forms, temporarily filled with gravel to protect the extended bars, are used to form the joints.

Each slot is reinforced and filled with tremie concrete. The same concrete mix design and placement principles apply as those for cast-in-drilled shafts. After two adjacent segments have been concreted and gained strength, the gravel-filled joint is excavated

FIGURE 9.47
Slurry wall cofferdam, 100 m diameter and 80 m deep, for Kawasaki Ventilation Structure, Trans-Tokyo Bay Crossing, Japan.

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by air lift and then tremie concrete is placed. As excavation proceeds, reinforced concrete circular wales are constructed to ensure that local moments are transferred.

This scheme was employed for the Kawasaki Ventilation Shaft of the Trans-Tokyo Bay Tunnel, a perimeter wall 120 m deep and almost 100 m in diameter, dewatered to 80 m depth. Other methods of jointing are also available.

The Kobe Anchorage of the Akashi Strait Bridge and the access shaft for the French side of the Channel Tunnel are outstanding examples of this type of cofferdam. It is also planned for use on the Messina Strait Bridge (Figure 9.47).

9.4.9.5 Portable Cofferdams

Portable cofferdams are utilized primarily for modifications and repairs of underwater structures. They provide a local enclosure of the area which is watertight, enabling persons to enter and work at atmospheric pressure. Typically, they are a three-sided box that is lowered and affixed to the near-vertical side of a dam or bridge pier. Entry for personnel, transport of materials, and removal of debris is through vertical tubes extending above water. Services such as dewatering, communication, power, ventilation, and lighting are also run through these tubes.

The box is sealed to the face by deformable gaskets, compressed by the external hydrostatic pressure. To make these effective, initial compression is attained by tensioning the box against anchor bolts drilled and grouted into the face. Then dewatering will provide the required deformation.

Portable cofferdams have been employed for repairs to the Bonneville Dam on the Columbia River and to repairs to the ventilation structure for the Bay Area Rapid Transit System in San Francisco, also for underwater modifications on the Jubail Industrial Canal in Saudi Arabia.

The atmospheric portable cofferdam contrasts with the hyperbaric enclosure used to join or repair submarine pipelines. This is a chamber which fits around or over the pipeline and excludes water by air pressure above the external hydrostatic pressure (see Section 18.7).

9.4.10 Protective Structures for Bridge Piers

Since collision of large ships with bridge piers has proven in recent decades to be one of the greatest threats to bridges, protective structures are now being required on all major bridges over navigable waterways.

Two situations may develop. In one, the force of impact is cushioned and distributed into the pier at a reduced level. In the second type, the pier may be unable to resist the additional force, so the fender structure must be independent.

A number of solutions have been developed to resist the impact of large vessels with no damage to the bridge pier and minimum damage to the ship.

The solution most commonly employed is to construct an embankment of sand or stone around the pier and to protect it against erosion by riprap. When a heavily loaded ship hits the embankment, it plows into the sediments, developing passive resistance, increasing with the increasing volume of displaced material until it stops. If the ship is in ballast or trimmed down by the stern, it may ride up, using up energy by this rotation and by friction.

A major concern is the capacity of the soil against settlement and the downdrag imposed by this embankment on the bridge pier's foundation. Dredging of overlying sediments is usually required.

Scour by currents is another concern, which is usually countered by constructing an extension of the embankment as an apron or deepened section at the toe.

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Since sand is the preferred material for the embankment due to its high frictional resistance to the ship hull, protection against erosion by currents and wave action has to be considered both during construction and in service. Riprap on the sloping sides and on top is placed on top of a graded filter of stones and filter fabric. Various solutions also include the use of articulated block mattresses and falling aprons (Figure 9.48).

Another type of protective structure, employed when the navigational opening cannot be restricted by embankments, is that of sheet pile cells, filled with sand, densified, and capped by concrete. These resist the ship collision by deformation of the cell, tension resistance of the sheet piles, passive resistance of the sand fill, and ride-up rotation of the ship.

The sand infill must be densified and/or perforated by drains, in order to not liquefy under impact. Densification is usually accomplished by vibration along with the provision of stone columns or wells in order to allow the displaced water to escape. A horizontal drain of crushed rock, covered by filter fabric, may be provided above water level.

To increase the circumferential tensile capacity of the cell and thus resist the bursting of the cell due to pull-out of the interlocks, heavy circumferential steel may be incorporated in a concrete ring or slab at the top of the sheets.

Circular concrete cells may be used in similar fashion as sheet pile cells and sunk into the sediments as mini-caissons. Larger rectangular concrete caissons may be installed, with piled support when necessary.

The most difficult problem for both designer and constructor arises when the bridge pier is in deep water with substantial current and underlain by deep soft sediments. One solution, adopted by the Japanese for the Kawasaki Ventilation Structure in Tokyo Bay, is to use steel jackets and pinpiles, capped by a concrete slab. Another, used on large bridge piers, founded on massive footings is to cushion the force by multiple steel boxes. Ship impact causes the buckling of several horizontal steel plates, allowing restrained penetration and distributing the reduced force to be carried by the pier itself.

Steel jacket and pin pile dolphins have been used in a number of cases, with the ship impact force transmitted by the passive resistance of the piles in the soil. To increase this passive resistance, the lower end of the jacket may be framed by stiffened steel plates. After the jacket has been seated on the seafloor and vertical pin piles driven, it is pulled down by jacking on the piles, forcing the plated lower portion into the soil. This solution then has minimum resistance to waves and currents with maximum passive resistance to ship impact.

The reactive force of increasing buoyancy under deflection due to impact has been used in a number of innovative ways. To protect the East River Viaduct in New York, large cellular buoyancy chambers (tanks) were incorporated in steel jackets. The jackets were in turn pinned to the bedrock by drilled-in chains. Under impact, the entire dolphin, consisting of the jacket plus tank, will rotate, pulling the tank below water, thus developing a restoring force.

Independent structures such as jackets and dolphins may be joined by slabs or steel frames above water or spanned by floating "camels" of steel or concrete.

Chains to arrest the ship have been proposed in a number of instances, supported by floating spars or jackets, but they are not considered reliable. Depending on the ship's draft and bow configuration, the ship may ride over or be forced under.

9.4.11 Belled Piers

Belled piers were developed in the 1930s and used on a number of large bridges in the Chesapeake Bay Area. They were adopted in the second half of the 20th century for bridges in California and Oregon and for Narragansett Bay in Rhode Island. They are a form of box caisson in that they are prefabricated shells, containing all the reinforcing

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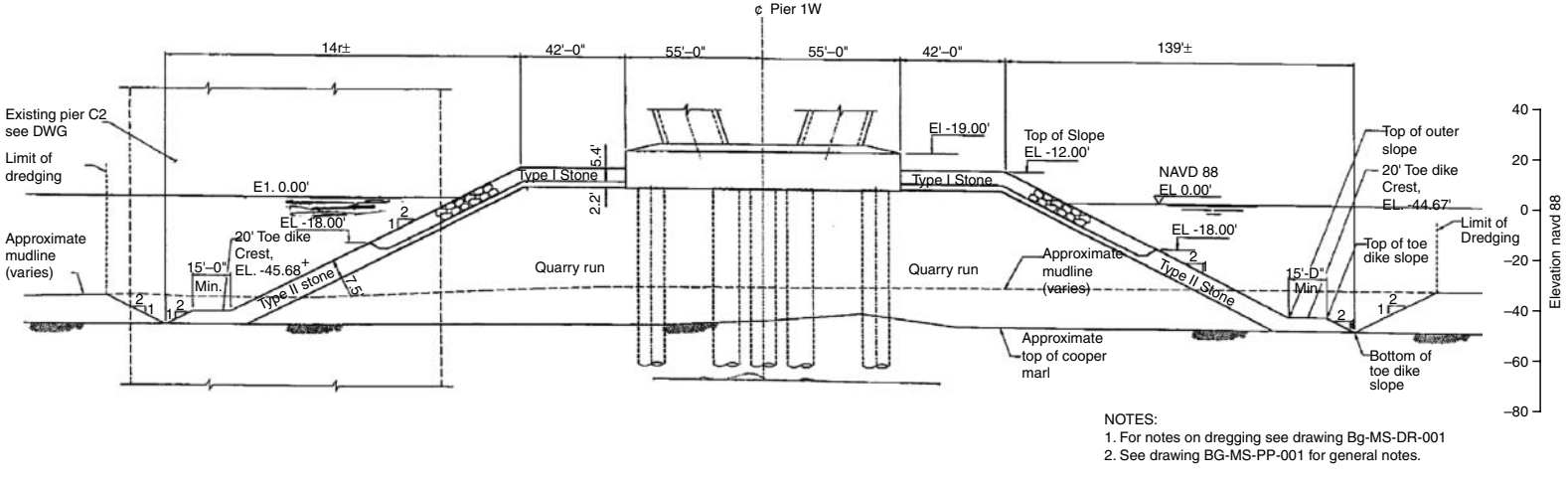

FIGURE 9.48
Sand and Riprap Island protect main piers of Cooper River Bridge, Charleston, South Carolina. (Courtesy of Ben C. Gerwick, Inc.)

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steel. The shells were initially stay-in-place steel forms and in more recent projects, precast concrete segments or removable steel forms. They take their name from their enlarged base and relatively small diameter shaft (see Figure 9.31 through Figure 9.33).

In most installations, the pier has been supported on steel H piles driven in a circular pattern, using a template placed in a shallow dredged excavation in the seafloor.

Then the shells are placed and filled with tremie concrete. This concrete ties the piles and shells together into a monolithic unit. This has proven to be a very effective solution, especially for long bridges with many piers, enabling repeated procedures and sequencing.

Lessons learnt include the following:

1. The tremie concrete must be "flowing concrete," self-leveling, with minimum bleed and segregation. It must flow around piles and reinforcing bars easily despite the fact that only the buoyant weight is consolidating it. Heat of hydration should be limited, e.g., by use of fly ash to replace cement by blast furnace slag-cement.
2. Reinforcing bars to be encapsulated by the concrete should have adequate clear space between bars and in the set-off from the shell to permit the flow. Clear space shall be at least five times the size of coarse aggregate.
3. Both concrete and steel shells can become slightly distorted, the concrete shells through creep in storage and the steel shells due to elastic deformation. "Spiders" should be used to retain shape.
4. Joints in the shells and forms must be tight and not allow grout leakage. Grout leakage is increased by the Venturi effect of flowing water in currents or waves.
5. Horizontal construction joints should be jetted clean at an early age so as to remove laitance.

9.5 Submerged Prefabricated Tunnels (Tubes)

9.5.1 Description

More than one hundred submerged prefabricated tunnels have been successfully constructed to cross estuaries, harbors, and rivers in the United States, Europe, Asia, and Australia. As a result of this repeated use, major improvements in construction and installation have been and are still being made.

These tube segments are large, typically 100 m long, 20–30 m wide, and 10 m high and with 1–2 m thick walls. They are prefabricated in a dry dock or basin, either as steel–concrete composite segments, favored in the United States and Japan, or as reinforced and prestressed concrete, favored in Europe. Each segment is then floated to the site, where a trench has been dredged and a stone base placed and screeded. The tube segment is supported from the surface by a catamaran barge and ballasted down onto the stone base or leveling pads. Connection is made to the previous segment. Finally, the tubes are backfilled and a protective cover of fill or riprap placed over the top. Regardless of which type of construction is used, the walls will be thick concrete, to give adequate weight on bottom when finally installed and dewatered.

The large size and mass of these structures requires careful consideration of the effect of currents and water density, as well as hydrodynamic and inertial forces while

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FIGURE 9.49
Prefabricated concrete tunnel segments ready for float-out from construction basin, Montreal, Canada.

submerging through the several strata of water. Scour during installation needs to be considered (Figure 9.49).

9.5.2 Prefabrication of Steel–Concrete Composite Tunnel Segments

The steel structure, consisting of the external skin plate and the internal stiffening and bracing, is typically fabricated in a shipyard. Heavy temporary bulkheads are attached to both ends. These are designed to resist the maximum hydrostatic pressure during submergence. Gaskets are fitted. Reinforcing steel studs are attached to the skin plate and other studs to the bracing. The additional reinforcing steel for the concrete may also be installed at this time, as may the internal forms. The steel hull is then launched sideways, using standard slipways, and then moored at an outfitting dock. Working through holes in the roof, workers progressively construct the reinforced concrete. Tunnel-type forms are used, placing a 16–20 m long segment at a time (Figure 9.50). Concrete is delivered by pumping. Careful control is kept of draft. When concreting is finished, the freeboard is usually about 1 m. A light structural tower and standpipe is installed on top of the tube at the loading end, to serve as a guide during installation and as access for the umbilicals carrying ballast lines, compressed air lines, electrical services, and

FIGURE 9.50
Tunnel segment forming and concreting machine, Øresund Tunnel. (Courtesy of Øresundsbro Konsortiet.)

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FIGURE 9.51
Segment of Øresund tunnel floating in assembly basin. (Courtesy of Øresundsbro Konsortiet.)

instrumentation. The standpipe also enables worker access. This structure may be fabricated and completed in a gravity dock, as was done for the Third Boston Harbor Tube (Figure 9.51).

The use of prefabricated double hull shells, filled with flowing concrete after launching, has been conceptually proposed by Corus-Bi-Steel of the U.K.

9.5.3 Prefabrication of All-Concrete Tube Segments

These are typically constructed as 100–120 m segments in a graving dock or construction basin, so that all construction may be completed before float-out. The reinforced concrete segment is constructed in sections, using a concrete mix especially designed for low heat of hydration. Thermal effects generally control the design of the reinforcement. Water tightness of the joints between large segments (125 m) is attained by the use of GINA seals. In some cases, the joints are made continuous after installation.

In many European projects, the structure has been heavily prestressed transversely. Because of the thick sections, a large number of tendons are required to keep a significant compressive stress across the section. Consideration must be given in the prestress design to the different stages through which the structure passes. During fabrication and prestressing, there is no hydrostatic load. At float-out, there is hydrostatic load on the bottom and sides but none on the roof. In the final case, the roof is also loaded. Thermal strains are locked in, due to the different stages and ages of fabrication, often resulting in cracks.

Careful control is kept of all dimensions during construction, since these determine the weight and displacement during float-out. Heavy temporary steel bulkheads are fitted to each end and gaskets placed. A tower and standpipe is attached, in order to provide for control and access. Waterproofing membranes are applied to the walls and roof. In the case of the roof, the membrane may be steel plate, attached by stud bolts, with grout injected underneath. A rubber membrane has also been used as the top membrane. The structure is floated in the dock and any needed adjustments in trim or heel are made by solid ballast.

Not all concrete tubes have been prestressed. The most recent tube segments in Hong Kong, Sydney, and Denmark have been constructed with reinforcing steel only. The reinforcing steel has been detailed to limit any cracking to very small widths. Special care has been taken to limit the heat of hydration of the concrete. A waterproofing coat of plastic has been placed on the sides and roof.

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In the case of the Øresund Tunnel between Denmark and Sweden, a unique form of fabrication was applied. Each large segment was subdivided into 5-m-long segments of full cross section. These were constructed above sea level, in a factory. Each small transverse segment was cast against the preceding segment. Each small segment was cast in one continuous placement to eliminate the differential thermal contraction and shrinkage which causes cracks. As the small segments were completed, they were progressively jacked out on rails. This factory production required that extremely close tolerances be maintained for the skid beams on which the segments were formed and joined. Six skid beams, pile-supported, were leveled to 2 mm tolerance. The sliding was on PTFE, Teflon bearing on a self-leveling epoxy compound. Friction was 3%–4% on breakout, dropping to 1% while moving. Gina gaskets were installed between each small segment. When all of the small segments for a 140-m-long segment were completed, they were temporarily post-tensioned together. Temporary end bulkheads were affixed as before. Using levees and a temporary bulkhead gate, the water level was raised around the large segment until it floated. The segment was then moved seaward to a deeper basin, where the water level was lowered to that of the adjacent sea. Now the segments were floated out, towed to the site, and submerged onto a preleveled bed (Figure 9.31 and Figure 9.32). When the segments were seated and backfilled, the temporary post-tensioning was removed. Thus the segments were free to accommodate limited differential settlement.

9.5.4 Preparation of Trench

While the segments are being prefabricated, the trench is being excavated by dredging. It is then partially backfilled with crushed stone or gravel to bring the bottom to grade. In the earlier tube projects, especially those in Europe, no attempt was made at exact grade, but the stone or sand bed was leveled sufficiently by drag screeds to ensure that no place was above grade. This method requires that temporary seating pads be constructed to exact grade upon which to seat the tube segment.

In the case of the Webster Street Tube between Oakland and Alameda, California, the temporary pad was pile-supported with a special cushioning cross girder on which the tube segment temporarily rested. This cross girder was fabricated of timber and steel, to be strong enough to support the tube during initial seating but to crush when the tube had been backfilled and fully ballasted so as to transfer the load uniformly to the sand. In another instance, polyurethane pads on top of concrete slabs were used, designed to transfer the load in similar fashion. In the case of several Danish-designed tubes, concrete pads were set, on which jacks operating from within the segment adjusted the segment to exact grade. Then sand was flowed underneath.

On most recent U.S. tubes, the stone bed has been screeded to exact grade. A special semisubmersible catamaran barge is employed, with pull-down devices at each corner, enabling the screeding barge to be kept at constant elevation despite the tidal changes. The pull-downs react against pre-set concrete weights. From the catamaran, longitudinal girders are hung to exact grade and profile. Running on these girder rails is a transverse hopper and screed. The hopper is fed by tremie pipes from the surface. Typical tolerance achieved is ± 20 mm.

On the Øresund Tunnel, the stone bed has been similarly screeded with a dredge ladder, fitted with a special head that distributes the stone. The head is computer controlled to exact elevation by electronic position and depth indicators. The dredge ladder and head sweep back and forth on a semicircular pattern to cover the entire area.

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9.5.5 Installing the Segments

In most recent construction, the segments, 80–140 m long, are installed by means of a catamaran barge (see Figure 9.52). This barge consists of two hulls which straddle the tube segment. The catamaran is moored in position with taut moors and the segment floated in and made secure. However, in the case of the Øresund Tunnel, pairs of pontoon barges, joined by a gantry truss overhead, in effect, two mini-catamarans, one pair on each end were used to minimize the longitudinal bending moments in the hull.

Stability during immersion is assured by keeping the center of gravity of the tube below the center of buoyancy, since the contribution to stability of the water plane is lost as the top passes below water. The ballast for overcoming the positive buoyancy which the tube had during float-out is usually placed as dry ballast, e.g., gravel, or is placed in compartments, so as not to develop a free surface, which could reduce stability. In the case of the Webster Street Tube in Oakland, California, water was sprinkled on sand beds, to add weight with no free surface effects.

The tube segment, as it is lowered, is both affected by the current and, in turn, affects the current by causing it to speed up under the tube as it nears the bottom. This can erode the trench sides and even the screeded base. The tube segment should therefore be set during a period of low current, e.g., slack tide. Horizontal positioning is controlled by mooring lines, run from platforms on deck or from the shore or dolphins to the segment itself, so as not to impose any rotational movement on the tube segment. Water density may increase as the structure nears seafloor due to salinity and suspended silt. Cases have occurred where the tube segment would sink no further until additional ballast was added.

A new tube segment is typically lowered 1–2 m seaward of the end of the previously placed segment. When it is 1 m above final landing, lines from fairleads on the previous segment pull the new segment to a 0.5 m gap. This gap may be surveyed by direct measurement by divers or by short-range sonic transducers on the tube end. The tube segment is then lowered to rest on steps protruding from the previous segment. At the same time, the leading edge of the tube is kept 0.5 m above the stone bed or leveling pads at that end.

Come-along jacks, similar to railroad car couplers, are attached at each side of the previous tube and used to pull the new tube tight, compressing the soft portion of the GINA gasket. The leading edge is then set down on the leveling pads. Working from inside the previous tube, the space between the two closure bulkheads is drained. The

FIGURE 9.52
Installing prefabricated tunnel segment by catamaran, Third Harbor Crossing, Boston, Massachusetts.

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full hydrostatic pressure acting on the leading end bulkhead is now available to squeeze the gasket tightly together. Additional ballast can now be added. When full seating and complete drainage has been verified, the two adjoining end bulkheads are cut out and the joint made good. For steel-composite tubes, this is accomplished by welding plates and grouting. For the reinforced concrete tubes, an inner Omega-type gasket is added.

9.5.6 Underfill and Backfill

When the tube segments have been seated on the steps and leveling pads, just above the stone bed, sand is flowed or jetted to fill the gap. Care has to be taken to avoid too great a hydraulic head of fluidized sand, so as not to raise the tube if the exits become blocked. Weak grout has similarly been injected under tube segments, the edges being sealed by sandbags or backfill. Initial backfilling around the sides of the tube must be done with care and consideration of symmetry to avoid sideways displacement of the tube. For that reason, placement through a large tremie tube is used rather than bottom dumping. Sand so placed will be of low density. In seismic regions, densification may be required by vibration or stone columns. This densification is carried out in small increments, balanced between the two sides so as not to displace the tube segment.

A sand blanket is then placed over the top of the tube. On this rock riprap or an articulated concrete mat is placed, in order to prevent damage to the tube from a dropped anchor. This riprap should be placed by skip or bucket and lowered to the seafloor so as not to damage the tube.

9.5.7 Portal Connections

The ends of the tubes are connected to portal buildings, in which the ventilation fans are housed. These are constructed separately, usually in a cofferdam of steel sheet piles. In the case of the Third Boston Harbor Tunnel, a circular concrete slurry wall cofferdam was used. Joint details are similar but more complex than those between segments, since they must accommodate thermal and seismic expansion and contraction as well as differential settlement.

9.5.8 Pile-Supported Tunnels

For some submerged tunnels, the existing soils are too weak and too deep to prevent substantial differential settlement, especially when the tunnel is covered for protection against ship anchors and locking backfill is placed on the sides. This latter develops not only a significant settlement but also exerts downdrag on the tube. If the weak soils are relatively shallow, they may be removed and replaced with select fill, suitably densified. If deep, piles may be required. The required pile capacity is determined primarily by accidental conditions such as tunnel flooding.

On the Webster Street Tunnel across an area of San Francisco Bay, in the areas of very soft and deep clay, piles were driven in the trench and cut off underwater. A tremie concrete slab was then placed, subsequently covered by 2 m of sand, on which the tube segment was laid.

For the Busan–Geoje (Korea) Tunnel, currently in design, even weaker soils occur, as well as deeper water. Seismicity and typhoon-generated wave forces plus high tidal

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currents indicate that it may be necessary to found the tunnel elements directly on piles. The tube invert is, in some cases, above the existing seafloor. It has been tentatively planned to install pipe piles, driven underwater and socketed into the underlying rock. Using a casing cutter, they would be accurately cut-off underwater. Then, a precast concrete plug will be placed and grouted. The 130 m long tunnel elements would be ballasted down, the loads equalized by flat jacks and the segment slid into connection with the previously installed element. Gina gaskets will be compressed using external jacks. After dewatering the space between end bulkheads, the bulkheads would be cut out and the joint made.

9.5.9 Submerged Floating Tunnels

These have been proposed for many years for deep-water crossings and have been engineered in considerable detail for the crossing of the Hogsfjord, in Norway. Proposed construction procedures are similar to those for standard submerged prefabricated tunnels, except that each buoyant tube segment must be held in position while tethers are attached.

The tethers run to anchor blocks, either gravity blocks placed on the seafloor or piled anchors. Because of the depth, the tethers will usually be pre-attached and fabricated with upper fittings to exact length. In the case of the piled anchor, a seafloor template can be secured to the piles at the exact design elevation. In the case of the gravity anchor block, the settlement in the soil may be somewhat indeterminate, so instead of pre-attaching tethers, they may be fabricated after the block location is known. Then, the tethers can be run down pre-attached guide lines to lock into the block. Tolerances and weight require closer monitoring than with the bottom-founded tubes, because the tubes are in a state of positive buoyancy; yet this must not overstress the tethers.

Various alternative solutions to the installation have been proposed, including towing the entire tube, with all segments preconnected on the surface with tension maintained by a trailing tug. This appears to pose severe risks. A tube segment may be lowered from a catamaran to its prescribed elevation and the prefabricated tethers attached, or the tethers may be equipped with linear winches to pull the segments down.

Dynamic bending moments at the end connections will be severe and must be accommodated, both in service and while under construction.

9.6 Storm Surge Barriers

9.6.1 Description

These barriers represent a hybrid in construction technology between bridge piers and submerged prefabricated tunnels. They are prefabricated, typically in a construction basin, and floated to the site, sometimes partially supported by external buoyancy tanks or a catamaran. On arrival, they are set down on a prepared base and joined to the previously set segment, much like a tube. Gates are either pre-installed or are placed afterward, fitting in recesses.

Since they act as a low-head dam, underflow must be prevented. In most cases, this is accomplished by the installation of driven steel sheet piles, although in the case of the Oosterschelde storm surge barrier, the Dutch elected to lengthen the path of permeation horizontally so that underflow pressures would be fully dissipated. They placed filter

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fabric and mattresses over extensive areas each side of the barrier. This meant that the permeation would be limited even if settlements occurred.

9.6.2 Venice Storm Surge Barrier

This mammoth project, involving the closure of the three openings to the Venice Lagoon, has been quite thoroughly planned and engineered, and reportedly the initial phase has been placed under contract (see Figure 9.53).

Designed with optimum hydraulic shape for the sill, and carrying the heavy steel gates, these highly irregular structures presented a serious problem of stability during submergence. Use of temporary vertical auxiliary buoyancy tanks appears practicable. Control of position at the site, with substantial current running, had been originally planned to be by means of lines from the towers. This would exert tilting forces on the segments, so attachment at the center of rotation was selected instead.

The opening is to be first bounded by steel sheet pile walls at either side, and then dredged to 1 m below founding grade. The heterogeneous lenses of clay and silty sand will be pinned together and densified by the driving of closely spaced reinforced concrete piles, driven with their heads terminating 1 m below founding elevation. A 1 m thick blanket of crushed rock will be then placed.

The concrete sill segments, with gates attached, will be ballasted down to found, very much in the manner described for submerged prefabricated tunnel units (see Figure 9.53 through Figure 9.55). Like them, each end will be seated on leveling pads of precast concrete, accurately preset to exact grade. Then the new segment will be slid sidewise, while it still has little net weight and hence little friction, squeezing the GINA gaskets between segments. Subsequent dewatering of the space between the two end bulkheads will make the full hydrostatic head on the leading end available to compact the gaskets. A steel sheet pile wall will then be driven on the seaward face to prevent piping and flow under the sill.

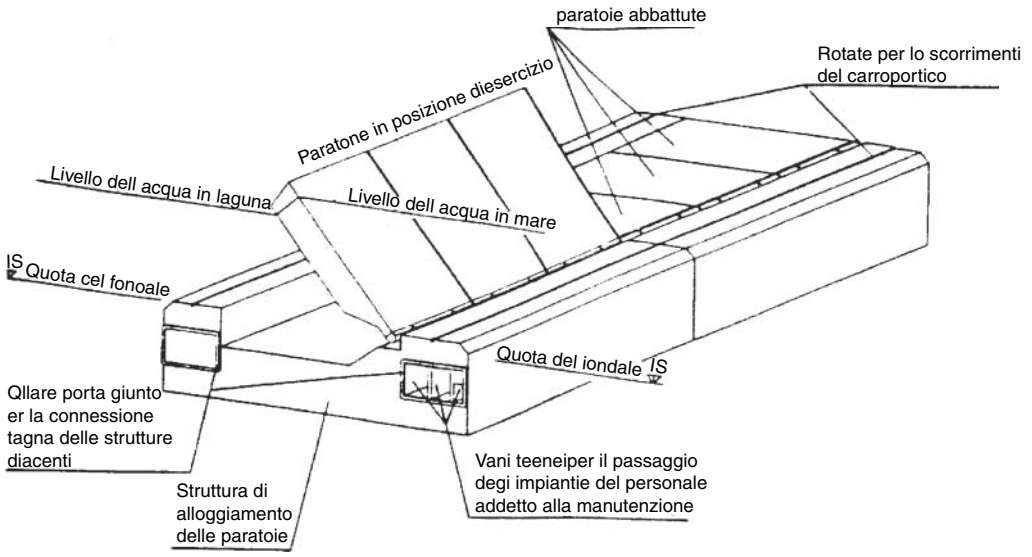

FIGURE 9.53
Venice, Italy, storm surge barrier. Concept.

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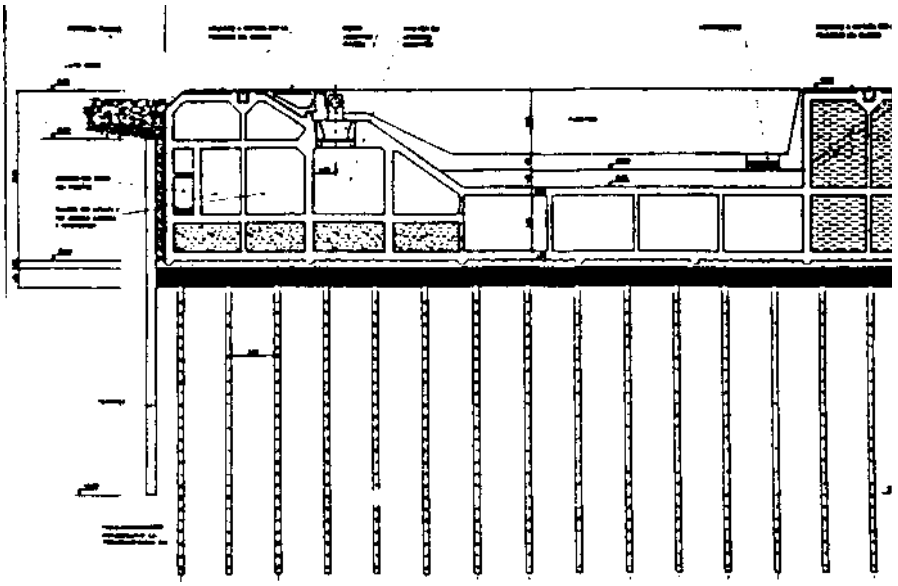

FIGURE 9.54
Transverse cross-section of storm surge barrier, Venice, Italy.

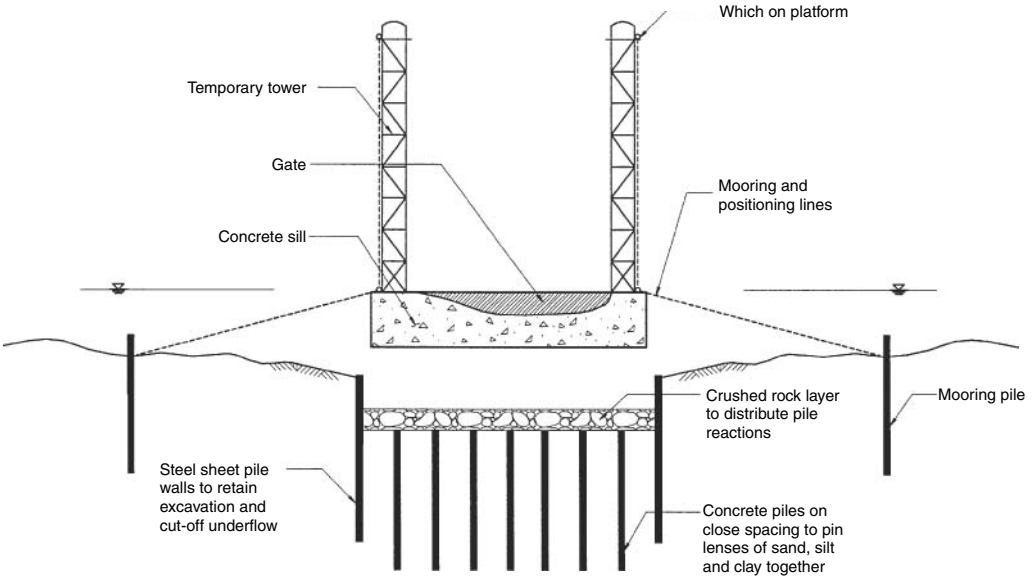

FIGURE 9.55
Original concept for installation of prefabricated storm surge barrier elements, Venice, Italy. Note erroneous location of fairleads and lack of stability during immersion. Procedures subsequently revised.

9.6.3 Oosterschelde Storm Surge Barrier

The Oosterschelde Storm Surge Barrier, across the mouth of the Eastern Scheldt Estuary in The Netherlands, must rank as one of the major offshore engineering and construction achievements of the decade (see Figure 9.56 through Figure 9.69). Although

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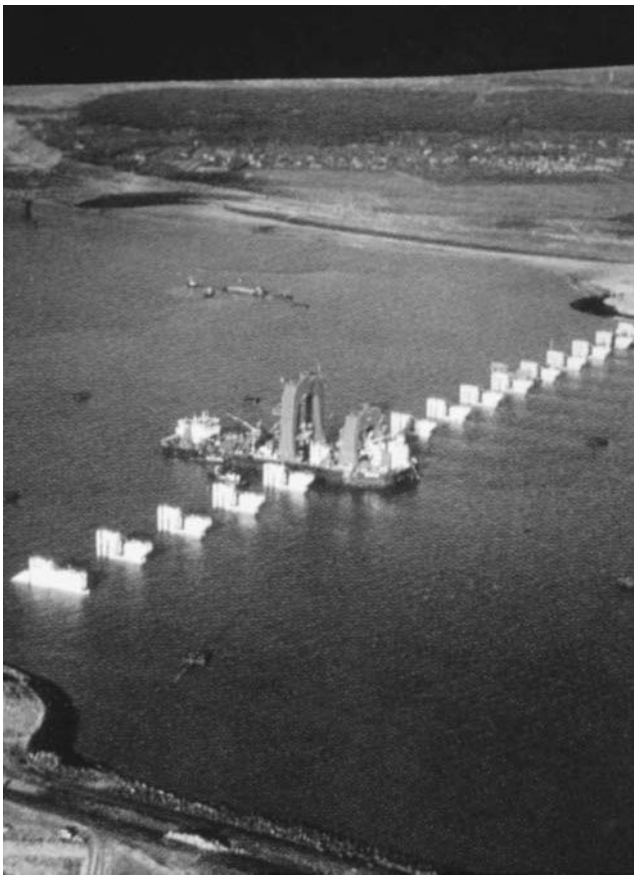

FIGURE 9.56
Oosterschelde storm surge barrier under construction. Giant catamaran crane is placing a 24,000 tn. precast concrete gate piers. (Courtesy of Ballast-Nedam.)

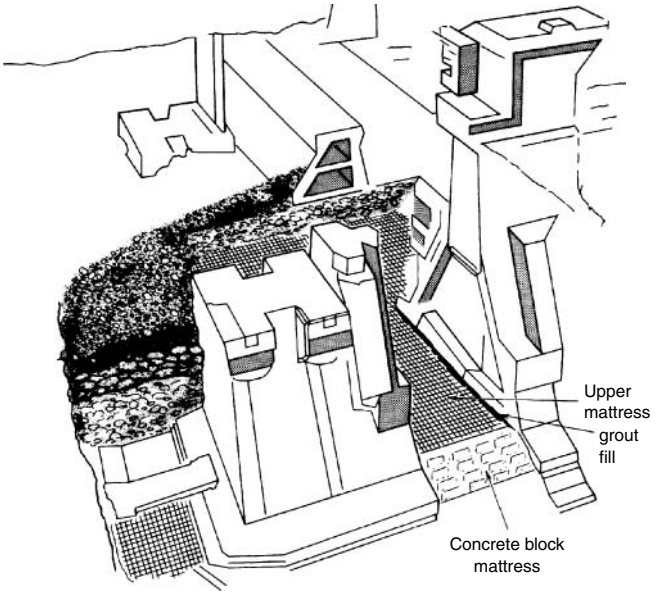

FIGURE 9.57
Isometric view of gate piers and foundation protection for Oosterschelde Storm Surge Barrier. (Courtesy of Ballast-Nedam.)

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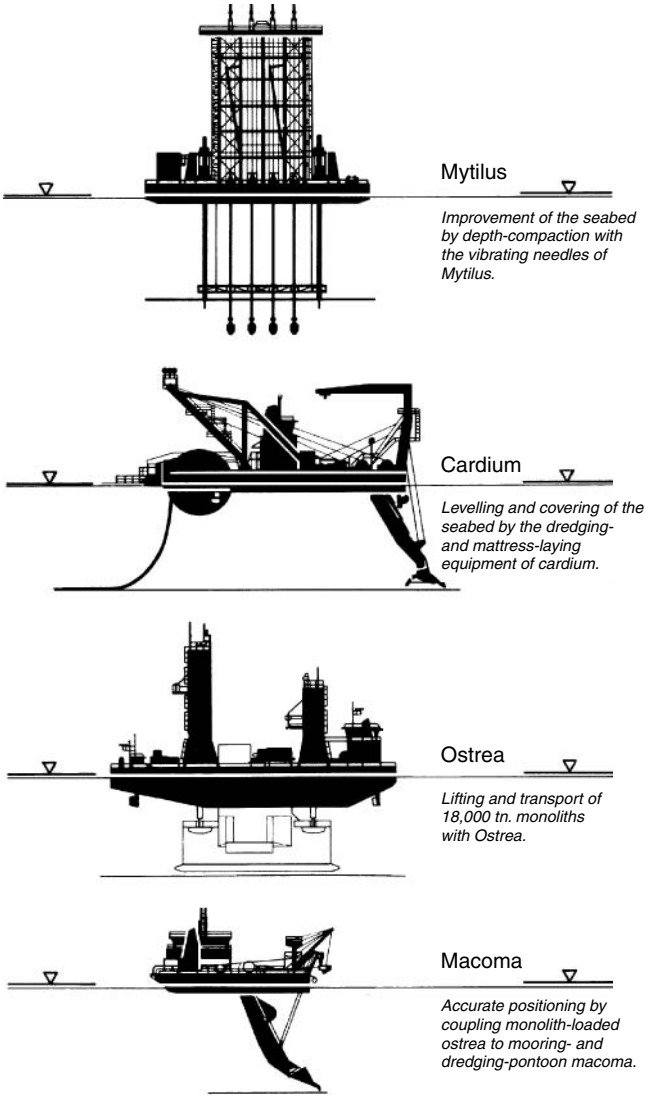

FIGURE 9.58
Specialized construction equipment used to construct the Oosterschelde storm surge barrier in The Netherlands. (Courtesy of Ballast-Nedam.)

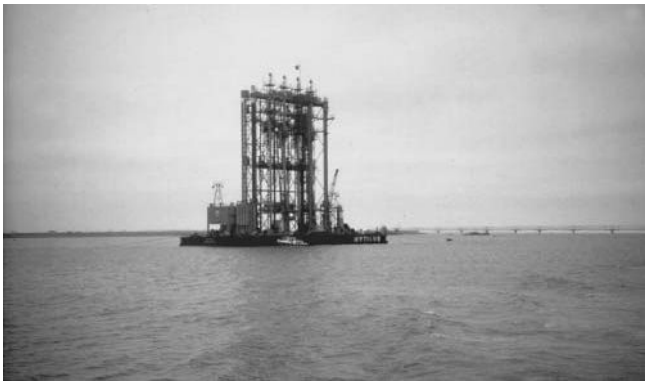

FIGURE 9.59
Mytilus densifying sands by intensive vibration.

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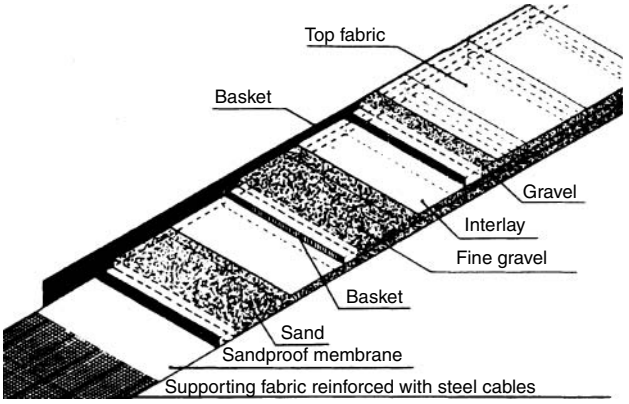

FIGURE 9.60
Foundation mattress for storm surge barrier. (Courtesy of Ballast-Nedam.)

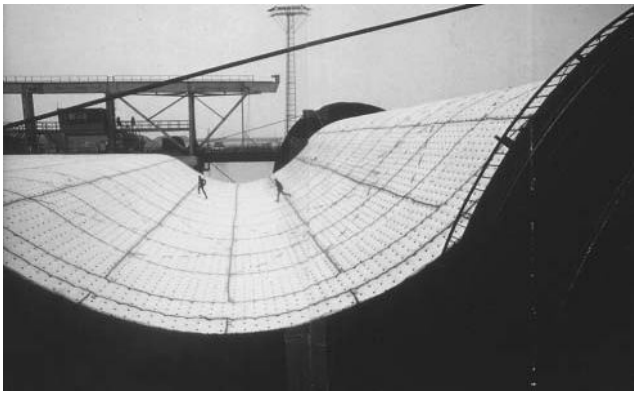

FIGURE 9.61
Rolling-up foundation mattress for transportation to site where it will be laid to prevent erosion of seafloor by strong tidal current and waves. (Courtesy of Ballast-Nedam.)

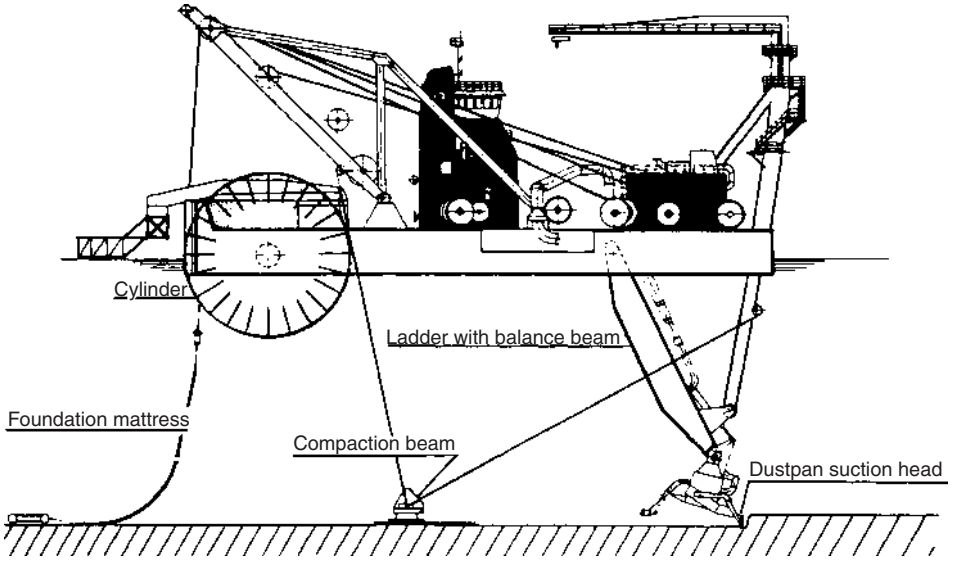

FIGURE 9.62
Clearing, compacting, and laying foundation mattress. (From World Dredging and Marine Construction.)

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FIGURE 9.63
Cardium dust-pan dredge removes silt from seabed and lays protective mattress. (Courtesy of Ballast-Nedam.)

constructed in water depths only 20–40 m deep, it is exposed to waves and winds from the North Sea and to high tidal currents, which required the development of new techniques for offshore construction that have application to future projects. The project is also notable because of the innovative development of specialized construction equipment (see Figure 9.58).

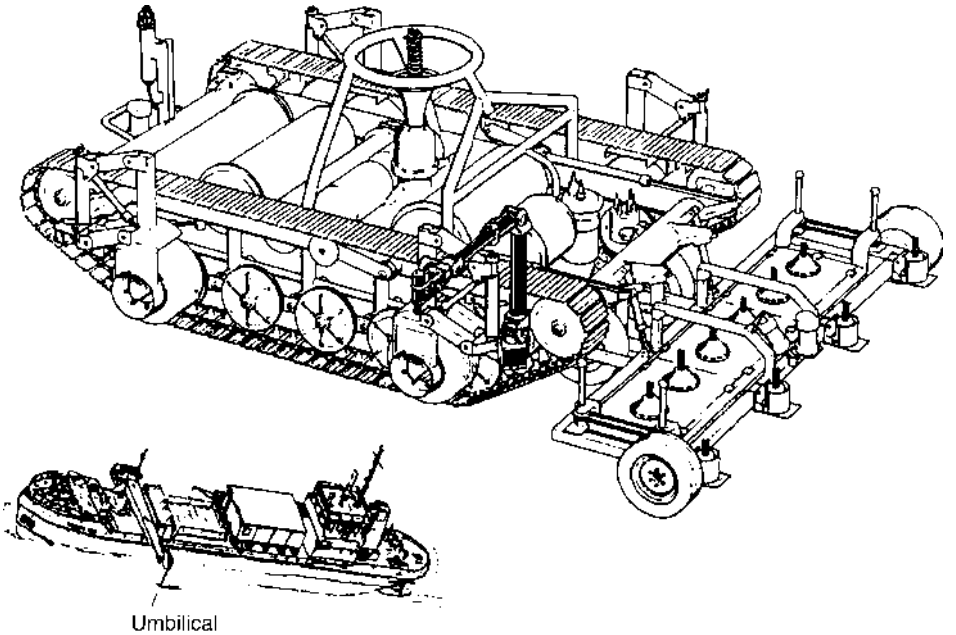

FIGURE 9.64
Underwater inspection vehicle used to ascertain variations in mattress elevations so that filler blocks can be sized and laid. (From World Dredging and Marine Construction.)

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FIGURE 9.65
Prefabricated gate pier for Oosterschelde storm surge barrier.

For the project, 66 mammoth concrete gate piers have been installed, seated on a prepared foundation on the sands of the river delta. Initial operations offshore commenced with construction of an island near the center of the project, from which all construction work was carried out. Three large basins were excavated, diked off, and dewatered to enable fabrication of the concrete piers in the dry. Concurrently, the extensive slope protection work was carried out on the beaches and dikes adjoined the barrier proper. Aprons of asphalt-filled stone, sand-asphalt, and articulated concrete mats were laid.

Meanwhile, at the site, large-diameter dolphin and anchor piles (steel cylinder piles) were driven to serve as moorings for the extensive floating construction operations to come. Lines from the anchor piles were run up to mooring buoys. The loose sands in the top 10–20 m of the foundation under the barrier were then compacted by vibratory compaction. A special floating rig, the Mytilus, jetted and vibrated four large-diameter steel tubes down to a depth up to 50 m below sea level and then actuated heavy internal vibrators as the tubes were withdrawn and sand ejected (Figure 9.53 through Figure 9.59). Spacing of the vibrated probes was $6 \times 6\ m^2$ each (see Figure 9.59).

The surficial sands of the seabed were then removed by a "dustpan" dredge, which also dragged a screeding compactor and laid out a heavy mattress behind. This mattress, consisting of reinforced geotextile fabrics and graded stone layers (see Figure 9.60) was

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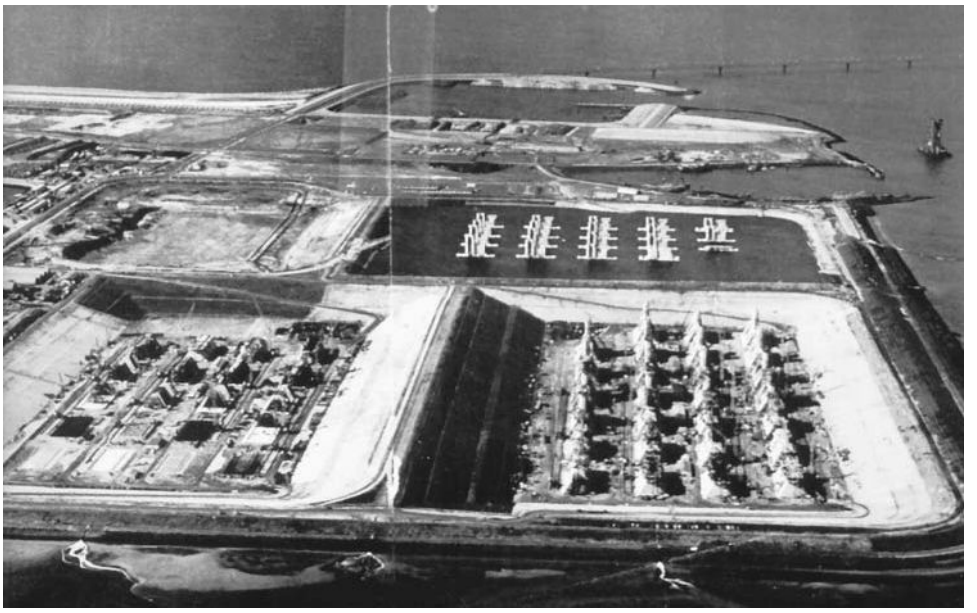

FIGURE 9.66
The 66 gate piers for the Oosterschelde storm surge barrier were prefabricated in three large construction basins.

prefabricated in an onshore plant and reeled up on a huge floating reel which was floated out and hooked up to the dredge (see , Figure 9.61 through Figure 9.63).

An underwater tracked inspection vehicle then crawled over the mattress while being tended by a floating survey vessel above (Figure 9.64 through Figure 9.66). Sonic and

FIGURE 9.67
Goliath Crane Ostrea lifting pier for transport to site.

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FIGURE 9.68
Ostrea can lift 12,000 tn. while buoyancy supports remaining weight of pier.

electronic instrumentation, plus feeler probes, enabled an extremely precise survey of each mattress to be made (see Figure 9.64). From the information so determined, an articulated concrete block mattress was tailored so that when it was laid over the lower mattress, the surface was level within a few centimeters.

Meanwhile, the concrete piers were completed in a basin (see Figure 9.65 and Figure 9.66). Then the basin was flooded, the dikes were removed, and a giant catamaran crane barge, Ostrea, moved in over a pier, raised it, and transported it to the site (see Figure 9.67 and Figure 9.68). The crane barge was mated to the mattress-laying barge, which was already properly positioned, and the pier lowered into place.

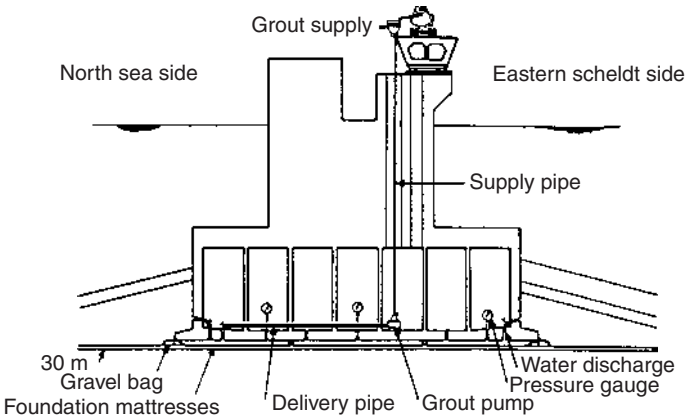

FIGURE 9.69
Undergrouting system for gate piers. (Courtesy of Ballast-Nedam.)

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After seating, grout was pumped in under the base, working from a compartment deep inside the pier so as to limit the pressure (see Figure 9.69). Extensive scour protection was placed outside each pier. Larger stones were run down an inclined ladder so as not to damage the concrete by impact. The pier was filled with sand to provide stability.

9.7 Flow Control Structures

9.7.1 Description

These include the structural steel temperature control structure for existing dams, the intake shafts in the bottom of existing lakes and reservoirs, and riser shafts for outfalls.

9.7.2 Temperature Control Devices

Temperature control devices are installed on the upstream face of existing dams in order to regulate the discharge to make optimum use of the cold water stratification in the reservoir. The discharge during winter and spring is taken from the warmer layer, saving the cold water pool in the lake bottom for discharge during the fall, thus enhancing the survival of migratory fish (see Figure 9.70).

An example is the large steel structure installed behind the dam in Lake Shasta, California. It is a structural space frame, incorporating gate guides and gates, extending to over 100 m depth. The structure is hung from massive anchorages at the top, anchored by prestressing bars grouted into holes drilled in the top of the existing dam. At intervals

FIGURE 9.70
Temperature control device to regulate discharge of water to benefit fish migration.

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below water, the structure is tied back to the upstream face by drilled and grouted dowels, with huge turnbuckles to prevent lateral distortion of the gate guides.

Consideration was given to fabricating the entire structure on shore, floating it into location on pontoons, and upending it. However, the contractor chose an alternative method, which proved more adaptable to the varying conditions on the upstream face and the varying lake levels. The first element of work was an accurate survey of the bottom of the lake immediately adjacent to the dam face, a task which had proved impossible from the surface due to multiple reflections. This work was accomplished by the use of an ROV, "flying" about 30 m above the bottom and using an acoustic imaging device. This system not only obtained an accurate profile of the bottom but also, by flying at different levels, an accurate profile of the back face, which proved to be far from a regular surface.

Saturation diving was then employed to set anchors on this upstream face. Vertical rails were mounted on which a sophisticated manipulator rode, with a battery of drills. It drilled holes in the back of the face of the dam, placed rods, and grouted them. The structural steel frames were then lowered in segments, bolted together segment by segment as they were lowered through the water plane. Turnbuckles were hung in temporary vertical attitude, hinged, so they could later be lowered to engage the anchors on the back face.

Because of the complexity of the underwater assembly of the turnbuckles and their torquing, including the limited space between the steel framework, a full-scale mock-up was made with which to familiarize the divers with what they would encounter in the limited visibility at depth. Saturation diving was employed throughout.

The work, unfortunately, had to be carried out during some of the highest lake levels on record. Nevertheless, due to the extremely careful and thorough planning, the work was completed on schedule.

Whisp'ring wind, soaring bird, gently rolling sea;
Dancing wave, flying fish, beckoning to me.
Shining sail, steady ship, heaven is my chart;
Guiding star, silver moon, call me to depart.
The rolling sea is keeper of my heart.

Kahuna Kai (traditional Hawaiian song)

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10

Coastal Structures

10.1 General

Coastal structures are characterized by a turbulent surf zone, local longshore currents, and "riptides," the latter being seaward escaping water. They are shallow, preventing or at least limiting the use of floating equipment. The soils are typically unstable deposits of sand and gravel, which may be seasonal in nature, piling up on a beach in summer and migrating to form an offshore bar in winter. In some cases, structures are built on steep rocky shores.

Access is a primary problem for equipment, personnel, and materials. Steel pile trestles are often built through the surf zone in order to provide access from above the waves. Trenching or excavation in the beach sands usually requires the building of steel sheet pile walls to keep the sand out. The beach sands move parallel to the beach under the action of the prevailing longshore current, and when sheet pile walls are built normal to the beach line, sand will build up on the upstream side and erode downstream.

The waves typically refract around in the shallow water to break more or less normal to the beach line. They also refract laterally from a natural or man-made trench to concentrate their energy along the sides. The sands, being loosely deposited, although subject to densification under the pounding of the waves, are subject to liquefaction when their pore pressure builds up, as it may from the continued vibration of sheet piles by the breaking surf. Large stones and seawall caissons may similarly cause the sands beneath to liquefy locally due to their cyclic rocking. All of these actions are, of course, intensified during storms.

Offshore terminals are typically constructed in semiprotected waters, although some are exposed to violent storms during a typhoon. Thus, they represent a transition between inland marine and offshore construction, with lateral forces dominating the design, and constructability dominating the access and construction.

Coastal pipelines for intake of seawater and for discharge of industrial wastes are typically of large diameter (e.g., 3–4 m) and made up of concrete pipe segments. Those for oil and gas are of steel and are typically less than 1 m diameter. The construction of these latter are covered in Section 12.4.

10.2 Ocean Outfalls and Intakes

These are usually large-diameter reinforced concrete or steel pipelines, although in a few instances fiberglass pipes have also been used. They are laid in a trench out to the depth

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where the storm waves break, typically 10–20 m. From there on out, they may be laid on the surface or in a trench. Riprap is placed on both sides up to the spring line to prevent lateral displacement, and may also be placed over the top to prevent uplift and damage from ship anchors.

Through the surf zone, trestles and sheet pile trenches are constructed. Although the water may be shallow, the trench increases the effective depth substantially. Hence, the trestle requires bracing, especially in the transverse plane. Prefabricated templates are often used, with sleeves through which the trestle piles may be driven and to which they may be fixed. The trestle often requires bracing in the horizontal plane as well, to make the structure perform as a lateral girder to resist the unbalanced and out-of-phase forces from the waves as they travel down each side.

The sheet piles for the trench are driven along each side and tied to the trestle, since forces will be both inward and outward as the crests and troughs pass. The end of the trench may have to be closed by a temporary sheet pile closure to prevent surges inside that would prevent the setting of pipe sections. The presence of the "wall" of sheet piles causes waves to be out of phase as they travel down the sides (see Figure 10.1). Thus, the trough and crests oppose each other, causing the entire structure to oscillate back and forth. The sheet piles may develop fatigue. Thus, bracing in the horizontal plane is needed.

Once the trestle and trench are complete, the pipe sections are set, with open ends. They are lowered just ahead of their design position, and then moved end-0 to enter the spigot into the bell. Bell and spigot pipe is typically laid with the bell leading. The forward end of the pipe is adjusted to grade, often by placement of a small amount of rock beforehand, or by sandbags afterward. After entry of the spigot into the bell, the new pipe must be forced into the bell the proper distance to engage the O-rings. The tightness of this joint is verified, first, by marks on the new pipe to ensure it has been fully entered and, second, by an air pressure test on the space between the double O-ring gaskets (see Figure 10.2). Backfill is then placed uniformly on each side to ensure against displacement.

Once the pipeline is through the surf zone, it may be laid directly on the seafloor. A uniformly screeded bed has to be constructed on which to set the pipe. This can be best done by a screed-frame and screed, riding on rail girders which have been set to proper profile. The stone is fed down a tremie pipe from a hopper. It is then distributed and

FIGURE 10.1
Heavy seas pound steel sheet pile trestle for construction of power plant outfall, Moss Landing, California.

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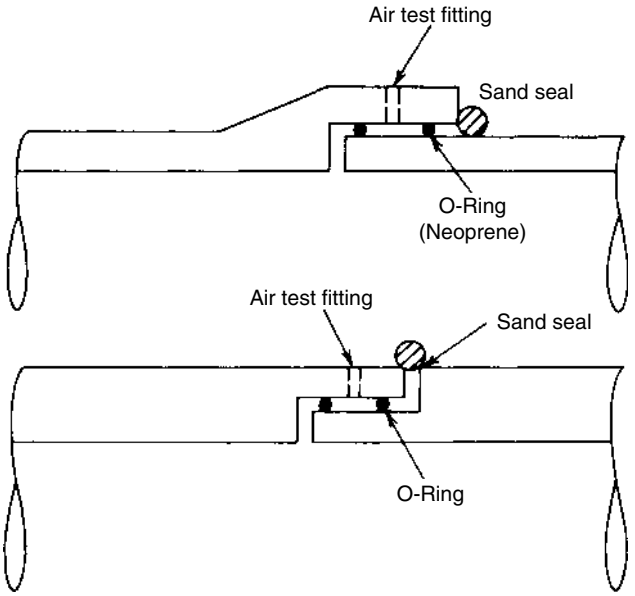

FIGURE 10.2
Sealing joints of concrete pipe segments.

graded by the screed. In the case of the very deep (100 m) outfall extension at San Diego, the existing seafloor was screeded by a horizontal augur riding on girder rails, removing excess stone from the bed, which had been placed by tremie pipe.

Pipe sections in the open ocean have to be laid by means of a cradle or ("horse") which is supported on the seafloor. The horse has hydraulic controls, which enable it to adjust and move a pipe section in all degrees of freedom. The horse is equipped with video and with cages in which the divers can ride.

A crane barge brings the horse above water and seats it over the next pipe segment. The pipe is clamped into the horse. Pockets on the side of the horse are filled with stone. Then, the crane barge lowers the entire assembly onto the seafloor, just seaward of the last section of pipe. Using video and diver control, the pipe is aligned and seated into the bell of the previous section (see Figure 10.3 and Figure 10.4). To apply a longitudinal force with which to seat the spigot in the bell, an innovative solution has been to install an end bulkhead in the new pipe segment. This is inflated to seal. A small pump is activated after the pipe segment has been initially seated; this creates an underpressure so that the excess external pressure drives the joint to full closure. Divers or video from an ROV should then verify a uniform gap around the circumference. The end bulkhead is then deflated and pulled out for re-use. An alternative system is to run a wire line through the pipe which the diver pulls to the seaward end of the next segment and attaches to a crossbar. This can then be used to pull the new segment into its bell.

Crushed rock (50 or 75 mm max) is discharged from the hoppers on each side and worked underneath by jets or vibrators attached to the cradle. The pipe is now released from the cradle (horse). The joint is tested and verified by pressurizing the water between a double O-ring at the joint. Then the cradle is raised to the surface and landed on the barge deck over the next section of pipe. After the pipe has been laid and bedded, backfill of small rock is placed up to the spring line. In some special cases, the pipe may be completely covered. Riprap is then placed to protect the rock backfill from wave erosion and from ship anchors.

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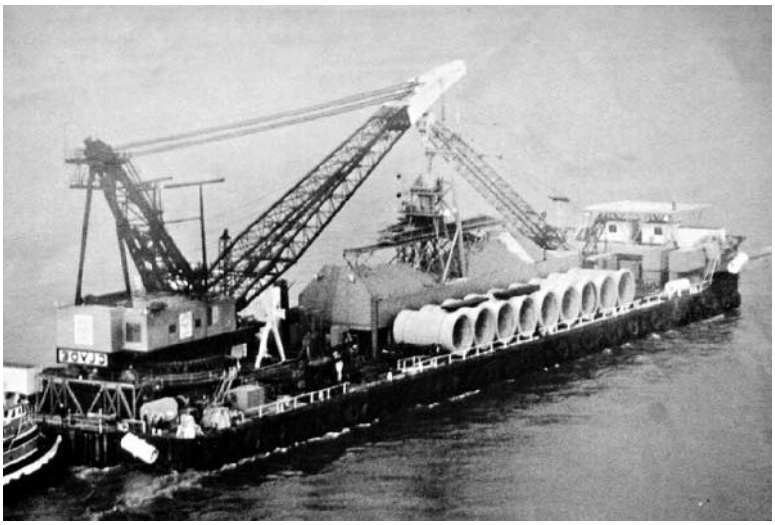

FIGURE 10.3
Crane barge loaded with concrete pipe segments enroute to installation site, San Francisco, California, Ocean outfall.

While the rock backfill may be placed through large-diameter (e.g., 1 m) tremie pipes hung from a barge, it may also be placed by skips. The skips are also used for placing the riprap. Large rocks allowed to free-fall from the surface will severely damage the pipe on impact.

North of Mumbai, India, a contractor attempted to lay pipe directly from a floating catamaran barge. This procedure has worked well in inland lakes where there is no swell nor wave-induced motion of the barge. The catamaran floats over a narrow trestle at the shore, on which a pipe section has been positioned. The catamaran picks up the pipe

FIGURE 10.4
Cradle ("horse") with pipe segment. Crane barge lowers this to seafloor and guides pipe into joint, San Francisco Ocean Outfall.

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section, moves to the site, and lowers it to near the seafloor. Guides and a line extended from shore, through the already laid pipeline, pull the new section into the joint.

In the open sea, however, this method is inherently flawed. Long-period swell energy from distant storms will cause the pipe section and the barge to surge, even when the sea surface appears to be almost calm. In the case of the Mumbai outfall, joints were repeatedly damaged by impact, and the procedure eventually had to be abandoned. Similar difficulties have occurred when attempting to place pipes directly from a crane barge. Several attempts have been made to install large-diameter lines by preassembling them in long lengths on a beach or in a protected harbor, then floating them to the site and lowering them by a combination of ballasting and support from surface pontoons. Unfortunately, catastrophic failures have occurred. While ballasting with no internal subdivisions, the water tends to run to one end or the other, in effect a very extreme "free surface" effect. Even if control of the two ends is maintained equally, deflections at the center may cause very large bending movements as the water increasingly "ponds" in the sagging center. Swells produce high bending stresses as well as hydrodynamic lift or pull down. Eventually, the joints have failed and the line has been abandoned.

When fiberglass lines are used, they are usually laid with open ends, and ballasted down by concrete saddles installed on top. Fiber straps with softeners are used to fix the saddles in place so they do not chafe through the fiberglass. Similarly, lowering slings must be softened. Being flexible, if they buckle during installation, they regain their shape when laid on the seabed (see Chapter 16).

Some outfall pipelines have been constructed by means of a jack-up, which in effect acts as a giant "horse." It is then necessary to control the pipe segment from lines reacting against the legs. Alternatively, a modified cradle can be lowered from the jack-up. Floating a barge with pipe segments between the legs of a jack-up has inherent risks in the open sea, since the barge or pipe may impact a leg and bend it. Another problem is the time required to jack-down and move ahead after laying a series of three or four lengths.

Laying of outfall pipelines on pile-supported underwater grids or caps also has inherent problems (see Figure 10.4). During a storm, the uplift forces from the upward orbital trajectory of the wave particles tend to lift the line and may either dislodge it or cause its eventual failure in fatigue. Hence, the pipeline needs to be adequately tied (locked) to the piles to resist uplift.

Diffusers are typically incorporated into special pipe segments, often purposely offset from the pipe centerline, and hence are not only heavier than the normal segment but have an eccentric distribution of load; thus, special rigging is required. Once laid, riprap is usually deposited around them to give them protection from ship anchors. This riprap has to be laid very carefully to avoid damaging the diffuser riser. Timber protectors may be temporarily affixed. A skip or bucket is used, usually guided by video. Then a diffuser cap is placed.

Increasingly, outfalls are tunneled under the surf zone, partly for practical construction reasons, such as the difficulty of construction in such a dynamic zone, but mainly for environmental and ecological reasons, to avoid disturbance at the beach crossing. Then risers are run up to seafloor diffusers.

In the case of the Boston outfall, which extended nine miles into the ocean, an additional reason for tunneling was to ensure stability of the line in the relatively shallow water during a hurricane. In the project, fifty-one 1 m diameter risers were first drilled, cased, and completed, using a jack-up rig and high-order surveying. The tunnel was then driven past them, with 2 m clearance. Probe holes established initial connection to each shaft, which had been filled with dyed seawater. Then the connections were hand mined. Temporary caps at the seafloor isolated the casing from the sea during construction.

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In the case of the South Bay outfall, on the border between the United States and Mexico, the tunnel extended 5000 m under the sea. While the tunneling was in progress, a riser shaft was constructed from a temporary steel platform set in 30 m of water. After glory hole excavation of 10 m of sediments, a 4.5-m casing, thick-walled (50–60 mm), was set and driven and drilled down 45 m, excavating internally to remove the dense siltstone, which was interspersed with lenses of gravel and cobbles. Although the actual work was performed by airlift removing the gravel and divers removing cobbles by hand, it is believed that the removal of cobbles could have been done more efficiently by hammer grab. It was then cleaned out to near its tip. A second casing, extending above the sea level, was lowered into the first and driven on down to a final tip of −68 m. Driving was done by the hydraulically actuated drop hammer described in Section 8.4. It developed a theoretical 270,000 kg-m (1,900,000 ft.-lb) of energy per blow, reduced by about 40% due to friction and eccentricity of blow. The tips of both casings were thickened to form an internal driving shoe.

A full-face rotary drill was now set on the casing and drilled out 9 m below the tip of the casing using bentonite slurry, then belled out with a belling tool to 6 m diameter. This hole was then filled with a tremie concrete mix, 10 mm coarse aggregate, designed to have a strength of at least 7 MPa at 28 days but not above 14 MPa at 90 days. This upper limit turned out to be a difficult specification to meet but was finally met by use of a mix containing minimal cement, fly ash, and limestone powder. The reason for the limitation on the maximum 90-day strength was to enable the TBM tunneling machine to drill through it so that the connection to the riser shaft casing could be hand mined within the protection of the concrete plug. The reason for the minimum strength requirement was to ensure safety even if full water head developed at the periphery, since the dense siltstone-mudstone was very susceptible to degradation when in the presence of water. In actuality, the full water head did develop. Some remedial grouting was employed. The connection was successfully completed. An alternative solution to the tremie plug would have been freezing but would have required more time.

Seawater intakes are constructed at coastal sites to provide cooling water for power and industrial plants and clean seawater for aquaculture. They are designed to minimize the intake of sand, seaweed, and other marine life.

Intake pipelines are typically installed through the surf zone and on out to a water depth of 5–20 m, where the intake structure proper is emplaced. The construction techniques are similar to those for outfalls. Joints must be gasketed tightly to avoid sucking in sand, since the flow velocities are usually greater (Figure 10.5).

To prevent sucking in of fish, turtles, and other marine life, velocity caps are often installed above the mouth of the intake, designed to reduce the velocity of the water flow. These velocity caps are flat concrete plates set on pedestals above the mouth.

At the St. Lucie Power Plant on the east coast of Florida, the caps were broken loose and thrown on the adjacent seafloor by the cyclic uplift from storm waves and swells. The replacement utilized much thicker and hence heavier slabs.

Divers are typically required to aid in making and inspecting joints. They must be protected against the surge of the surf in the shallow water. Use of heavy suits and tethering to a taut wire line or support in a cage on the horse may be needed.

Intakes are sometimes built on steeply sloping rock shores. The trench is excavated through the rock by drilling and blasting. After the pipeline has been laid, riprap, tremie concrete, or grout-filled bags may be specified to backfill the trench. Tremie concrete can be stiffened by addition of silica fume or other viscosity-enhancing admixtures to the mix and reducing the high range water-reducing admixtures, thus reducing the slump, to allow it to be laid on a moderately steep slope. However, a minimum slump of 75–100 mm is generally required to ensure flow. Thus, the tremie concrete cannot be

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FIGURE 10.5
Submersible pump removing sand from seafloor trench. (Courtesy of Toyo Pump Co.)

placed with a top surface sloping more than about 1–5. Where the slope is steeper, it will have to be retained at intervals, e.g., by a rock dike or grout bags. The intake structure, velocity cap, and duct segments are generally set by a crane barge, moored to heavy clump weight anchors. In the absence of any significant swell, taut wire guidelines and a setting tower may be used for setting the segments. Trench surveys can best be carried out by wideband acoustical imaging, which is unaffected by turbidity. Agitating sand pumps as well as jets can be used to remove silt just prior to setting a segment.

Setting from shore to seaward on a steep slope may require hold-backs for each segment, until it is well embedded. However, this permits more tolerance for alignment. Setting the intake structure first and then setting shoreward require very careful survey to ensure accurate profile and alignment to obtain a designed position at the landward end. Moderate-size intakes (up to about 1500 mm diameter) may be constructed by directional drilling, as described in Section 15.12.

Where there are neither swells nor significant waves, as in a lake or existing reservoir, the setting of pipes for either outfalls or intakes is not subjected to the heave of the setting barge. Then, a job-built catamaran or barge-mounted crane may be used in water deep enough to float. The new pipe segment may be set and the joint made up, using some divers or a guide to automatically ensure proper position. There will still be some impact, so some form of cushioning, such as neoprene pads, should be employed. However, in the open ocean, some low-height but long-period swells almost always occur, even in calm weather, with the result that heave makes it very difficult to mate the pipe. As a new section is laid, impact has often damaged the joints of it or the previously laid pipe. Since

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long-period swells extend to the depth of typical intakes, the oscillating surge action will also cause difficulty and endanger any diver who is working near the joints. Thus, the use of a cradle or "horse" usually becomes necessary.

In the case of intakes in existing lakes or reservoirs, the concept may consist of a vertical shaft, which is subsequently intersected by a tunnel. The same construction problems may be encountered as with a riser shaft joining an outfall pipeline. The shaft may need to be constructed in deep water.

That in Lake Mead, Nevada, formed by the Hoover Dam, was constructed for the Southern Nevada Water Authority: it starts at a depth of 60 m and extends down through granitic rock another 30 m. In this case, a down-the-hole drill was used through a casing. After the seafloor had been leveled, a large steel frame was set, using four drilled-in guide piles to enable it to be fixed on an accurate level plane. Then a template was set on the frame. A down-the-hole drill, working through the casing, which was successively positioned in the holes in the template, drilled closely spaced holes to completely perforate the area of the shaft. The drill cuttings were raised by air-assisted circulation, and then flowed down a tremie pipe for discharge on the seafloor. Deposition on the seafloor was permitted since no drilling fluid other than water was used. Repositioning of the drill to each successive hole in the template was controlled by an ROV with video image at the control shack on the barge. The rock in the interstices between the holes generally broke down without additional action. However, if any ribs did remain, a spud and heavy clam shell bucket such as a hammer grab were employed (see Figure 10.6 and Figure 10.7).

When the shaft had been excavated to final depth, the permanent steel casing with a temporary dome on the tip was set into place by progressively ballasting with water. Since upending by water flooding creates high bending stresses as well as hoop stresses due to hydrostatic pressure, the process had to be designed to ensure against buckling. Then, the casing was anchored to the walls of the shaft by tremie grout around the tip. When that had hardened, a short height of annulus was progressively filled with grout placed through tubes which had been welded to the outside of the casing (Figure 10.8).

After this short length had been concreted and gained strength over two or three days, the casing was ballasted so that it would not float in the grout. Centralizing fins on the casing ensured a minimal annulus width. The annulus was filled with grout, using additional tubes welded to the casing. A collar was then placed at the top. The tunnel

FIGURE 10.6
Drilling rig constructing shaft in 60 m of water. (Courtesy of Southern Nevada Water District.)

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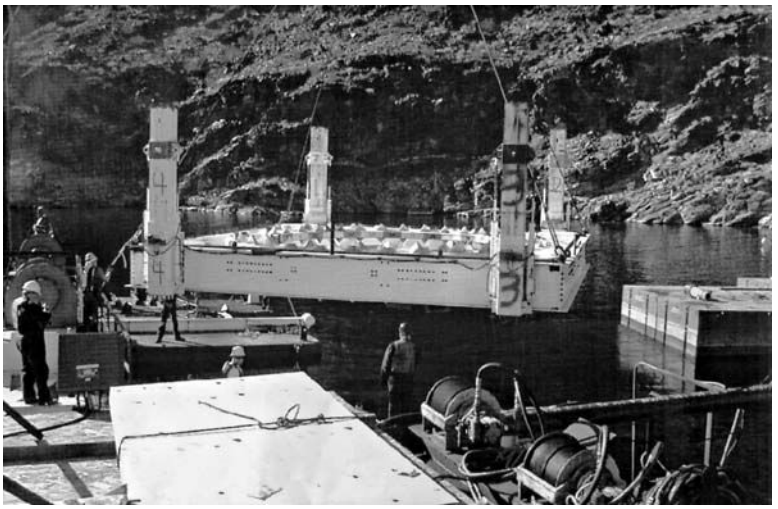

FIGURE 10.7
Frame for drilling template in deep water. (Courtesy of Southern Nevada Water District.)

was then driven to the intersection, with its invert just below the tip of the shaft casing. Grout holes were drilled to encase the connection zone and grouted. Then the connecting elbow was hand-mined.

At the similar Roosevelt Lake intake, the elbow was pre-attached to the casing. The shaft excavation was ovalled out to allow the elbow to pass.

An outfall, also in Lake Mead, but in a different, deeper area, is to be built so as to discharge waste just below the thermocline. The pipe itself will probably be HDPE, with concrete saddles to keep it in place, but the discharge has to be adjustable so as to maintain its proper location despite changes in the lake level. Support from the surface, the obvious

FIGURE 10.8
Installation of steel casing into pre-drilled shaft in 200 ft. of water, Lake Mead, Nevada. Note spacers to maintain annulus for tremie concrete infill. (Courtesy of Southern Nevada Water District.)

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solution, is not currently environmentally acceptable, so other means are being investigated, including telescoping and hinged devices placed perhaps 500 ft. below lake level.

For an intake to a nuclear power plant south of Shanghai, four relatively short pipelines were laid in a rock trench on a 20° downslope. An intake structure was constructed at each of the four lines. At this location, only partially protected from the Pacific Ocean, tidal currents reach 7–9 knots, producing tidal bores and moving a bed load of suspended loess silt.

Heavy concrete pipe segments were prefabricated on a large barge, as were also the intakes. Weights were up to 800 tn. At the site, 100-tn. concrete block anchors were laid for the crane barge moorings. The rock trench was drilled and shot, using line drilling and cushion blasting, with compressed air, so as to reduce excessive overbreak, fracture of the rock and damage to the already constructed shore-sited plant.

After installation the trench was backfilled with tremie concrete. Concrete was designed with high viscosity so as to stand on a slope and the concrete placed from seaward up.

10.3 Breakwaters

10.3.1 General

There are many types of breakwater structures, depending on the wave climate, the slope of the beach and the depth at the desired location of the breakwater, and the availability and cost of the materials. The most common types are the rock mound breakwater and the concrete caisson breakwater, or a combination of the two.

Since breakwaters are being constructed in ever more severe environments, their designs are becoming increasingly sophisticated as a result of advanced understanding of hydrodynamics of wave interaction with the structure and the sloping bottom. Experience has been accumulated worldwide and translated into these improved but more complex designs. For the constructor, this means the positioning and placement are very demanding, especially in an environment where the waves are typically steepening and refracting.

Concurrently, there have been significant improvements in methods of control and survey. Crane barges are larger, with more stability. Cranes have greater reach. Mooring systems are available with increased holding capacity for taut-line moors. Buckets (grabs) can now be fitted with electronic position indicators connected to GPS or DGPS and read out in the operator's cab. Acoustical profilers can give real-time two-dimensional cross sections, automatically corrected for roll, pitch, and tidal variations, while photogrammetry can give accurate three dimensional pictures of the above-water portions.

10.3.2 Rubble-Mound Breakwaters

These are usually constructed with a core of quarry-run rock, overlain by one or two layers of larger rock carefully sized to prevent leaching out of the fines from the core. On the top and sea sides, large riprap or concrete armor units are installed.

In selecting the quarry for production of the required rock, primary consideration has to be given to ensuring that the requirements as to durability and abrasion resistance can be met. Rock will be required in different sizes: the most critical will be the large armor rock. The development and blasting plans for the quarry have to ensure that an adequate quantity of each size can be obtained. These plans must include temporary roads so that the hauling to the sorting and stockpiling areas can be carried out efficiently.

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Blasting will usually be done with widely spaced, large-diameter holes, and slow powder, such as ammonium nitrate, in order to produce large individual pieces for the armor stone. The other rock is put through a grizzly (grid) in order to separate the Class B rock, which is the next largest size. Subsequent grizzlies used during loading can remove unwanted fines from the core rock.

Rock tends to break along preexisting fractures. Examination of the quarry face will indicate whether the stone will break in the desired polyhedral shape, or in unusable slabs. Abrasion resistance is normally determined by the Los Angeles Rattler Test. The handling of rock in the quarry is normally by loader, although a crane may be used to separate and stockpile the armor rock. Care must be taken in handling and loading the larger rock to prevent excessive breakage of the edges and corners. Transport of the rock is either by barge or truck. When a barge is used, the deck is protected by timbers.

Where the core is placed on a sand seabed, either small stone particles or a filter fabric must first be laid, to prevent the sands from migrating up into the interstices of the material above. During storms, excess pore pressures develop in the underlying sands, causing local liquefaction with the consequence that if a proper filter has not been placed, the breakwater "sinks" into the sand.

Filter fabric is a good alternative to the finely graded stone but requires special means in order to place it and to keep it temporarily in place. It can be fixed to large frames made of steel angles, which stay in place, or made up as a mattress, with articulated concrete blocks attached. This latter is the practice in The Netherlands, where they first roll up the mattress on large buoyant steel drums, then later unroll it on the seafloor.

The core may be placed by bottom-dump or side-dump barge. Tests and experience show that lateral dispersion and segregation may be greatly reduced by pre-saturating the material on the barge to eliminate the entrapped air. Placement in a mass also tends to prevent segregation, although the impact in deep water will cause lateral windrows to form. The core can also be pushed off barges, but the smaller rocks tend to glide laterally and segregate, whereas the larger stones go straight down.

Designs often call for relatively thin layers of progressively larger stones. In some wave climates, accurate control of these separate layers of different gradations becomes almost impossible. Pre-blending of two different layers may then be done, with the approval of the engineer. The layers then tend to automatically grade from finer inside to coarser outside provided the blended gradation is placed through a pipe or lowered to the seafloor in a bucket or skip. The next layer, usually referred to as Class B rock, typically 400 mm maximum, may best be placed by a skip or net although a large bucket is also used. Finally, the largest rock is placed, typically by grab from a crane (see Figure 10.9 and Figure 10.10).

Because the maximum obtainable size of riprap is limited, breakwaters on exposed coasts often use precast concrete armor units (see Figure 10.11). These armor units are designed to be stable against roll and displacement by the waves, and at the same time, to give a porosity that will dissipate the wave energy. A great deal of experimental research in both laboratory and field trials has been carried out. As a result, a Corps of Engineers Manual listed more than forty different configurations. In recent years, much attention has been directed toward the actual performance under severe storms. It has been found that many of these sophisticated shapes tend to break up into large fragments, which then act to batter the remaining units.

It was found that much of the cracking originated during the casting, due to thermal strains and the restraint of the forms. As a result, the steel forms have been redesigned to accommodate this thermal shrinkage during subsequent curing, but the friction of the support still causes significant strains. Many tests and actual installations have been made with the use of reinforcing steel. This has served to pull the cracks closed and to hold the legs of the unit to the body, but has not completely solved the problem. It is, of course, very

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FIGURE 10.9
Placing Class B rock in breakwater, Los Angeles Harbor, California.

costly. Other steps, which should help, would be the use of blast-furnace slag cement and/or fly ash, to reduce the heat of hydration, precooling of the concrete mix, and insulation of the steel forms and the unit itself during the curing period. Due to the thickness of the members, drying shrinkage is a minor problem.

During concreting of shaped units containing reinforcing steel, there is a tendency of the steel to displace upward, i.e., to "float" during placement and vibration. This results in the steel being either exposed or having insufficient cover to prevent corrosion. Concrete dobe blocks should be fitted to hold the bars in place, both from their own deadweight and from uplift. Steel fibers have also been used to improve the apparent tensile strength of the concrete with relatively good results. However, the quantity of fibers required makes the costs very high. As a result of these problems, Dutch engineers have returned to the use of

FIGURE 10.10
Placing Class B rock in breakwater for nuclear power plant intake, Southern California.

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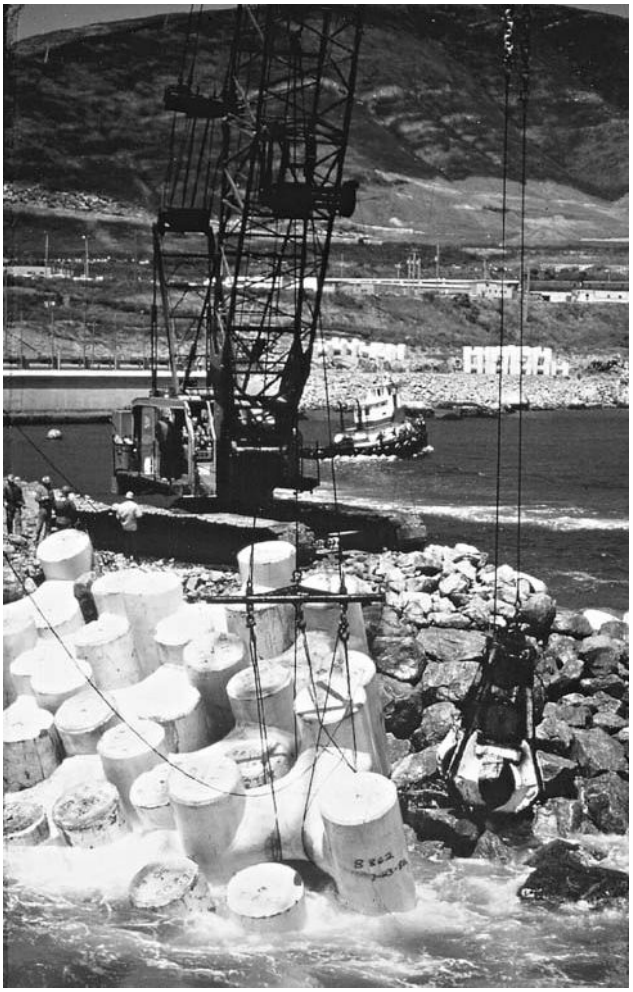

FIGURE 10.11
Precast armor units for breakwater.

unreinforced solid rectangular prisms, each weighing 1 MN (100 tn.). These are randomly placed on the slope.

Concrete armor units are placed with slings, so that the unit may be oriented and placed with precision working from the lower end upward. They are usually placed with one or two segments per layer. The slings are fitted with releases that can be tripped from above. A single line sling is used to a drilled-in rock anchor, with a bridle so that the positioning may be visually controlled from above the water. When laying from floating equipment, a large crane barge is positioned inside the working end of the breakwater to be partially protected from the seas and to minimize the risk of grounding (see Figure 10.12).

The U.S. Corps of Engineers has developed a special shape of concrete armor unit called Core-Loc which has good porosity and improved structural strength. The Accropod, of similar but heavier configuration, is currently believed to be the most reliable for extreme exposure (Figure 10.13 and Figure 10.14).

Survey control of the rock mound breakwater presents certain difficulties due to the fact that the critical points are in a zone of breaking and reflecting waves. Transverse profiling can be run on major breakwaters by side-scan sonar and a subsea profiler. Aerial

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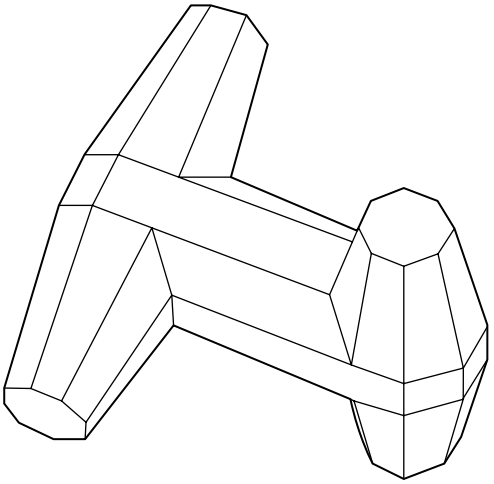

FIGURE 10.12
DOLOS breakwater armor unit.

photography can give contour of the above-water sections. On smaller breakwaters, more conventional means can be used. A crane on top can reach out to hold a lead line with a heavy sounding lead, large enough so as not to go down a crevice. The line is located in the horizontal plane by conventional theodolite. Further off the centerline, helicopters can be used in a similar fashion. The lead line should have a weak link so that if it gets caught, it will not endanger the crane boom or the helicopter. If the sea is calm, then, of course, a boat can be used with both lead line and sonar. Sonar must be narrow beam, and even then, correction must be estimated for the spread of the beam since the sonar will measure the nearest point in its cone.

For breakwaters that must traverse shallow water or tidal zones where floating equipment cannot work, the access is "over-the-top," employing a trestle or working on top of a widened section of core. Provision of sufficient width for the crane on top often requires additional material in the breakwater, not only core rock but also in the other layers over

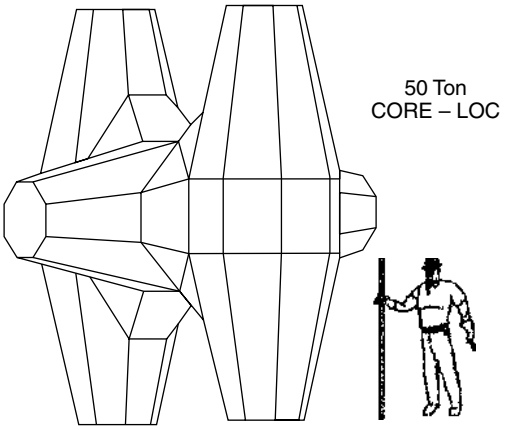

FIGURE 10.13
Core-Loc breakwater armor unit.

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FIGURE 10.14
Three different placement methods of breakwater construction: jack-up, crane, and crane barge. (Courtesy of Weeks Construction Co. and PG&E.)

the cap as well. Core rock and stones are transported over the core by large truck. At intermittent locations, the core is widened still further to provide a turnaround, or a turntable is provided.

When trestles are employed, the crane may be mounted on a rail undercarriage and the rock delivered by rail car (see Figure 10.15).

Frequently, concrete caps are constructed on top of the breakwater. These must have adequate relief holes since otherwise, the hydraulic ram effect of plunging waves on the open interstices below will generate large internal pressures that will blow the cap off, as it

FIGURE 10.15
Rail-mounted crane placing tetrapod armor unit.

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did on the initial installation of the breakwater for cooling water intake of the Diablo Canyon nuclear power plant in Southern California.

Similarly, the interstices in the above-water outer layers are often chinked with smaller stones, and then grout or concrete is placed to prevent rats. Sufficient pressure-relief holes must be left open, or otherwise, they will open themselves during a storm with disruption of the surrounding stone. For placement of grout or concrete in the tidal zone, silica fume admixture gives both stiffening and anti-segregation capabilities; that is, the concrete is more cohesive. Antiwashout admixture will prevent segregation but the set time will be lengthened.

The outer corners and exposed bends in the breakwater receive much more damaging attack, in part by wave concentration but also aggravated by the wave-induced currents. Additional armor and a thicker cross section are usually adopted.

The stability of individual rocks in the breakwater is a function of their specific gravity and is proportional to the cube of the underwater (buoyant) weight. Thus, trap rock, with a unit weight of 3100 kg/m^3 (195 lb/cu. ft.) is more than twice as stable as normal siliceous rock at a unit weight of 2600 kg/m^3 (165 lb/cu. ft.).

10.3.3 Caisson-Type Breakwaters and Caisson-Retained Islands

Concrete caissons are frequently installed on top of a prepared stone bed. Typical segments, rectangular in cross section and from 20 to 60 m in length, are manufactured in a fabrication facility, launched or lifted out, transported and installed in much the same manner as previously described for quay walls (see Section 9.2.3), bridge piers (see Section 9.4), and submerged tubes (see Section 9.5). Care must be taken in the installation to ensure that they are adequately supported at all stages of construction. One concrete caisson for the Tarsiut caisson-retained island in the Beaufort Sea was inadvertently seated with a large overhang, due to a surveying error. This resulted in negative moment and shear cracks, which fortunately did not prevent its successful functioning.

The major new element is the sealing of the gaps between adjoining caissons. The caisson-to-caisson tolerances have six degrees of freedom, so seals or gates must be designed to accommodate such differentials in each dimension and attitude. The best method is to install sheet pile arcs across each gap, one on the inside face, one on the outside face, and to fill between with stones, using a filter course on the bottom, if needed, to prevent washout in the inherent large turbulence that will occur at these locations.

A sheet pile can be half-embedded in each opposing corner, using the heaviest flat sheet piles with the strongest interlock. These are anchored back into the concrete by welded reinforcing bars. Then an arc similar to that employed in sheet pile cellular construction, is installed (see Section 9.2).

Caissons are employed on very large and deep breakwaters to form the core, and filled with additional core rock. Class B and A rock and concrete armor units are installed on the outer face.

Caisson-retained islands are similar, except that for these, the interior fill is usually sand fill. It is essential that no gaps occur under the caissons or at the gap closures that could allow the sand to wash out. Thus, graded stone fill on the inside corners, with filter fabric at the joints, is appropriate.

Caissons are frequently overtopped in service by storm waves or heavy spray and run-up. The protection on the top, whether by large stones or by concrete, is crucial to their successful service. This can saturate the fill and also cause erosion due to runoff (Figure 10.16).

See also Section 14.13.

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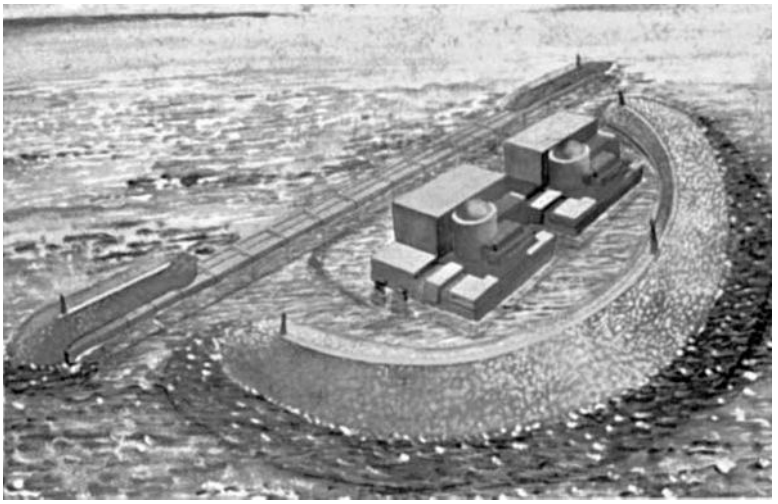

FIGURE 10.16
Proposed offshore nuclear power plant would be protected against hurricanes and ship collision by massive breakwater of concrete caissons plus riprap. (Courtesy of Frederick R. Harris.)

10.3.4 Sheet Pile Cellular Breakwaters

Sheet pile cellular breakwaters are constructed in the same way as described in Section 9.3.2. However, in the case of breakwaters being constructed offshore, there is always cyclic surge motion due to swells and often local wave action as well. A sheet pile cell is not stable until it is fully interlocked all around and at least partially filled with sand.

Installations off the coast of Brazil and in the Bering Sea have proved almost unbuildable by the conventional system of setting sheet piles, a panel at a time, around a temporary ring wale or wales. In the case of Brazil, the solution adopted was to first make up a complete cell on a barge, with several ring wales joined in a space frame. The sheet piles were temporarily fixed to the frame and wales. This entire cell was then set by a large offshore crane barge and immediately filled part way with sand. Then, the piles were released, a few at a time, and driven down into the underlying sands.

The Bering Sea case was more easily solved by making the frame and wales much stronger and extending it to give support to the sheet piles over the full length. The sheet piles were tied against inward and outward movement. A similar problem may arise in strong currents where an incomplete cell may be subject to the full current force, prior to the cell obtaining stability as a gravity structure. A special compound, Adeka, can be brushed into the interlocks of steel sheet piles before setting the sheets. In place after installation, the compound swells to become essentially watertight.

Prestressed concrete sheet piles have been used to construct a number of breakwaters within harbors, in order to give protection to small craft such as fishing boats and pleasure boats. These sheet piles have to develop considerable bending moment, and so are usually thick, wide, and heavy as compared to conventional foundation piles. They are often prestressed eccentrically, which means that they may not have sufficient resistance against tensile rebound cracking during driving (see Section 9.2.2.3). Added unstressed reinforcing steel in critical zones can prevent macro cracks.

These piles have to have sturdy support during installation. They have to be set vertically and tightly against one another. To accomplish this latter, the tip of the sheet pile should be angled so that, as it is driven, it forces the toe back against the preceding pile. At

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the same time, the top, which is above water, is pulled back against the preceding pile. One method is to use a roller with two lines and an hydraulic Turfur hoist.

Sealing of the joints between concrete sheets is often required to prevent sand from leaching out due to tidal changes and waves. The groove formed between the two sheet piles is cleaned out by jetting. Then a polyethylene tube is pushed down and filled with grout. Alternatively, a stiff grout with antiwashout admixture is forced down the grooves, to both seal and bond with the sheets.

Experience shows that damage often occurs during installation to the "wings" that form the female groove of concrete sheet piles. To prevent this, the wings should be reinforced, preferably with headed bars or hooked bars to ensure anchorage at their ends. The male edge should always be leading. This eliminates the wedging action which occurs when the female edge leads; it fills with sand which is then wedged apart by the male edge, breaking the wings.

10.4 Offshore Terminals

Offshore terminals are typically built in water depths exceeding 20 m in order to accommodate very large crude carriers (VLCCs) and deep-draft ore carriers. Wherever feasible, of course, these have been located in protected or semiprotected waters, but on many continental margins, adequate water depth is found only offshore, in a partially or fully exposed location. Therefore, the construction operations must be carried out in the ocean environment, subject to the normal waves, wind, and current for that season, and with suitable precautions for possible storms.

The typical offshore terminal consists of a loading platform, two (or four) large breasting dolphins, and four mooring dolphins (see Figure 10.17). Catwalks usually join all these structures and may require intermediate supports. A trestle may connect the loading platform to shore, or submarine pipelines may be used instead.

The initial structural concept employed was an extension of the harbor-type structure of independent piles supporting a deck and fender system, adapting dimensions to the more severe design conditions of the open sea. Such structures have been extensively used in Japan, in the Arabian Gulf, and along the coasts of Brazil and Australia, for example.

Pilings are typically large-diameter steel pipe piles, 0.75–1 m in diameter, 40–60 m in length, having wall thicknesses of up to 50 mm. High-yield steel (350 MPa) is normally employed. Both vertical piles and batter piles are employed, intersecting at the deck level to react against each other under lateral loads. Batter piles are also called raker piles. As the size of ships and the exposure conditions become more severe, the proportion of batter piles increases so that typically, they dominate the construction. Relatively high axial loads are used as the basis for design, 400–600 tn. in compression and 50%–100% of that in tension. Therefore, it is important that the piles be driven to their design penetration and that they be accurately located so that the axes of a batter-vertical pile group intersect at a single point, this latter to minimize bending. The connection at the intersection must be adequate to develop the flow of forces.

The construction problems for offshore terminals arise principally from the following:

1. The offshore site is usually moderately remote, involving logistical, personnel transfer, and survey problems.
2. The piles are large by harbor standards (although not if compared to those of deep-ocean production platforms).
3. Very close tolerances are required for positioning of the piles at their heads.
4. The connections at the head of piles must not fail due to fatigue.

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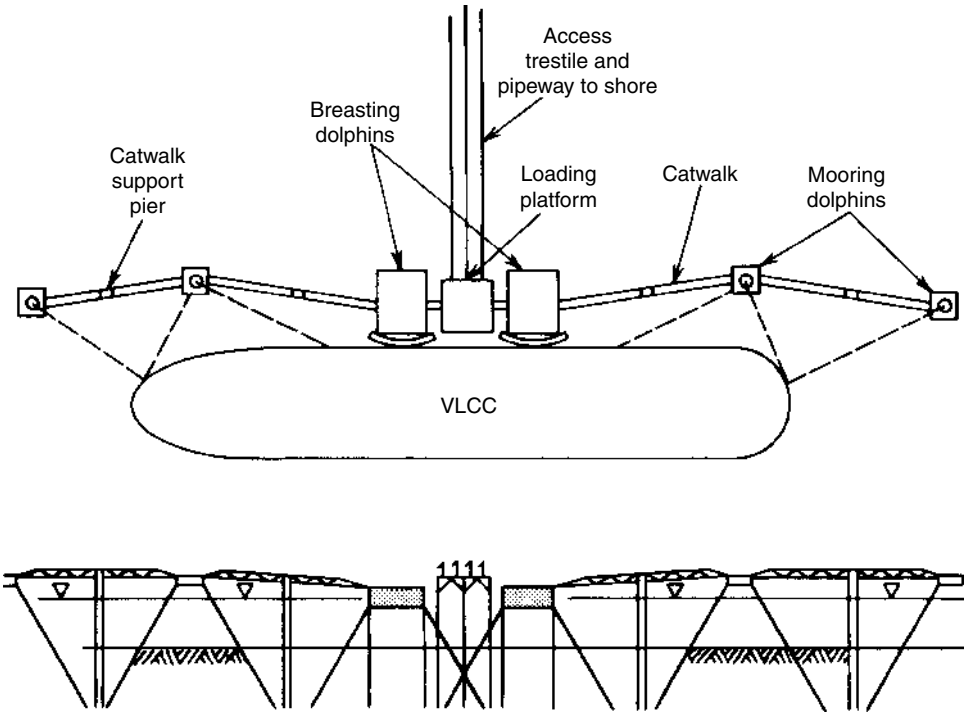

FIGURE 10.17
Typical offshore loading terminal.

5. The relatively shallow water (20–30 m) causes refraction of waves and changes the wave and swell characteristics, making operations difficult even in moderate sea conditions.
6. Last, but not least, the economic constraints and the relatively short distance offshore tempt the contractor to utilize harbor-type equipment and methods, which often prove too small and limited for the exposure conditions and work involved.

The contractor needs to study carefully the bathymetry, which may be changing seasonally on a steep coast; the geotechnical information, which may indicate the presence of hard or firm strata or boulders and coral that may be difficult to penetrate; the wave refraction and breaking patterns, which may influence the work at the site; and the currents, which may affect the route of supply services.

Among the many problems which have been encountered are

1. Sand waves, resulting in slowly changing bottom bathymetry;
2. Refraction of waves causing intersections and concentrations of wave energy at certain zones, with consequent pyramidal waves and confused seas;
3. Opposing currents causing steeper and higher waves;
4. Caprock on or near the surface that is difficult to break up and to penetrate yet is underlain by loose, almost liquefied sands and silts;
5. Weathered rock, which gives widely varying resistance to piles, even to adjacent piles;
6. Boulders on and under the surface;

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7. Sloping hard surfaces, on or below the seafloor, causing the pile tips to tend to run downhill during driving;
8. Calcareous sands, requiring special methods in order to develop uplift and bearing capacity;
9. Overconsolidated silts and clays, which are extremely difficult to penetrate.

An initial construction requirement is to set up a shore base for support of the construction. Ideally, there will be an adjacent harbor with dock and craneage facilities. Unfortunately, this is usually not the case; therefore, such a support base must be established.

In the case of most offshore terminals, there are a very large number of structural and mechanical elements to be transported out to the construction site. An analysis of the number of barge loads and lifts will usually indicate that efficient transfer at the shore base is essential. Some protection must be provided against the breaking waves. Swells not only lead to much larger breaking waves, even in calm winds, but also can lead to severe surges in harbors and channels, where mooring lines may be suddenly snapped. Adequate draft must be provided not only for the barges but also for the tug and crew boats. Services (power, water, and fuel) must be provided to the shore base, as well as communications, both local and to centers of supply, and especially to weather-forecasting services. Unfortunately, in the past history of offshore terminal construction, the contractor has often failed to set up an adequate shore base initially and has had to progressively improve it as the job went on, meanwhile suffering from the inadequacy.

The second step is to set up survey control. Horizontal control may consist of ranges, using focused brilliant lights visible for several miles at sea, even in the daytime, or lasers. Electronic-positioning systems are usually also installed. The shore stations are established on the coast. Tidal gauges are installed, preferably in protected wells. Levels can be run by laser, corrected for curvature of the Earth if the structure is distant off shore. GPS may be used as a verification of position. If real-time rapid satellite positioning is desired, differential GPS systems may be used.

As so often happens with all types of offshore structures, the geotechnical investigation made by design engineers for their purposes may be inadequate for the constructor's needs. Therefore, additional site information may need to be developed regarding the seafloor sediments, boulders, and obstructions. Through use of a "sparker" survey, jet probings, and borings, more information can be obtained on the upper soil strata through which the piles must be driven. For example, the contractor may want to handle and set a 60-m-long steel pile as a single piece on its designed batter. The contractor needs to have a reliable estimate of how far the pile will run down under its own weight. Will jetting be required? At the construction site proper, the contractor will now set up moorings, to facilitate moving the floating vessels and derrick barges along and around the terminal. Pre-set moorings minimize the time of moving and the problem of handling and resetting anchors. In many cases they eliminate the problem of crossed anchor lines when two pieces of floating equipment must work in close proximity.

Another early step may be the construction of an offshore survey tower, which will provide visual reference for close-in surveying. The availability of electronic-positioning devices of high resolution has minimized but not eliminated the need for such towers in recent years. Their proximity allows the towers to act as visual guides and enables the crane barge superintendent to move rapidly to approximate position.

The next decision is what to do in case of storm. Presumably, the boats will run to safety in a harbor. If reasonable weather-forecasting services exist, with proper planning and judgment, the contractor may avoid being caught with supply barges at the site. But what about the major floating equipment: for example, the large derrick barge? This will always

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be at the site, and hence, may be vulnerable to a sudden storm. It will not always be practicable or safe to try to tow it to a harbor; the harbor entrance may have breaking seas, shoals, or cross currents which make it exceedingly dangerous to enter in a storm. The tugboats may not have enough power and size to handle a large derrick barge under severe sea conditions.

Experience has shown that it is often safer to ride the storm out at sea on a pre-set storm mooring. Such a mooring will consist typically of a single long catenary wire line from the barge to a mooring buoy (spring buoy), which in turn has a line or chain leading to a heavy anchor. The anchor is usually two anchors, piggybacked (one behind the other joined by a halfshot of chain), or one large anchor, with a shot of chain to ensure that the pull is horizontal (see Section 6.2). During any season when a sudden storm is possible, the storm mooring line is kept connected, but usually slack, to permit normal maneuvering on the short, taut operating lines. When the taut moorings are slacked, then the derrick barge will ride to the storm anchor. An auxiliary anchor, dropped underfoot off the end of the barge will prevent excessive yaw.

When construction proper begins, the piles are delivered, either on a barge or self-floating, and are picked up for driving (see Figure 10.18). The first pile in each dolphin

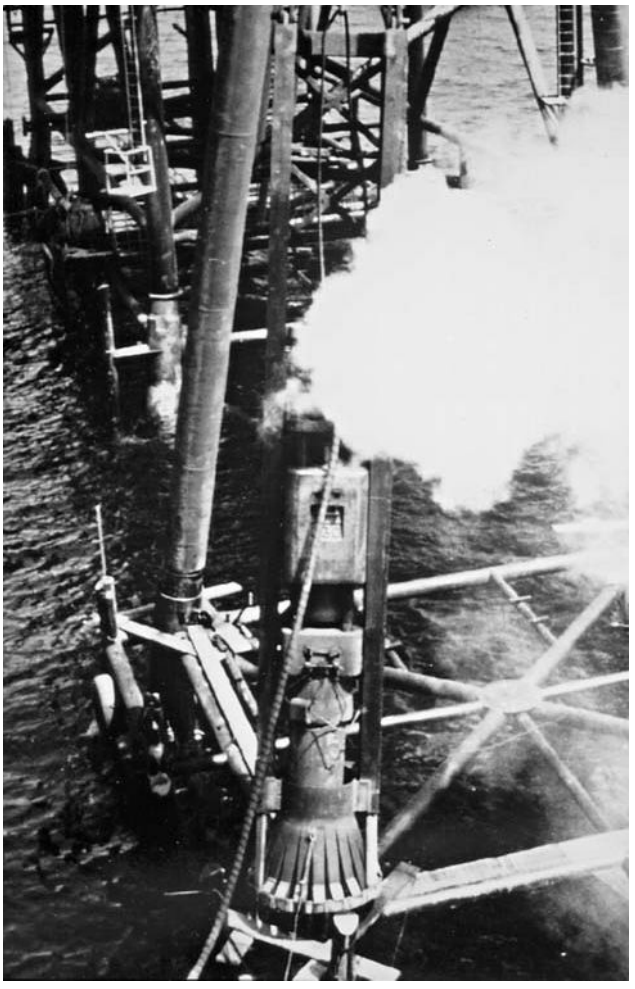

FIGURE 10.18
Driving piles in jacket of offshore terminal. (Courtesy of J. Ray McDermott.)

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should preferably be one of the vertical piles in order to give a point of reference and support for subsequent batter piles. Yet many mooring dolphins are designed with all batter piles, no vertical piles!

A second problem is that the piles are driven alongside each other and then cut and pulled to final position. With heavy-walled, large-diameter pipe piles that are driven, for example, through hard seafloor material, they may be quite inflexible and difficult to position. Pulling the heads may impose bending stresses that will be permanently fixed in the piles.

There is another serious problem with the subsequent connecting of the piles. If these tubular members have to be cut, fitted, and welded at their point of intersection, this may prove impracticable to carry out in a satisfactory manner. The piles may be vibrating due to waves and vortex shedding from the current. The joint area may be wet with spray, the steel below optimum welding temperature. Weld positions will be unfavorable (see Figure 10.19). These joints are subject to cyclic loading and dynamic loads; thus they often fail from fatigue.

As a result of the twin problems of positioning batter piles and connections, the prefabricated template has been evolved. In this case, a template, typically extending from high-tide level up to the top of the dolphin (a height of 5 m or more), is prefabricated of tubular steel members (see Figure 10.20 and Figure 10.21). All welding is done in the shop, where conditions are optimum and proper procedures and nondestructive testing can be carried out. The template will have sleeves, at the proper angle, through which the piles may be set and temporarily supported. The template is then held over the stern of the derrick barge in proper plan location, and one or more vertical piles driven through their sleeves (see Figure 10.22). They are not normally driven to final penetration at this stage but only as needed to provide lateral and vertical support to the template. If the final design does not contain a vertical pile, then the contractor adds one, with its sleeve, as a temporary support pile, usually in the center of the dolphin. Now the template is raised vertically to proper elevation and temporarily welded off to the vertical piles. Its position and orientation are checked.

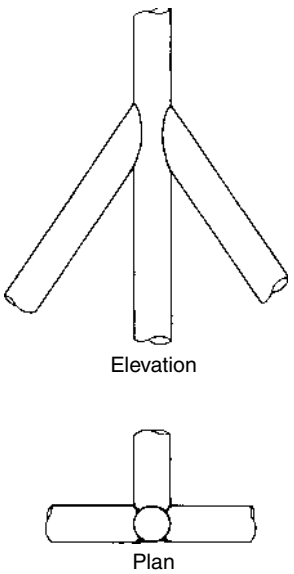

FIGURE 10.19
Typical intersection of vertical and batter piles of mooring dolphins.

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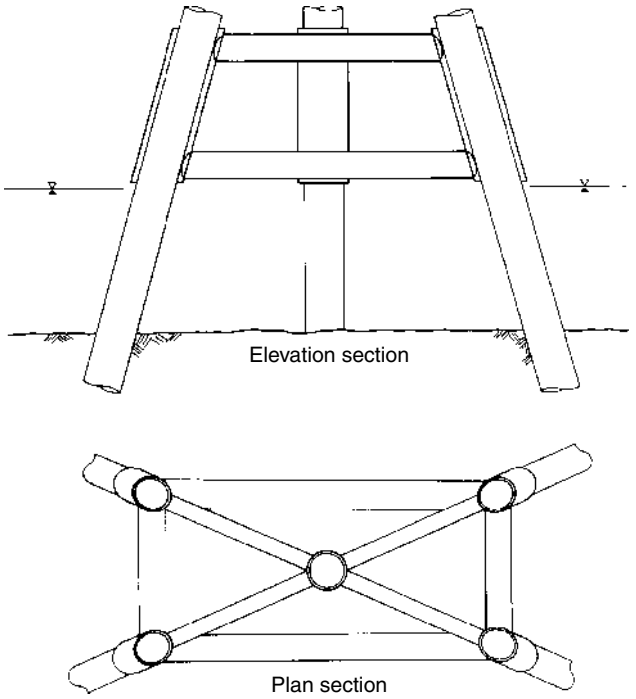

FIGURE 10.20
Use of prefabricated template for construction of mooring dolphins.

Batter piles are successively set through the sleeves, alternating directions to avoid dislocating the template. As they are set, they are driven to grade (see Figure 10.23). Finally, the vertical pile is cut loose, spliced, and driven to its final penetration. If the vertical pile is not in the design, but was only supplied by the contractor for temporary purposes, then it is cut loose and removed.

The connections between piles and sleeves of the template are usually by a combination of shear welds, on a scalloped profile, along with cement grout injection. This latter is needed, in any event, to prevent vibration.

Where a jack-up construction barge is used, as has been the case for a number of offshore terminals in Japan and elsewhere, then the jack-up platform can provide the initial support for the template. However, the template should not be rigidly attached to the jack-up platform but be free to slide as dynamic lateral loads are applied by the driving of the batter piles; otherwise, the stability of the jack-up rig may be endangered.

Breasting dolphins must resist the heavy loads imposed during docking. Thus, they are often full jackets, similar in concept to although smaller than the offshore oil drilling and platform jackets described in Chapter 11. Similarly, the loading platform may be a jacket, often with all vertical piles, since none of the loads from the vessel are transmitted to it (see Figure 10.24).

During the construction of an offshore terminal, with its many independent structures, the derrick or construction barge will have to move many times in order to be able to handle the batter piles at the many different angles. In turn, it must have its mooring lines out at various angles, which change as the barge is moved. It is, of course, undesirable to have a line run around a dolphin, as it may apply a high lateral force in a direction other than that for which the dolphin was designed, displacing it. More likely, it will break the line just at the time it is most needed.

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FIGURE 10.21
Prefabricating jackets for offshore terminal, Kharg Island, Iran.

FIGURE 10.22
Setting large jacket, Kharg Island, Iran.

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FIGURE 10.23
Driving pile through jacket leg.

Jack-up construction rigs in clay soils must be careful not to reset their legs too close to a previous hole. On the Ise Bay terminal, near Nagoya, Japan, some 65 positions were required for the jack-up. An extremely careful layout was required to prevent overlap of leg holes in the clay soil which might cause bending of the legs or loss of support. These many positions, orientations, crane reaches, and mooring arrangements all need to be laid out in sequence on construction drawings.

The positioning of the mooring dolphins is usually not critical; a meter or two each way is usually acceptable. The relative positioning in and out of the breasting dolphins and loading platform is very critical, because when the ship berths and lies against the

FIGURE 10.24
Conveyor truss being installed. Port Latta Iron Ore Terminal, Tasmania.

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breasting dolphins, it must not hit the loading platform, even with temporary deflection. At the same time, it must be close enough to allow hose connections to be made or to stay within the shiploader's radius. Hence, great care must be taken in establishing the front face for the breasting dolphins and the setback for the loading platform. Massive fenders are provided on the breasting dolphins to absorb the impact energy during docking. Typically, the 250,000-DWT tanker docks at 15 cm/s (6 in./s), and all this energy must be absorbed by the fenders plus the elastic distortion of the dolphins. These fenders therefore are large, massive, energy-absorbing devices with a predetermined load deflection response. Many different types have been developed, utilizing deforming rubber fenders, springs, hydraulic rams, the deflection of high-strength steel tubulars in bending or torsion, or the potential energy of gravity weights. The impact of the ship produces high comprehension in the fender and also a longitudinal force dragging the fender along the face. This is usually resisted by chains.

Regardless of these details, the fenders must be properly and accurately set and installed by the contractor. Fender units are prefabricated in the largest segments that can be conveniently handled. Temporary guides should be installed, which will automatically position them in proper position for bolting or welding.

The previous remarks about the difficulty of welding in the splash zone apply here. Because of the impact forces involved, for example, 300 tn. (3 MN), very extensive shear welds are required.

High-strength bolts are, therefore, usually preferable. To aid in fit-up, slotted holes should be provided, slotted in one direction on the jacket frame, the other direction on the fender bracket. If proper bearing plates are used under the bolts, a good connection can be rather quickly made. Tolerances are very important, in that the face of the fenders on the two or more breasting dolphins must line up to be engaged simultaneously and equally by the ship's hull. The main emphasis by the constructor, therefore, must be on maximum prefabrication, provision for tolerances, and adoption of all practicable expedients to aid in installation.

The superstructure of the loading platform is now installed (see Figure 10.25). To the greatest extent practicable, it should be prefabricated in large modules. The hydraulic loading arms are especially time-consuming to set, because of the accuracy required and their awkward shape. Prefabrication (preassembly) of these in a modular frame will save many hours of time at the site.

Similarly, erection of a shiploader on the loading platform is a major task due to the heights involved and weights that must be lifted. It may be necessary to erect a stiff-leg derrick on the loading platform in order to set the higher segments. Special planning is

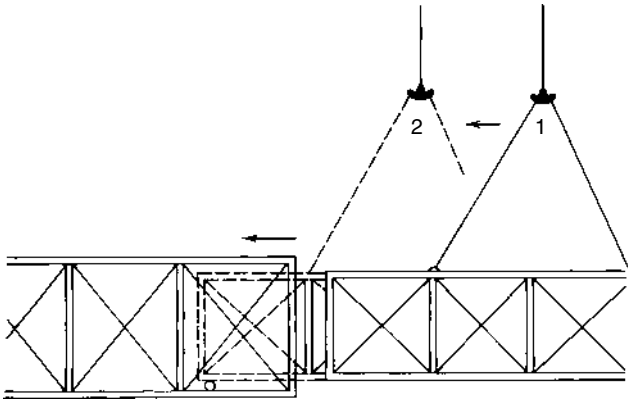

FIGURE 10.25
Erecting telescopic boom of ship-loader.

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FIGURE 10.26
Gravity base berthing structure being prefabricated, Hay Point Terminal, Queensland, Australia.

required for the rigging when installing a telescoping shiploader boom; not only is the boom heavy and awkward and required to be lifted high above the deck and the sea, but it must be traversed into the closely fitting housing. In turn, the lifting slings tend to foul on the housing frame and may require relocation during erection (see Figure 10.26). For these reasons, complete prefabrication and assembly of a shiploader in the shipyard or harbor is, of course, preferable to assembly over water, whenever this is practicable.

Caisson-type (GBS) offshore terminal structures have been built in order to permit complete outfitting in a protected harbor, including shiploader, conveyor stacker and fenders, and the transport and setting of this complete terminal as a single unit in manner similar to that described in Chapter 12. For the Hay Point Terminal in Queensland, Australia, the terminal consisted of three berthing caissons, one carrying the shiploader completely erected, another the conveyor stacker. Short temporary buoyancy tanks were attached to the caisson bases in order to give positive stability during the critical stage of submergence below the top of the base raft.

The initial submergence for tow was carried out in the harbor so the tow was performed with the structure riding with the waterline up on the shafts. These caissons were each towed out and installed in a single day, the actual set-down taking only a few hours. Mooring lines were run from each caisson to preset mooring buoys. Winches on the caisson pulled the structure to exact location and held it there during set down (see Figure 10.27 and Figure 10.28). One short mooring line was required, leading to the previously set structure. This was affixed to a rubber cushioning device that was designed to absorb shock loading. A fiber line could have been similarly used to accept the dynamic force variations.

The structures were designed to arrive at high slack water, be positioned during the fall in the tide, ballasted down to seat at low slack, and then ballasted to stay on their pads during the subsequent high tide (Tidal range was 5–6 m).

Dolphin caissons had superstructures of tubular structural steel; they required substantial temporary buoyancy tanks to be attached to ensure stability and control during seating. The buoyancy tanks were removed after each dolphin was placed and re-installed

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FIGURE 10.27
Towing GBS terminal structure to site. Note short temporary buoyancy tanks to maintain stability during submergence.

on subsequent dolphins. One tank connection failed due to fatigue, allowing the dolphin to tip. Fortunately, it was only two meters above the seabed, so did not prevent installation (see Figure 10.29).

After seating, the spaces under the caisson bases were filled with grout containing a thixotropic admixture (methylcell). Scour protection, in the form of articulated concrete block mats, was placed on the periphery of the caissons.

Import and export terminals are required for the transfer of LPG and LNG to and from specialized ships. In many cases, the terminals are combined with storage of these cryogenic substances. In these cases, specialized problems arise and new requirements must be

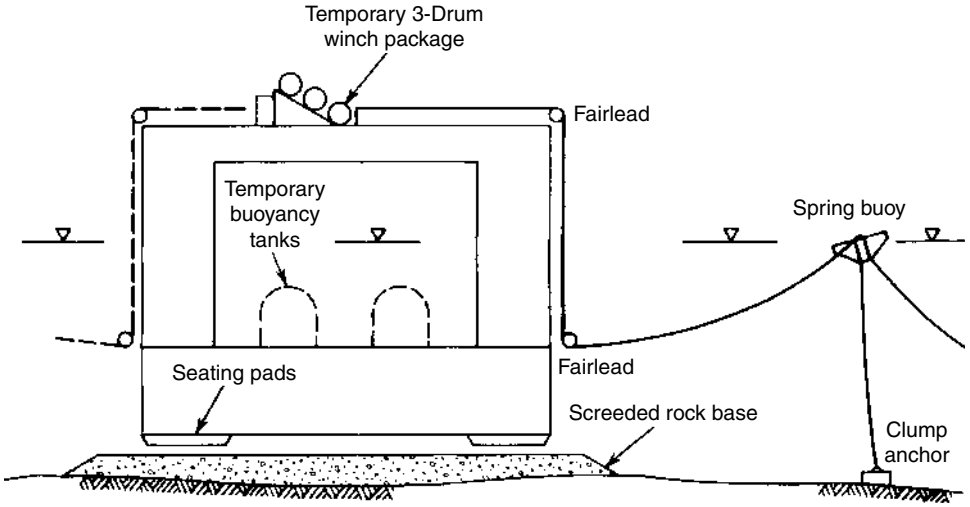

FIGURE 10.28
Positioning of offshore GBS for Terminal, Hay Point, Queensland, Australia.

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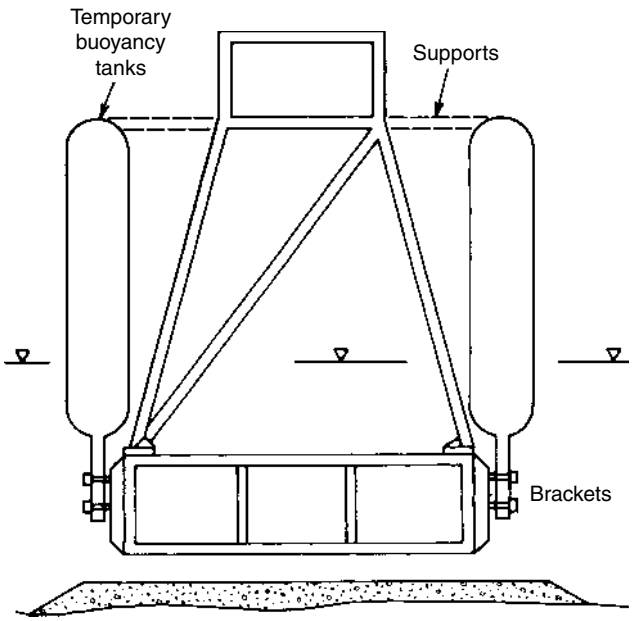

FIGURE 10.29
Temporary buoyancy tanks to provide stability and buoyancy during installation.

met. These terminals may be pile supported, with jackets or may be GBS caissons or even floating structures.

Several import LNG–LPG terminals exist, and more are planned. Most are conventional pile-supported structures, connecting to gasification facilities on the nearby shore. The LNG import facility at Cove Point, Maryland, is founded on prestressed concrete piles.

The LNG is transported to a shore-based gasification facility by an insulated pipeline in a submerged tunnel, constructed in the same manner as an underwater submerged vehicular tunnel. An offshore LNG facility is under construction at Costa Azul, Baja California. LNG will be piped on a trestle to shoreside re-gassification facilities. The breakwater is being built using concrete caissons other GBS structures are being planned for installation offshore in the Gulf of Mexico. Re-gassification would take place on the GBS and the gas transmitted by submarine pipeline to the land system (see Section 14.8 and Section 13.10).

Another type of design for offshore terminals is that which employs large-diameter cylinder piles 2–4 m in diameter, usually of thick-walled steel. This type has been extensively employed in Cook Inlet, Alaska, where ice loading dominates the design criteria. They have also been used in the Mideast. More recently, this concept has been adopted for North American terminals, where seismic criteria as well as overload ship impact require ductility. This type of structure consists essentially of all-vertical steel tubular piles. Each pile represents a major installation process, by jetting and driving, or by drilling and grouting, as described in Chapter 8. Adequate penetration is required in order to develop the required lateral resistance; typically, a penetration of at least five diameters is needed. Very large driving heads must be fitted in order to distribute the hammer blow over such a large-diameter cylinder. Often these are specially fabricated, with stress relieving of welds to prevent cracking under impact. Jets, if required, are usually preinstalled in the piles to permit their operation simultaneously with the impact driving.

In many cases, the installation has been carried out using large offshore-type impact hammers. In other cases, multiple vibratory hammers have been employed. A major construction problem for this type of construction is the handling and positioning of

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FIGURE 10.30
Large diameter tubular pile, Nikiski Terminal, Cook Inlet, Alaska. Pile head tapered to minimize ice forces.

such a large-cylinder pile, say 50–60 or more meters in length, 3 m in diameter, weighing several hundred tons. Tidal currents, such as those in Cook Inlet, which range up to 7 knots (4 m/s), tend to displace the cylinder pile both in the direction of the current and laterally due to vortex shedding (see Figure 10.30). The pile must have special slings fitted so that it will hang vertically. Once set into the soil, the problems of current are reduced.

The next problem is that of driving such a large pile in the heterogeneous soils that are typically encountered: glacial till and overconsolidated silt in Alaska; caprock, limestone strata, and calcareous sands in the Middle East areas. The procedures required are described in Chapter 8. The subsequent superstructure erection is carried out in similar manner to that for the more conventional terminals, except that prefabrication is made easier by the all-vertical pile arrangement. Large prefabricated deck sections or bridges may be set by a crane barge. These may incorporate piping and equipment.

The iron-ore loading terminal at Port Latta, Tasmania, had hard and deeply convoluted igneous rock covered by fine sands. The persistent wave environment made installation of the batter piles difficult. To accurately position the piled piers, a prefabricated steel frame was pinned to the sands by four vertical piles driven to but not into the bedrock. The frame was leveled by hydraulic jacks. Then, a prefabricated template was hung onto the frame. The inclined steel tube casings were set in the template, driven to the bedrock, and cleaned out. By means of a drill and the pile hammer, the casings were seated in the rock, and sockets drilled. Steel tubular piles were driven and locked to the bedrock by accelerated grout. The original seating frame was lifted off for re-use.

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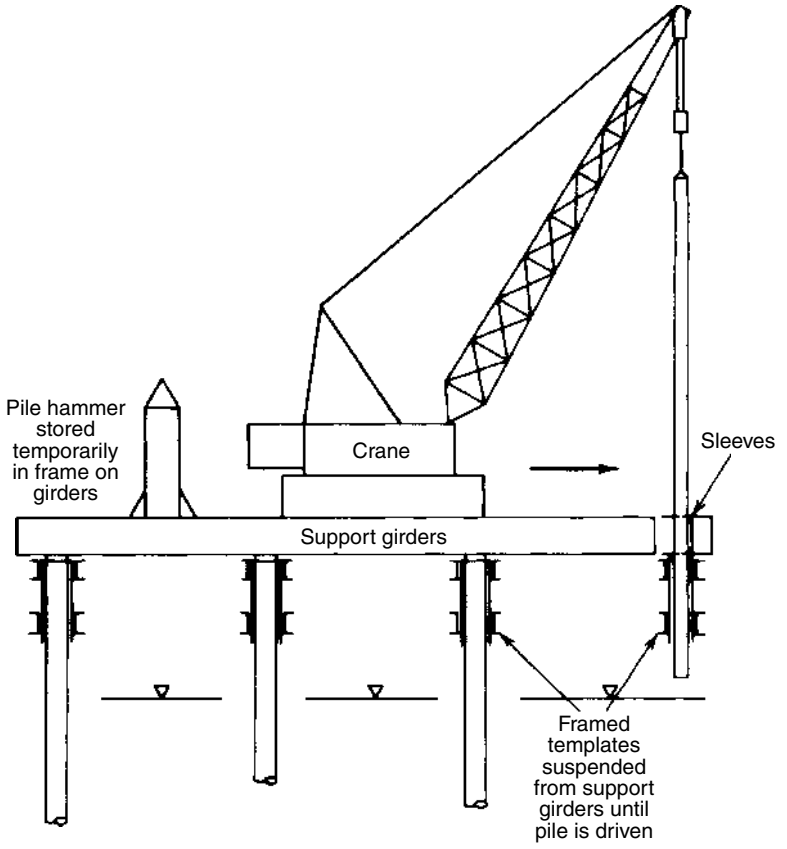

FIGURE 10.31
"Over-the-top" construction of access trestle to offshore terminal. Entire crane including supporting girders, slides forward over each pile cap as it is completed.

The Arco Terminal at Cherry Point, Washington, employed 2-m-diameter steel tubular vertical piles, driven to 25-m penetration. Smaller-diameter stub piles were concreted into the top of the large-diameter piles to give greater flexibility and ductility under severe earthquake. Prefabricated deck sections were completely outfitted to reduce the subsequent topside work to essentially that of connections.

On some offshore terminals, for example, the LPG terminal at Ju'Aymah and the petrochemical terminal at Jubail, both in Saudi Arabia, large-diameter prestressed concrete and steel cylinder piles have been set in predrilled holes and then driven and grouted to develop proper bearing and lateral support. These have varied from 1.6 to 4 m in diameter.

Trestle connections from shore are relatively standard in their construction operations. Because they typically cross through the surf zone and over shallow-water areas, part or all of their construction is carried out over the top. By such methods, the lifting, driving, drilling, and framing are essentially independent of the sea state and current (see Figure 10.31). Typically, the upper works of a large crawler crane are mounted on long girders, for example, double-wide flange beams designed to span one bay and having extensions that cantilever out to the next bay, where a template is set on the ends. Spud piles are dropped and the girders are jacked to grade. The crane then sets the piles through the template and drives them. The template is now welded to the piles. Longitudinal stay beams are dropped into place and bolted. The rig can now skid forward to the next bay. Another crane follows up behind, placing prefabricated deck sections, completing all

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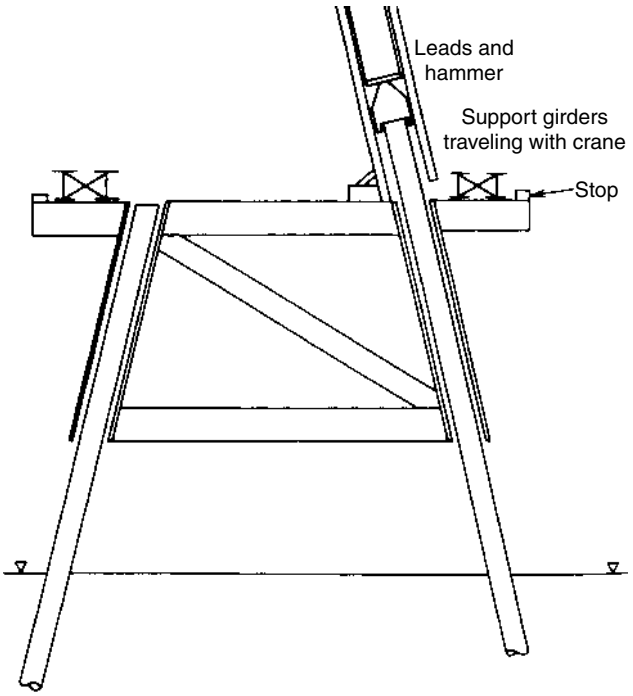

FIGURE 10.32
Prefabricated bent templates for offshore trestle also sense as permanent bracing.

bracing and framing. The piles in the template need to be fully fixed to prevent excessive vibration and assure proper interaction. Shims are inserted to fix the pile head, and a seal made at the lower end of the sleeve, followed by grout injection and welding at the top. Accelerators may be used in the grout to achieve early strength (see Figure 10.32). A long trestle will require an anchor bent every 300–500 m; this often consists of a double-bent, adequately braced. Because of weight limitations, the prefabricated template may have to be set in two half-segments and bolted together. Launching forward of the rig is done by pulling against the longitudinal girders. To guard against accidental sideways displacement, lateral stops should be installed on the end of the cap girders of each template. Such a sidewise runoff actually occurred when launching the crane rig forward on the access trestle to the Hay Point terminal, Queensland, Australia; fortunately, the rig did not run completely off, and no damage or injury occurred.

Such over-the-top methods have been used with spans of 20 m. Designs have been prepared showing their practicality for spans as great as 30 m or more. In planning for a very long access to a terminal off the Ivory Coast, where heavy swells from the Southern Ocean would make work afloat very difficult, spans of 30 m showed significant economies. The major difficulty with this type of construction is that all materials must be delivered over the trestle, rehandled by the deck construction crane to the stern of the pile-driving rig, and then swung around by it to the next bent. Proper packaging and planning of such deliveries, perhaps using a night shift, will simplify this problem.

Even when working on top of a trestle or offshore terminal platform, provision must be made for safe access by personnel. Walkways, safe step-downs for when the crane swings, and adequate lighting need to be incorporated into the design. In deeper water, trestle supports may be constructed afloat, with small jackets and pin piles (Figure 10.33). Water safety also needs special consideration. A person may fall overboard. Life jackets should

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FIGURE 10.33
Template and bent frame for pile access trestle to offshore terminal. Port Latta, Tasmania.

be mandatory. Lifelines of nylon or similar floating material should be strung. From a modern terminal, with deck at +10 or +12 m, and nothing but tubular piles in the water, it can be very difficult to rescue a person who has fallen overboard in a choppy sea with a current running. The person needs to have a line to hang onto while awaiting rescue. The discussion of personnel access and transfer in Section 6.4 is especially relevant to offshore terminal construction.

An alternate to trestle construction, usually adopted for offshore terminals that are far offshore or where the waterway cannot be impeded, is the use of submarine pipelines. Their installation is generally described in Chapter 15. However, there are available a number of special options. The pipelines can be pulled from shore, with the pulling winches on the terminal structure itself, or by a pulling barge located seaward of the terminal structure. Small-diameter lines can be stacked and welded vertically on the terminal, fed down through J-tubes, and pulled to land. Where the terminal is far offshore, standard offshore lay barges can be used, laying the end alongside the terminal. The lines may be laid in a predredged trench; even if it partially silts in, it often may be practicable to sink it down later with a jet sled operation. Near to the terminal itself, at the very least, the pipeline should be buried and the trench backfilled to prevent damage to the lines from boat anchors. In the zone of breaking surf, even conventional riprap may not be adequate to protect the pipeline. Stones of high density, even iron ore, or drilled-in anchors may be needed.

The Lord sent out a great wind into the sea,
So that the ship was likely to be broken.
Then the mariners were afraid, and
Cried every man unto his god—
So they took up Jonah and cast him
Forth into the sea.
And the sea ceased from her raging.

Jonah, Chapter 1

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11

Offshore Platforms: Steel Jackets and Pin Piles

11.1 General

This chapter addresses the typical offshore platform, originated in the Gulf of Mexico and now spread worldwide. Its range extends from water depths of 12 m to over 300 m and from relatively benign climates in Southeast Asia to those of the North Sea and North Atlantic. More than 4000 such platforms have been constructed. Jackets, the main component of the system, range in weight from a few hundred tons to over 40,000 tn. Structures for very deep water, greater than 300 m, are presented in Chapter 22.

The principal structural components of the offshore platform are the jacket, the piles, and the deck (see Figure 11.1 and Figure 11.2). The concept is very simple: the jacket is prefabricated on shore as a space frame, and then it is transported to the site and seated on the seafloor. The piles are then driven through sleeves in the jacket, and connected to the sleeves. The deck is now set. Jackets are also employed for offshore terminal construction, especially for the loading platform and breasting dolphins of petroleum terminals.

The typical offshore drilling and production platform does not exist for its own sake but rather is thought of as a necessary but expensive support for the primary functions that are the reason for the project. These functions are to drill wells, produce oil and gas, process it as necessary, and discharge it to pipelines to shore or a loading terminal.

From the platform, conductors are installed, held by conductor guides bracketed from the jacket. On the deck, derrick and drilling modules are installed, so that the wells can be drilled. Processing modules are installed on the deck, and all the necessary support modules for accommodations, power and water generation, sewage disposal, communication, and heliport. Cranes are installed to handle drill collars and casing, and all consumables from barges or supply boats to the deck. On the deck are stored drilling mud, cement, fresh water, and diesel oil, and contaminated drilling slurry and cuttings. Other functions, such as re-injection of water or gas, may also be performed from the platform. An emergency flare stack is provided in order to flare excess gas. While diesel oil is used initially to fuel operations, produced gas may be used after production and processing are established.

This chapter addresses the construction phase of a jacket for an offshore platform, including the fabrication, loadout, transport, launching, upending and seating, as well as deck installation, module erection, and the installation of the pin piles described in Chapter 8.

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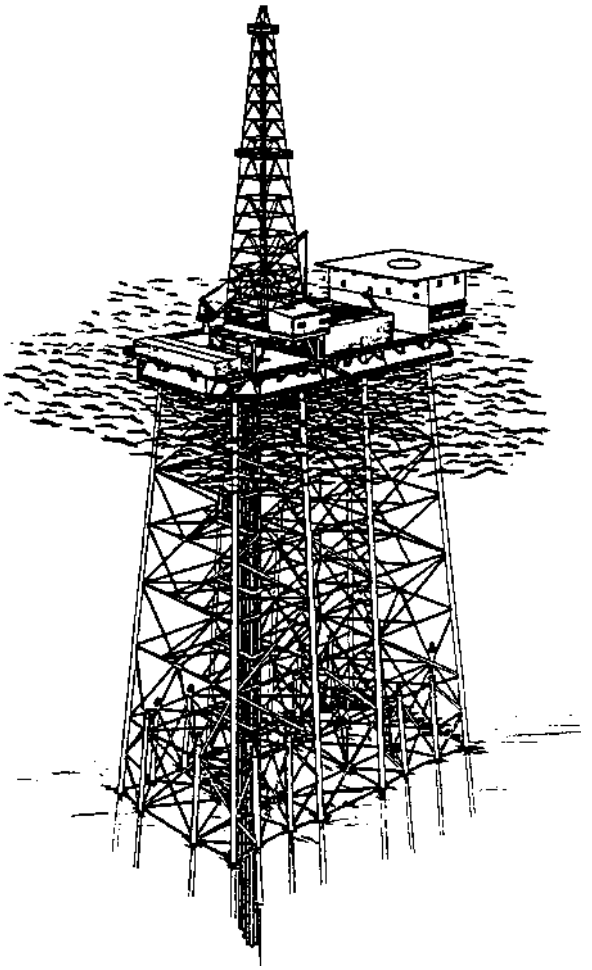

FIGURE 11.1
Steel jacket and clustered skirt piles. (Courtesy of J. Ray McDermott, S.A.)

11.2 Fabrication of Steel Jackets

The typical jacket is subdivided so that the two narrowest sides are fabricated first, each laid out flat on the surface so that they may later be rolled up to position, the jacket itself being horizontal.

Cutting and fitting of the tubular intersections require precise work. In today's modern yard, this cutting is numerically programmed to ensure that the final weld gaps will be of the order of 3 mm. Fitting is done in the early morning, when all the steel is at a uniform temperature. Rollup is accomplished by several large cranes, positioned so that they can initially lift the outside main leg, and then walk toward the jacket centerline as the side is raised (see Figure 4.5 and Figure 4.6). This requires that the ground under the cranes have adequate support. Once vertical, or at the design tilt, sides are guyed off. Then individual cross members (bracing) which connect the lower and upper legs are set, fitted, and welded. Daily survey checks are run to prevent the development of cumulative errors.

Since the weight of the jacket has to be temporarily borne by the lower legs, additional vertical support is often required during fabrication. Jackets that are large in plan and only

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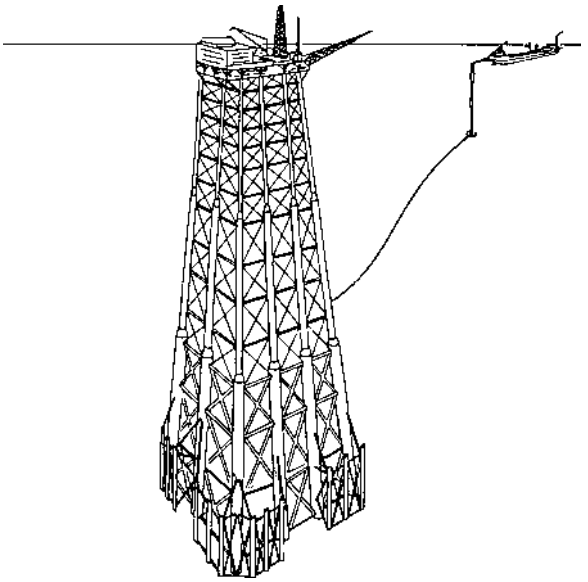

FIGURE 11.2
Steel jacket with sleeves for pin and skirt piles. (Courtesy of J. Ray McDermott, S.A.)

moderately high may be fabricated in their upright condition. In this case, temporary runner beams are needed to support the jacket during fabrication and launching. Large jackets often have complex intersection of tubular bracing, where anywhere from three to thirteen braces intersect at a point, yet each must transfer the full force through the node. In this case, the nodes may be fabricated first, detailed so that their subsequent connection in the field to each brace is a right-angle joint, to be connected by a full-penetration butt weld. This same concept of prefabricated nodes is beneficial when a complex jacket is to be fabricated in a remote area of the world. Then the nodes can be shipped separately and the braces fitted at the site. For example, the brace may be beveled on one end, and allowed to run 300–500 mm long on the other end. At the final fabrication site it can be cut to fit.

All temporary attachments, such as lifting eyes, should be welded with the same procedures as the permanent members in order not to cause cracking or heat affected zone (HAZ) defects in the primary steel. Once their use is ended, these temporary attachments can be burned off 6 mm or so from the primary steel and then ground flush.

All personnel must be fully aware of the catastrophe of the Alexander Kjelland semi-submersible floatel, where a temporary entry, sealed with a substandard weld, later developed a fatigue crack in the main brace which resulted in the capsizing of the vessel with a large loss of life.

Further and more detailed discussion of fabrication can be found in Section 4.1.

11.3 Load-Out, Tie-Down, and Transport

The jacket, having been fabricated on shore, must be transported to the site. Typically, it is skidded onto a launch barge (see Figure 11.3). The launch barge is usually grounded at the dock, on a prepared and screeded sand pad. Water ballast is placed, sufficient to hold the empty barge on the bottom even at high tide.

For very heavy jackets and where the water is too deep to ground the barge or the tidal change too great, the barge must remain afloat. In this case the barge must be continuously

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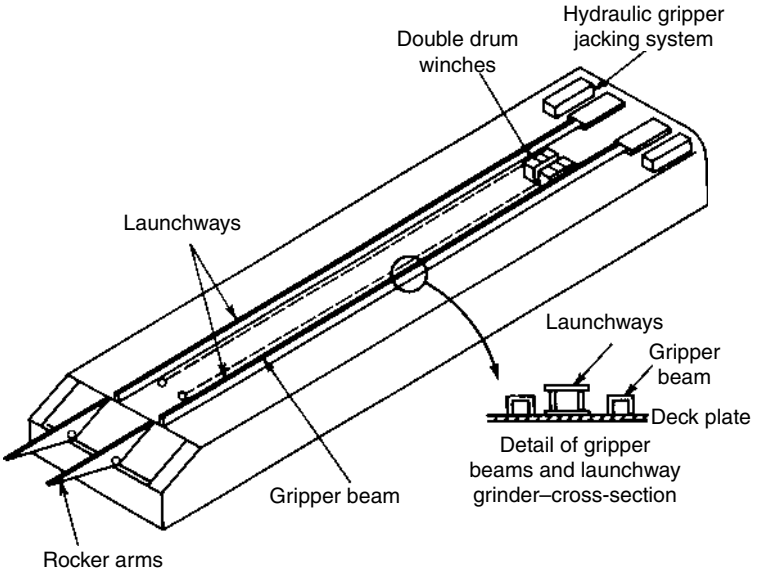

FIGURE 11.3
Launch barge.

ballasted to remain level and at the proper elevation relative to the skidways on shore, while the weight of the jacket is progressively transferred onto the barge. This can best be done by computer-controlled ballasting (see Figure 11.4).

API RP2A Section 2.4.3b requires that "structures be moved horizontally onto the transportation barge by means of ways or wheeled dollies on track supported by cribbing and should be checked for the effects of localized loading resulting from the change in slope of the ways or tracks and the change in draft of the transportation barge as the weight of the structure moves onto it. Since movement is normally slow, impact need not be considered." With the barge positioned end-on to the dock, the jacket is pulled onto the barge, usually by winches on the outboard end of the barge itself, since this automatically holds the barge tight against the dock. Alternatively, the barge may be moored taut against

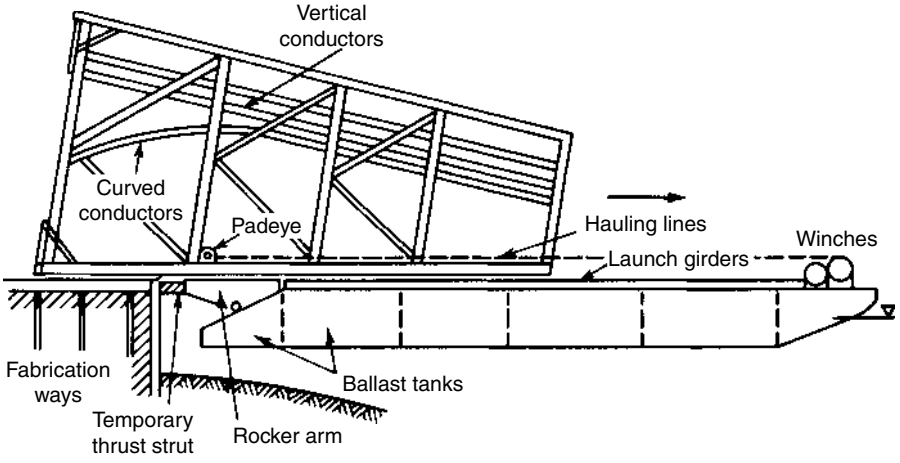

FIGURE 11.4
Loadout of jacket onto launch barge.

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the dock, with lines strong enough to resist the friction force from the jacket. Jacks or screw-rods or onshore winches are used to pull the jacket out onto the barge. If spacer struts are used between the onshore fabrication ways and the launch beams on the barge, these will be placed in heavy compression. They must be strong enough to take the forces without buckling and supported against "kicking out" sideways.

The barge must be secure against transverse movement induced by current, wind, or the wakes of passing boats. The alignment of the jacket must be accurately maintained while the jacket is pulled out onto the barge. For this reason, it is preferable to ground the launch barge before loadout, whenever such factors as water depths make this practicable. Large modern launch barges are equipped with multiple ballast tanks and pumping capacity to enable the barge to be maintained at proper trim and draft.

During loadout, the jacket is supported on the fabrication ways, usually on two inner legs of the jacket. These are strengthened by plates to act like girders, able to carry the jacket weight with some free span between points of contact. These girders are also converted into the bottom chord of a large truss, by using the basic platform bracing, often supplemented by additional diagonals, for example, to enable it to span between points of support, especially when part of the jacket is on the barge and part still on the fabrication ways (see Figure 11.5).

Initial friction of the jacket on the ways may be as high as 10%, especially if the jacket has been erected with its weight bearing on the ways continuously. In many cases, the initial fabrication is carried out slightly elevated above the ways by means of sand jacks. Alternatively, hydraulic jacks are used to permit removal of a filler piece. At time of launching, the jacket is lowered onto the skidways. To reduce the sliding friction, grease on hardwood, or heavy lubricating oil on steel, or even fiber-filled Teflon-faced pads, are used to reduce friction to as low as 1% or less.

FIGURE 11.5
Heather jacket ready for loadout onto grounded launch barge, Inverness, Scotland. (Courtesy of J. Ray McDermott, S.A.)

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A check list of the operations relating to loadout of jackets follows:

1. Is jacket complete? Has the structure been analyzed for loadout stresses on the basis of the actual structure as fabricated at the time of launch?
2. Are the conductors, both straight and curved, in the same configuration and support condition as has been assumed in the analysis? Conductors, especially curved conductors, are often installed during the onshore fabrication and fixed to the jacket frame, as opposed to the vertical conductors, which are often installed offshore, through conductor guides. Since decisions on the number, direction, and time of installation of conductors are often changed during the fabrication process, their support and tributary loads may differ from those used in earlier design.
3. Is the launch barge securely moored to the loadout dock, so that it will not move out during the loading? Is the barge properly moored against sideways movement?
4. If compression struts are used between the barge ways and fabrication ways, are they accurately aligned and supported so they will not kick out during launch?
5. Have the pull lines, shackles, and padeyes been inspected to ensure that they are properly installed and cannot foul during loadout?
6. Is the barge properly ballasted? If the tide will vary during loadout, are ballasting arrangements made? Will ballast be adjusted as the weight of the jacket goes onto the barge? Are there proper controls?
7. If the ballast correction is to be made iteratively, step-by-step, as the jacket is launched, are there clear paint marks so that each stop will be clearly identified?
8. If the loadout is taking place in an active or potentially active waterway, has the Coast Guard been asked to issue a Notice to Mariners to stop all traffic? Has a boat been stationed to stop the private power cruiser or tug that may not have received the Notice to Mariners?
9. Are the tugs on station? Are standby tug or tugs available in case of tug breakdown?
10. Has the weather forecast been checked? Squalls are especially dangerous due to their sudden occurrence.
11. Have clear lines of supervision and control been established? Are the voice radio channels checked?
12. Have the marine surveyors been notified so they can be present? Owner's representatives? Verification agent? Have their approvals been received?

Once the jacket is on the barge, the barge must be ballasted for sea. During loadout, many tanks will be partially full only, in order to control deck elevation and trim. Now, with the jacket fully supported on the barge, these considerations are no longer active, and the tanks can be deballasted to suit the demands of the sea voyage.

Tanks should normally be either "pressed-up" full or else completely empty, to eliminate free surface and sloshing effects. The draft and freeboard will have been carefully selected to maximize stability and especially to prevent the outrigger-like legs of the jacket from dipping into the sea during roll of the barge. Trim will be adjusted to optimize tow speed and to give directional stability during tow; usually the barge will be trimmed down by the stern.

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The above remarks apply when the barge has no restrictions from the loadout dock to the open sea. Many interesting variations arise in inland channels and bays, which have to be dealt with as special site-specific and jacket-specific operations. Examples follow:

1. Shallow water or a bar may limit draft and necessitate even trim.
2. A narrow channel may require that the overhanging legs of the jacket be high enough to clear dolphins, boat slips, even docks.
3. Fixed bridges may limit the height and necessitate ballasting down to deeper draft and occasionally to severe trim-down by the stern, even to the degree of submerging the stern of the barge. Since this reduces transverse stability (the water plane is reduced), this condition has to be checked with extreme care. This procedure was brilliantly executed in connection with the transport of platform Eureka under the Richmond-San Rafael Bridge in San Francisco Bay, with only 1 m clearance below the bridge deck girder.
4. The tides can be selected to give the greatest benefit at these critical stages. Tidal currents must also be taken into account, and adequate reserve tug capacity must be available to abort the tow and pull back if proper conditions are not maintained.
5. Wind from the side can cause the barge to heel; if the spread of the jacket legs at the bottom is 50 m, then even a 2° wind heel can cause a 1 m increase in height of the jacket leg, or perhaps a half-meter increase in draft. Once past these constraints, the barge can be ballasted for sea.
6. The barge, while very stiff, is nevertheless a flexible member. The jacket is typically even stiffer than the barge. Therefore, adjusting of the ballast of the barge should preferably be done prior to tie-down for sea. If one scheme of ballasting was used for the inner channel tow and another will be used for sea, the tie-downs should be freed during the change in ballast to prevent imposing bending deformations on the jacket legs.

Tie-downs are installed after loadout and prior to entering the open sea (see Figure 11.6). They are major structural systems, subjected to both static and cyclic dynamic loads. Therefore, the gravity and inertial forces involved must be calculated for all anticipated barge accelerations and angles of roll and pitch during the design storm adopted for the tow, usually the ten-year return storm for that season of the year and location. Since the loads are dynamic, impact must be minimized and fatigue in a corrosive environment must be considered. The tie-downs will see approximately 14,000 cycles of fully reversing load for each day at sea. Fatigue has become a major concern in long transpacific tows (see Figure 11.7).

Inertial forces are due to acceleration in heave, roll, and pitch and are therefore dependent on the period of response of the barge with the jacket loaded on board. Gravity loads depend on the maximum angle of pitch or roll. Wind loads must also be considered, although they will normally be a much smaller component of the total load.

For typical short-term tows in temperate seas, the following criteria have been used for design of tie-downs:

- Single amplitude roll, 20°
- Single amplitude pitch, 10°
- Roll or pitch period, 10 s, double amplitude
- Heave force, 0.2 g

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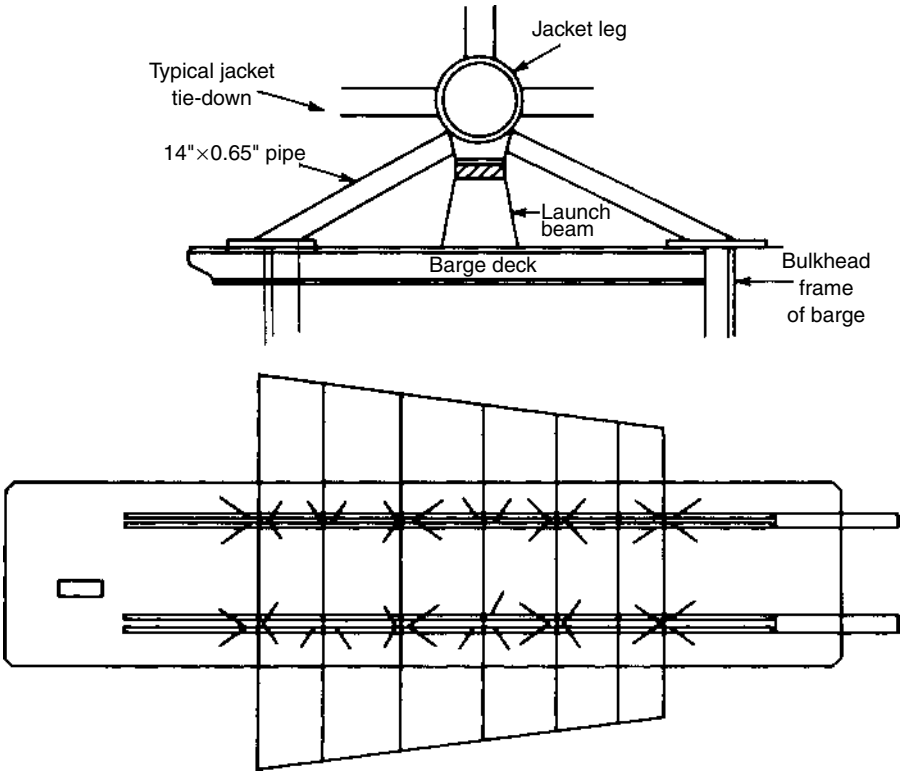

FIGURE 11.6
Tie-down of jacket on barge.

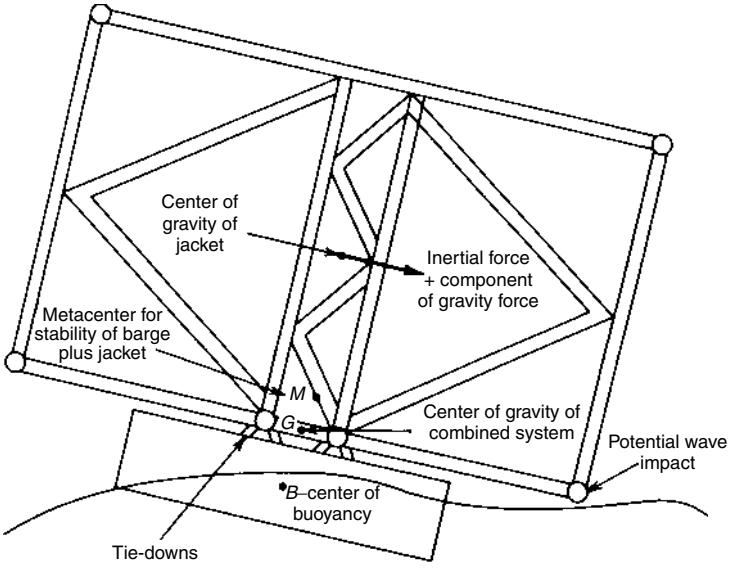

FIGURE 11.7
Launch barge with jacket under heavy roll during storm.

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The design load will then be calculated on the forces due to heave plus pitch or heave plus roll.

For long tows, or tows at seasons of the year when storms are likely, special studies should be made of weather conditions and the resultant force combinations imposed. Model tests in both regular and irregular seas are used to measure motions and forces, for various headings of the barge. Typical model scales are 1:50 or larger.

Computer programs have been developed which have been correlated with model tests as well as actual performance. They are quite reliable when applied to relatively standard jackets on conventional barges. These give significant values of the responses of the barge for various headings and sea conditions, from which the extreme values can be computed. If elastic analyses are used, then it is normal practice to use the American Institute of Steel Construction (AISC) allowable stresses for the significant value of the responses, without any increase for the short-term duration of the load. If load factor analysis is employed, then the load factors should be chosen appropriate to the significant or extreme value adopted. If cold weather will be encountered during the tow, then the sea fastenings must be constructed of steels having suitable impact values for the temperatures involved.

Tie-downs are structural members, connecting the jacket to the barge. Therefore, the point at which they are connected to the jacket must also be able to resist these forces. Similarly, the structure of the barge to which the tie-downs connect must have proper strength. Usually, this means penetrating the deck and making a shear connection to an internal bulkhead or even to the side of the barge. The penetration itself must be sealed to prevent water entry.

Wire rope is normally not usable for tie-downs, since under cyclic loading it stretches and starts to work loose. Therefore, chain is employed where bending flexibility is required. Wedges, even though driven tight, must be welded in place; otherwise they may work free under repeated loads. However, in the great majority of cases, fixed structural members such as heavy walled pipe are used, rigidly welded to the barge. Punching loads on the barge deck can be spread out by suitable doubler plates, stiffeners, or bearing beams.

Shallow-water jackets, such as those employed for offshore terminals, are short and squat. They often may be loaded out and transported vertically, rather than on their side. In this case, they may be skidded onto a barge, supported on temporary steel girders under or alongside the jacket legs. Since the weight of such jackets is usually less than 1000 tn., the loadout forces are not excessive. However, because the launch beams on the jacket are temporary, they must be checked for possible eccentricity and also for web buckling; adequate lateral support must be provided. Once on board, the effects of barge response must be fully checked as to the loadings on the temporary girders, since the loadings will now have dynamic and lateral components, and the webs of girders will no longer always be vertical.

The third method of loadout is that of the self-floater. In this case, the jacket is fabricated in a dry dock or shallow basin. The legs on one side are typically made much larger in diameter to provide flotation for the entire jacket. Alternatively, extra legs or buoyancy tanks may be provided. Thus, for example, if in-service leg diameters of 2 m are required for structural purposes, the legs on one side may be 8 or even 10 m in diameter to provide the necessary buoyancy so the jacket can be self-floating.

This system has been successfully employed on several platforms off the California coast, on the Brent A, Thistle, Ninian South, and Magnus platforms in the North Sea, on the Maui A platform in New Zealand, and also on the Drift River Offshore Terminal in Alaska.

The British Standards Institute Code of Practice, BS 6235, emphasizes that it is important to ensure that when self-floating jackets are built in a basin or dry dock, as the basin is

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flooded and open to the sea, the structure will not "bottom out" on a subsequent low tide. To assure this, the structure will usually be ballasted to negative buoyancy before the basin or dock has been fully flooded, thus remaining on its supports until the day of float-out. Then it is deballasted on a rising tide and floated out at or near high tide.

Alternatively, temporary buoyancy in the form of a raft of tubular members temporarily affixed to the jacket structure may be provided and later removed after installation of the jacket. This system was successfully employed on the platforms for the BP-Forties Field in the North Sea and the North Rankin A platform in Australia. It has now become common practice to include temporary buoyancy tanks as part of the jacket; for example, this method was employed for the Heather platform in the North Sea and Eureka on the California coast in order to facilitate their control during up-ending and placement.

The obvious disadvantages of the self-floating method of transport are the increase in diameter of the permanent jacket legs and the increased wave and earthquake forces generated in service. The obvious advantages are in eliminating the forces imposed during loadout and launching, as well as the costs of these operations. The requirements for a launch barge and for tie-downs (sea fastenings) are eliminated.

Use of temporary buoyancy tanks or rafts eliminates many of the disadvantages of the concept but imposes the new problems of making temporary connections that will neither fail due to the dynamic impact forces of transport and installation, nor suffer corrosion-accelerated fatigue in the saltwater environment, yet will still be readily disconnected after the jacket is installed and pinned to the seafloor.

Properly designed, the self-floater tanks can act as the hull of a barge, stiffened by the total jacket framing. Roll response is minimized by the typically wide spread of the legs. The Maui A platform, en route from Japan to New Zealand, successfully rode out a typhoon with only minor damage. Self-floaters have also been built on ship-launching ways and launched down into the water, just as is done with a ship.

There are, of course, severe bending stresses induced during launching, since for a short time, the jacket is partially supported by buoyancy and partly by the ways. Even more serious may be the concentrated bearing forces exerted on the upper end of the jacket and on the ways as the jacket rotates upward at the outer end. Side launching, of course, produces less severe loadings during launch but the jacket must be prevented from rotation during the launch. A self-floater must be analyzed for the hog-sag and quartering responses to the sea, the first set producing bending and the second, torsion. The large legs on which flotation will take place must be checked for watertightness and internal subdivision provided, just as with a barge.

The tow of a launch barge with jacket tied down or of a self-floating jacket must be planned with great care because of the sensitivity of these awkward structures to sea and wind conditions. Tows have been successfully carried out from Japan to California, Alaska, New Zealand, and Australia, over distances of many thousands of miles. Since towing speeds are inherently slow, a delivery voyage may extend up to thirty days or even more. This makes it probable that at least one major summer storm will be encountered en route.

Tow routes are selected to minimize adverse weather and storms, to avoid complex channels if feasible to do so, and to take advantage of favorable currents. Often two tugs will be employed, as insurance against breakdown and to provide better control of the tow in close quarters. In any event, having one additional boat in attendance while at sea appears wise. The tow route is also planned to have good sea room to lee in event a major storm is encountered.

The advice of a competent weather-forecasting ocean-routing service should normally be obtained to be able to select the most favorable routing for the tow, first from the point of view of safety—i.e., storm avoidance—and second from the point of view of minimum

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time of transit. Such a service will usually provide daily advice to the tow master regarding both weather to be expected over the subsequent seventy-two hours and recommended changes, if any, in routing. In turn, the tow master advises the ocean-routing service daily of the position, speed, course, weather, and sea state.

Tow boat size is determined partly by its horsepower and bollard pull and partly by the hull form of the boat itself. For long tows in the open sea, the boat should have substantial length and draft, whereas for working through crowded channels or in close proximity to other structures, a short boat, with minimum draft, preferably equipped with a bow thruster, will be found best. For the long open sea tow, the boats should be able to continue towing the barge along the tow route at about 2–3 knots, even when heading into a 15-foot sea with a 40-knot wind and 1-knot current.

Under severe storms, the boat(s) should be able to keep the heading even though the forward speed may be zero or negative. Under some circumstances, the tow boat may cast the tow free, allowing it to drift to leeward while the boat rides out the storm. When the storm abates, the tug picks up the tow again. This is one reason for having a trailing nylon or similar rope which can be picked up by a tug in moderate seas and used to haul the spare bridle and pennant to make up with the tow line. In narrow channels, where tidal currents may be adverse and tricky, or in the vicinity of islands or land from which gusty winds may strike, larger boats, and sometimes more boats, are required.

When the BP-Forties platforms were towed out of the Moray Firth of northeast Scotland, two pulling tugs were supplemented by two steering tugs, being pulled astern through the water, arranged so they could act to slow or turn the self-floating jackets. Such tugs must be selected, or modified, to have high enough stern sheets that they will not take in water over the stern. The engine room hatch must be battened down securely and provision made to drain the stern well in the event water is shipped.

Stability against capsizing is a major design consideration for a jacket on a barge, primarily because of the high center of gravity of the jacket, but also because of the possible sudden wave slam impact if the overhanging leg of a jacket is engulfed by the crest of a wave. Metacentric height is a valid measure of stability only for small angles of list; it can be considered a first approximation for purposes of determining accelerations and stability. For survival in a storm, and for assurance of stability against capsizing, righting moment vs. angle of heel charts must be prepared. From computations based on the center of gravity of barge, ballast, and jacket, corrected for the free-surface effect of any partially filled compartments, a righting moment curve can be drawn, representing the resistance to overturning.

The angle of down-flooding is that at which water will enter the vents in the ballast tanks. To provide extra safety against such an event, if approved by the marine surveyor, the vents should be capped after the ballasting has been completed for the sea voyage. A word of warning: the vents must be uncapped during ballasting or else overpressure and structural damage may occur. They are then capped until the barge returns to port. Caps should be painted red.

The wind heeling moment is generally based on the maximum velocity to be expected during the tow: steady state plus gusting. Alternatively 100 knots (50 m/s) is arbitrarily assumed for a summer voyage. The vertical center of gravity is very sensitive to the actual weight of the jacket plus sea fastenings.

With large and important jackets, strict weight control is exercised, both for purposes of transport and for launch, this latter being a special concern. These include measurements of actual O.D. of all tubular members and wall thicknesses; the latter will usually be found to be near the upper limit of tolerances. Padeyes, sea fastenings, conductor guides, instrumentation panels, mud mats, and so on must all be accounted for. Weld material presents a special problem: calculations should be made on the basis of sampled weld profiles, which

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again are usually greater than the minimum shown on the drawings. The reason for such detail is, of course, the magnitude of the effect that a small additional weight has when it occurs on the upper side of the jacket, perhaps with a large $\overline{KG}$.

Before leaving harbor, an inclining experiment will usually be required by the marine surveyor. A known weight is placed on deck and moved a known distance off the centerline. The angle of heel is then used to determine the metacentric height $\overline{GM}$, from which the position of the vertical center of gravity $\overline{KG}$ can be determined. $\overline{KB}$ and the other factors in the equation, I and V, are geometric properties which can be determined by carefully measuring the draft at all four corners of the barge, after assurance that there are no internal compartments with free water surface that have not been fully accounted for.

Increasingly, marine surveyors are concerned about stability under damaged conditions, when one external compartment of the barge may be flooded. This could occur, for example, by collision from a tugboat, or from hitting floating ice or debris, or from rupture of a pipe in a ballast compartment of the barge. Evaluation of stability and heel under this condition is usually made while neglecting wind, or assuming only a moderate wind, and may use as a criterion that the edge of the deck on the low side should not go below water, i.e., that the peak of the righting moment curve under this condition should not have been exceeded.

As stated earlier, computer programs have been developed to predict the loads on a jacket-barge combination during oceanic tow and the resulting stresses in these two bodies. One of these, by Noble-Denton and Associates, Inc., known as OTTO, predicts the maximum stresses during a design storm as well as the fatigue due to cyclic loading during the entire tow. It takes into account gravitational and buoyancy forces, wind and current forces, wave-induced inertia forces, and wave-induced hydrodynamic pressure forces. Fatigue during tow is becoming of increasing concern as tows become longer, through more severe seas, and with larger jackets having greater inertia. During the tow of a jacket from Japan to California, for example, many nodes may use up a significant portion of their total fatigue endurance. This is especially critical for self-floating jackets.

When towing a jacket in semirestricted waters such as the North Sea, contingency planning should be carried out, including designation of storm lay-by areas, with a plan to proceed from area A to area B and so on only on the basis of favorable weather reports.

Jackets have been lost during tow, a pertinent case being one under tow from Scotland to Brazil, which sank in the North Sea shortly after the start of the journey. The impact of such a loss far exceeds the value of the jacket; it means the loss of at least a year in placing the field into production, with serious cash-flow and interest cost implications. Most losses of jackets can be traced to one of two causes: either the sea fastenings are ruptured, leading to increased inertial forces and shifting of the jacket, or the watertightness of the deck is violated, usually at a sea fastening but sometimes at a manhole or vent, resulting in shipping of water, free surface in the hold, and consequent loss of stability. Manholes that are partially hidden under a jacket leg are a frequent culprit.

11.4 Removal of Jacket from Transport Barge; Lifting; Launching

Smaller jackets, designed for shallow water, are often lifted directly from the barge by one or two crane barges and set on the seafloor. The slings are attached and then the tie-downs and connections to the temporary skid beams are cut loose (see Figure 11.8). Where long-period swells are being amplified and shortened by shallow water, significant differential movement may occur between crane barges and transport barge.

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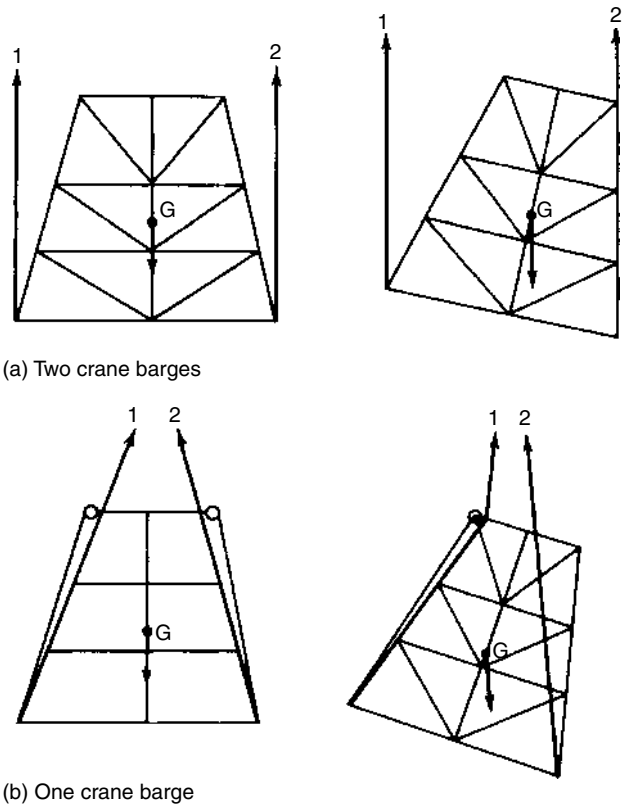

FIGURE 11.8
Lifting jacket with slings attached below center of gravity: (a) Two cranes; (b) One crane.

Appropriate slack must be left in the lines during the period of cutting loose. The cutting-loose operation must be carefully pre-planned in order to prevent endangering the personnel, since most of the cuts must be made by hand. Short vertical guide posts may have been pre-installed at the loadout site to prevent lateral shifting of the jacket once it is cut loose. These braced vertical posts can form part of the tie-down frame; they must be adequately braced for impact. The jacket may also have chain stoppers acting as supplemental tie-downs. When the primary tie-downs are cut, the chain stoppers still hold the jacket laterally. These chains can then be severed remotely by power-actuated (explosive) cutters. Hydraulically operated pins can also be pulled.

Slings for the jacket will preferably have been attached above the center of gravity of the jacket, so that the jacket will hang more or less vertically as it is lifted. In this case, it is only necessary to try to catch a group of lower swells or waves, and then hoist as rapidly as possible as the barge starts to rise on a crest. The dangerous time is the first wave crest after lifting off, when the jacket may once again be contacted by the barge or by the guide posts. These posts therefore should be only the minimum height necessary to prevent lateral displacement during cutting loose and have inclined protector plates welded across their top ends to minimize punching if the jacket leg should contact them on the second rise.

Occasionally, the height of the jacket as compared to the length of the boom will prevent direct lift from points above the center of gravity. The slings have to be attached below the center of gravity in order to get a reasonable angle of spread. This is a dynamically unstable lifting mode, since if the load rotates, the righting moment decreases.

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Recognizing this deficiency, the system can still be used safely, especially if there is a wide spread between the points of attachment of the slings. It must be recognized that a high proportion of the entire load of the jacket may then occur on one sling and one point of attachment, and this in turn severely stresses the jacket frame. Such critical lifts have been made, for example, by two crane barges lifting from opposite sides of the jacket. The slings or lifting lines can also be run through the jacket legs in such a way that any tilting of the load results in a reactive force from the line itself; this line, of course, must be guided in such a way that it cannot be frayed or cut on a sharp edge.

In one case, for a shallow-water jacket in the Gulf of Mexico, the jacket was skidded on its side onto the launch barge. At the site, it was lifted and set on the seafloor. Pre-attached slings then permitted re-rigging of the hook so that the jacket could be re-lifted from its end, causing it to rotate to the vertical for placement. Note again that certain slings must take the entire load of the jacket during rotation, which in turn reflects on the padeye design and the stresses in the jacket frame.

For any offshore jacket lifts, it will usually be found expedient to pre-attach the slings at the loadout site. Then when the barge arrives at the site, the eyes of the slings may be quickly raised by the whip line and placed over the horns of the hook, ready for the lift to take place. Tag lines must be used during the initial phase of lifting clear of the barge, to keep the jacket pulled slightly inward toward the lifting barge and thus preventing it from swinging.

Because of the heavy weight of a jacket and complications of such a lift, the crane barge(s) is usually pre-positioned and moored on site. The transport barge is brought in across the stern of the crane barge and secured. Then the jacket is lifted free. The transport barge is cut loose and pulled clear. Now the load is lowered to the seafloor. This minimizes the need for swinging either the barge or the boom, and keeps the crane barge picking over the stern where it has highest capacity and minimum roll response.

Jackets over 30–40 m in height are launched end-O from the transport (launch) barge. Jackets have been launched which weigh over 50,000 tn. and which are over 400 m in length (see Figure 11.9 and Figure 11.10). This is one of the most dramatic operations in

FIGURE 11.9
Lifting jacket with slings pre-attached. (Courtesy of Santa Fe International.)

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FIGURE 11.10
Commencing launch of platform Heather. (Courtesy of J. Ray McDermott, S.A.)

offshore construction, yet has been successfully performed many hundreds of times. Jackets have also been damaged or even lost during launching, emphasizing the critical, dynamic nature of this operation.

The procedure itself is relatively straightforward. With relatively calm seas, the barge is headed into the sea. The sea fastenings are cut loose. It is ballasted down by the stern so that it has an angle of 3° or more. The jacket is then pulled off the stern by lines from the winches rigged around blocks at the stern and back to the bow of the barge. With larger jackets and dedicated launch barges, the jacket may be pushed off by hydraulically operated gripper jacks. As the jacket moves end-O, off the stern of the barge, it finally reaches a point at which its center of load is beyond the pin of the rocker arms. The rocker arms then rotate to their limit (usually about 30°). The jacket now slides off the rocker arms into the sea.

There is a strong horizontal reaction imparted by the jacket to the barge, causing the barge to surge forward, at the same time as the stern kicks up due to the release of the jacket load. If this is a manned operation, the personnel, stationed near the bow, must have a safety line to avoid being thrown off the barge by this rather violent reaction of the barge. In most modern cases, this operation is carried out by remote control, unmanned, to avoid the danger. An umbilical cord from the tug or radio may be used to actuate the launching system.

The jacket, leaving the barge, has combined downward and rotational momentum. It will therefore usually plunge, with some jackets plunging even deeper than nominal diagonal length, before slowly returning back to sea level in a horizontal attitude (see Figure 11.11). Most jackets are designed to ride, self-floating, on the upper side legs, with these about half immersed. This means a freeboard of only half the diameter of a jacket leg.

To return to the launching operation, starting friction may be relatively high. This will require the use of high pulling forces or thrust from jacks, opposite in direction to those applied in loading the jacket. As the jacket moves down the launching ways, its weight is imposed progressively on a smaller and smaller length of the two central jacket legs, until finally all the load is that at the rocker arm. The jacket now rotates partially into the water so that it is supported over two zones: the water and the rocker arm. The jacket continues to slide, the two legs still carrying a high portion of the total load until the jacket finally slides free. Thus the jacket legs will normally require reinforcement to take the bending and local concentrations of load. Note that while the vertical load at the time the rocker arm rotates will normally be the maximum, there is in addition a friction force acting parallel to the jacket leg.

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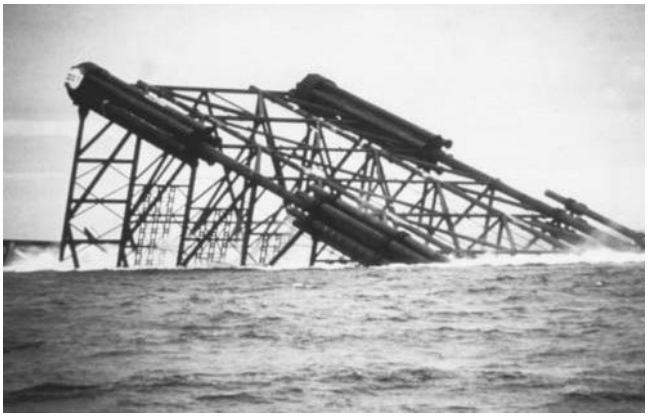

FIGURE 11.11
Jacket being launched from launch barge. (Courtesy of J. Ray McDermott, S.A.)

The worst thing that can happen during a launch is for the jacket to skew sidewise and thus not only tilt the barge to cause the jacket to roll but also cause loads on the jacket frame at points and in amounts for which it is not designed. The proclivity to roll is in part due to the raise of the bow of the barge out of the water as the center of gravity of the load, i.e., the jacket moves aft, thus reducing the water plane moment of inertia.

During initial movement of the jacket on the barge, moving astern until its center of gravity reaches the rocker arm pin, the jacket can be kept properly aligned by controlling the jacking/pulling system and by steel plate side guides on the two center legs of the jacket, which hold them on the launching ways. Feedback instrumentation must be installed to verify that the jacket is correctly aligned as it reaches the rocker arms.

As the jacket rotates into the water, there are impact forces (similar to wave slam) on the legs and cross members and any temporary buoyancy elements, tending to tear them loose. There are also inertial forces acting on any piles that were preinstalled in the jacket legs or sleeves, tending to cause them to plunge downward. In the case of the launch of the Magnus platform in the North Sea, for which the piles had been preplaced in the jacket legs, several piles ruptured the temporary supports in the legs. The piles plunged to the seafloor and were severely damaged by impact.

Jackets have been loaded and launched with the lower end (base) launched first, and also the reverse, where the top is launched first. Present practice appears to favor launching with the top of the jacket first (see Figure 11.12).

Many of the tubular members of the jacket will have been subdivided to be watertight and empty, in order to provide the needed buoyancy to cause the structure to float properly. These are subjected to hydrostatic forces, principally hoop stresses but also complicated by axial compression due to hydrostatic force on the end. Supplemental hoop reinforcement may be needed in order to resist the combined stresses (see Figure 11.13). The tubular members and temporary buoyancy tanks may also experience ovaling forces due to drag as the water rushes past the bracing and legs during launching. The design against buckling must consider initial out-of-roundness of the tubulars.

It is obvious that control of the jacket weight and its distribution is very critical for the launching process. This is why detailed accounting must have been carried out regarding variations in wall thickness and diameters, weld material, temporary attachments, mud mats, conductors, piles, and so on. On the buoyancy side, outside diameters must be thoroughly checked; usually circumferences are more readily measured and can form an adequate basis for buoyancy calculations. In a few cases in the past, improper and inadequate calculations have led to jackets plunging deeply due to inertia at launching,

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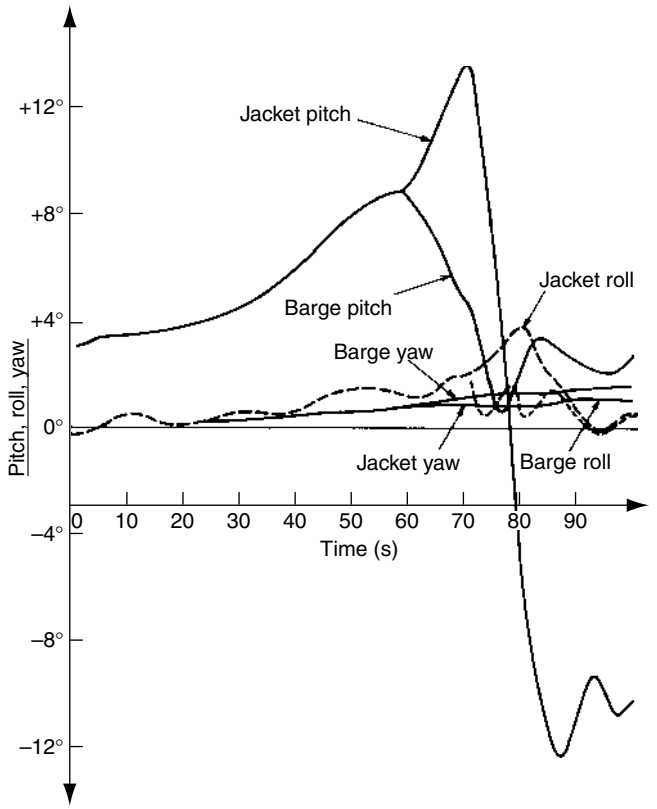

FIGURE 11.12
Motion responses during launch of large jacket. (Courtesy of J. Ray McDermott, S.A.)

then imploding due to excess hydrostatic pressure acting on members which were never intended to be deeply submerged. As noted above, the legs which plunge most deeply are subjected to the combined stresses of hydrostatic pressure acting both circumferentially and axially on the end closure.

The jacket must be launched in sufficient depth of water so that there is no danger of it hitting the bottom, again taking into account the momentum of launch and the diagonal length across the jacket (see Figure 11.14). On several occasions, jacket legs have been damaged by hitting the seafloor.

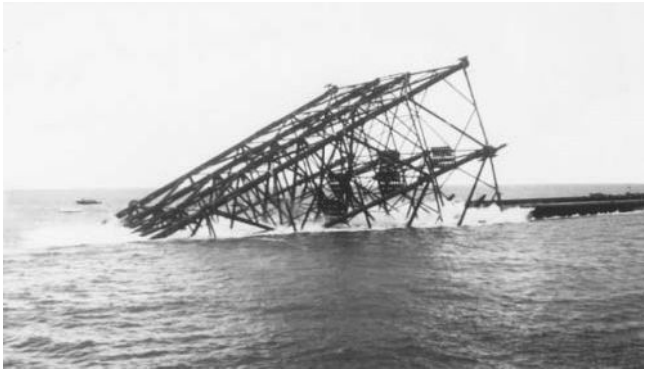

FIGURE 11.13
Launching of jacket.

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FIGURE 11.14
Circumferential stiffening of lower legs of platform Hondo jacket.

Computer programs have been developed that portray the entire launching process graphically. They also give the stresses on the jacket members and launch barge rocker arms during launch and enable the entire dynamic process to be examined in detail. Note, however, that such programs are only as valid as the input data. The actual behavior is very sensitive to relatively slight variations in the amount and distribution of weights and buoyancies. Such programs permit controlled launching, which is becoming increasingly important for deep-water platforms.

The tubular members designed for buoyancy must be positively sealed. Cross-bracing will normally be welded closed. Filling holes should be left and vents provided for those members that are to be free-flooded; plugs can be installed for those that are to be temporarily buoyant and flooded after installation. Attention is called to the fact that the location and details of the holes must be determined and/or approved by the jacket designer; since these may become stress raisers or initiate cracks under cyclic loading in service. The ends of sleeves or legs through which piles are later to be driven are usually sealed with reinforced neoprene jacket leg closures, so that they can be penetrated by dropping the first pile section.

The above descriptions have been addressed to launching off the end of the barge, "end-O launching," which is the common method employed because of the typical jacket configuration. However, just as side launching of ships imposes less severe structural demands on the hull, so sideways launching of jackets, when applicable, imposes less severe forces. For guyed towers, with their rectangular profile, giving a uniform cross section throughout, or for the loading and breasting platform jackets of offshore terminals, which often are rectangular in cross-section, side launching is very practicable. Recent Japanese analyses indicate that side launching may also be practicable for tapered jackets, as long as guidance is provided while still on the barge.

Several short athwartships launching girders can be used, thus reducing the load acting on any one point of the jacket frame. The barge can be heeled 5°–7° by differential ballasting. Relatively small rocker arms can be employed, pinned at the side of the barge. The jacket is pushed or pulled to the downside of the barge; then all restraints except one on each end are released. These two must then be cut simultaneously. The jacket slides and rolls off, in turn kicking the barge sideways. This concept of sideways

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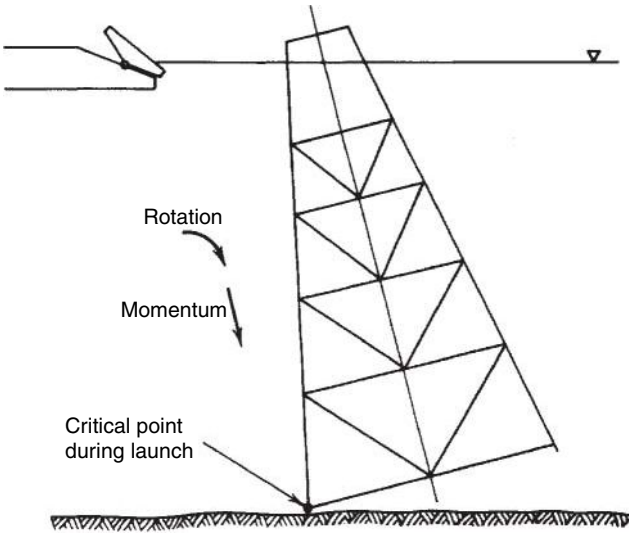

FIGURE 11.15
Impact of jacket on seafloor during launching.

launching is especially attractive for very long jackets, such as the 500-m jackets proposed for use in the guyed tower system.

Brown and Root has developed a system for launching a long jacket from two barges, in which the barge supporting the lower end of the jacket is towed end-O, until the rocker arms release the support of the lower end. This now drops into the water, rotating and therefore allowing the upper end of the jacket to slide off the second barge. This system places a heavily concentrated load at the lower end of the jacket, and hence both this location on the jacket and the rocker arms on the first barge must be heavily reinforced (see Figure 11.15).

Recent schemes have been proposed in which the heavy end of the jacket is to be supported on a launch barge and the upper end is to be made self-floating by means of temporary buoyancy tanks. In this case, the lower end supports of the jacket and the rocker arms must carry very heavily concentrated loads. The rocker arms must be arranged so they can rotate through 90°. Sliding shoes are built into the ends of the jacket support. The temporary buoyancy tanks at the upper end are easily removed after upending.

A significant variation of the above is to provide self-flotation at the lower end of the jacket and support the upper end on a conventional launch barge. Now launching can be combined with upending, and the upper end of the jacket will rotate in conventional fashion. Since the enlarged legs or tanks are now deep below the surface, they attract minimal wave forces and may be left permanently in place, filled with water (see Figure 11.16 and Figure 11.17).

McDermott has proposed joining a second barge astern of a conventional launch barge, using an articulated connection. This second barge would then rotate downward as the jacket moves aft, providing support as the jacket tilts into the water.

Since proposals are being made to install conventionally framed single-piece jackets in depths of 500 m or more, and compliant towers up to perhaps 1500 m, it appears that innovative concepts of launching such as those outlined above will be required. Side launching from one or two barges appears appropriate for deep-water structures. See Chapter 22.

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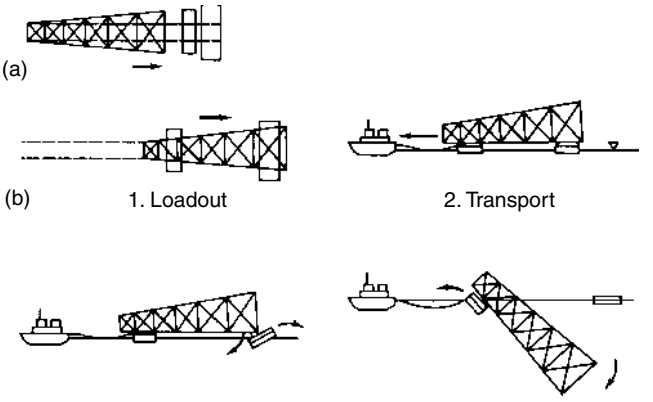

FIGURE 11.16
Twin-barge launching of deep water jacket. (Courtesy of Brown and Root, also Exxon.)

11.5 Upending of Jacket

The upending of smaller jackets has been often accomplished by a combination of differential ballasting, augmented by the lift from the crane boom of an offshore derrick barge. Although this provides excellent control, it involves several potentially dangerous dynamic aspects.

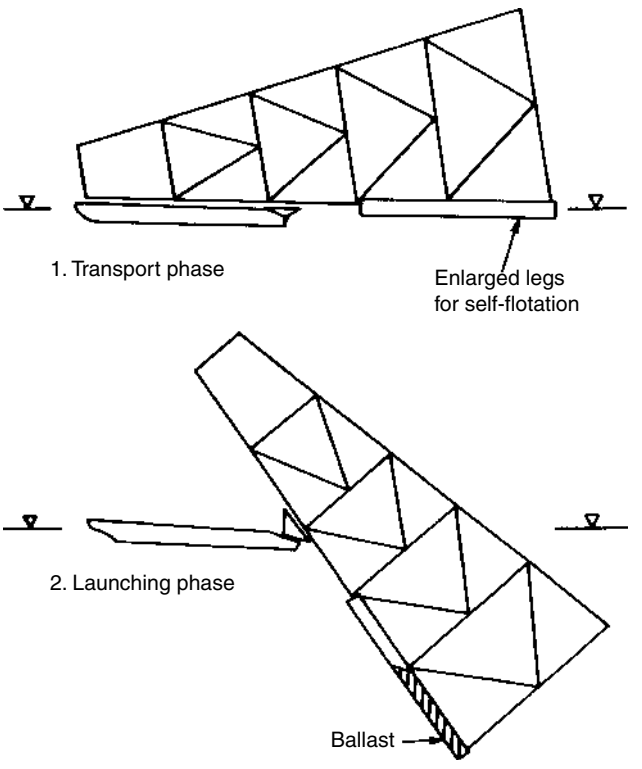

FIGURE 11.17
Launching of partially self-floating jacket.

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First, the jacket, having a large actual mass plus an added mass (hydrodynamic mass) of almost equal magnitude, cannot respond to the accelerations induced in the boom tip by the heave and pitch of the barge. These latter have a typical double-amplitude period of six seconds, which means that it is the boom and derrick barge that are pulled down when the wave crest passes, rather than the jacket being pulled up. There is, of course, elastic stretch in the wire rope falls; hence use of as many parts as practicable is desirable. There is also the flexibility in the boom and stretch in the topping lift lines. Nevertheless, this procedure is safe only in a very calm sea. The slings for this upending should have been pre-attached, to be readily accessible above water, for hooking on.

The crane boom can provide control of the jacket attitude; but the primary upending moment must come from differential ballasting in which water is flooded into the lower portions of the jacket legs on the high side. As the jacket rotates, water may be drained out of upper bracing.

API RP2A provides in part: "Generally, the up-ending process is accomplished by a combination of a derrick barge and controlled or selective flooding system. This up-ending phase requires advance planning to pre-determine the simultaneous lifting and controlled flooding steps necessary to set the structure on site. Closure devices, lifting connections, etc., should be provided where necessary. The flooding system should be designed to withstand the water pressures which will be encountered during the lifting process."

Large jackets have extensive ballasting and control systems installed, to permit flooding and venting, as well as hydraulic lines with which to operate valves. No cranes are used for the large jackets; there is too much danger of overload.

The bending moments and forces induced in the jacket during upending must be determined, in order to prevent overstress in the jacket frame. Any tubular members that are empty or partially empty during the upending process must be able to withstand the combined hoop forces and axial forces induced by the water pressures at the depths involved; these conditions and forces may not necessarily be the same as those during launching or in service. Failure to recognize the effect of combined stresses is believed to have been partially responsible for the collapse of the temporary buoyancy tubes on the Frigg DPI platform, which resulted in loss of the jacket. Note especially that self-floating jackets will first experience significant hydrostatic pressures during upending (see Figure 11.18).

One means of countering high hydrostatic pressures is through internal pressurization with compressed air. On the BP Forties platforms, nitrogen gas, released from liquid nitrogen, was used to internally pressurize the temporary buoyancy tanks.

The British Standards Institute Code of Practice for Fixed Offshore Structures, BS 6235, states, Whenever possible, the use of internal balancing pressurization should be avoided due to the constraints upon design and handling that it produces. If it is used, the following should be noted.

a. The rate of pressurization should not exceed the structure's ability to withstand stresses induced by the increased temperature due to compression of internal air.
b. The process has to be capable of being arrested at any stage without the need for power.
c. Note that the expanded gas from a liquid, e.g., liquid nitrogen, is extremely cold and may freeze valves. Compressed air, on the other hand, can get very hot and interfere with controls and computers.

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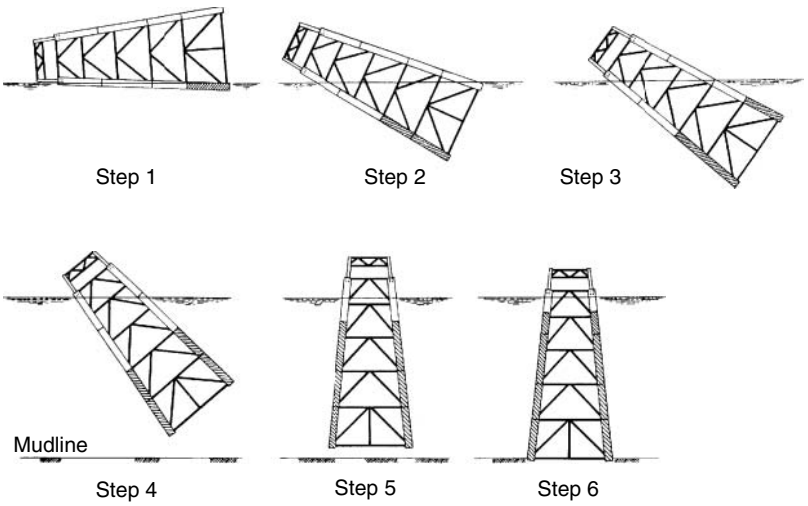

FIGURE 11.18
Installation of self-floating jacket.

Control of relatively small jackets has usually been by umbilical (electric-hydraulic) from the derrick barge, actuating the opening and closure of valves, and feeding back information on progress of flooding. Usually, the valves are equipped with spring closures to automatically close in event of power or hydraulic failure. Screens are provided over intakes to prevent entry of debris that might prevent closure of valves.

Pressure sensors, sensing the rise in pressure of the air compressed at the top of a member as it is flooded, or of the water at the bottom, provide necessary information. As more-sophisticated and larger jackets are installed, valve-position indicators may also send the signals back to the control station on the barge.

As jackets have become larger, the upending process has usually been carried out remotely, without involvement of the derrick barge for lifting control. Three-legged jackets usually roll during upending, making it unsafe to have a line from the boom, while deep-water jackets usually traverse too great an arc for the boom to follow (see Figure 11.19). Remote control has been exercised as before, through an umbilical. However, umbilicals have been broken by the extended sweep of the upper end of the jacket as it rotates. Radio control has thus been found more reliable and is now the state-of-the-art for major jackets.

As a backup, there is usually a station on the upper end of the jacket, where manned controls can be activated in an emergency. The personnel are usually not on board the jacket during the initial part of the upending but may be transferred later by helicopter or boat. For this latter purpose, a rope ladder is arranged to hang down from the control station.

Large jackets, designed for deep water, obviously require a more-sophisticated plan for upending in order to avoid overstressing of the jacket frame. The large legs of self-floaters can be subdivided both in plan and length. Similar subdivision can be carried out for those jackets in which only skirt piles are employed, thus permitting the legs to be divided by transverse closures. Large legs and temporary buoyancy tanks may be pressurized internally to resist hydrostatic pressure.

Upending is usually planned by means of a computer program, which takes into account the constantly changing configuration of submergence and the changes imposed by ballasting. Once a suitable plan has been developed, physical model tests

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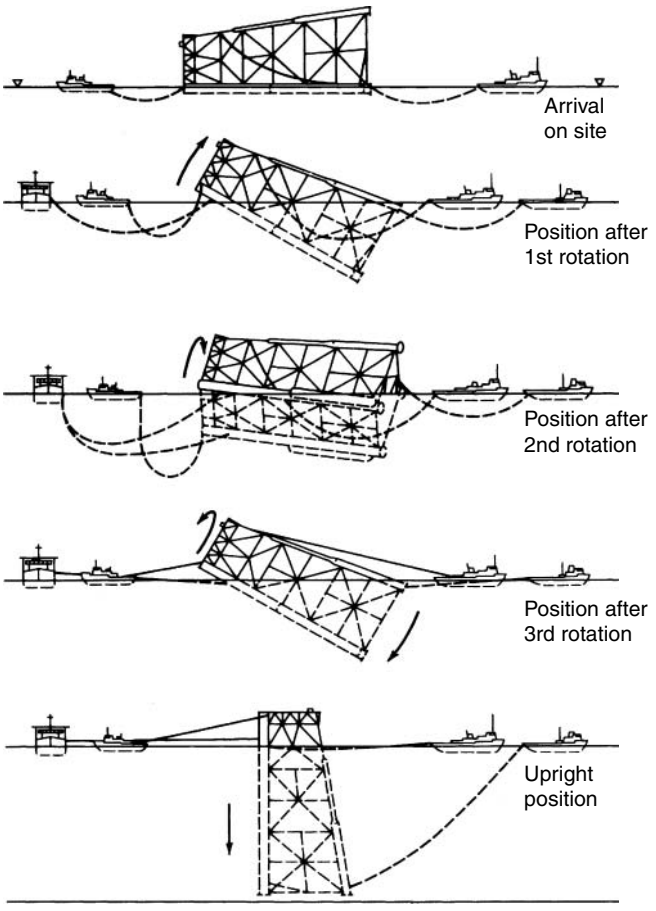

FIGURE 11.19
Upending of Thistle platform jacket. (Ballasting Sequence not shown.)

are run. These serve two purposes: one, to verify the behavior of the jackets during upending and, two, to acquaint and train the key people—barge superintendent and offshore engineer—in this complex dynamic operation (see Figure 11.20).

After upending, the jacket, now in vertical attitude, and with a draft only 3–5 m less than that at the installation location, is towed slowly to its final site location. Wherever feasible, the upending is, of course, carried out at or in the immediate proximity of the final site. However, the diagonal depth of the jacket may exceed the final draft. Where seafloors are very uniform in depth over a large extent, as in parts of the North Sea, this may necessitate upending it some distance from the site and then towing it to final location. Such final tows will have a bridle pre-attached near the center of rotation of the jacket, so that the jacket will remain vertical in the final tow. Towing force and speed is purposely reduced to a minimum. To eliminate or reduce this extra step (final positioning tow), the present trend is to use temporary buoyancy tanks which will enable uprighting near the final site.

11.6 Installation on the Seafloor

To ensure that the jacket will be installed in its proper location, an offshore derrick barge is normally moored on location. In shallow water, this mooring is accomplished

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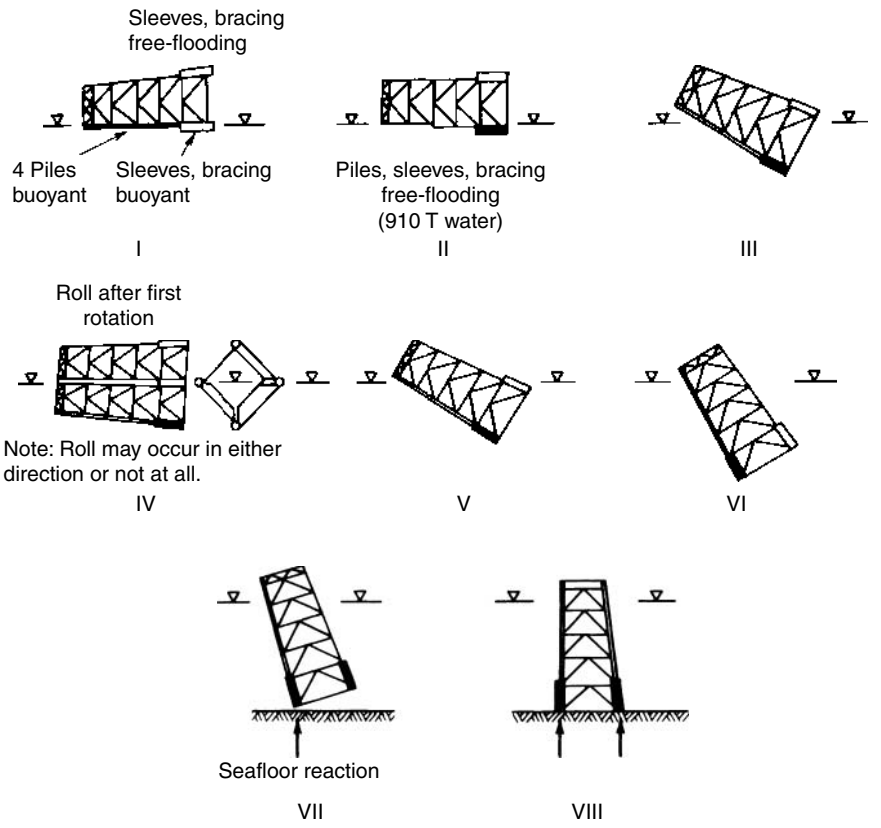

FIGURE 11.20
Upending of self-floating platform. (Ninian Southern, U.K., North Sea.)

by the derrick's own anchoring system, with the anchors being carried out by anchor-handling boats. Once set, a pull is taken successively on each anchor line to ensure the anchor is properly seated. The final location and orientation of the derrick barge is then established by means of survey, principally DGPS and electronic survey, but often keyed in to any preset acoustic transponders on the seafloor.

API RP2A requires that the anchor lines be of sufficient length for the water depth at the site and that the anchors and lines be of the proper size (weight) and shape to hold against the maximum combination of wind, current, and waves.

In deep water, the derrick barge's own anchoring system may have inadequate length of lines and hence mooring buoys are preset to which the derrick barge lines can then be run.

API RP2A also contains a rather curious section suggesting that, where holding ground is poor or the mooring system cannot be made fully adequate, the derrick barge should be located so that if the anchors do slip, the barge will move away from the platform. This provision may be appropriate for small platforms being constructed with marginal equipment, or may have been intended primarily for application at later stages of construction, when the jacket is firmly seated in place. However, for installation of major jackets, it would seem more appropriate to orient the barge so that it would have the minimum boom tip motion. Further, as piles are driven and later, as deck sections are erected, the barge has to locate itself within the limiting radii and sectors.

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Fortunately, the second set of criteria will sometimes match the first, in that the derrick barge will have its stern to the platform. The jacket location will then be guided by lines from the derrick barge, controlling not only its location but also its orientation.

To install a platform over an existing subsea well template, great precision and care are required in order to prevent damage to wells. The template will normally be held in place by piles although a gravity base could conceivably be employed. Two of the piles (or spuds from a gravity base) are fitted as guideposts, with tapered tops to engage cone-shaped funnels from the jacket. These guideposts will usually be decoupled from the template at this stage. Independent "bumper" piles may be used to protect the template during final positioning.

The jacket is brought into proper position, floating with several meters of clearance between the bottom of the jacket and the top of the guideposts. To provide full control of the jacket position during this operation, a second derrick barge will normally be moored on the far side of the jacket.

In early development of this technique, guidelines from the jacket top were attached and tensioned to give a visual indication of location and verticality. Today, with sophisticated sonar (acoustic) locators and transponders, plus video cameras and inclinometers that can be mounted on the legs, it is feasible to dispense with the guidelines and place full reliance on the instrumentation. Redundancy in instruments must, of course, be provided in the event of malfunction of any instrument. The jacket is now slowly ballasted down to engage the guideposts, and then on down to seafloor contact with the mud mats.

Whether set independently on the seafloor or over a well template, the jacket now must derive temporary support from the mud mats bearing on the soils at or just below the surface. The jacket must be self-supporting until pin piles can be driven. It is important that the jacket be level and remain so within a small tolerance until the piles are installed. The effective weight of the jacket on the bottom may be controlled by ballasting. This permits moderate adjustment of level of the jacket, which may be supplemented by moment induced by lines from the controlling derrick barges.

For large jackets, where precise leveling is required, jacking devices can be built into the connection between jacket legs and the mud mats. Commercially, available pile-supported leveling systems are available. These hydraulic leveling tools work in conjunction with temporary gripping devices which hold the jacket in position while pile-to-jacket connections are being completed.

The jacket must be approximately level when the piles are driven to avoid introducing unacceptably high bending stresses in the piles. Thus, leveling after piles are installed must be limited to relatively minor corrections. A careful evaluation must be made of the soil loadings during this phase. The jacket will be bearing at this stage either on the bottom bracing or on mud mats or a combination of both. The weight of the jacket must include any piles or conductors that are being supported by the jacket during the installation.

The bearing pressure on the soil must be within allowable limits under the combination of direct load and the pressure due to waves and current during the piling phase. API RP2A allows a one-third increase in allowable soil-bearing values during this phase if wave action is considered. This may be roughly acceptable in smaller installations. A much more thorough analysis is required for major structures, taking into account short-term consolidation settlements, and the effect of cyclic lateral and vertical strains. Scour around and under the mud mats must be prevented. This may require filter fabric and stone placement.

All structural elements bearing on the soil or supporting the mud mats must be adequate for the maximum bearing loads anticipated, including those due to storm. The design of the mud mats should also address the failure mode, to be sure that any

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structural failure will take place in the mat proper, rather than by damage to the permanent jacket legs or braces.

Mud mats were originally timber planks affixed to the bottom bracing to increase the bearing area. With major jackets, these mud mats are now structural steel, heavily reinforced flat plates, carefully designed to provide proper bearing. They are frequently tailored to fit the bottom contours; in the case of the Hondo platform off the Southern California coast, there was 20 m difference in elevation from the deepest to the shortest leg. This means that the jacket must be accurately oriented as well as positioned.

The jacket must also have resistance to lateral displacement. In competent soils, this may be increased by added ballast. Especially if a storm comes up or the derrick barge has to suspend operations before the jacket is adequately secured by piles, then the addition of ballast may be indicated. Another and perhaps better means of increasing lateral resistance is by having the pile sleeves or jacket legs extend a few meters below the mud mats, to act as spuds.

In mudslide areas, in areas of sand waves, and in very weak soils, jackets are being designed to penetrate well below the seafloor to provide frame action at depths up to as much as 15 m. This may also be required for mud slide zones and for unconsolidated sands, which may be subject to liquefaction in an earthquake. To enable jacket legs and pile sleeves to penetrate into soft soils, the addition of water ballast is normally sufficient. Bracing, however, is more difficult to penetrate because of its large area. Jets can be pre-installed, with nozzles acting along the underside of horizontal bracing to wash out material from under the bracing and lubricate the sides.

For self-floating jackets, which typically have two enlarged legs, or where pile sleeves are of large diameter, supplemental means may also be necessary to cause them to penetrate. Jets can be arranged inside to break up the plug and an airlift or eductor system employed to remove the material. These can be designed to operate below the pile closure.

For the Maui A platform off New Zealand, jet and airlift systems were built into the two enlarged legs, to enable the material within the large-diameter legs to be progressively removed.

In poor soils, one other way around the dilemma of providing vertical and lateral support to the jacket during this early phase is by driving four temporary piles to a short penetration only. The jacket can then be leveled by jacking, lifting, or ballasting, and temporarily welded off to these four piles.

The "temporary piles" may be four of the permanent piles, driven initially only to a small penetration. They would typically have been transported with the jacket, to expedite their release and installation. Sometimes these short piles are made permanent and used as spuds for lateral support. In most cases, after the remaining permanent piles have been driven, these temporary piles are cut loose, and raised as necessary to release any bending stresses. Add-ons are welded on and the lengthened piles are now driven to final penetration.

11.7 Pile and Conductor Installation

The jacket, now temporarily supported on the seafloor, is ready for pile installation. Some pile sections may have been transported with the jacket. The initial add-ons are welded and the piles driven, as described in detail in Chapter 8. In some cases, only a few piles are driven from a floating derrick. A work deck may have been pre-installed on the jacket or

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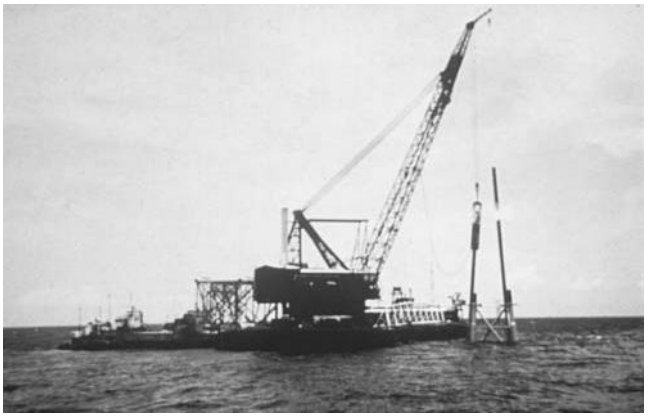

FIGURE 11.21
Setting piles through jacket legs.

may be now placed. On this deck, cranes may be set, so that all further operations may be carried out from the platform itself (see Figure 11.21).

Fully self-installing platforms have been designed; these have a stiff-leg derrick pre-attached to a work deck, the whole built into the jacket, so that upon upending, the stiff leg may erect itself, then pick and drive piles. Whether such a solution is an acceptable one in any specific situation will depend on the remoteness of the location, availability of offshore derrick barges, sea and weather states during the installation period, and ability of the soil to support the temporary loads of the jacket, with work deck, stiff-leg derrick, and live loads.

The piles will penetrate the jacket closures as they are initially dropped. The grout seals at the base of the sleeves will keep mud out of the jacket leg as the piles are driven to final penetration. After the piles are driven, the jacket is leveled and the final connection is made (see Figure 11.22 and Figure 11.23). Grouting is described in Chapter 8. Grouting of piles in jacket legs is also an effective way of stiffening the gross section and preventing local buckling, as, for example, at nodes where the bracing members intersect the legs.

Piles that extend on through legs of the jacket to the deck can also be secured to the jacket by welding; this system has been much used in the past where piles extended up above water. It is also currently employed in offshore terminal construction where jackets and pin piles are employed. The transfer of high cyclic axial loads from the top of the pile

FIGURE 11.22
Driving pile through sleeves of jacket.

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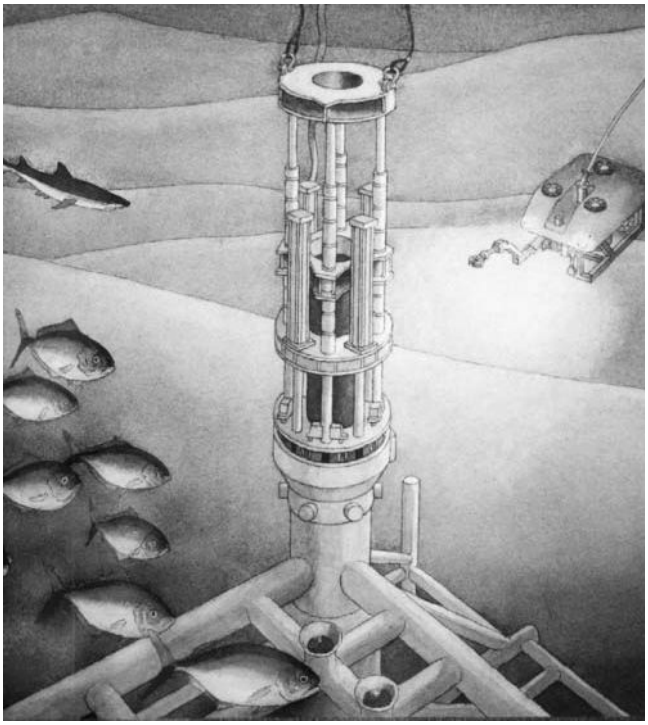

FIGURE 11.23
"Latch-lok" tool for leveling jackets. It reacts against driven pile. (Courtesy of Oil States Rubber Co.)

into the jacket leg requires careful consideration of weld details, since the welding will have to be carried out under adverse conditions of wetness (spray), perhaps low temperature, and while the jacket may be vibrating under wave action. Grouting helps to reduce vibration and limit excessively high stresses at the welds.

Steel shims are used to center the pile in the jacket leg; these are usually one quarter or one third segments of steel pipe of the proper radius. The welds are best designed as shear welds, from the pile to the shims, then from the shims to the jacket leg. A developed section of this detail is shown in Figure 11.24.

The lateral resistance of the installed platform is developed by the P/y (lateral load-deflection) of the pile-soil system, which in most soils takes place over the top five embedded diameters of the pile. Since this is normally the zone of weakest soils, lateral resistance may be critical. Several schemes to enhance the properties of the soil in this zone have been proposed, including built-in drainage systems at the tip of the jacket legs (or sleeves). To prevent annular gaps from forming around the pile under cyclic wave loads, pea gravel has been dumped on the seafloor around the pile: as a gap opens on one side, the pea gravel works down and wedges the pile, preventing progressively increasing displacements.

For the same reasons, it is important to prevent scour around the piles and bottom of the jacket, both when the jacket is temporarily supported by the mud mats and in service. Some prospective areas for platforms, for example, Sable Island off Nova Scotia, have sandy seafloors and high bottom currents, and have shown rapid scour around the legs of jack-up drilling platforms. Shallow-water areas, with sandy bottoms, where wave action may be severe, are especially suspect, since scour due to eddy action may be augmented by the pumping action of the jacket vibrating and rocking under the waves.

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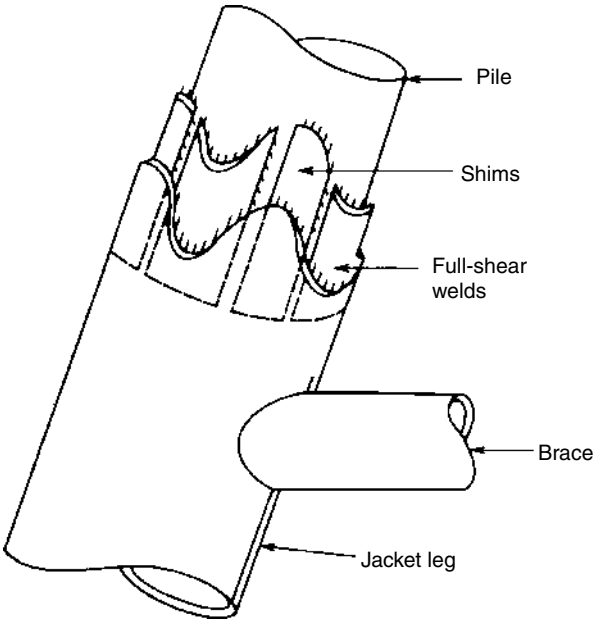

FIGURE 11.24
Scalloped connection for joining pile to jacket leg.

Scour protection around jacket legs can probably be most expeditiously and practically accomplished by the placement of graded rock through a long tremie pipe. Obviously, the depth of practicability is limited, but fortunately, so is the depth at which scour action usually occurs. Alternatively, controlled dumping from the surface has been utilized with generally satisfactory results but is obviously difficult to achieve without excessive quantities of material. Articulated concrete mats can also be employed.

Conductors are now installed, in much the same manner as piles. The lower section of some of the conductors may have been carried out with the jacket, but for the most part, they are transported by barge, threaded in through the conductor guides, extended by add-ons, and driven to the required penetration. Since they are usually of smaller diameter and thickness than piles, that is, about 750 mm diameter and 25 mm walls, and usually penetrate to less depth than the piles, they are easier to drive, and smaller hammers can be used. Their penetration requirement is determined primarily by the ability to seal off flow during drilling, so that drilling mud will not escape to the sea. They must also be driven to a sufficient depth to prevent escape of shallow gas, which could form a flow path for future release. Conductors also must provide vertical support to the wells. Conductors may be installed by the drilling rig, which may use either a pile hammer or drilling and jetting techniques. In mudslide areas, the conductors may be enclosed within a larger-diameter tubular, which provides the strength and stiffness to resist the lateral forces from the moving mass of mud. In other areas, such as Cook Inlet, Alaska, the conductors were drilled through the supporting piles, using the latter as protection against ice.

11.8 Deck Installation

API RP2A requires that the deck elevation be within plus or minus 75 mm from the design elevation and shall be level. The degree of level is usually limited to about 300 mm

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FIGURE 11.25
Lifting deck onto top of piles jacket. Note stabbing guides on legs. (Courtesy of J. Ray McDermott, S.A.)

differential height across the longest dimension of the platform, but in any event should ensure proper drainage and proper operation of processing equipment.

Deck sections are now to be lifted on. With smaller platforms, the "pancake" concept was often adopted, in which some of the permanent equipment was pre-attached to decks, with each deck of the platform being lifted on in succession. After each deck was erected, remaining equipment for that deck was set.

With larger platforms, the "deck" now consists basically of module structural support frames, consisting of girders and trusses onto which large modules of assembled and integrated equipment are set. The initial sections have legs extending below them, with stabbing guides to fit into the piles or jacket legs. The stabbing guides are so configured that they also act as backup plates. Since the mating leg is the same diameter and wall thickness as the extension to which it is to be joined, a full penetration girth weld is made, similar to the splice in a pile (see Figure 11.25).

To aid in stabbing four legs into four sleeves, the stabbing guides may be made slightly different in length so that one can be entered first, then the module rotated so the second can be entered and then the whole lowered to fit the remaining joints.

Transport of a large deck module will usually be by barge, although smaller modules and equipment may be transported by supply boat or on the derrick barge itself. The weight of modules has grown in recent years, to 500 tn., then 1000, and most recently, over 10,000 tn. Mammoth derrick barges are now available to lift this weight and more. The purpose is to enable more complete assembly onshore and reduce the time and cost of offshore hookup. These monstrous lifts require calm seas and a derrick barge with minimum response to the seas.

The latest generation of heavy-lift derrick barges is of the semisubmersible type. This, of course, is a trade-off between the reduced motion response of the barge and the concomitant reduction in stability, especially in roll, as the load is lifted. For this reason, heavy lifts are generally made over the stern, with the swing being used only for minor adjustments in position to engage the stabbing guides; in fact, for heavy lifts such as these, a sheer-legs crane barge is also suitable, with the minor positioning adjustments being made by the deck engines. The largest offshore derrick barges in the North Sea now are fitted with two huge cranes, one on each stern quarter, so that their combined capacity may be used. For heavy lifts, boom tip response is very critical. Onboard minicomputer programs have been developed to optimize the heading and boom angle.

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The derrick barge is pulled back from the platform, and the cargo barge, with the large deck unit on deck, is pulled in across the stern. The lift is made and the cargo barge pulled clear. The derrick barge now pulls astern to the platform, where it sets the deck unit. Because of the numerous parts of line needed for such heavy lifts, getting rid of the load after having landed it on deck is often a problem, even with free overhaul release. The stabbing guides must be designed to remain engaged, once entered, since otherwise on the next heave cycle, they may disengage.

Loadout and transport of these large deck units and modules on a cargo barge requires procedures and sea fastenings similar to those of the jacket, with added complications. The unit, with its four or more legs extending downward, is difficult to support. Large reinforced plate-bracket assemblies are needed to distribute the load (static plus dynamic) over the deck so the leg will not punch through, and can be supported both on the skidway and on the barge. The center of gravity of the unit is high above the deck; the tie-downs must provide adequate lateral support to resist the lateral forces due to roll angle and accelerations during transport.

A typical deck module tie-down arrangement is shown in Figure 11.26. This shows one method of supporting the leg. Supports may also be built up to provide direct support to the module frame itself. Further description of module erection and hookup is given in chapter 16.

Decks have thus undergone a series of major evolutionary developments, spurred by the recognition that the greatest portion of the total costs of an offshore platform is generally in the processing and support equipment and the greatest labor demand is in hookup and testing. Operating with high volumes (50,000–300,000 barrels of oil per day), high pressures, and high volumes of gas requires precision assembly and thorough testing. Hookup is a major demand on skilled manpower and on a large platform can require 2–2.5 million man-hours. Significant savings in cost and time can be achieved by carrying out this work under favorable conditions at inshore shipyards.

At the same time, the total deck "payload" has grown from 7000 tn. to more than 50,000 tn., partly because of increased requirements such as gas re-injection, and water

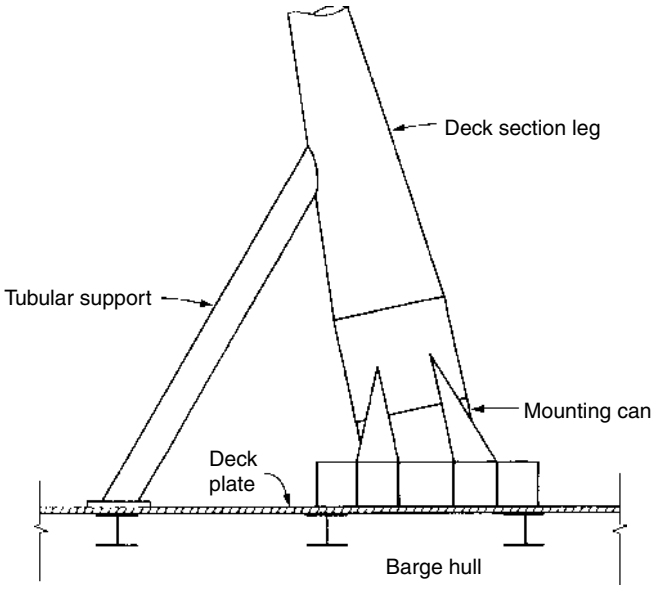

FIGURE 11.26
Sea fastening for transport of jacket.

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flooding, partly because of more remote and demanding environments requiring weather protection, which in turn requires more ventilation, large helicopter services, and greatly enlarged quarters and support. The typical jacket-pile structure is very sensitive to total deck load. One way to reduce the total is by integrating the deck or at least the modules to make more effective use of the deck structure. The evolution then has been from individual deck sections and individual pieces of equipment, to module support frames and large integrated modules, to completely integrated decks.

Float-over decks are a dramatic new development which enables the prefabrication of the complete topsides, so that it may be transported by barge and set as a complete unit on the preinstalled jacket. These are discussed in Section 17.5.

11.9 Examples

In the following pages, a description is given of the installation procedures employed for three landmark offshore platforms: Hondo, Cognac, and Cerveza. Each of them employed important new techniques which will have an influence on future platforms, especially those in deeper water.

11.9.1 Example 1—Hondo

The Hondo platform installation off the coast of Southern California in 270 m of water was carried out in a unique fashion, by launching the jacket in two halves, then mating them afloat, and upending as a single jacket (see Figure 11.27 through Figure 11.32). The

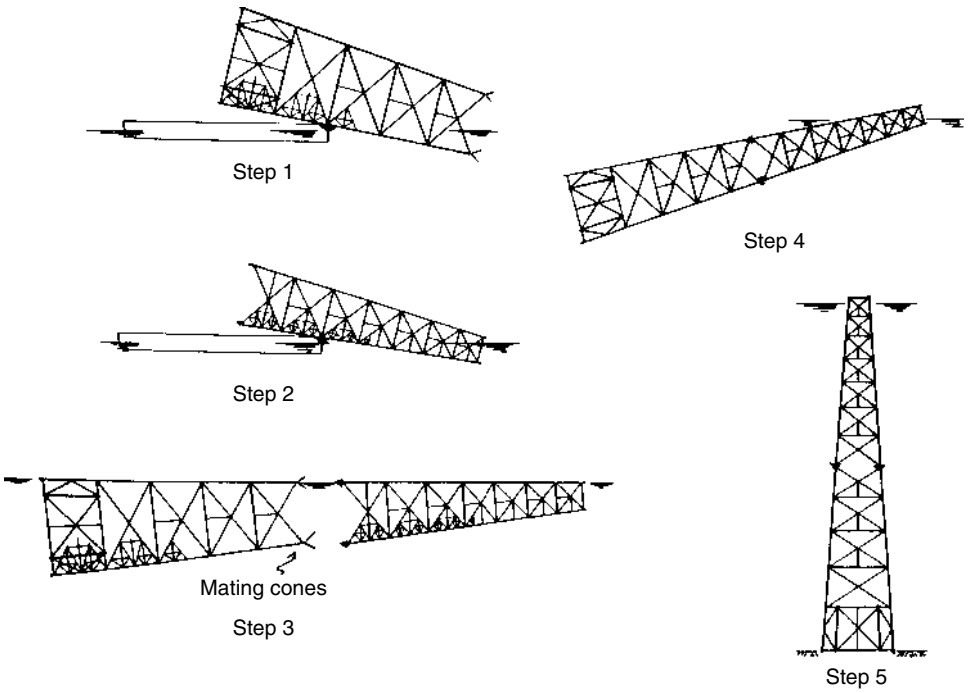

FIGURE 11.27
Assembly of jacket of Hondo platform.

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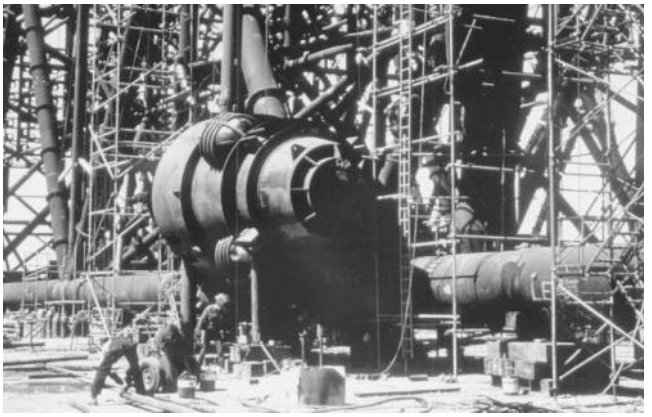

FIGURE 11.28
Female mating cone for Hondo jacket.

FIGURE 11.29
Male mating cone for Hondo jacket.

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FIGURE 11.30
Loadout of lower half of Hondo jacket. Note steel mud mats at lower end.

two halves of the jacket were constructed on a single long fabrication ways at Oakland, California, to ensure exact match of the huge space frame. At the juncture between the two sections, mating cones were provided, each with a series of hydraulic ram connectors.

FIGURE 11.31
Tow of lower half of Hondo jacket.

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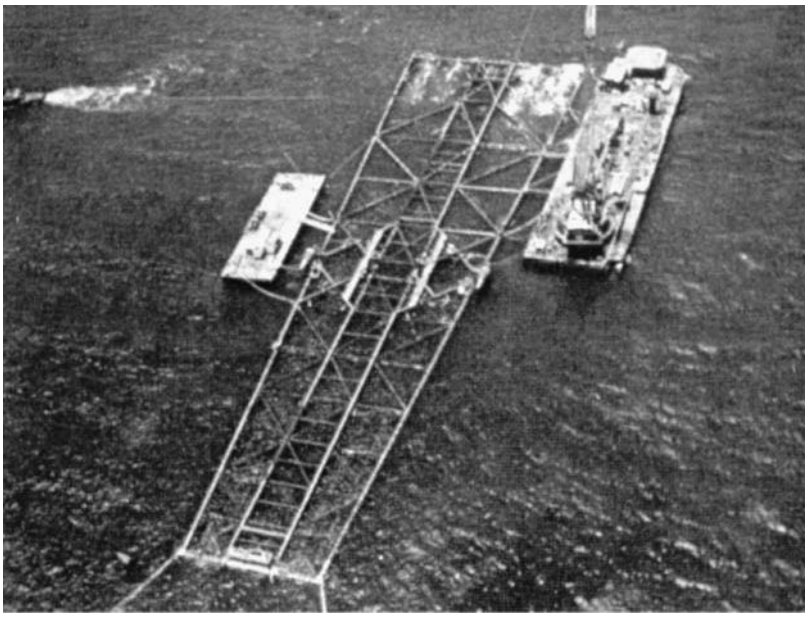

FIGURE 11.32
Mating of two halves of Hondo jacket in open sea. (Courtesy of J. Ray McDermott, S.A.)

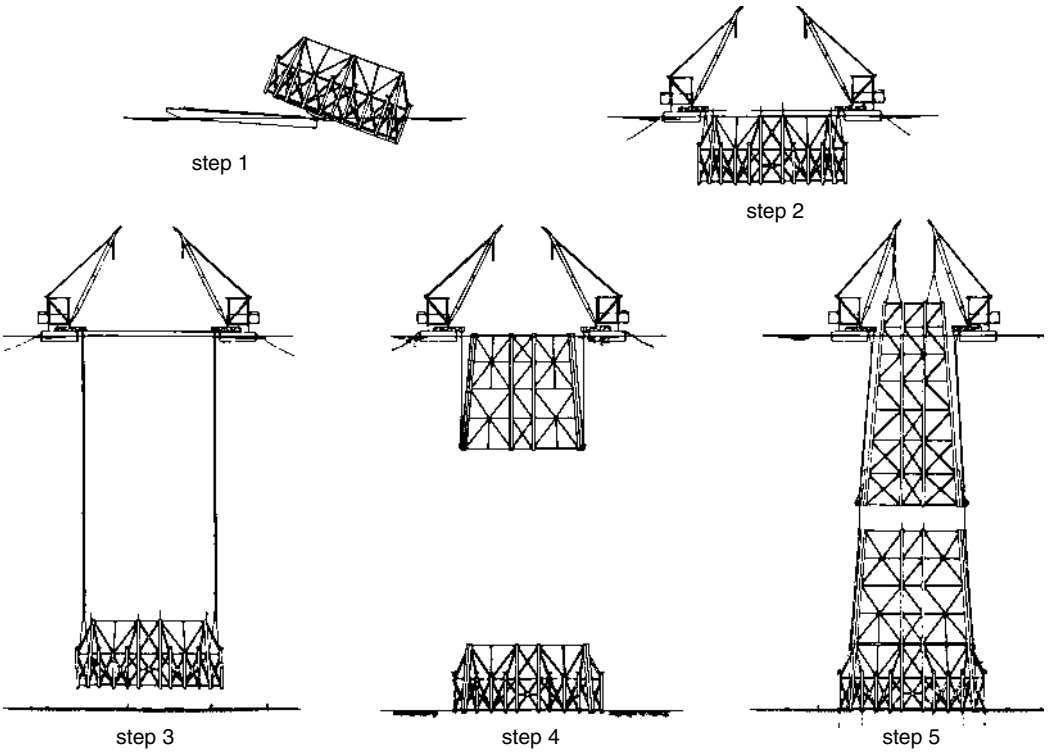

FIGURE 11.33
Assembly of jacket for Cognac platform. (Courtesy of Shell Exploration and Production.)

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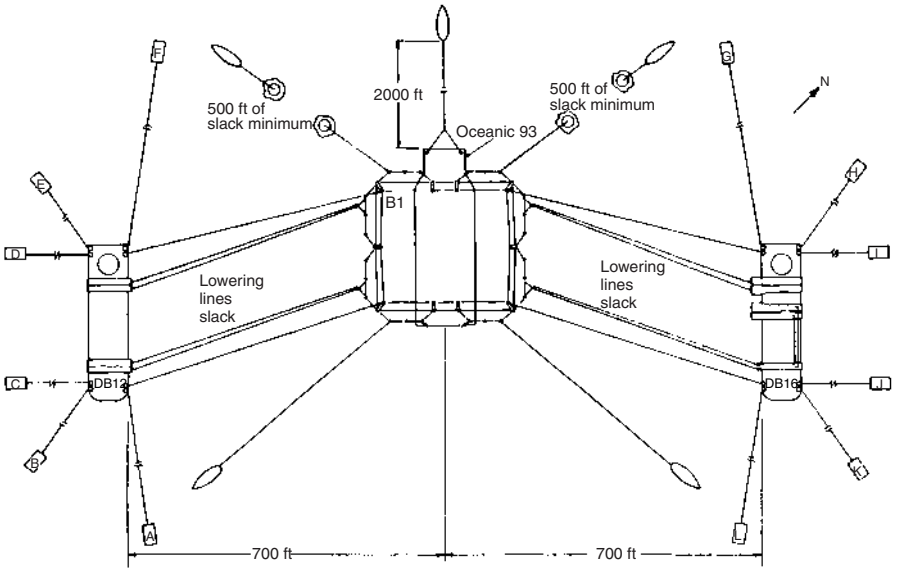

FIGURE 11.34
Launch configuration of mooring spread for Cognac. (Courtesy of Shell Exploration and Production.)

Each half was separately loaded onto a large launch barge, towed to a semiprotected site east of Santa Rosa Island, and launched in conventional fashion. The two halves now floated on their upper legs, with a freeboard of only about 1 m. The two halves were aligned and pulled together, the mating cones engaged, and the hydraulic ram wedges

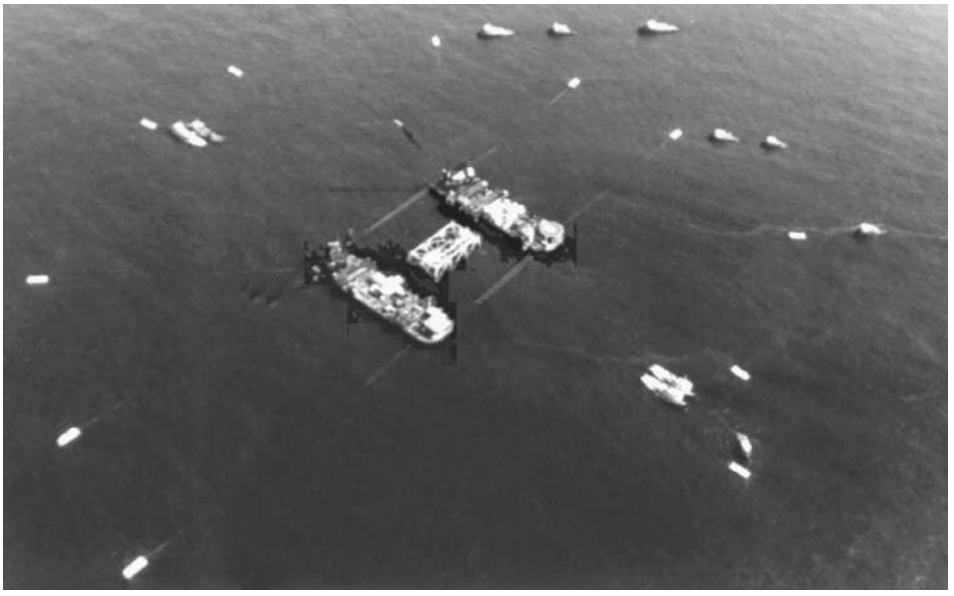

FIGURE 11.35
Mooring buoys and crane barges for assembly of Cognac jacket. (Courtesy of Shell Exploration and Production.)

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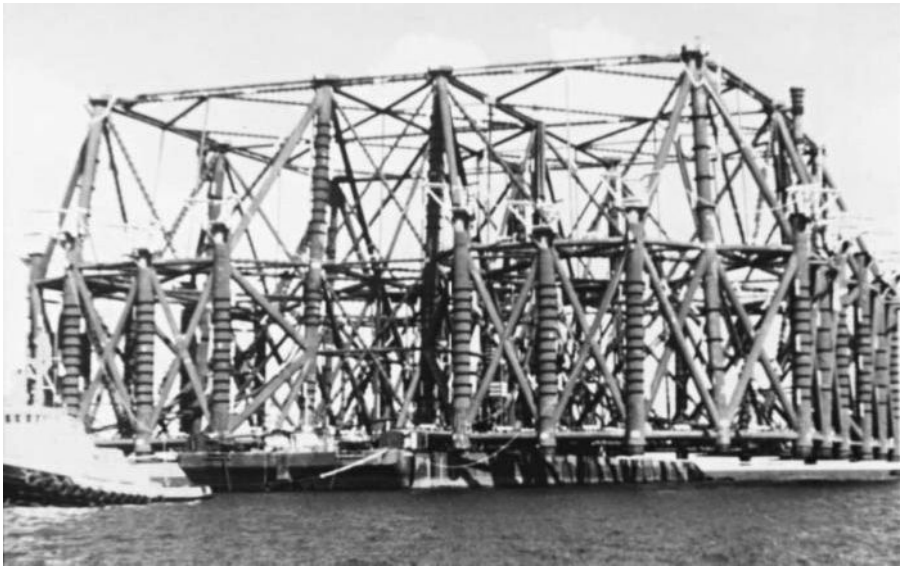

FIGURE 11.36
Base segment of Cognac platform on launch barge. Note overhang. (Courtesy of Shell Exploration and Production.)

activated. Full-penetration welds were run on the inside of the corner legs of the platform. Access to the two legs 50 m below water was through special tubes, which also provided ventilation, power, and light for the welder. A similar mating procedure has been planned for the guyed tower concept when applied to water depths beyond those practicable for transporting and launching a single-piece jacket.

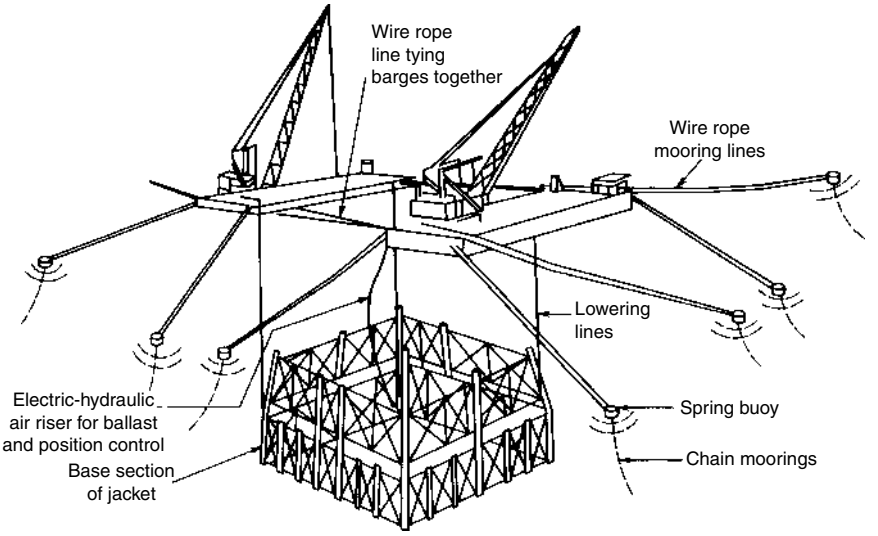

FIGURE 11.37
Installation of Cognac jacket base. (Courtesy of Shell Exploration and Production.)

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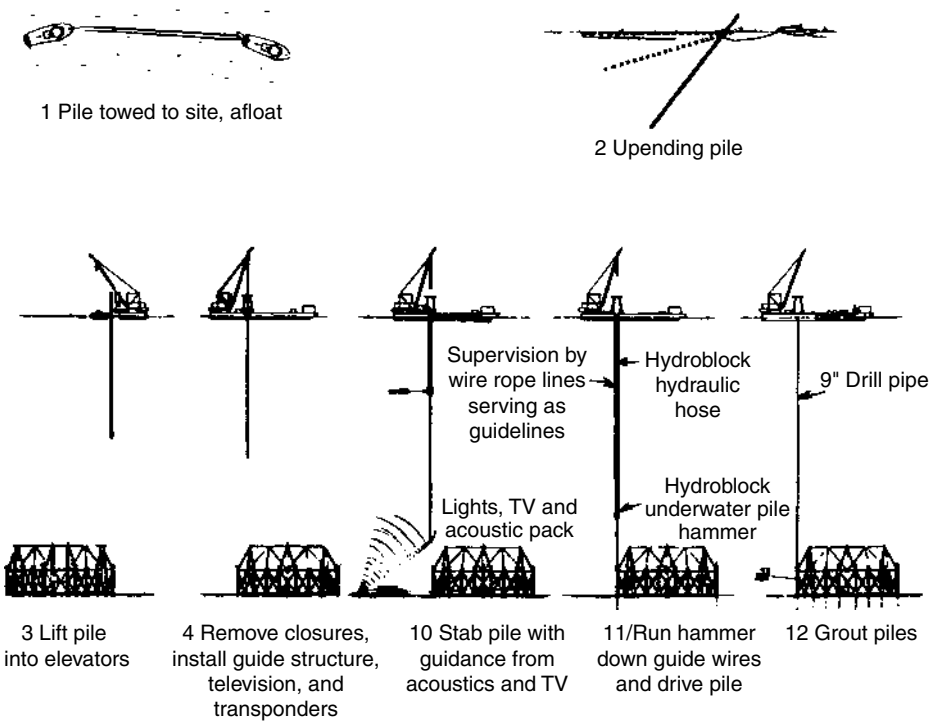

FIGURE 11.38
Pile installation for Cognac. (Courtesy of Shell Exploration and Production.)

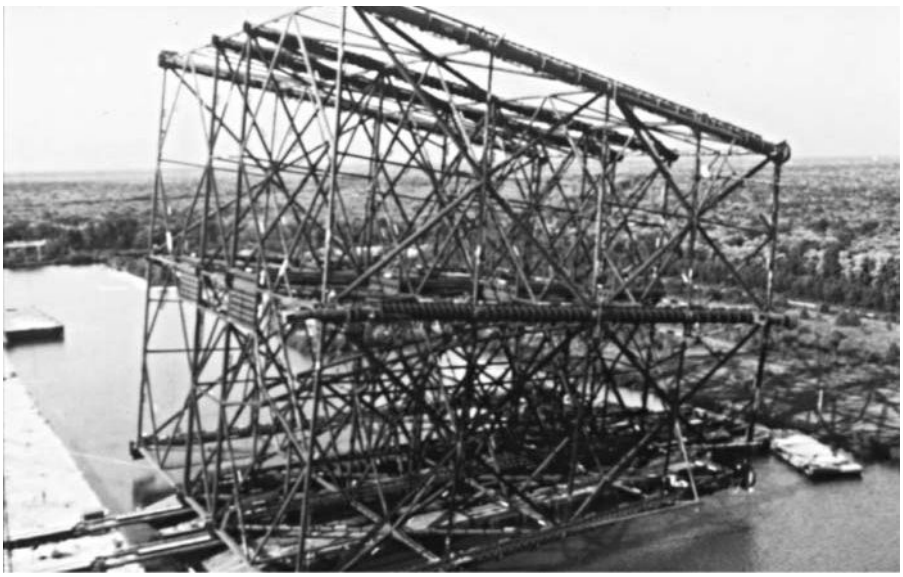

FIGURE 11.39
Mid section of Cognac on launch barge. (Courtesy of Shell Exploration and Production.)

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FIGURE 11.40
Launching of mid section of Cognac. (Courtesy of Shell Exploration and Production.)

FIGURE 11.41
Installation of mid section of Cognac. (Courtesy of Shell Exploration and Production.)

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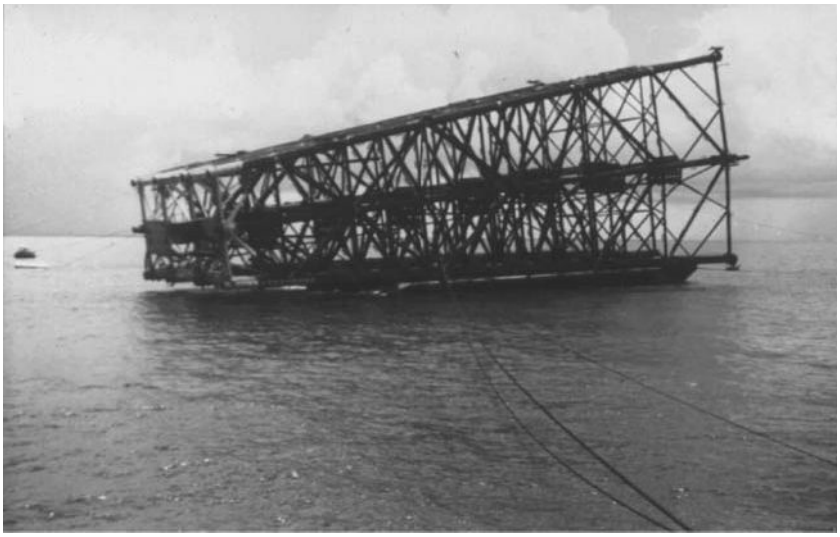

FIGURE 11.42
Launching of top section of Cognac. (Courtesy of Shell Exploration and Production.)

For the Hondo platform, pile sleeves were sealed with a double set of heavily reinforced neoprene pile sleeve closures designed for the hydrostatic head at 270-m depth. When the piles were later released, they did not penetrate under their own weight and had to be driven through the closures. Subsequent piles were cut with a serrated bottom edge which facilitated penetration of the closure.

11.9.2 Example 2—Cognac

This platform was the first truly deep-water platform, breaking the 300-m-depth limit. The jacket was constructed in three sections: base, middle, and top. The installation of this

FIGURE 11.43
Preparing to launch top section of Cognac. (Courtesy of Shell Exploration and Production.)

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FIGURE 11.44
Up ending top section of Cognac. (Courtesy of Shell Exploration and Production.)

platform also served as a proving ground for many advanced deep-water techniques: acoustic and video positioning devices, deep diving, and underwater hydraulic hammers. The sequence of construction is shown in Figure 11.33.

At the site, twelve large mooring buoys were set, each with three anchors. Two offshore derrick barges were positioned, one on each side of the platform site (see Figure 11.34 and Figure 11.35). The lower section was built vertically (in the same orientation as its final position), transported, and launched (see Figure 11.36). It was then positioned between the derrick barges, given slight negative buoyancy, and lowered to

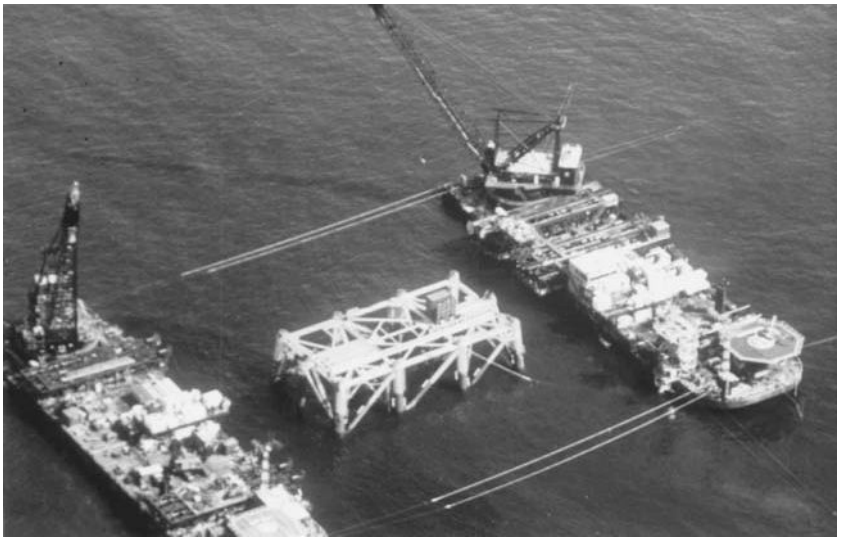

FIGURE 11.45
Placing top section of Cognac. (Courtesy of Shell Exploration and Production.)

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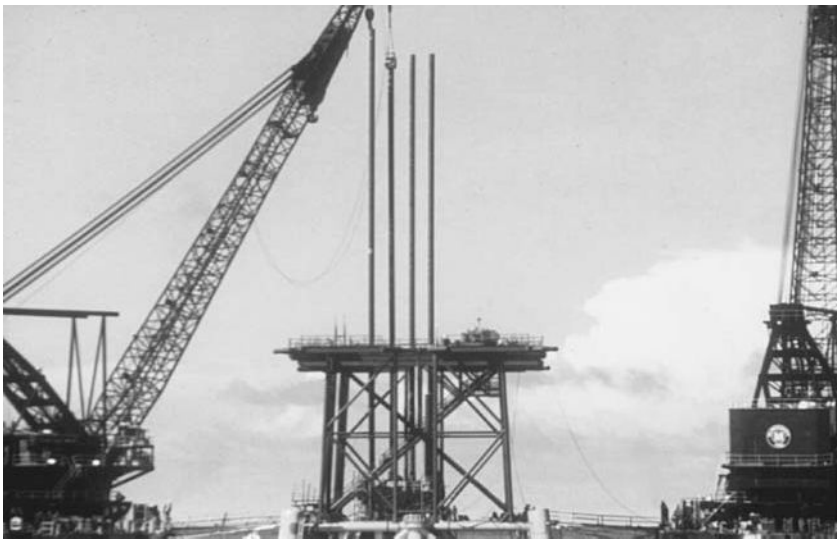

FIGURE 11.46
Installing conductors in Cognac. (Courtesy of Shell Exploration and Production.)

the seafloor by means of four 3½ in. (89 mm) and four 3-in. (75 mm) wire lines. Control of the structure was by selective ballasting, by means of the electric hydraulic riser (see Figure 11.37).

Intensive studies were made of the dynamic responses during this lowering, and strict limitations imposed on the sea conditions which would affect the derrick barges during this critical operation. All lowering lines were equipped with motion compensators.

The piles were now transported full length, self-floating. Each was upended by the derrick, and then inserted in the sleeve almost 200 m below water, guided by sonic and video devices and entered into the funnel. The hydraulic hammer, an HBM Hydroblok

FIGURE 11.47
Completed Cognac platform in production.

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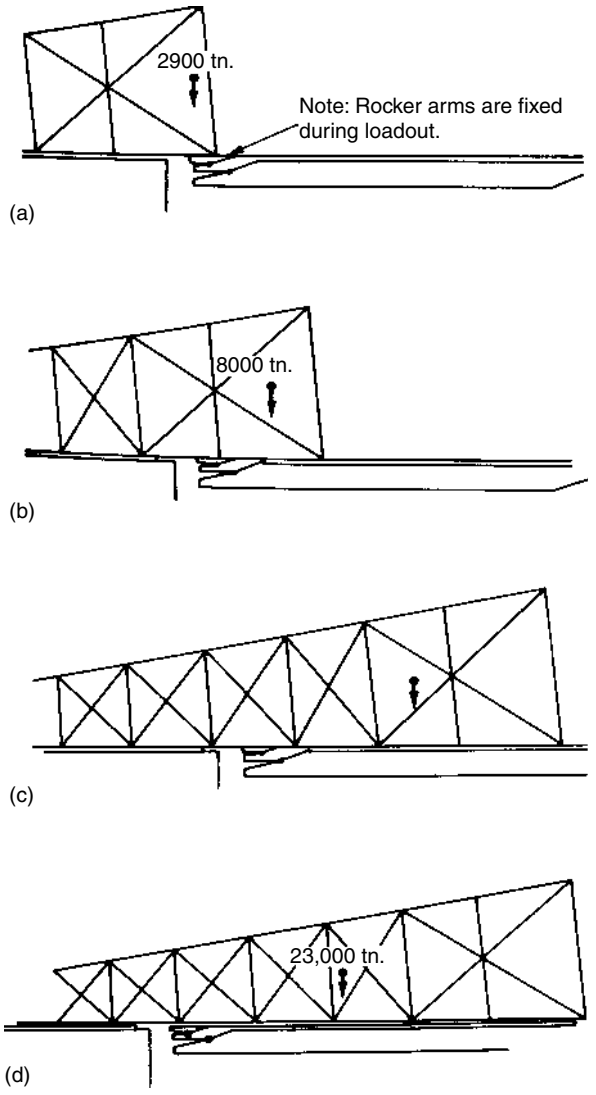

FIGURE 11.48
Load out of jacket for Cerveza.

hammer rated at 800,000 ft.-lb per blow, was then lowered onto the pile and guided by tensioned guidelines and acoustic transponders. The pile was driven to full penetration. There were twenty-four piles, each 2100 mm in diameter and 140 m long, weighing 465 tn. (see Figure 11.38).

Piles were then grouted to the sleeves of the lower section using a drill pipe. This completed one season's work. The next season, the middle and top jacket sections were transported, launched, and upended. Each was then lowered to mate, with mating cones, into the previously set jacket section. Guidance was by acoustic transducers, video, diver visual reports, and mating funnels. Hydraulic clamps were activated to temporarily fix each section to the other. To join these sections together, ten large pile-like tubular dowels, each 1800 mm in diameter and 312 m long, were inserted through the jacket legs and grouted to all three sections (see Figure 11.39 through Figure 11.47).

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Observation and monitoring of all underwater operations was carried out by a remote-controlled vehicle (ROV) and by divers, using TV as well as direct visual observation.

11.9.3 Example 3—Cerveza

This jacket-pile platform was located in water almost as deep as Cognac, yet was fabricated and installed as a single piece. The jacket was 250 m high and weighed 24,000 tn.

For loadout, the launch barge was ballasted down to the draft of 10 m so that the top of its launchways was 200 mm below the top of the fabrication skidways. Then, as the jacket was pulled onto the launch barge, the barge was progressively deballasted to pick up the load of the jacket (see Figure 11.48). Once the jacket was fully on the barge but still overhanging the fabrication skidways, the barge was further deballasted to lift the jacket clear. During loadout, closely spaced paint marks on the jacket served to guide the ballast controller. The rocker arms of the launch barge were locked during this operation.

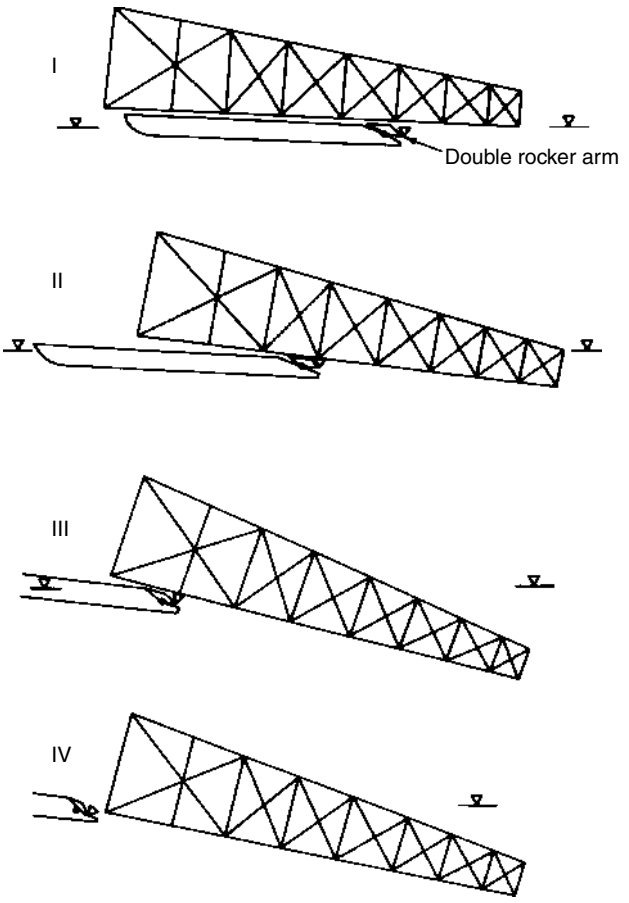

FIGURE 11.49
Launching of jacket for Cerveza.

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The jacket was positioned on the barge for minimum draft and level trim during tow, with a 280 ft. overhang on one end. The launch barge was 200 m long, 50 m beam, and 12 m in depth.

In preparation for launch, the barge was ballasted with 29,000 tn. of water which trimmed it down by the stern 3°. The static coefficient of friction of the jacket on the launchways was 0.11 and required a jacking force of 1400 tn. to initiate launch. The dynamic coefficient of friction was 0.05 (see Figure 11.49).

The jacket attained a launch velocity of 3 m/s, a maximum dive angle of 13°, and a maximum dive depth of 80 m. During this launch, the maximum submergence of the barge keel was 24 m, an indication of the need to ensure that launch and similar barges can withstand excess hydrostatic heads without imploding.

Personnel on the launch barge were kept clear of the outboard side of the launchways; in between they were safe as the jacket moved over their heads. The key feature of the launch was the use of the double-rocker arms, which reduced the forces on the jacket legs and on the barge. The jacket when launched had a reserve buoyancy of 10%, that is, 2400 tn. Upending was carried out by ballasting two pile sleeves with tops closed to trap air

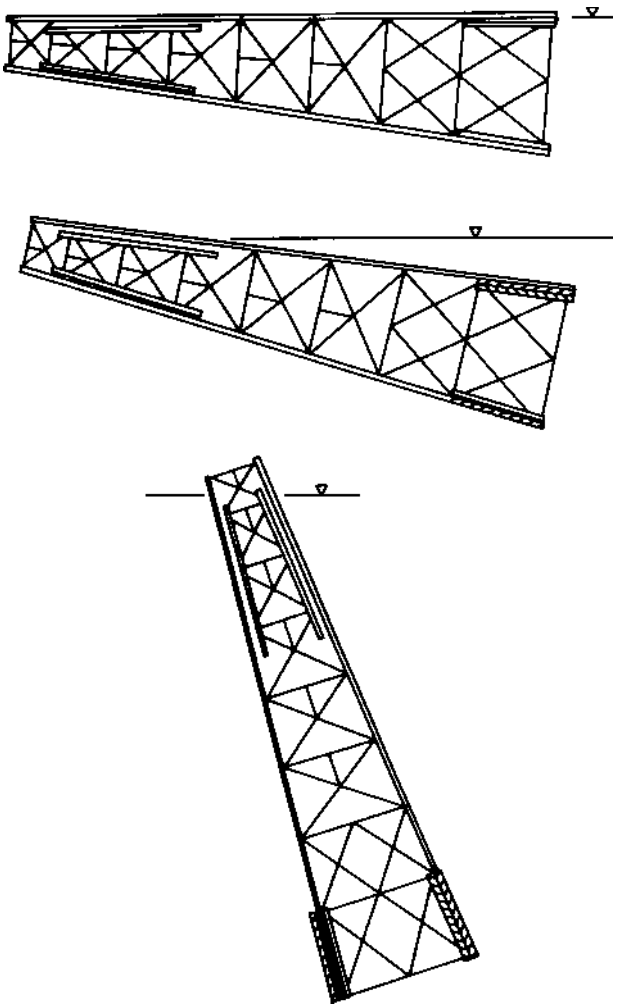

FIGURE 11.50
Upending of Cerveza jacket.

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and then two more which were open to the atmosphere. The jacket then upended to a stable vertical attitude (see Figure 11.50). The controls for upending were on board the jacket, on a temporary control skid. The jacket was upended against the current, at a site 1 ½ miles away, in 400 m of water, giving the tugs time to correctly orient the jacket as it drifted back to location.

Control of final lowering and touchdown was by the use of sonic depth sounders located on each corner leg. All skirt sleeves had one flooding valve and one venting line. The grout lines were available to inject compressed air to deballast if necessary. Valves were hydraulically operated, with a spring return. Indicator tubing from each compartment led to the control board, enabling the state of flooding to be monitored.

A contingency plan was prepared to be able to deal with all postulated emergencies during upending. In addition to the air pressure indicator tubing, inclinometers gave continuous readouts on attitude. It was recognized that variations in the current with depth might affect verticality. Any damage to pile sleeves or in-leakage was able to be offset by ballasting, valve closure, or air pressure. In case of other damage, for example, to bracing, the jacket would be brought back to horizontal for repair.

Thus this mysterious divine Pacific
zones the world's whole bulk about;
Makes all coasts one bay to it; seems
the tide-beating heart of Earth.
It rolls the mid-most waters of the
world, the Indian Ocean and
The Atlantic being its arms.

Herman Melville, Moby Dick

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12

Concrete Offshore Platforms: Gravity-Base Structures

12.1 General

Offshore platforms of the gravity-base category are designed to be founded at or just below the seafloor, transferring their loads to the soil by means of shallow footings. Such gravity-base platforms have usually been constructed of reinforced and prestressed concrete, but a few have been built of steel or a hybrid of concrete and steel.

Concrete platforms are almost always constructed in their vertical (final) attitude, enabling much or all of the deck girders and equipment to be installed at an inshore site and transported with the substructure to the installation site. These structures are usually self-floating. When necessary, additional lift forces may be developed by temporary buoyancy tanks or special lifting vessels.

To minimize soil-bearing loads, these structures have a large base "footprint." To provide buoyancy, they have large enclosed volumes. They thus generate much greater inertial forces under waves, earthquake, and impact from vessels or iceberg, 30,000–100,000 tn. of lateral force being typical, with individual structures developing even more. Thus, sliding tends to become the dominant mode of failure, at least for water depths up to 200 m.

To transfer this lateral load into the soil and thus prevent sliding, concrete or steel skirts and dowels are employed, designed to penetrate and thus force the failure surface farther below the seafloor. Such skirts also provide protection against scour and piping. While the skirts are typically fixed to the base of the platform during fabrication, in special cases where shallow water limits draft, spuds may be installed through sleeves after the structure has been seated on the seafloor. A typical gravity-base structure (GBS) structure is shown in Figure 12.1 and Figure 12.2.

The construction of a typical concrete gravity-base platform takes place in a well-defined sequence of stages. For each stage, there are several important criteria that must be met:

1. While afloat, the structure must be watertight and have stability and freeboard at all stages of construction.
2. The loading conditions and combinations acting on the structure are significantly different from one stage to the next. Structural integrity must be assured at each stage.
3. Ballasting and compressed-air systems (if these latter are employed) must be carefully and positively controlled at all stages.

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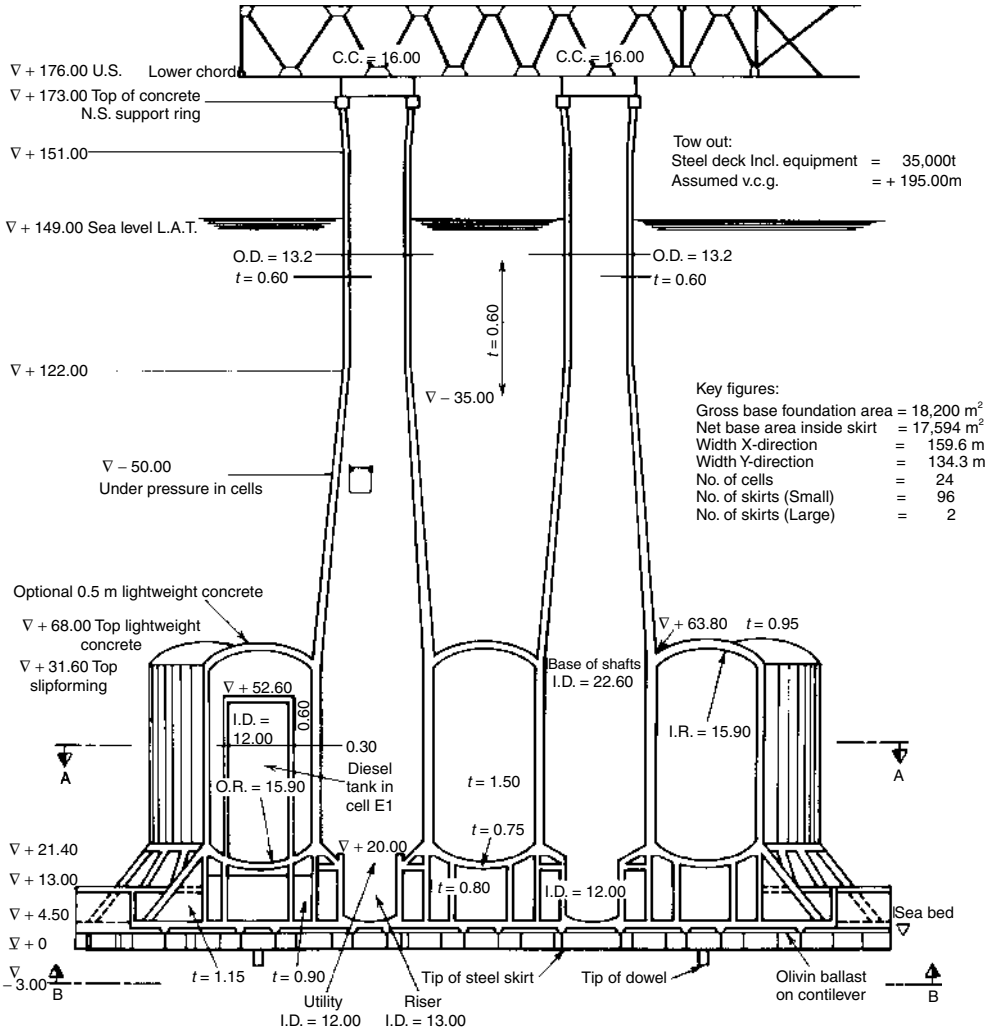

FIGURE 12.1
Statfjord B GBS offshore platform.

To meet the above criteria, it becomes necessary to control weights and dimensions with great care. These structures are often very large and massive, extending 100–200 m or more on each of the three axes.

The sequence shown in Figure 12.3 through Figure 12.7, inclusive, shows fifteen stages of construction. The number of stages has been purposely abbreviated in order to give the overall pattern of construction. There are numerous substages in each main stage. Each such stage must be carefully analyzed to be sure all criteria are met from the beginning to the end of that stage.

Most errors to date have been due either to overlooking an intermediate stage or to combining two or more stages to save computational effort. Detailed sketches of each stage, along with evaluation of the pertinent hydrostatic, hydrodynamic, and structural loadings, must be prepared to enable visualization by both design and construction engineers.

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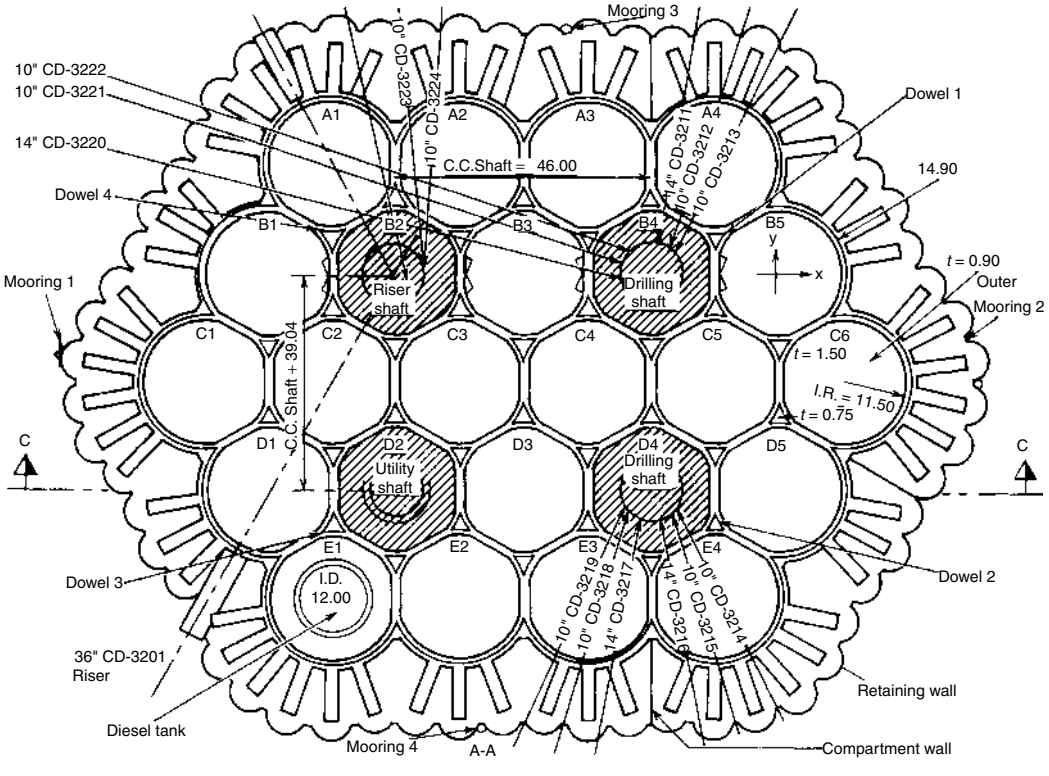

FIGURE 12.2
Base raft of Statfjord B platform, showing struts to cantilevered slabs. (Courtesy of Mobile and Aker Maritime.)

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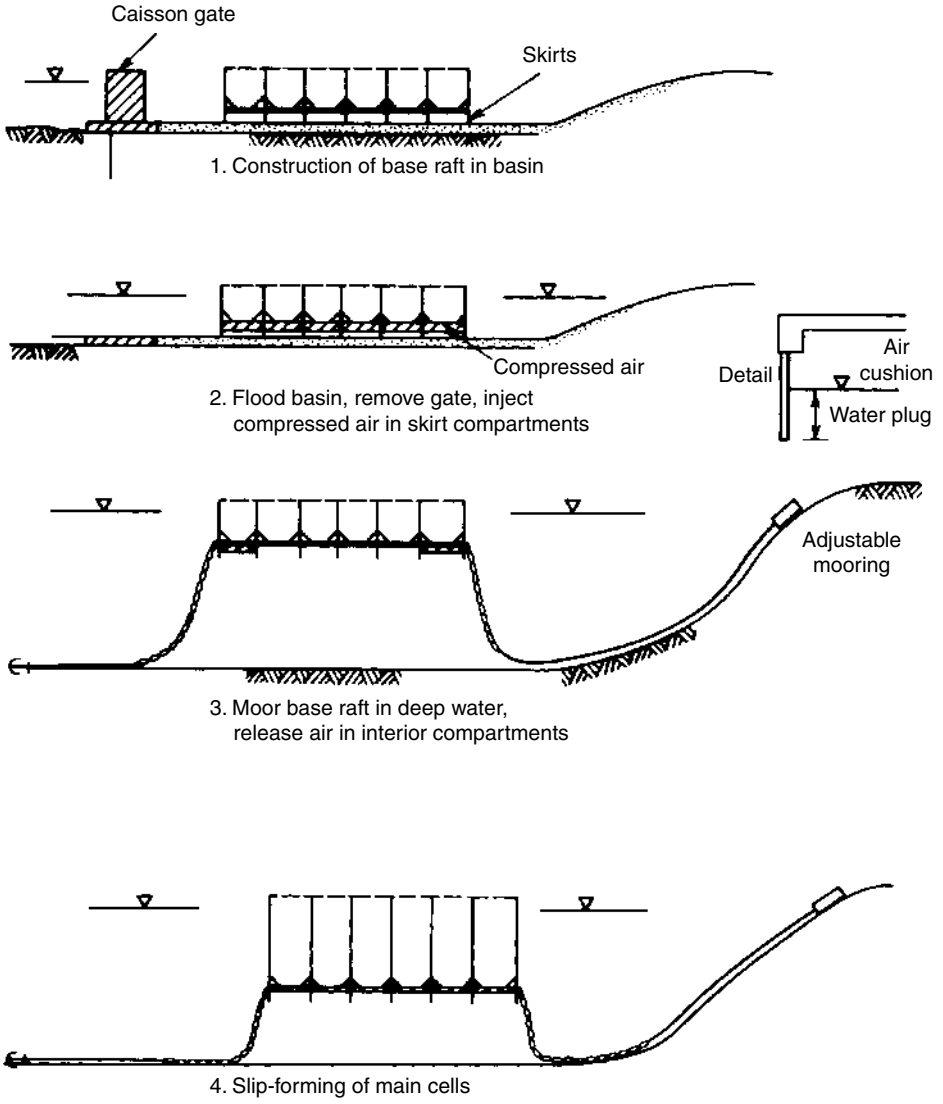

FIGURE 12.3
Construction stages of GBS (stages 1–4).

The internal subdivisions of the structure are subjected to differential pressures, primarily due to the different ballast water heads acting on each side. Compressed air may be used on occasion to pressurize a compartment; the forces occasioned thereby must be considered.

Accidental conditions must also be considered: the loss of compressed air from under the base skirts on one side, rupture and flooding of one compartment due to collision from a boat, a broken ballast pipe, failure of a valve to close, or a failed penetration. Under such accidents, the structure may be permitted to suffer minor local distress so long as its integrity, stability, and buoyancy are maintained. Progressive collapse—for example, where one compartment floods, overloading the adjoining bulkhead which in turn fails, and so on—cannot be permitted.

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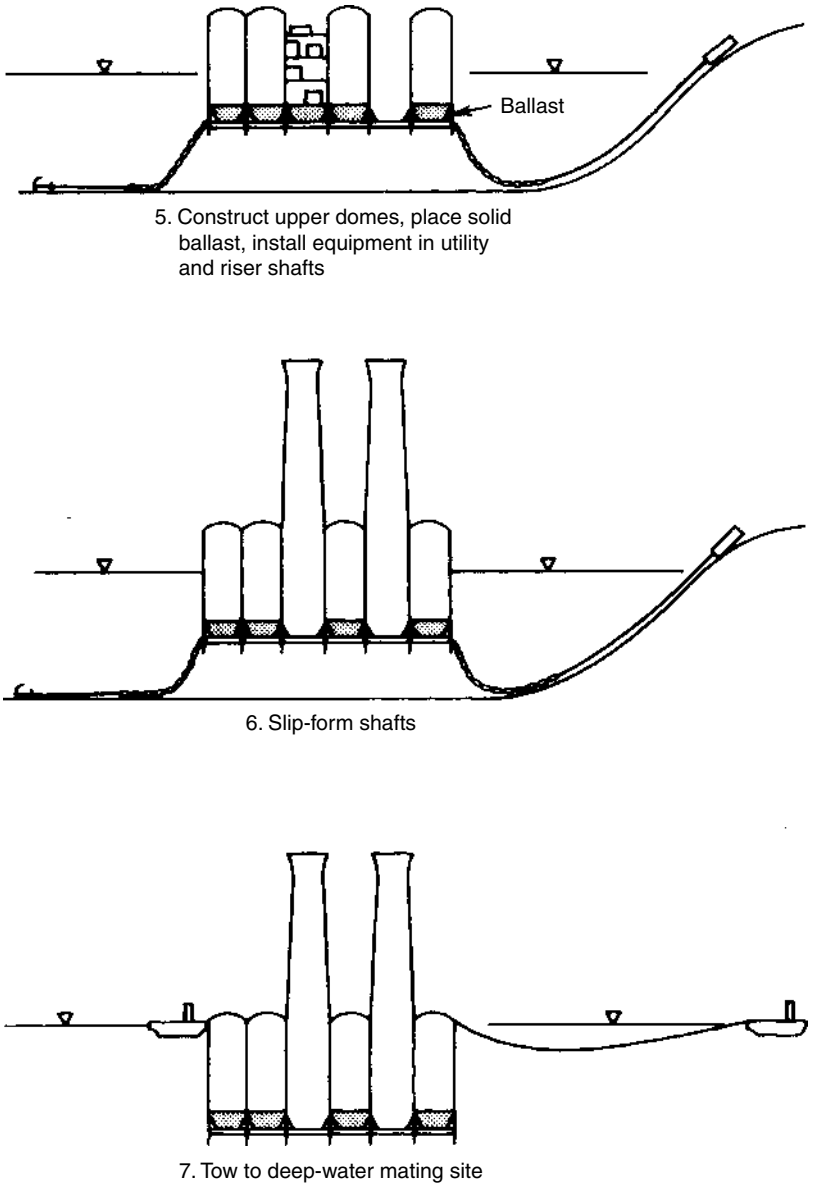

FIGURE 12.4
Construction stages of GBS (stages 5–7).

A more-detailed description of the special requirements and considerations at each step and stage will now be presented.

12.2 Stages of Construction

12.2.1 Stage 1—Construction Basin

In stage 1, the basin must be capable of being safely dewatered against the maximum high tide plus storm surge and under the maximum rise in water table and rain runoff. In a basin

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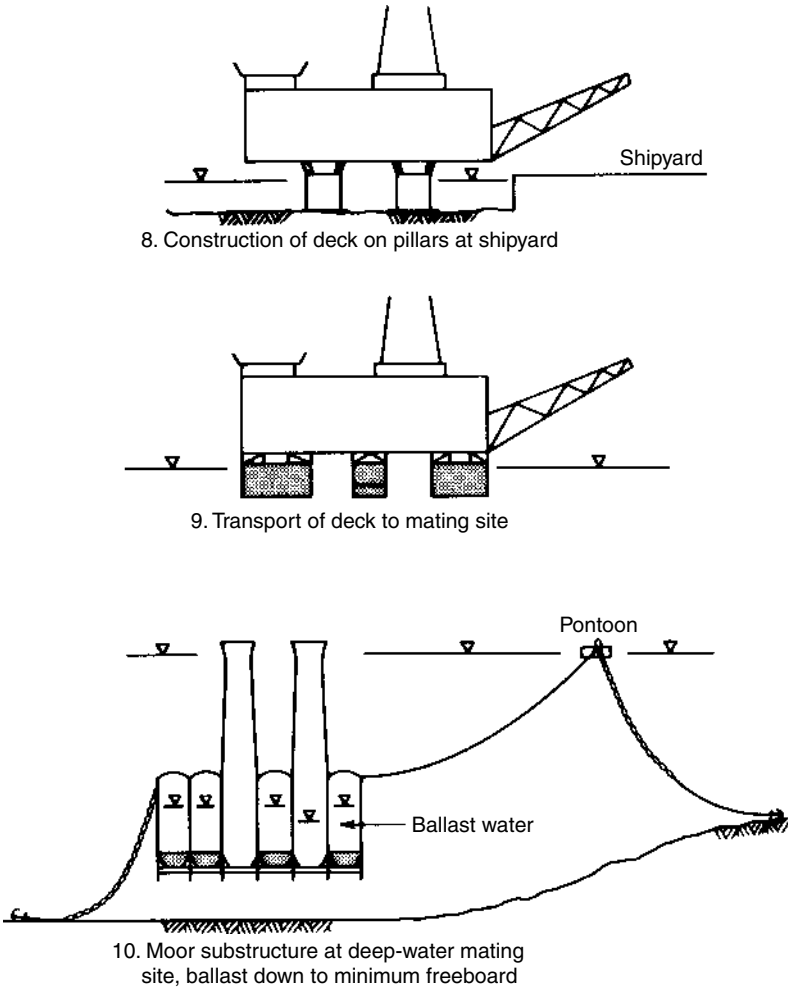

FIGURE 12.5
Construction stages of GBS (stages 8–10).

constructed in Queensland, Australia, the initial gate was made of steel sheet piles, supported by a sand berm. Some of the sheet piles had been driven out of interlock. Piping through these gaps eroded the sand berm and led to failure of the gate wall under extreme high tide, which in this case was 7 m above MLLW. The flooding and subsequent rebuilding of the dike, along with cleanup, delayed the project 4 months.

Similar dikes of sand have been protected in The Netherlands by well points and rock slope protection or steel sheet piles. Dikes used in the basin for the construction of the Condeeps in Stavanger, Norway, have been of steel sheet piles backed by a substantial berm of rock.

The base course under the new gravity-base structure's base slab must be free-draining; perforated pipe or vitrified clay pipe drains are usually installed under a crushed rock base. The surface itself may be a concrete slab, provided with adequate bleed holes in case of a rise in the water table. This same provision of free water movement will later be important when the basin is flooded, to assure equal water pressure acting upward on the base of the GBS.

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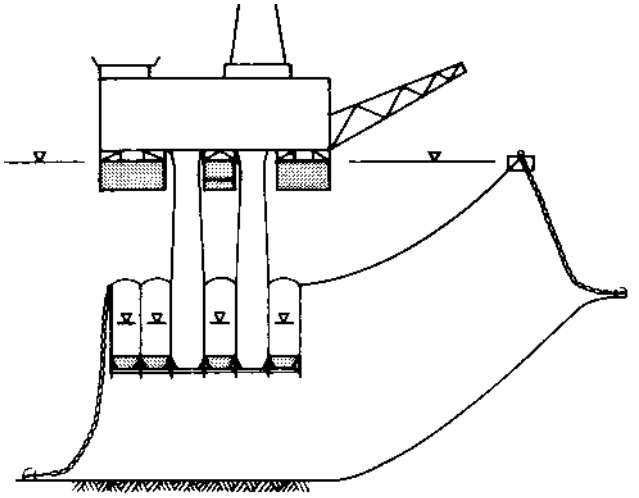
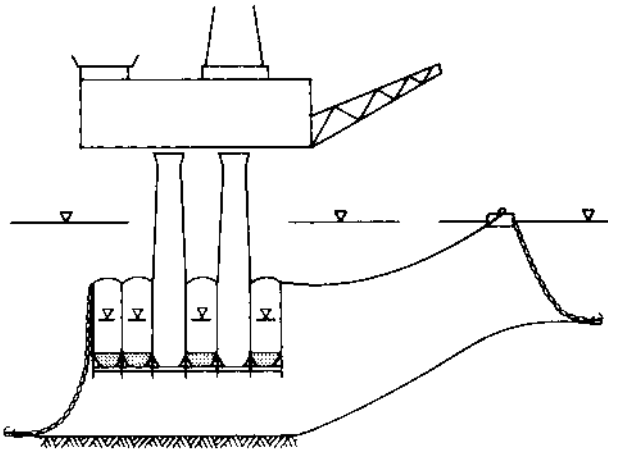


11. Maneuver deck over substructure and transfer deck to substructure



12. Deballast and lift deck off barges complete outfitting and hookup

FIGURE 12.6
Construction stages of GBS (stages 11–12).

Where the basin has been constructed in silty clay soils, as in the Concrete Technology Corporation basin in Tacoma, Washington, a pile-supported concrete slab, with a full underdrainage system, was constructed.

Side slopes must be protected against excessive erosion under heavy rain and, more importantly, against slope failure. Horizontal drains, rock berms at the toe, or well points can be used to prevent slides; gutters at the top and plastic, bitumastic or shotcrete coverings applied to the slope may serve to prevent erosion.

Access is frequently underestimated. At least two well-surfaced roads should lead into the basin or else trestle access should be provided at the top. Similarly, the roads around the base raft on the floor of the construction basin should be well-drained and properly surfaced—e.g., gravel—since a basin tends to collect large quantities of water and mud.

While a dike may be used to close the basin during the caisson's fabrication, gates are constructed where several uses of the basin are planned. They can be removed and replaced

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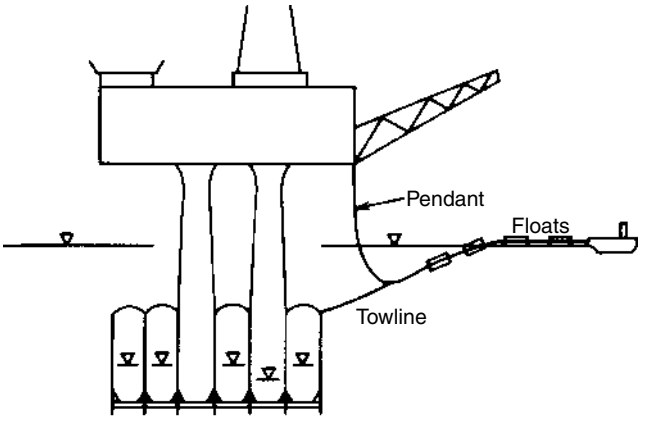
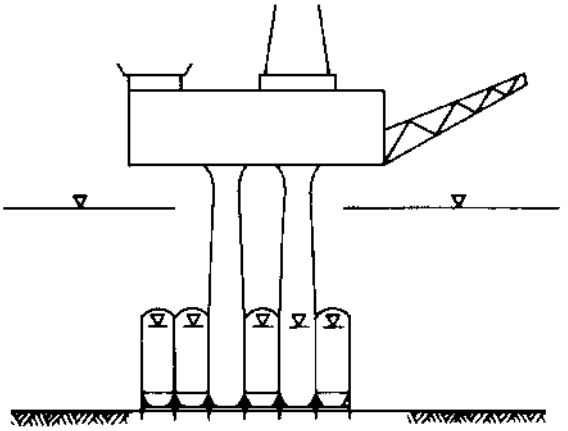
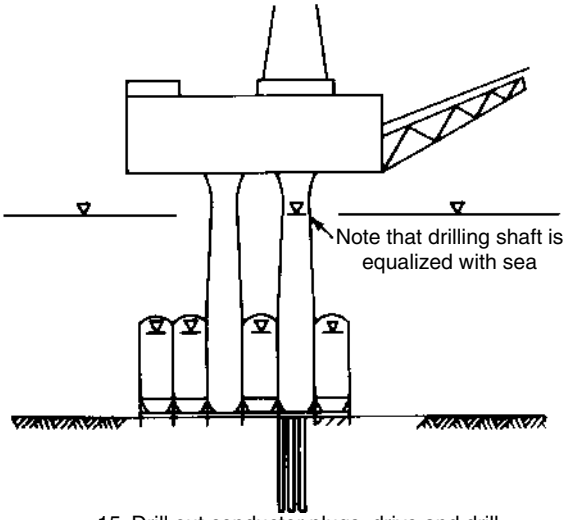


13. Tow to field



14. Ballast down to seafloor, add ballast so as to penetrate skirts, grout under base within skirt compartments



15. Drill out conductor plugs, drive and drill conductors

FIGURE 12.7
Construction stages of GBS (stages 13–15).

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in one to two days. Prestressed concrete gates were installed after the piping incident in Queensland. Gates were installed at the Highland Fabricators basin in Nigg Bay and Howard-Doris's basin at Loch Kishorn, both in Scotland. Two details need to be verified: first, the provision of drainage from under the gate caisson so that no uplift can occur; second, the connections at each side, where a sheet pile wall is usually extended well back into the bank.

12.2.2 Stage 2—Construction of Base Raft

The base slab or raft of the GBS caisson is now constructed. The first items installed are the skirts, which are constructed of either steel or concrete depending on the foundation soils at the installation site. In some cases, such as at McAlpine's yard at Ardyne Point, Scotland, the precast concrete skirts were set down into slots. Then the base raft was directly supported on the basin's foundation slab. The skirts in this case must be free to pull clear when the base raft is floated.

Conversely, in Norwegian Contractor's basin at Stavanger (see Figure 12.8 and Figure 12.9), where longer steel skirts are employed, the skirts are supported on the basin's foundation slab and the structure's base slab is constructed on falsework scaffolding. In the latter case, anodes for cathodic protection, filtered drains to relieve the pore pressures in the surficial sands, and grouting nozzles all have to be installed under the slab. Access by personnel must be considered. This is usually provided by temporary "doors" through the skirts. Later, the "door plates" can be replaced and welded. Some of the larger GBS base rafts have had skirt compartments, all more or less identical, covering a total of perhaps 13,000 m of area and 12,000 m^2 over 500 m of perimeter (see Figure 12.9). In order to ensure an orderly flow of materials and efficient access of personnel and equipment, these skirt compartments must be clearly marked; otherwise a significant portion of any worker's time may be spent hunting for the right location.

Concrete base slabs are usually 1–2 m thick, and they typically generate high heat of hydration, leading to thermal expansion. If the subsequent cooling is restrained, as by the skirts, scaffolding, or adjacent slab pours, substantial cracking may occur. To minimize this

FIGURE 12.8
Fabrication facility of Norwegian contractors, Stavanger, Norway.

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FIGURE 12.9
Base raft under construction.

problem, the sequence of pouring and the time between pours needs to be carefully determined, based on thermal analyses. The concrete slab may be placed in a checkerboard pattern. Thermal probes may be used to monitor the actual temperature in the interior of a slab. The construction of the base on scaffolding, raised above the floor of the dock, minimizes the problems of constraint and cracking, but the scaffolding must not collapse when laterally displaced.

Chapter 4 discusses many of the practical problems of congested reinforcement, prestressing ducts, and concreting.

Base slabs are usually post-tensioned with long tendons (see Figure 12.10). Once again, the concrete slab must be free to shorten as it is not prestressed until the shortening actually

FIGURE 12.10
Ducts for post-tensioning tendons in base raft of Statfjord C.

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takes place. If it is restrained by friction, this will not occur until the base raft is floated. For this reason, when there are no long skirts to deflect, special means are provided to reduce friction—for example, a sand layer between polyethylene sheets, on which a plywood soffit is laid. This was the method used during the construction of the CIDS concrete structure at NKK's shipyard dock in Tsu, Japan.

The top sheets of polyethylene may adhere to the concrete bases underside. If plywood sheets are used, some of them may stick. Where corrugations are required on the underside of the base slab, galvanized corrugated sheets may be used. Time is required for water to penetrate the sand or under the flexible sheets: allowing several hours or even one day will facilitate lift-off. Where there are skirts, these may be supported on thick neoprene pads to permit outward and inward shear deformation. However, long steel skirts usually have adequate flexibility in themselves to accommodate the expansion and contraction, rendering the special supports unnecessary. For platforms destined for sites of very soft soils, long concrete skirts are constructed (see Figure 12.11).

The lower raft walls are now constructed, forming cellular partitions, capable of developing the necessary shear when the base raft is floated. If there is an upper slab on the base raft as an integral part of the design configuration, it will provide the deck of a barge-like structure. Then any hog-sag moments during floating will be resisted by both slabs. Unfortunately, having such a top slab on the base raft is the exception rather than the rule. Usually, there is only the multitude of intersecting cell walls. There is no upper flange, no "deck," for moment resistance during floatout. The base raft is then a large "barge" with minimal depth and strength.

Unequal moments arise, for example, because of a thickened outer wall or because of cantilevered extensions to the base slab which will be submerged during floatout and/or mooring. Because the center of gravity of a semicircular annular ring lies outside that of a complete semicircle, a significant hogging moment may be induced. This may be aggravated by the weight of the skirts, since these are usually concentrated around the perimeter, and by cranes, mooring chains, and the vertical component of mooring forces.

Reduction of the hogging moment may be attained by increasing the air cushion under the outer skirts and by adding ballast to the center cells. However, this latter usually

FIGURE 12.11
Draugen platform has 25 m long concrete skirts to penetrate soft clays.

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conflicts with the requirements to take all possible steps to reduce draft during floatout. A more positive solution, often adopted, is to prestress the top of the cell walls and/or add reinforcing steel to offset or resist the hogging moment and reduce residual stresses. Note that since many of the larger GBS base rafts are not symmetrical, it is usually necessary to check bending around a number of possible axes, both orthogonal and oblique.

With the base raft structure nearing completion, the mechanical systems are installed, principally consisting of the saltwater ballast piping, underbase grouting system, skirt drainage and skirt venting, instrumentation such as strain gauges on skirts, bottom clearance acoustic sensors, and base mat pressure cells.

A careful inspection is now made of all skirt compartments to remove the accumulated debris, forms, and scaffolding that inevitably are left. The doors in the skirts are now welded closed. Ballast water is pumped into the cells to hold the raft on its foundation slab.

12.2.3 Stage 3—Float-Out

With the structure ready for floatout, final checks are made of weight and displacement to ensure that flotation can be accomplished within the design tolerances of draft and heel. Note that for a 120-m-diameter raft, it takes only 1° heel to increase draft by 1 m. A minimum underkeel clearance of 0.5 m is usually required. If a tidal cycle is involved, the clearance must extend over a sufficient period to accommodate potential delays in the floatout operation. The basin is now flooded.

To reduce draft, air cushions are sometimes pressurized under the skirts. Compressed air may be introduced through the underbase grouting system, for example. To prevent the air from escaping under the tip of the skirts, a "water plug" about 0.75 m high is usually left. If the skirts, for example, are 4 m long, then the use of the air cushion can reduce the draft by 3 m or so, depending on the area covered by skirts which are capable of holding an air cushion. The air cushion creates a free surface in each compartment, thus reducing the stability.

In the case of the Andoc Dunlin base raft, draft limitations were so critical that it was desirable to eliminate the water plug completely to gain another 0.75 m of bottom clearance. Therefore, large rubber inflatable "rafts" were placed in the skirt compartments, allowing complete filling by compressed air and preventing any unwanted escape of the air caused by the drag effects from the water.

When the flooding of the basin is complete, the air cushions are tested for each set of skirt cells to be sure there are no leaks. The ballast water is removed, and the base raft floats up to its floatout draft. All systems are checked.

Meanwhile the exit channel has been re-sounded to verify adequate draft, and dragged or profiled (with an electronic-acoustic profiler) to ensure against any rocks, debris, piles, or other obstructions above grade. If all is in order, the gates are now removed. If the "gates" consist of sheet piles and a rock dike, this area must also be carefully swept to ensure against obstructions.

Navigational aids will have been established to guide the exit and to mark the route to the deep-water construction site. Laser or light ranges are useful in monitoring the effect of side currents. Electronic distance-measuring and position-plotting systems are usually installed on a temporary control deck on the base raft, supplemented by a sextant or theodolite.

Moorings will have been set at the deep-water site, secured to mooring buoys. All boat traffic in the area will have been stopped. Weather reports will have been double-checked, especially with regard to wind. Tidal predictions will have been verified for the specific exit site.

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Floatout from the basin has to be very carefully controlled because, as noted earlier, the typical raft is very unstable and weak in bending and hence sensitive to accidental loads and events. Winches are mounted on the walls or sides of the basin control lateral movement, while highly maneuverable tugs pull the base raft out into the channel. The intact freeboard should be a minimum of 1 m above the local wave crest height with allowance for run-up. Because mooring lines can jam in a sheave or otherwise be fouled, contingent means are available to cut the lines if necessary. Good practice is to use fiber lines for the land lines, so that they can be cut by an axe.

12.2.4 Stage 4—Mooring at Deep-Water Construction Site

The base raft is now towed to the deep-water site. A harbor crane barge will aid in securing the mooring chains from the mooring buoys to the base raft.

The air cushion will still be contained within the skirts. The release of this air must follow a carefully calculated procedure to ensure against overstressing the base raft in bending. In general, the air cushion under the central portion can be released, initially, while that under the periphery cells is kept until additional freeboard is attained by the extension of the peripheral walls. Alternatively, a short cofferdam can be built on the outer walls (see Figure 12.12).

Control of the air pressure under skirt cells is very tricky. If pressure gauges are used in different segments of the base, the readings may lead to the wrong action. When the Statfjord A base was floated out, pressure gauges were the only instrumentation used (see Figure 12.13). As the base raft tilted slightly, the pressure on the downside (B) rose, and that on the upside (A) fell. So the natural reaction was to bleed air from the downside (B) and add air to the upside (A). This caused increased heel, generating even higher pressures on the downside and lower pressures on the upside. So, in effect, the intuitive "corrective" action was slowly jacking the platform to ever greater heel.

Earlier reference was made to the low stability of the base raft. This is largely due to the air cushion during floatout, which creates a free surface in each skirt compartment, reducing the gross water plane moment of inertia by the sum of the individual water planes. Any ballast water in a partially filled internal compartment will also shift to the downside, adding to the overturning moment.

The proper means of control, therefore, is not pressure but volume. Water level indicator gauges—for example, based on electrical resistivity—or hinged floats should be installed in the skirt compartments to give a measurement of the height of the water plug relative to the base slab and hence a measure of the air volume. Base raft level indicators of suitable sensitivity should also be installed at the control station.

During floatout, mooring, and initial construction operations at the deep-water site, there is always the possibility for loss of air pressure under one set of skirt compartments. The subdivisions must therefore be carefully selected to minimize the adverse effects of loss of air pressure or of local flooding.

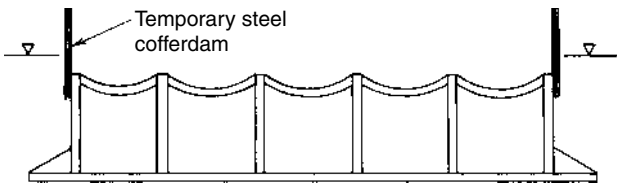

FIGURE 12.12
Temporary cofferdam walls to permit increased draft during float out and early construction stages.

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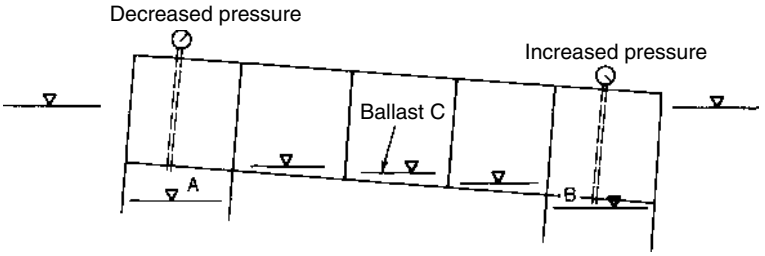

FIGURE 12.13
Effect of platform heel on air pressure in skirt compartments. Intuitive but erroneous response to lower air pressure in B will further increase list of platform.

With deep concrete skirts and a deep air cushion, such as those for the Draugen platform, the air pressure at the top of a compartment will be substantially higher than the outside water pressure at that level. The structure must be checked for this resultant internal overpressure, which creates uplift on the roof and tension in the cell walls.

Either three or four mooring legs will have been pre-installed, terminating in mooring buoys arranged around the GBS deep-water construction site. These will typically consist of a heavy clump anchor on the seafloor or drilled-in anchor on the beach, several shots of chain, and then a wire rope mooring line or chain to the buoy. The buoy is held in approximate location by a separate small clump anchor and pendant.

From the buoy, a chain is run to the base raft and connected by shackles. Now there is a continuous mooring from base raft to anchor, with a float supporting it in the center, giving it some spring under load and reducing the tilting of the base raft. At least one line will have provision for progressive take-up as the construction proceeds and the mooring sinks lower in the water (see Figure 12.14). In addition to moorings, a water line and a power line are usually run out from shore. By mating these lines to a wire rope, adequate strength can be provided.

12.2.5 Stage 5—Construction at Deep-Water Site

The remaining construction activities for the platform now take place while the structure is moored in adequately deep but protected water. During this period, the structure must be safely protected against accidental flooding. Detailed consideration must be given to the sequences of construction, since temporarily unbalanced loads will result in structural stresses, which may be locked in as residual stresses.

Shortly after the base raft of the Ninian central platform was moored, a storm arose. Continuous wave action beating against temporary bulkhead closures near the waterline caused working and then fatigue of the bolts holding the closures on. The external compartments on that side flooded. The water pressure inside one compartment ruptured the temporary internal closures to two adjoining compartments. Fortunately, all other closures held and the raft did not heel further.

However, the concrete batching and mixing barge had been moored tightly to the far side. Its manholes on deck were not secured. When the base raft listed into the waves, the near side of the base raft rose, lifting the adjacent side of the concrete barge, consequently submerging its deck on its far side. The water poured into the manholes, and the concrete barge broke loose, turned over, and sank. All temporary closures, therefore, must be designed for both compression under wave crests and tension during wave troughs. The bolts should be designed to resist fatigue. Double nuts with lock (spring) washers can be used. Large bearing plates can be installed to prevent local crushing of wood.

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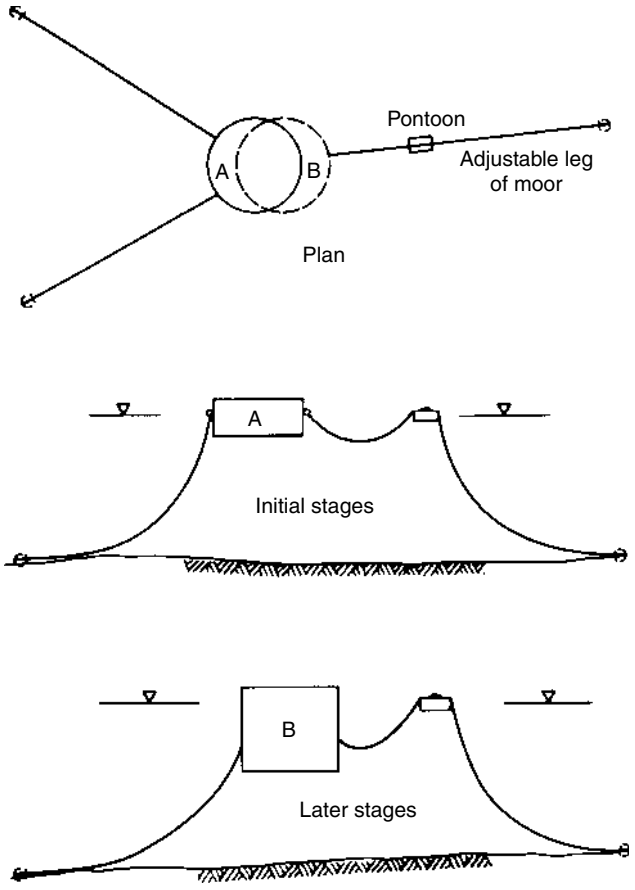

FIGURE 12.14
Mooring of substructure at deep water site.

A procedure that can be usefully employed to increase the freeboard during the critical stage of floatout and initial caisson construction is to extend the outside walls higher than those on the inside. On the Ekofisk and Hibernia caissons, for example, the outside walls were kept built up above the inner concrete. This can also be accomplished by extending a steel cofferdam up from the base raft, cantilevered from the base raft to give adequate freeboard.

During construction afloat, the platform is required to maintain its freeboard under the hundred-year storm wave plus run-up for that location and season of the year, as well as one-compartment stability, that is, to be safe with any one compartment flooded. If this cannot be maintained, then the intact freeboard must be increased as necessary.

Construction can now take place at the deep-water site (see Figure 12.15). If the GBS is being constructed of reinforced concrete, as has been most common to date, then transport and installation of reinforcing steel, prestressing ducts, and concrete must take place in substantial volume and on a more or less continuous basis.

Typically, slip forming is used to raise the walls, using retarding admixtures in the mix because the large volumes involved necessitate a very slow rise, perhaps only 1 m per day (see Figure 12.16 and Figure 12.17).

A typical base raft will have 1200 linear meters of 80-cm-thick walls, so that a continuous production of perhaps 1000 m^3/day is required to raise them at the rate of 1 m/day.

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FIGURE 12.15
Statfjord B base raft moored at deep water site. (Courtesy of Aker Maritime.)

Meanwhile, perhaps 300,000 kg (300 tn.) or more of reinforcing steel must be placed, along with prestressing ducts and embedded plates. As many as six hundred workers per shift are required, around the clock, for up to 60 days of continuous operation (Figure 12.18). There are many variants to this construction scheme, each of which has applicability under special circumstances.

Where labor supply is limited, where there is an unusually large concentration of reinforcement, or where embedments must be set with great accuracy, then slip forms may not be applicable. Panel forms, cantilevered up, or flying forms are then employed, with each pour being defined as to extent and duration (Figure 12.19). This often necessitates the use of window boxes and tremies ("elephant trunks") to place the concrete. It also means that there will be a large number of horizontal construction joints. Both these requirements

FIGURE 12.16
Slip-form construction of cell walls of base raft of Statfjord B.

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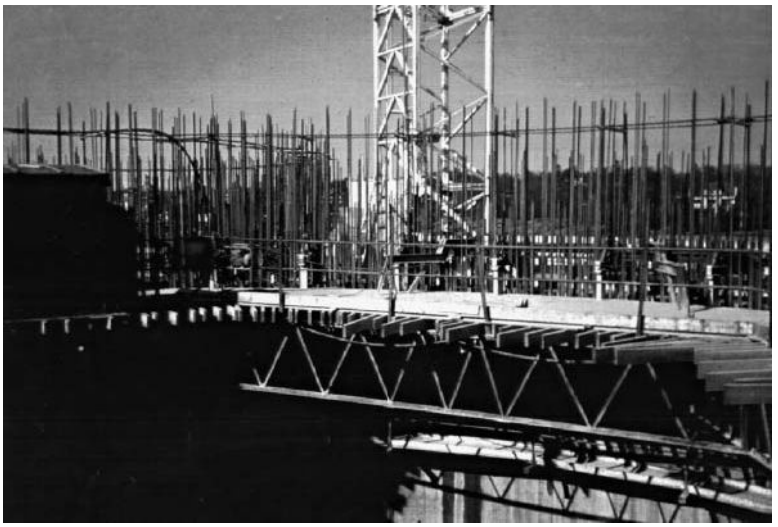

FIGURE 12.17
Slip-form construction of cell walls.

can and have been successfully met, not only in base raft construction for offshore platforms, but also in the rather similar walls of nuclear reactor containment structures.

Slip forms have also been employed in such cases, subdividing the structure into segments, with vertical construction joints. The Ninian central platform, with its seven concentric walls, was so subdivided. Precast concrete elements have been extensively incorporated into base raft construction, the Ninian central platform, and Global Marine's Super CIDS being prime examples. These should be cast in as large segments as practicable; 150-tn. segments were used on the Ninian central platform. They can be incorporated into the

FIGURE 12.18
Extending reinforcing steel during slip forming.

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FIGURE 12.19
Placing reinforcing steel and prestressing ducts for domes (caps) over base cells.

remainder of the structure to act in monolithic fashion by means of dowel extensions and cast-in-place concrete joints or by post-tensioning.

Tolerances in the cell walls are very critical, especially where the cell walls are designed to carry hydrostatic pressures in ring compression. Tolerances must be constantly monitored and controlled. These include local relative deviations, deviations from a true circle, and deviations from the design radius and deviations from the vertical axis.

Match-cast jointing techniques appear applicable in the case of many precast assemblies; these can follow the technology now perfected for long-span bridges, where units up to 30 m wide have been successfully joined by this method. Match-casting consists of casting a second segment against the first, thus insuring a perfect fit when later the two segments are joined. For this joint, each face is coated with epoxy glue, and then the two segments are pressed together by prestressing. The combined segments behave monolithically, as proven by tests and experiments. Match-casting was employed with great success for the breakwater wall of the Ninian Central Platform, where horizontal joints were employed. Match-casting not only led to rapid construction but also assured perfect alignment of the complex post-tensioning ducts.

Other construction expedients include the use of precast concrete soffit forms to support horizontal slabs and domes during construction of the base caisson. The Seatank platforms used precast concrete pyramidal forms which were domed shells constructed of shotcrete. A number of the Condeep platforms have used precast shells to form the lower dome. The Hibernia platform used precast beams and slabs to form the roof.

Steel soffits with internal support beams built in can be similarly used, the whole designed for composite action with the cast-in-place concrete. These steel soffits can have superimposed trusses, which support them and which are later embedded to serve as reinforcement.

When extremely thick walls are required, as in some designs for ice-resistant structures, precast concrete hollow box units may be employed, to be filled later with cast-in-place concrete. Examples of the effective use of combined precast concrete and cast-in-place concrete are the outer walls of the Ekofisk caisson and the Ninian central

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platform. The combination of slip forming for the walls and precast segments for shells and slabs has been used successfully on many platforms.

With steel GBS structures, large prefabricated segments are being lifted, guided into place, aligned, and joined by high-strength bolts or welded. They must be given temporary support against wind. Welding locations must be protected from rain and spray. Columns and shafts must be adequately stiffened to prevent out-of-roundness distortions. Erection pins and bolts are needed to assure proper fit-up.

For all structures being constructed afloat, access must be provided for the large number of construction personnel. This matter has often been underestimated in the past. Personnel must move from shore to a barge or floating dock alongside, up and over the outside walls, and across open cells and the internal walls to their site of work. These walls are constantly being extended up, with closely spaced reinforcing bars, ducts, and climbing rods (for the slip-form yokes) all projecting above the walls. The freeboard of the structure is also changing. To add to the difficulties, distances between cells and walls are large, extending the reach of cranes to their limits and sometimes necessitating re-handling of reinforcing steel and other materials.

Clear identification of cells and walls is essential so that there will be no confusion of workers about where they are to go. Access routes must be planned and safe walkways provided. Reinforcing steel bars may purposely be spread at certain specific locations; this should be incorporated in the working drawings and checked by the designer rather than being improvised in the field.

The most recent gravity-base platforms have required very dense concentrations of reinforcing steel. Bars have been bundled and spacing between bars reduced. Mechanical connectors have been used to splice bars, rather than laps. With such concentrations of steel, up to 500 kg/m^3, the placement has controlled the rate of progress. Shear resistance through the walls is best attained by the use of T-headed bars rather than the closely spaced hoops originally employed. Concrete mixes must be designed that are sufficiently workable to flow around and through these tightly placed bars. Coarse aggregate sizes have been reduced to 8–10 mm and the sand content increased to 50%. "Flowing concrete" is well-suited, provided it is compatible with the forming method adopted.

Placement of concrete has generally been accomplished by a combination of pumping and bucket or even wheelbarrow delivery, the latter for distribution into the slip forms. Retarding admixtures are added as necessary and adjusted for temperature, to match the rate of rise of the slip forms.

Freeboard can be kept constant by selective ballasting of the cells. However, these operations must be carefully planned and checked at every stage and substage. On the Beryl A platform, main cells were ballasted with water up to 40 m deep. No ballast water was placed in the small star cell interstices between the primary cells. No notice was taken of this because these interstices were covered over by temporary timberwork platforms that rose with the slip forms. Small in plan as they were, they had never been designed or checked for a hydrostatic head of the water ballast inside the main cells, which placed tension and shear on the connecting walls. Only after conclusion of concreting of the base caisson to a height of about 52 m, when the work platforms were removed, was the large and extensive cracking in the walls made apparent. The integrity of the structure was finally restored, but only through a mammoth effort of concrete repair. Similarly, the Sleipner failure was due to hydrostatic over-pressure in the interstices.

This illustrates the need in planning to portray each of the many incremental steps of construction, stage by stage, to reflect the step-by-step addition of concrete, which changes weight, trim, and draft, the progressive changes in ballast, change in stability (usually not critical), the changes in hydrostatic loading, both external and internal, and the structural response to the different load combinations.

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In particular, the matter of differential hydrostatic head acting on the internal and external walls must be addressed. In the usual case, the net load is the difference between two large numbers, each the square of the water head. Mere application of a load factor to the differential may be totally inadequate to cover tolerances in these heads. Conversely, application of a load factor to the larger of the two heads of, say, 1.3, along with 0.9 to the lesser head, can result in impracticably, almost ridiculously, high design loads, especially as the total head (depth) increases. Therefore, it is necessary to establish realistic tolerance ranges for the water ballast in the various compartments and to ensure that the ballast differential be physically limited by provision for overflow. All ducts and open pipes must be grouted; since these can transmit hydrostatic head between compartments.

During construction afloat, the draft of the base structure gradually increases. This changes the scope of the mooring lines. Usually, the necessary tension can be maintained by taking in on one, or at the most two, of the mooring lines. This is facilitated if one of the lines can be run to shore, assuming it is nearby. If not, adjustment means can be provided on the caisson itself or by use of an intermediate pontoon or weight.

Taking up tension on a leg of a mooring line presents practicable difficulties. The lines should pull at the center of rotation of the caisson, but that is underwater and changing. The line can be best adjusted by installing a multi-part block and tackle arrangement to a sheave and hence up to the deck. The sheave has to be progressively raised (see Section 9.4.2).

The moorings must be designed to hold the substructure during any storm wind plus waves and currents having a significant probability of occurrence during the construction period. Usually this is selected as the ten-year return storm, but considering the seriousness of the consequence of a rupture of the moorings and the delay and loss to the total project, a longer-return-period storm should perhaps be adopted. Fatigue must be considered since the wave-generated currents generate vortices, which lead to vibrations, small in magnitude, but large in numbers.

The substructure almost always has several large barges moored to it, and it may be impracticable to move these away prior to a storm that arises suddenly, as was the case at Ninian. Therefore, these should be included in the calculations.

Crew changes are another practical problem due to the large number of workers per shift and the need to carry on work continuously. During rough weather and high winds, there arise the problems of getting personnel on and off the boats, seasickness, and finding position and course in fog, rain, or snowstorm. These must all be addressed in the planning stage.

Provision must also be made for emergencies. What is to be done if a worker is injured? Are there pallets or basket stretchers available that can be handled by a tower crane to hoist the injured person to a boat? What about man overboard? A continuous safety boat has been provided twenty-four hours a day around some of the larger platforms. Life preservers and trailing floating lines should be provided. Life jackets should be worn by all personnel working over the side and during transport. It is unnecessary and perhaps counter-productive to try to have personnel wear them while working in the cells; they may catch on the reinforcing steel. However, if there is ballast water in the cells, life jackets and safety nets will be appropriate.

Cells eventually are 60 m or so deep. Safety nets may be needed where personnel are working across their top and to catch falling debris when personnel are working underneath. Usually these nets are provided at least for the principal shafts. Projecting reinforcing bars are a puncture hazard and an eye hazard. They should all be fitted with a red plastic cap. Prestressing ducts, especially vertical ducts, become convenient receptacles for tools, bottles, aggregate, and the remains of lunches. They should all be covered with red plastic caps, which will also serve to keep out the rain. On one project, a heavy rain was followed by

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freezing weather; the rainwater ran into the ducts and then froze and split the concrete walls in multiple fractures.

Grouting of these long vertical ducts presents the problem of sedimentation and formation of a void at the top, due to the bleed water driven by gravity and using the prestressing strands as wicks. Voids can be 2 m or so deep and the potential source of long-term corrosion beneath the upper anchorage. At early ages, the water in these voids may be subject to freezing, although later it is re-absorbed by the grout. Various steps can be taken to minimize or prevent such voids. One is to design a grout mix that minimizes bleed, e.g., low W/CM ratio and contains an anti-bleed admixture such as silica fume. The second is to drill through the upper anchorage plate to allow the bleed water to be expelled. A third is to use a thixotropic gel admixture that gels as soon as pumping stops. Vacuum grouting can be used: it sucks the bleed water out as the grout is placed. In any event, any remaining void should be topped off and filled with grout.

Fresh water and electric power must be supplied to the structure during construction afloat. If the deep-water site is relatively near to shore, a hose and power cable may be run, attached to a wire line. Otherwise a water barge and generator barge must be moored alongside.

During the final installation process, many stages later than those being discussed at the present, the platform will be ballasted down onto the seafloor. As it nears touchdown, the water trapped under the caisson tends to cause the platform to skid laterally, more or less uncontrollably, due to the thrust generated. While venting of the skirt compartments and reducing the rate of descent will minimize this problem, a more effective method has been developed to prevent dislocation which uses three or four dowels that engage the seafloor while there is still 2–3 m of bottom clearance.

These dowels cannot be installed in the basin because of draft limitations, so they are lowered and fixed in position at the deep-water construction site. A typical dowel is a steel tubular 2 m in diameter, with 75- to 100-mm-thick walls, extending 4–5 m below the skirts. It is designed to provide adequate lateral resistance to hydrodynamic skidding by developing passive resistance in the soils, which, of course, produces shear and moment in the dowels. The dowel is designed so that failure, if it occurs, will first be by plowing through the soil and then by bending/buckling of the dowel, so that the structure itself will remain undamaged. So as soon as the structure is moored at the deep-water site, each dowel is lowered and secured in place by grouting within a sleeve.

During this deep-water mooring phase, major mechanical system installations take place, primarily the crude oil piping and oil level indicators. Several complex pumping and ventilating modules may be placed in the utility shaft. In addition, risers for flow lines from satellite wells and for crude oil transfer lines are installed in or on the periphery of the base caisson. The risers may include J-tubes and, in some cases, curved conductors. In other cases, entry tunnels may be constructed, through which the pipelines will eventually be pulled. Experience has shown that the mechanical installations can occupy a disproportionately long period in the overall construction schedule. To minimize this time requirement, as well as to permit more efficient assembly, modularization should be performed to as high a degree as practicable, with large modules being set into the utility and riser shafts. Similarly, conductor guides and support frames should be preassembled for rapid installation by a large crane barge. Modularization and prefabrication permits coatings to be applied under shop conditions. At the site, conditions of moisture and temperature are usually adverse, and hence, coatings at the site should be limited to touch-ups.

The next substage at the deep-water site is the construction of the domed roof of the base caisson. The construction of this domed roof, perhaps 10,000–20,000 m^2 of area, covering over the 15–30 cells, often requires more time on the construction schedule than the

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construction of the walls of that base caisson, even though the ratio of concrete may be as low as 1:3 (see Figure 12.19). The delay, of course, is due to support of the forms. Prefabricated dome soffits of concrete have been employed as well as the more conventional steel trusses and plywood. A steel soffit, containing all the necessary reinforcing steel and having multiple studs for composite action, could conceivably be placed as a single lift, permitting rapid concreting thereafter (see Figure 12.20).

Once concreted, the domes are often covered with a meter or so thickness of lightweight concrete to absorb the impact of objects dropped from the deck during drilling and production operations. Over-dome pipe supports are also provided for any crude oil transfer piping that may have been installed on the exterior of the base caisson and which now must be led horizontally into the riser shaft.

As noted earlier, the dome is under tension due to the upward pressure from the oil. The oil will tend to find any small crack or permeable zone through which to escape into the sea. To prevent this, a steel membrane is sometimes used as a watertight membrane, attached by studs embedded in the concrete when it is cast. The steel membrane also serves as a form and is in term supported by temporary falsework. After the roof is self-supporting, the temporary supports are removed and then the crevice between the concrete and steel is pumped full of grout at low pressure. However, this last operation requires very careful execution and multiple injection points, because accidental local overpressure may cause a new fissure.

Since the substructure may later be towed to another site for deck mating, towing attachments and notches are fitted around the periphery of the walls where they intersect the domes. Towing attachments are designed to withstand the failure load of the towing lines, but to have a weak link themselves, which fails before pulling out and damaging the structure.

Circumferential prestressing is also usually required at this intersection between domes and exterior walls. Proper staging should be applied to facilitate this operation. Most

FIGURE 12.20
Construction of domes on Statfjord B GBS. (Courtesy of Aker Maritime.)

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offshore platforms require solid ballast in order to lower the center of gravity as much as possible. Solid ballast may consist of concrete placed above the base slab, within the cells.

Mass concrete typically generates high heat of hydration and hence leads to thermal expansion which may adversely load the adjoining walls and crack the slab. Therefore, the mix should be selected for low heat and low modulus of elasticity. High density is desirable; this may be enhanced by selection of aggregate of high specific gravity. Blast furnace slag-cement, low-heat cement, or a Portland cement–pozzolan mix should be employed. Strength is normally not a criterion; hence total cement content can be kept relatively low. To prevent undesirable structural interaction between the ballast concrete and the walls, sheets of crushable material (polyurethane foam) are often placed on the boundary walls. Where the ballast concrete is also designed to act structurally in composite action, it may be necessary to place it in sections to avoid causing distress in the walls due to heat of hydration expansion.

Iron ore has also been used effectively as solid ballast, placed by mixing with fly ash and pumping in as a slurry. Provision must be made to decant the slurry by drainage. Finally, where solid ballast is to be placed on exposed slabs, such as cantilevered base slabs, high-density rock can be placed. There are several methods available for placement of rock on exposed underwater slabs: placement through a large-diameter (e.g., 1 m diameter) tremie tube, placement in a skip or bucket that is lowered to the slab before discharge, and discharge from the surface in relatively small batches. Impact on the slab has to be considered.

12.2.6 Stage 6—Shaft Construction

The next stage is typically the construction of the shafts, which typically may be from one to four in number. These are usually tapered, with varying wall thickness, necessitating the use of sophisticated adjustable slip forms (see Figure 12.21 and Figure 12.22). Shaft slip forms, like chimney (stack) slip forms, tend to rotate. To correct this after it has started is difficult. Chain jacks may be used to react against the climbing rods, or a more rigid steel beam stub may be embedded in the wall and used as a reaction point.

The large single shaft of the Ninian central platform was constructed using match-cast precast concrete segments, later post-tensioned to act monolithically (see Figure 12.23 and Figure 12.24).

FIGURE 12.21
Slip-forming shafts on Condeep platform. Note temporary plastic curtains to protect workers and fresh concrete from dry November winds. (Courtesy of Aker Maritime and Mobile, Norway.)

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FIGURE 12.22
Shafts at Statfjord C platform near completion.

FIGURE 12.23
Shafts for Ninian platform under construction using precast concrete segments for breakwater.

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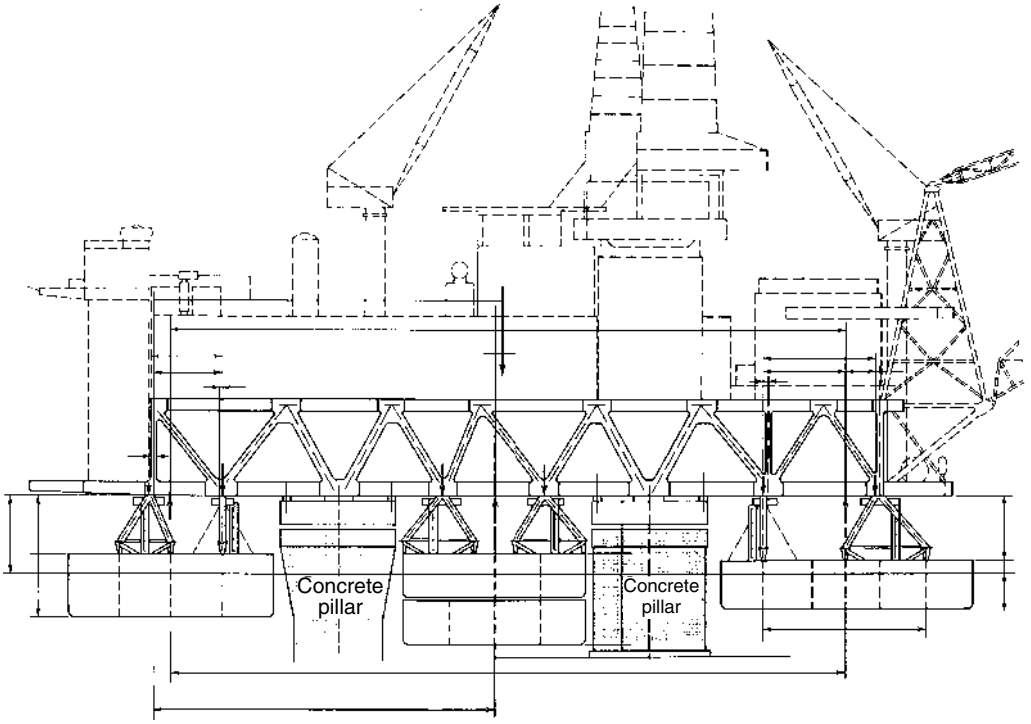

FIGURE 12.24
Arrangement of barges for tow of statfjord B deck to mating site. (Courtesy of Aker Maritime.)

Control of verticality is by lasers, set accurately on the base slab. Unlike chimneys on land, the vertical axis of a floating structure changes with list and trim of the structure, so all points must be relative to the base slab. Control of tolerances is discussed in Section 21.10.

During construction of the shafts, vertical access is provided by a construction elevator, which is laterally supported at intervals. Inserts for support must be located in concrete that will be adequately strong by the time the elevator tower needs to be connected (see Figure 12.25).

Post-tensioning of the completed shaft is carried out, most often from a gallery in the base raft. At this stage, the gallery will be deeply submerged and hence subject to very high hydrostatic pressures and differential pressures from the ballast water. Since this is a temporary facility for the purpose of jacking the tendons and grouting the ducts, additional internal supports of structural steel or even timber may be found more practicable than permanent concrete struts.

The grouting of the ducts must be carried out under sufficient pressure to reach to the top of the shafts. This, and the wicking action of the strands, leads to considerable sedimentation and bleed, carrying water to the top of the ducts. Provision of means for vacuum grouting or for refilling at the top and use of a thixotropic gelling admixture are usually both necessary.

On some gravity-base platforms (e.g., the Andoc Dunlin platform) the upper portion of the shafts was made of structural steel in order to reduce weight and overturning moment. The large "cans" of the shaft were set by use of a large floating derrick after the base structure had been ballasted down to reduce the height above water. They were then post tensioned to the concrete structure.

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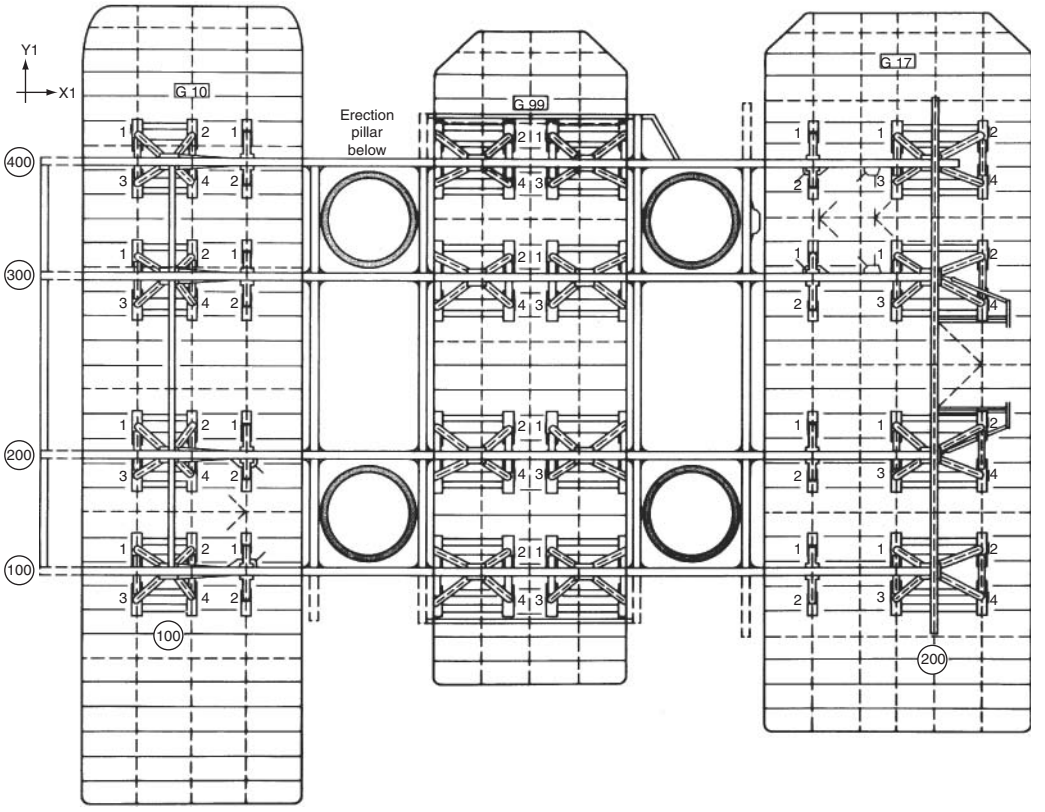

FIGURE 12.25
Plan arrangement of tow to mating site. (Courtesy of Aker Maritime.)

Gravity platforms have also been constructed of steel; examples are the four small Loango platforms off the mouth of the Congo River and the large Maureen platform in the North Sea. Their construction generally follows that of the concrete platforms except that, being light, most of the structure can be constructed in the construction basin. Large "bottles," cylindrical tanks, attached to the edge of the base, provide both buoyancy and stability during tow and installation.

An especially daring hybrid concept has been proposed by Saga Petroleum of Norway. The mating sequence proposed involves joining the top section of steel tubular framing to the lower section of concrete cellular structure while both are inclined at 30° to the horizontal. Obviously, control of attitude, orientation, and rotation are all required, yet this same operation has already been successfully carried out for an articulated loading column of somewhat similar concept, employing selective ballasting.

Mechanical outfitting must continue within the shafts during and after their construction. Temporary doors may be left to facilitate personnel entry and provision of services. The "doors" subsequently have to be concreted closed and must then be able to carry the stresses imposed by hydrostatic and gravity loads and by the global bending of the shaft under wave action. They must, of course, be watertight. Detailing of the closures needs to be carefully worked out by the constructor's engineers in coordination with the design engineer. Particular attention has to be devoted to the upper horizontal joint where bleed water and settlement may allow water to permeate. A secondary injection of grout or

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epoxy may be utilized. Consideration must be given to residual stresses and differential prestress.

There is typically a large, heavy ring beam or girder to be constructed on top of the shafts. This may be of reinforced and prestressed concrete, as with the Condeep platforms, or of fabricated structural steel, as used at the Ninian central platform. In either case, tolerances, level, and distances must be controlled to within a very few millimeters. Since at the time of installation of the deck, when the concrete structure has been submerged to its deepest draft, it will have ballast distributed differently from that at the time of deck mating, causing bending in the structure as a whole, correction factors must be calculated and applied to the different inclinations and hence distance apart of the ring beams.

12.2.7 Stage 7—Towing to Deep-Water Mating Site

At this stage, the structure may be towed to a different site for deck mating, since this requires even deeper water than at the final installation site. The structure will still be floating with waterline near the top of the large base caisson. Since the tow will be partially or wholly within restricted waters, the boats will tow with short scopes of their towlines. Unfortunately, the thrust of the propellers will react against the base structure and the net forward speed will be significantly reduced or even prevented. In the case of the Statfjord B platform, the tow had to be aborted and rearranged. Use of two pusher tugs then proved far more effective.

Control of the structure within narrow channels has been effectively carried out by having one or more lead boats, auxiliary boats on each side, and either pusher tugs at the stern or small tugs headed the opposite direction and being pulled astern. For such a tow, a temporary control platform is mounted. A generator must be provided for power and lights, with fuel for the trip, freshwater supplies, radios, and navigation gear. A life raft is provided, plus fire extinguishers.

The Statfjord platforms were towed about 100 km to their deep-water mating site; the Ninian central platform was towed only 30 km, but the Andoc substructure was towed from Rotterdam to Bergen, Norway.

A principal environmental concern en route is that of currents, which will act on the very large submerged mass. Winds are of concern only because of their effect on the boats and their ability to hold or change heading.

The deck mating site should be selected to provide adequate depth to enable the substructure to be almost fully submerged by ballasting yet provide good protection from winds and good mooring positions. An ideal location would be at the head of a relatively narrow and deep inlet, protected from high winds.

Moorings are installed both in the seabed and onshore, the latter facilitating the needed adjustments in lengths of line as the structure is ballasted down and deballasted up. A second set of moorings may be established in order to permit mooring of the deck-barge complex upon its arrival or it may moored to the legs of the gravity base caisson, using polyester lines. In this latter case, the moorings of the GBS must be designed for the combined loading.

12.2.8 Stage 8—Construction of Deck Structure

While the substructure is being constructed, the integrated deck is fabricated (see Section 17.5). This may be done in a shipyard, on girders, or on trusses spanning a graving dock or overhanging a trestle (jetty). These support conditions will be different from those that will be acting when the deck is finally mated on the substructure, and hence

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FIGURE 12.26
Erecting precast concrete breakwater segments.

deflections and twist deformations must be calculated and accounted for. These can amount to as much as 100–150 mm in extreme cases and affect the equipment and piping as well as the girders themselves. Most recently, therefore, pillars have been constructed in shallow water near the shipyard. The pillars are short segments identical in configuration and relative location to the top of the shafts of the substructure, enabling erection of the deck under exactly the same support conditions as it will have when mounted on the substructure. It is therefore essential that these pillars be correctly located, with a tolerance in plan of only 20 or 30 mm and in level of 10 mm (see Figure 12.26 and Figure 12.27).

The deck structure will typically start off with large, heavy structural steel ring girders, perhaps designed to be filled with concrete later, after the deck has been finally mounted on the substructure. Then, very heavy steel trussing or plate girders are built up with prefabricated sections. Because of the spans and loads involved, plate thicknesses often run up to 100 mm. Welding procedures will probably require both preheat and postweld heat treatment. Since these joints are subjected to cyclic loading, fatigue considerations will probably require grinding of the weld profiles and checking not only of the welds by NDT but also of the adjacent heat-affected zones for hardness.

As the support structure is built up, various modules of equipment are installed. Scheduling is obviously a very complex matter, since equipment modules take time to

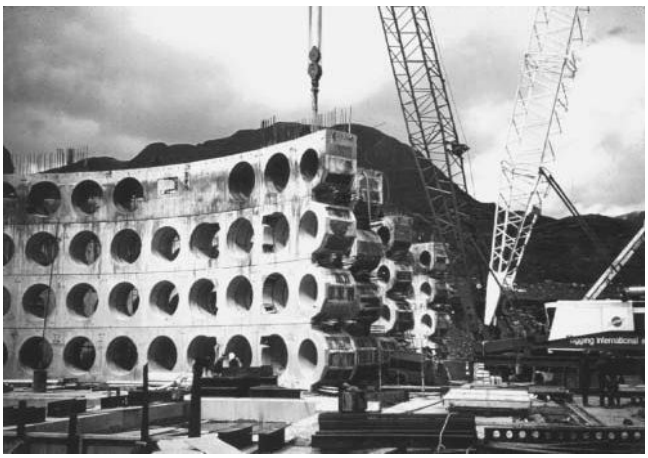

FIGURE 12.27
Fabricating precast concrete segments.

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assemble, yet the support structure must also be completed. Careful planning will enable equipment modules to be lifted over the support structures, lowered, and in some cases skidded to final location.

The deck, now supported on several pillars or other temporary supports, is a very large structure that will undergo expansion and contraction with the temperature and will experience rotations as weights are installed. Therefore, it is usually supported on heavy laminated neoprene and steel pads, allowing shear deformation and rotation. At the same time, the deck must be adequately secured against a windstorm. Stops must be provided so that the structure cannot possibly slide off. Wind forces can reach 500–1000 tn. or more under a severe storm. Provision must also be made for jacking if the combination of temperature cycles and wind cause the structure to "crawl" sideways. Bearing pads should be regularly inspected visually.

During this period, a great many people will be working in confined spaces and with possibly conflicting demands for scaffolding, craneage, ventilation, lighting, and access. Since the deck is almost always on the critical path and the labor costs are high, careful planning and scheduling can produce significant savings.

Appropriate provision must be made for fire protection. Also, since the work is over water, either floats or nets should be installed to protect anyone who falls. Proper provision must be made for grounding of electric welding equipment. Welding cable insulation should be maintained in good condition and repaired or replaced when damaged by abrasion.

Housekeeping and cleanliness, along with proper lighting, will have positive benefits on safety and efficiency. Cleaning of insulation, metal particles, and debris is especially difficult but, of course, is no different in nature from that necessitated by large ship construction. Some operations will require protection from rain and wind; fireproof canvas, plastic, or similar shelters may be required.

Detailed weight control of the superstructure must be maintained, as it must for the substructure, with the added impetus that weights on the deck are high above the center of buoyancy and hence will have a disproportionate effect on the stability. For example, to counter an extra 100 tn. on deck may require adding 1000 tn. of solid ballast in the base raft.

When the deck is ready for transfer, it is essential that the weight and its distribution be accurately known, since the deck structure will be transferred by floating equipment. This structure may differ from the final configuration, since one or more modules may have been delayed or postponed. Temporary materials and equipment may be on board and must be accounted for.

12.2.9 Stage 9—Deck Transport

The deck is now to be lifted off the pillar supports by means of large barges. These "barges" may be conventional offshore barges, halves of surplus tanker hulls, or special pontoon combinations designed to float in under the deck, between the pillars. They are then deballasted to rise up under the deck, so that sea-fastening supports may be affixed between the barge and the deck girders at appropriate lift points. This operation requires calm water.

A careful analysis and evaluation has to be made regarding where to use fixed connections, where to use pinned connections, where to allow flexibility, and where to use ties or struts. With fixed connections, high moments will be developed in the deck trusses or girders, requiring evaluation. The two or three barges, together with the deck, will form a

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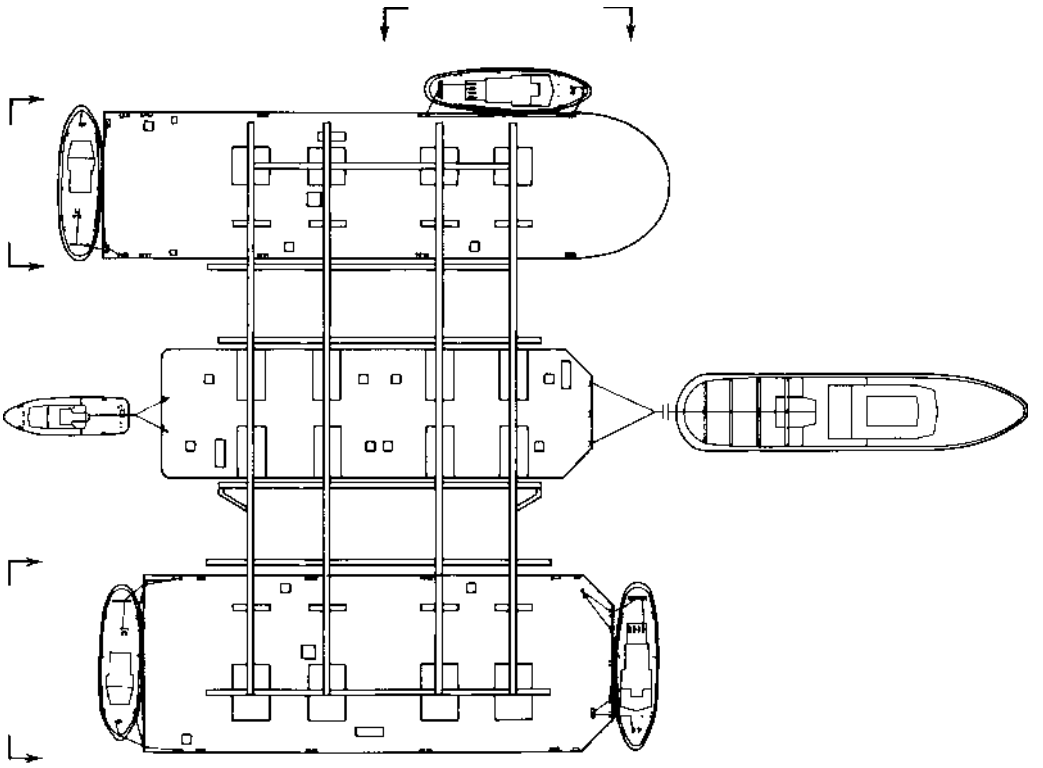

FIGURE 12.28
Configuration of towing spread for Statfjord B deck. (Courtesy of Aker Maritime.)

complete structural system and must be analyzed as such, under both static conditions and the dynamic conditions (waves) expected during this particular movement.

When all barges have been properly ballasted, the sea fastenings connected, and the weather forecast is favorable, the barges are progressively deballasted until just before liftoff. Load cells may be used to verify forces in key members. Then, the three barges are further deballasted, raising the deck clear of the pillars.

During this operation, the barges are moored to the pillars by taut lines of sufficient length to accommodate the vertical movement. Fiber lines may be used to absorb shock. If blocking is used between barges and pillars, it should have Teflon coating or similar provisions to enable sliding to take place.

The boats then take the structure in tow, providing both pull and lateral guidance (Figure 12.28). The major concern during such a catamaran or trimaran tow is wind across the channel. Boats must have adequate power. Bow thrusters are desirable if the tow must be made in narrow channels; this will enable the boats to operate on very short scope if necessary. Contingent plans should be made in case a wind storm is suddenly forecast; lay-by areas should be selected.

Temporary facilities will have been mounted for the tow similar to those provided for the substructure in order to provide for control, communication, lighting, and emergencies.

Upon arrival at the site, a decision must be made, depending on weather forecasts, whether to moor the deck-barge complex or to proceed directly to the mating. When the deck and barge complex is finally committed to the mating, many of the sea fastenings can

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be loosened so that, while support is still provided, there are no tension connections between barges and deck.

12.2.10 Stage 10—Submergence of Substructure for Deck Mating

Before arrival of the deck-barge complex, the substructure will have been subjected to two tests. One is a standard inclination experiment, in which a known weight is moved a known distance. The resultant angle of heel is measured. From this, the true position of the metacentric height can be determined and hence the location of the center of gravity. The second is to test the ballasting and deballasting systems and control systems and to verify watertight integrity of the substructure. It also is an excellent opportunity to train the crew in this all-important operation. In some cases, watertightness depends on temporary closures; in the case of Ninian, several hundred plugs were used to close the holes in the perforated breakwater. These must all be double-checked to ensure they are secured tightly; the failure of only one could endanger the entire platform.

The substructure is then submerged in a series of steps, checking weight of added ballast water vs. resultant draft. As the roof of the base raft becomes submerged, any vertical openings such as star cells, duct openings, etc., are immediately subjected to hydrostatic pressure throughout their full height.

In a number of cases, compressed air has been introduced into the main cells during deep submergence in order to provide additional safety against implosion due to the high hydrostatic forces. It is recommended by both the Federation Internationale de la Precontrainte (FIP) Recommendations for Concrete Sea Structures and the American Concrete Institute ACI 357 State of Art Report on Concrete Sea Structures that the structure have a safety factor of at least 1.05 in the event of loss of air; thus, the use of compressed air is determined by the need to further raise the safety factor.

Care must be taken to ensure that when compressed air is introduced into a cell, it is done in a step-by-step pattern matching the external hydrostatic pressure during both deep submergence and subsequent deballasting. The air pressure is constant over its full height, whereas the hydrostatic pressure not only decreases due to deballasting but also varies in a triangular diagram along the height of the cell. Uncontrolled air pressurization could result in an outward explosive force on parts of the structure designed only for external load. Compression of air raises its temperature which will later cool back to that of the external water, in turn reducing the pressure. The typical structure is very large. It is not unusual for the compressed air volume to add several hundred tons of air to the total weight of the structure during the filling process.

During the submergence, the structure is subjected to the largest external hydrostatic forces that it will ever experience; hence stresses are at their maximum. Any structural defects are brought into sharp focus.

In the case of Statfjord A, it was an apparent small crack and leak in the diaphragm wall of a cell, which later turned out to be a serious but repairable delamination of 10 by 20 m in size and up to 6 mm in width.

The tragic catastrophe of the Sleipner failure occurred during its test submergence. When at its deepest draft, the diaphragm wall at a star cell (interstice between the larger cells) failed in shear. This was followed a few seconds later by a second similar failure and the entire structure rapidly sank, violently imploding at depth. Fortunately, the small crew on board was safely evacuated.

This failure was, of course, the subject of intensive investigation, which found the specific technical error to be the lack of reinforcing steel adjacent to the throat of the intersection of

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the two walls. The star cell was subject to the hydrostatic head of the external sea while the adjacent main cells were deballasted, thus developing a high shear across the wall.

The relevance to construction is that the originally designed long-headed stirrup bars that crossed the throat had been replaced during construction at the request of field personnel by short bars, in order to facilitate installation in this congested zone. While the overall causes included design and institutional failures as well, it is probable that the original long bars would have prevented failure.

The structure was rebuilt with substantial additional reinforcing steel in the critical areas. The replacement structure was successfully submerged for the deck mounting and is now in service in the North Sea.

12.2.11 Stage 11—Deck Mating

The scene is now set for the deck transfer operation. The weather forecast must, of course, be favorable, with minimum wind. All boat traffic in the vicinity is stopped. Water density measurements at depth are made to be sure that calculations are based on actual densities. The substructure is ballasted down so that only 3–5 m of shaft extends above the waterline. The deck-barge complex is slowly winched in around the shafts. Clearances are typically only 300–500 mm (see Figure 12.29).

Hydraulic winches can control the structure within 20 mm or so, but the lines have elasticity, making them sensitive to even minor surges. Blocking is therefore employed, arranged with hydraulic jacks, to effect the final horizontal control. The blocks are fitted with Teflon so they can slide vertically.

When horizontal positioning has been verified, it is time to deballast the substructure. There will usually be about 1 m of vertical gap. The substructure is brought up just short of contact; all points are checked. Then, deballasting continues. When 10% of the load has been transferred, a final position check is made. All remaining tension sea fastenings are now disconnected.

During this stage, a very complex interaction takes place. As the substructure starts to raise the deck, the barges also rise due to relief of weight. They tend to follow the deck up as it is picked up by the substructure. Since the substructure will now have the weight of the deck on it, the caisson sinks lower, which means that the net external hydrostatic heads are now the maxima which the structure will ever see. This condition typically is the most critical in the structure's life, as far as implosion is concerned.

FIGURE 12.29
Floating deck structure over submerged GBS. Note buoyed line for compressed air in lower right.

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Finally, the deck is picked clear of the barges and on up to a safe height (see Figure 12.30). This "safe height" is one selected to balance safety against implosion (with all compressed air removed), stability, and access for further work. The barges can now be pulled clear.

Since the conditions of support of the deck during transport are inherently different from those on the pillars or on the substructure, there will be differential vertical and horizontal deflections and twist (Figure 12.31). These must be accommodated either in the flexibility of the deck and shafts or else by the use of bearing devices which will permit lateral movement on the bearings. Sand jacks, neoprene pads, stainless steel, and sliding plates with Teflon are some of the systems used. Whatever is used, positive stops should be provided.

Vertical tolerance adjustment between the deck and the ring beams on top of the shafts has been provided by a number of means. C. G. Doris has employed flat jacks, which permit measurement and equalization of load. After such adjustment, the water in the jacks is replaced by cement grout. Norwegian contractors have employed soft iron pipe sections, thick-walled, which deform under load concentrations, equalizing the load onto adjoining pipe sections.

The deck now rests on the shafts by gravity alone. A check must be made to ensure safety in the event of accidental flooding, which would produce a heel. A typical requirement is that the deck not slide off, even if a significant heel (e.g., 7° or even 15°) is experienced due to accident. Such an accidental heel did occur at a later stage with the Statfjord A platform. During testing of the ballast control system, one valve was left open, causing an unplanned

FIGURE 12.30
Deck transferred and raised. (Courtesy of Aker Maritime.)

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FIGURE 12.31
Towing Beryl A in the open ocean. Note two tugs pulling astern. (Courtesy of Aker Maritime.)

shift of water to one side, resulting in an increasingly severe list. Alarms sounded and the workers started to abandon the structure. Fortunately, the quality control supervisor on the platform had the knowledge and courage to descend deep into the utility shaft, correct the valve error, and ballast the structure back to vertical.

The deck is now additionally secured to the shafts by prestressing. Prestressing is especially effective in eliminating cyclic fatigue and in providing safety against uplift under accidental conditions. These prestressing tendons are usually short bars. A system must be used that permits adjustment for initial seating losses, since in a 4-m-long tendon, a seating loss of 6 mm amounts to a loss of prestress of 350 MPa, almost half that input by the jacks!

These prestressing tendons must also be protected against corrosion, since this is one of the most vulnerable zones. In particular, sealing and drainage must be assured to prevent saltwater spray from being trapped in anchorage pockets.

During the entire deck-mating operation, careful checks are made of the ballast water in all the cells to detect any unexplained variances in water level or quantity pumped. Even relatively small discrepancies should be checked by visual examination within the cells. The relatively small in-leakage in one of the cells of the Statfjord A platform, referred to earlier, showed only about 300 l/min inflow, but turned out on more detailed investigation to be due to a very large laminar crack in a cell wall, caused by secondary bending of the large caisson as it was subjected to the high hydrostatic forces of the deep submergence. Extensive repairs were necessary.

The Ninian deck transfer was carried out in a different but very ingenious manner. The deck structure, weighing 7000 tn., was constructed on land, at Inverness, Scotland, and then skidded onto a single large barge for tow around the north end of Scotland to the mating site near the Isle of Skye. In a sheltered inlet, the deck was transferred to two barges which supported it only along its edges.

Meanwhile, the concrete substructure was ballasted down. In suitable weather, the twin barge assembly was brought to the substructure. By means of lifting rods supported from towers on the centerlines of each barge, the deck was raised so that it would clear the top of the single central shaft. The twin barges now were guided past the shaft, straddling it. The lifting rods now lowered the deck onto the shaft.

Temporary supports were flat jacks on neoprene pads. With the change in support conditions, the deflections of the deck trusses also changed, resulting in lateral movements

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which were accommodated by the pads. The flat jacks were then used to equalize the load at each of eight support points. Then they were pumped full of grout. For the new float-over method of deck installation, see Section 17.5.

12.2.12 Stage 12—Hookup

During this stage, which may require two or three months, the platform is deballasted to raise it to its optimum position, where pressures on the cell walls are reduced and where access can be provided to the shafts.

The next operations are those of hookup of all equipment and its testing. These can require many hundreds of thousands of man-hours. While the site of work is usually protected, nevertheless all access must be by water, and the deck is now 30–50 m or so above the water. Hundreds of personnel must move on and off at each change of shift. It is necessary to provide boat landings, adequate crew boats, elevators and stairways, lighting, and water and power supply. As soon as possible, the permanent cranes on deck are made operable so they can handle supplies, tools, and fittings on and off as required. Safety of the workers must be assured; this requires life rafts, a safety boat, and suitable temporary railings. Fire protection must be provided (see also Section 17.3).

12.2.13 Stage 13—Towing to Installation Site

The structure is now placed under tow to the site. Its displacement may be several hundred thousand tons; Statfjord B displaced almost 700,000 tn., Hibernia over 1,000,000 tn. Six or more of the largest tugs in the world are typically employed, each being rated near 20,000 IHP and having a bollard pull in excess of 150 tn. These are arranged with towing pendants and retrieving lines as described in Section 6.1. A more in-depth discussion of towing is given in that section. For the Troll platform, displacing almost 1,000,000 tn., eight tugs were employed.

Within restricted waters, two stern tugs are also employed, to prevent yaw and to keep the GBS platform fairing true behind the boats (see Figure 12.31). Then, when the tow emerges into the open ocean, the scope of towlines is lengthened and the stern tugs are cast off (Figure 12.32).

A pilot boat may lead the way during exit from the channel, available to warn other shipping and in some cases to verify clearance across relatively shallow waters. A safety

FIGURE 12.32
Towing Ekofisk caisson—first offshore oil storage caisson.

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boat may run alongside, for use by supervisory personnel, interboat transfers if needed in an emergency, and to pick up a person overboard.

The GBS platform is outfitted for the tow with a navigational bridge, radios for communications to boats and to the shore base, electronic position-finding equipment, safety and firefighting equipment, quarters for the riding crew, diesel and water supply, and generators for power and lighting. The heliport on the deck may be activated for emergency use. Duplicate radar, gyros, fathometers, and forward-searching sonars are installed, as well as emergency generators.

The route will have been carefully surveyed using side-scan sonar and profiler equipment to identify any pinnacles, wrecks, ridges, or shallows which may have been missed by conventional echo-sounding. The boats and the platform will be equipped with satellite position-finding equipment (GPS and/or DGPS).

Lay-by areas are provided along a long tow route, where the structure may safely ride out a severe summer storm. In the event of a storm, the structure may be ballasted down more deeply to increase stability.

Data concerning tows of some of the earlier gravity-based offshore platforms are given in Section 6.1 and in Table 12.1 and Table 12.2. Also, Figure 12.33 shows graphically the total tug horsepower in relation to displacement for large gravity-based structures in the North Sea.

During the tow, stability will probably be the controlling parameter. The draft, ballast, and pay load of equipment on deck will all be selected to give about one meter of positive metacentric height (Figure 12.34). While this may seem small, it must be remembered that the initial righting moment is the product of the $\overline{GM}$ and the displacement, and the displacement in this case is very large.

Damage control provisions may also govern design. Usually the structure will be required to be able to suffer flooding of any one exterior compartment. Internal subdivision of a shaft and fendering of a shaft are both rather impracticable in most cases; hence these requirements are usually waived if the shaft is specially thickened and reinforced to withstand the impact of a boat near the waterline at the towing draft. Note that the towing draft is normally less than the installation draft.

Dynamic response of the platform's motions must be considered under the action of the design storm. This is usually taken as the ten- or twenty-five-year return storm for the period of year involved. With its relatively low metacentric height, the structure will typically roll and pitch in a relatively long natural period, perhaps 60 s, developing maximal accelerations as high as 0.20–0.25 g. The accelerations, of course, affect the deck equipment and require that all attachments be capable of resisting the lateral forces which develop.

It is important to calculate righting moments for the various angles of heel and the heeling forces due to wind and waves, since a typical platform does not have the response of a conventional ship hull and metacentric height is a measure of stability only at very small angles of heel. Usually, a limit of 5° will be placed on the maximum heel during the 10-year storm, assuming the towline is slack or has been dropped.

12.2.14 Stage 14—Installation at Site

Upon arrival at the site, the structure is to be ballasted down onto the seafloor. The several boats fan out in star-like pattern to hold the GBS platform in location. Here is where bow thrusters are very helpful, enabling the boat to select its heading without necessarily exerting excessive pull.

Witt and Meurs in their paper "The Positioning of Offshore Constructions. Research and Training by Simulation" presented at the European Offshore Petroleum Conference in

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TABLE 12.1

Towage and Emplacement of Concrete North Sea Structures

Unit	Date of Tow and Departure Point	Tow Displac. (tonnes)	Draft with/without Dowels (m)	Distance (nautical miles)		Total Tow Time (days)	Mean Open Sea Speed (knots)	Total Tug IHP.
				Channels or Fjords	Open Sea			
Brent A (Condeep)	July 75 Stavanger	338,000	82	44	118.4	6	2.36	68,000
Brent B (Condeep)	August 75 Stavanger	384,500	76	44	184	7	2.20	68,000
Frigg CDP.1 (C.G. Doris)	August 75 Andalsnes	209,000	67	44	268	8	1.89	44,500
Frigg MCP01 (Doris)	July 76 Kalvik	206,000	66.5	25	350	11.0	1.6	43,000
Brent D (Condeep)	July 76 Stavanger	382,000	116/113	48	172	8	2.30	72,000
Frigg TP.1 (Seatank)	May–June 76 Ardyne	166,000/ 209,000	35 inc. to 64	147	620	11.5	2.78	54,000
Statfjord A (Condeep) (Base Tow)	August 76 Stavanger	370,000	65.2/60.5	75	40	3	2.70	78,000
Statfjord A (Condeep)	May 77 Stord	457,000	119/114.3	24	165	5	2.06	68,000
Dunlin A (Andoc) (Base Tow)	July 76 Rotterdam	232,000	25.2	65	508.5	7.10	2.98	78,000
Dunlin A (Andoc)	June 77 Stord	419,000	131.2/129 (locally 151.2/149)	32	136	5	2.3	68,000
Frigg TCP 2 (Condeep)	June 77 Andalsnes	292,760	91.54/86.04	44	307	14	2.15	68,000
Brent C (Seatank) (N. Channel)	July 77 Ardyne	298,000	38.4/38.4	199	738	13.12	3.42	84,000
Cormorant A (Seatank) (N. Channel)	June 77	346,500	37.5/37.5	173	687	8.58	3.34	95,000

(*continued*)

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TABLE 12.1 *Continued*

Unit	Date of Tow and Departure Point	Tow Displac. (tonnes)	Draft with/without Dowels (m)	Distance (nautical miles)		Total Tow Time (days)	Mean Open Sea Speed (knots)	Total Tug IHP.
				Channels or Fjords	Open Sea			
Ninian Central (Howard Doris)	May 78 Raasay	601,220	84.2	–	499	12	1.8	76,500
Statfjord B (Condeep)	August 81 Vatsfjord	825,000	130/127	70	164	5.5	1.7	86,000
Statfjord C (Condeep)								

Source: Courtesy of Noble Denton.

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TABLE 12.2

Towage and Emplacement of Concrete North Sea Structures

Unit	Total Static B.P. (tonnes)	No. of Towing Tugs	Speed of Immersion (m/h)		Time to Emplace (h)	Conditions at Touchdown: Wind Force/Wave Height, Period	Water Ballast Added to Emplace (tonnes)[a]	Distance Off Target (m)
			Max.	At Seabed				
Beryl A (Condeep)	584	5	8	8	45	SE'ly 2 0.5 m 5 s	90,000	32
Brent B (Condeep)	584	5	12	6	39	S'ly ⅔ 0.6 m 6 s	124,000	25
Frigg CDP.l (C.G. Doris)	372	4	5.6	2	39	WSW ¾ 1.0 m	6,000	14
Frigg MCP01 (Doris)	335	4	6	1.6	13	NW ¾ 2.0 m 6 s	7,000	8
Brent D (Condeep)	545	5	6	6	11	WSW 3 1.2 m 5 s	12,500	8
Frigg TP.l (Seatank)	440	4	10	Minimal[b]	11.7	SE 4 1.5 m	4,500	1
Statfjord A (Condeep) (Base Tow)	605	6	–	–	–	–	–	–
Statfjord A (Condeep)	565	5	8	8	7	SW 1–2 1.3 m 8 s	13,000	10
Dunlin A (Andoc) (Base Tow)	560	6	–	–	–	–	–	–
Dunlin A (Andoc)	632	6	10.5	5	4	NE 3 2 m 8 s	2,500	12
Frigg TCP 2 (Condeep)	565	5	6.2	6.2	6	NNW 3 1.5 m ⅘ s	5,000	2
Brent C (Seatank) (N. Channel)	645	5	–	–	–	–	–	–
Cormorant A (Seatank) (N. Channel)	730	6	–	–	–	–	–	–

(continued)

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TABLE 12.2 *Continued*

Unit	Total Static B.P. (tonnes)	No. of Towing Tugs	Speed of Immersion (m/h)		Time to Emplace (h)	Conditions at Touchdown: Wind Force/Wave Height, Period	Water Ballast Added to Emplace (tonnes)[a]	Distance Off Target (m)
			Max.	At Seabed				
Ninian Central (Howard Doris)	585	5	–	–	19	–	–	10
Statfjord B (Condeep)	715	5	7	1	6	W ½ 1.5 m 7 s	11,000	15

[a] Calculated from commencement of tug rearrangement to point where unit no longer considered in transit condition.
[b] Speed of immersion of TP.1 at contact with seabed was related to moving at slow speed onto a sloping seabed.

Source: Courtesy of Noble Denton.

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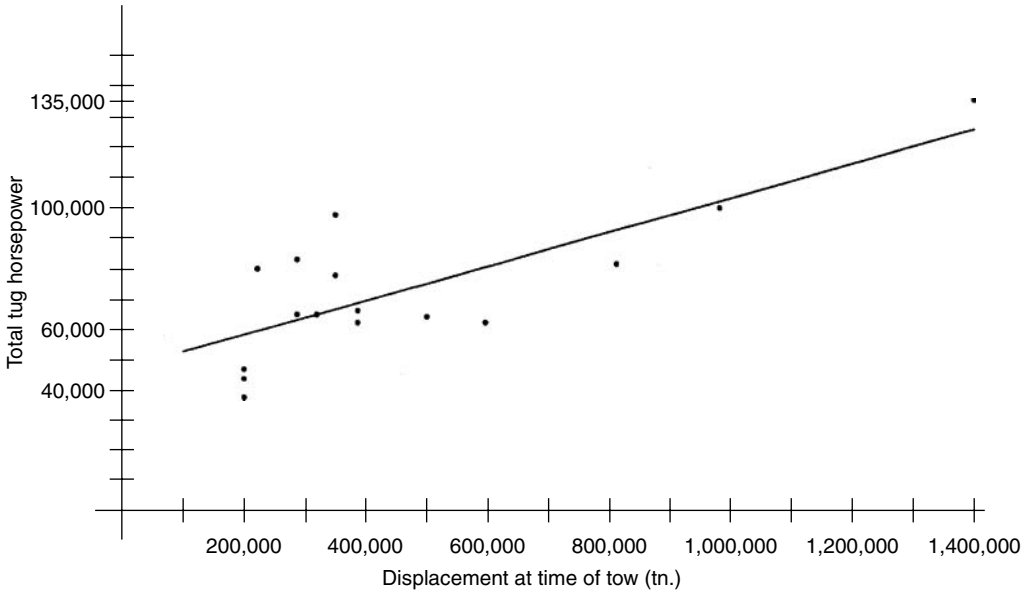

FIGURE 12.33
Total tug horsepower vs. displacement of North Sea gravity base structures as ballasted down to towing drafts.

Stavanger, Norway, 1978 suggest the following rules to govern tug manipulation and positioning:

1. Wait to give a new order until the former order has had noticeable effect. The structure moves with great inertia and little drag, so all braking must be done by tugs.
2. Use as few tugs as possible.
3. Redirect tugs back to their original position after each action.
4. Use little steps in changing power and course as opposed to radical changes.

It would appear that Rule 1 will need to be interpreted in context, since braking action may sometimes have to be initiated soon after a thrusting force is applied.

FIGURE 12.34
Statfjord A GBS Offshore Platform in initial service in North Sea.

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An even more precise method for positioning is the use of mooring lines, generally employing the inverted catenary mooring system, with wire lines leading from the anchor to a spring buoy, then to the tug which is anchored. The spring buoy ensures that the line leading to the tug is near horizontal; otherwise an inclined line might pull the stern underwater. From winches mounted on the deck of the platform, wire lines are run down to submerged fairleads, then out to the spring buoy and thence to anchors or to moored tugs. This system of moorings is appropriate when offshore structures are installed over subsea wellhead templates with predrilled wells. One or more large derrick barges may take the place of several spring buoys, serving as control vessels during the lowering, as was done in the case of the Cognac steel jacket platform. Final positioning is further discussed in subsequent paragraphs.

Control during installation is carried out from a control room within the structure. The instrumentation will generally include

- Echo sounders, to give bottom clearance at four or six corners;
- Pressure transducers to read draft at the same locations;
- Pressure transducers to read internal ballast water levels in each cell;
- Strain gauges to read axial forces and moments in dowels and in selected skirts;
- Differential pressure transducers to give water pressure in each skirt compartment;
- Biaxial inclinometer for tilt;
- Earth pressure transducers to give contact pressures on base slab;
- Strain gauges to give strain of reinforcing steel in base slab or domes;
- Pressure transducers in a closed hydraulic system to give skirt penetration.

The installation manual will contain a description of all the systems involved, background data, and drawings. It will give a detailed set of guidelines for installation, including:

1. Positioning and orientation
2. Dowel penetration at touchdown
3. Ballasting and drainage (venting)
4. Concrete skirt penetration
5. Base slab (or dome) seating
6. Underbase grouting

Ballasting down is carried out by pumping water in or by controlled free-flood. Stability generally becomes greater during submergence, at least with structures having generally vertical or outward-sloping sides. However, conical structures such as those often designed for the Arctic, lose water plane rapidly as they are submerged more deeply and may become unstable.

Another system, extrapolated up from the smaller barge-mounted facilities in shallow water in the Gulf of Mexico, is "tipping down." One end is purposely tilted down to engage the seafloor; then the entire facility is rotated down. In such a case, stability will initially depend on the inclined water plane and, later, on the support from the bottom as it engages the lower end and prevents sideways rolling. While such maneuvering has been successfully carried out for smaller structures, its use on large structures should probably be limited

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to relatively minor angles of tilt and be well verified by model basin tests before field implementation. Consideration must be given to the loading on the structure's edge during this landing and the disturbance to the seafloor. Skirts may suffer overstress in bending. This procedure was successfully employed on the Hunterston B platform.

For large structures and major offshore platforms, the use of temporary buoyancy would appear to be more conservative and appropriate. Temporary buoyancy tanks will have to be relatively large in diameter (e.g., 5–10 m) and high (e.g., 40 m) to raise the center of buoyancy without significantly affecting the center of gravity. Therefore, they will usually be made of steel, with heavy ribs, although one study has shown that a hybrid steel-lightweight concrete shell was even more efficient. Temporary buoyancy tanks also give a minor improvement in the moment of inertia of the water plane but unfortunately also increase the displaced volume, so that the net effect on stability may not be significant.

Temporary buoyancy tanks give their major contribution by raising the center of buoyancy. Therefore, they do not have to be symmetrically placed around the structure. For example, one such tank can be installed on one side only, if that is desired, and verticality maintained by selective ballasting.

These tanks must be attached to the structure at some point on the enlarged base caisson. These attachments are subject to the dynamic cyclic forces of the waves during the tow, which can lead to low cycle fatigue, especially in the corrosive environment. Prestressing of the tanks to the base is one good means of attachment, using post-tensioned unbonded tendons prestressed from the top of the tank at a stage when that top is above water. The tendons can then be released after installation, again working from the top of the tank. Alternatively, they can be cut by explosives, hydraulic shears, or underwater burning. To release such tanks, they should be ballasted to neutral buoyancy and then released so that they do not rise up suddenly. A towline to an attachment at the center of rotation of the tank will prevent the tank from rotating during release and rise, with possible damage to the shafts, deck, or tank.

Temporary buoyancy tanks were employed on ten caissons of the offshore terminal at Hay Point, Queensland, Australia. During the installation of the last caisson, one connection failed due to corrosion-accelerated fatigue; fortunately, this happened when the structure had almost touched down, so there were no serious consequences.

Returning to the structure at the site, it is being held in position by the tugs (or mooring lines) and being ballasted down by pumping or flooding in of seawater. All events and conditions are being monitored in the control room. As it nears the seafloor, the rate of descent is slowed to allow the water to escape.

At touchdown, the dowels plow into the soil to sufficient depth to prevent the caisson from further lateral movement. Bending stresses, converted from strain gauges on the dowels, are read out in the control room. The short-range, high-frequency echo sounders on each corner give the distance between base slab and seafloor. In lieu of dowels, spud piles may be employed.

To reduce the time to first production of oil or gas, a small number of wells may be pre-drilled, using a template to ensure proper spacing on the seafloor. This requires that the structure be floated over the top of the wells. This requirement also precludes the use of long skirts and pre-installed dowels. Bumper piles are drilled or driven in, against which the base of the GBS is engaged and held during final ballasting down to the seafloor. To hold the structure against the bumper piles during final descent, one or two spud piles may be dropped and caused to penetrate far enough to prevent short term dislocation. Alternatively, wire rope lines may be attached as to pull the GBS tightly against the bumper piles. To position the bumper piles relative to the pre-drilled wells, an extension from the well

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template may be used, (but removed before GBS set-down) or wire lines pulled taut. Video provides means of monitoring these operations.

The skirts now engage. These have been designed not to buckle, even if they hit a subsurface boulder; they will displace it laterally through the soil. Ballast continues to be added until the desired penetration is achieved. The initial penetration rate is kept slow—e.g., 150 mm/h—to avoid overpressure in the skirt compartments and consequent piping. Once skirts are well embedded, the rate of penetration may be increased to about 1 m/h. Meanwhile, water is being vented out from the skirt compartments.

Eventually, full penetration is achieved and the structure comes to rest on the base slab or on concrete skirts or sills purposely designed to halt further penetration and thus prevent excess local pressure on the slab. Without such arresting, the structure will continue to penetrate until it bears directly on the slab. High spots of clay will be displaced and squeezed out. Even a surface boulder will be forced down into the soil. A high lens of sand, however, may give very high local bearing resistance, thus overloading the slab. Pressure cells on the base slab and strain gauges may be used to monitor slab-foundation interaction.

If the skirts do not penetrate the required distance, even when the structure is fully ballasted, then the water trapped inside the skirt compartments may be bled off into the utility shaft at an underpressure, thus increasing the effective force causing penetration. Such selective ballasting and underpressuring allows very accurate control of level. The degree of underpressure must not be so great as to initiate piping under the skirts. Use of underpressure has proved to be essential for achieving penetration of long skirts such as those on Draugen, Gullfaks C, and Troll. For the Gullfaks C platform, 22-m-long concrete skirts penetrated the soft clays with interbedded lenses of sand. By reducing the pressure inside the cells, the necessary driving force was developed. A large-scale field test demonstrated the enhanced effectiveness of cycling the internal pressure.

Once the structure has been penetrated to its designed embedment, the spaces remaining beneath the base slab are often filled in order to ensure equal bearing on the soil. Weak grout fill has been used in many cases. Thixotropic admixtures can be added to the grout to reduce segregation and prevent excessive intrusion of the grout into interstices of stone and grout escape through minor openings into the sea. For example, for offshore caissons in Australia that were seated on a prepared base of crushed stone, tests showed that normal grouts were much too fluid and penetrated too far into the stone, whereas a thixotropic grout gelled as soon as the resistance to flow increased.

Grouts for underbase fill must always be placed under a very low head to avoid development of piping under the skirts and to avoid lifting the caisson. Use of a gravity feed through an open hopper at the correct elevation needed to just overcome the hydrostatic head by about 15 psi (0.1 MPa) is the best procedure. Provision must be made for venting of the displaced water. If pumping is employed, this must be at low pressure, with positive vents to prevent overpressurization.

When the void under the base is thick(for example, 0.5–2 m) as it is in many North Sea installations, then means must be taken to prevent washout of cement. An anti-washout admixture should be employed. Excessive heat of hydration is a problem. Some full-thickness tests showed that the temperature with normal cement mixes could rise to 100°C and more, creating undesirable strains in the base slab. Use of a coarse aggregate, about 10 mm maximum, coarse ground cement, high fly ash, or blast-furnace slag cement will allow reduction of cement content and hence lower heat.

In addition to selecting a grout mix for low heat, consideration must be given to the logistics involved: How will the materials be delivered, batched, and mixed out at the site, taking into account remoteness and sea conditions? In some cases, batching and

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mixing can be performed on the platform deck; other solutions involve prebatching with dried materials and transport in water-resistant containers of steel or plastic.

Quantities of underbase grout are typically large. The installation of 14,000 m^3 of underbase grout on the ELF–TCP2 platform required nine days.

Finally, each installation needs to be evaluated as to what is desired. In many cases, a low modulus of elasticity is desirable. Strengths need be only a little better than that of the foundation soils: 1–2 MPa may be a reasonable value in many cases. Bleed should be minimal. In the North Sea, underbase grout mixes have been developed by Norwegian contractors, which are a slurry of cement and seawater, with a foaming admixture such as sodium silicate, from 4 to 10% by weight of seawater, along with retardation and stabilizing admixtures. This presents minimum logistics problems and develops a low-strength, low-modulus mix that flows well and tends to fill up underbase cavities.

For caissons in relatively shallow water or where strong bottom currents are present, underbase grout must be protected from erosion either by skirts or an antiscour stone mattress. A higher-strength grout and quicker set are required. Cement-based grout fills were flowed into the underbase gap of the sixty-six gate support caissons of the Oosterschelde Storm Surge Barrier and under the caisson piers of the Øresund Bridge.

In cases where the platform will later be refloated, for example, an exploratory drilling structure for the Arctic, or where a permeable underbase fill is desired, properly graded sand may be slurried into the underbase spaces. The sand slurry is flowed in through piping, either internal or external, having minimum bends; those bends that are necessary should be of large radius. The sand tends to drop out of suspension and build up a little dam; this causes a slight rise in pressure, and a rivulet of sand slurry breaks through to create new fill and successively new dams. Spacing of discharge points is usually 3–5 m on center, assuming a 30-cm depth underbase void is to be filled.

Sand flow methods have been thoroughly developed by both Danish and Dutch engineers for use in their submerged tube and harbor construction. Offshore, they have been successfully applied under Arctic exploratory drilling caissons. In all cases of underbase filling, whether by grout or sand, the feed should be at low head and well controlled to prevent raising the structure by excess pressure.

The next item of work is that of scour protection, if required. Skirts which penetrate are one means of scour protection; however, these are not always practicable due to soil conditions or draft constraints.

Rock may be dumped around the periphery of a caisson, using a long discharge tremie from a rock hopper barge. This system was developed by Dutch dredging contractors for covering submarine pipelines in the North Sea. Uncontrolled dumping from barges caused significant impact damage to the concrete piers of the Oosterschelde Storm Surge Barrier. Placing by skip is another solution.

In sandy soils, a filter fabric of some sort is desirable under the stone to prevent the sand from leaching through the interstices in the stone. Fabric mats can be preattached to the edge of the base slab and rolled up along the sides of the lower walls. After landing, they may be cut loose and spread out radially from the base. To hold their outer edges, divers may place sandbags or drive a headed steel pin through them to anchor them in the sand. This procedure was employed on the Ekofisk caisson. The filter fabric was then covered by dumped rock. Shortly after completion of this work, the structure encountered a major storm. The scour protection proved to be fully adequate.

Filter fabric for this application should be heavily reinforced with nylon or even stainless steel wire. Such a heavily reinforced fabric, sewn together with a finer mesh geotextile, has been used extensively in coastal applications in The Netherlands.

Hence, one solution is to place prefabricated mats made up of heavily reinforced filter fabric to which concrete blocks have been preattached. Such mats can be placed by derrick

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barge, using spreader beams; large mats would have been laid out and stacked on a barge for easy handling. These were used to provide scour protection for the caissons of an offshore terminal in Queensland, Australia, and successfully performed through a major cyclone shortly after installation. Such mats can also be affixed to the periphery of the GBS base and stopped off up along the sides. After seating of the GBS, they can be cut loose and laid out on the seafloor with the aid of cranes or workboats. Such mats could probably not stand the action of the sea on a long open sea voyage and hence would not be attached until at or near the final installation site.

Articulated concrete block mats such as those used to protect offshore islands, were used on a large concrete caisson structure for the Arctic. At one installation offshore Alaska's North Slope, these mats were damaged by sea ice. Heavier ones have been installed as repair. Recently, some have been installed offshore Sakhalin.

Dumped rock may be impracticable where there are flow lines and other mechanical fittings near the base of the caisson or where it is impracticable to locate such a vessel in the vicinity of the platform, keeping in mind the many other activities which must be carried out during these first few critical months. In that case, placement must be by tremie pipe or skip.

12.2.15 Stage 15—Installation of Conductors

The final construction operation at the site is to install the conductors. These will usually have been transported with the GBS structure and already set in the conductor guides. In the typical platform base, conductor sleeves will have been concreted in the base slab and then sealed against water entry during transportation.

While a double set of reinforced neoprene pile closure sleeves might be utilized to seal these conductors, most closures to date have been a 1.5- to 2-m plug of unreinforced lightweight concrete, which is later drilled out by the drill rig.

The conductor sleeves, penetrating the bottom slab, have both internal and external mechanical shear strips or keys to prevent any possibility of shear failure, as this could cause the loss of the platform during tow. When the sleeves are later drilled out, the drilling water level inside the drill shaft should first be equalized with the external sea level. Once the drill penetrates, any significant head differential will either cause the foundation soils to flow in or piping to take place to the outside.

In the case of Beryl A, when the head inside had been pumped down about 15 m to permit installation of fittings in adjoining cells, as soon as the drill penetrated, several hundred cubic meters of sand flowed into the shaft. Piping in from under the outer cells occurred. Satisfactory support and filling was restored only by extensive underbase grouting and external filter plus rock protection.

For subsequent structures, skirts have been provided around the drilling shafts and water heads have been kept balanced during this operation. Skirts around the drilling shafts will also limit the flow of drilling mud which may spill over from the wells into any adjoining underbase voids.

The conductors are now driven and drilled down to the prescribed penetration, following which the surface pipe is run and the wells drilled. The annuli between well casing and surface pipe and that between surface pipe and conductor are cemented. Biocides maybe added to the open shafts.

The wells remain free to move vertically, independently of the structure, as the platform settles over time. Figure 12.34 shows a typical offshore gravity platform, Statfjord A, in service. The disposal of drill cuttings must be planned so as not to create an

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unbalanced force on the structure, Drill cuttings maybe utilized for scour protection or ballast.

12.3 Alternative Concepts for Construction

Where a construction basin is impracticable, relatively narrow concrete or steel barge-like structures may be fabricated on a slipways and launched sidewise into the water. There they are joined together and post-tensioned afloat. Steel or precast concrete skirts may then be hung from the sides and joined to the deck.

Side launching produces less extreme moments and forces than end-o launching. It has been employed for many years for launching of steel barges and even ships.

A steel sheet pile cofferdam around the periphery can then be constructed so as to increases free board. Working inside the barges, any needed reinforcing structures (e.g., for enhancing shear capacity) can be constructed so as to act in composite action with the original hull.

Now the cell walls can be constructed and the rest of the structure completed as described for construction in the basin.

Many smaller GBS base rafts have been fabricated on a large steel barge, and then the combined structure submerged in a natural or artificial basin onto a shallow sand seafloor of known depth. During submergence, the lower barge is ballasted to negative overall buoyancy. Interior walls as well as exterior walls and the deck must be capable of withstanding the differential heads. Stability is maintained by the water plane of the concrete base raft. When the lower barge seats on the seafloor, the base raft is floated off. To refloat the lower barge, it is deballasted. While rising with free surfaces in its compartments, it will have negative stability, so practice is to install 2 or more steel columns fixed to the lower barge, which, if placed well apart, give sufficient increased moment of increased moment of inertia, primarily $I = Ar^2$, to offset the negative stability factors.

In some installations at sites with very weak soils, it may be necessary to augment shear transfer from the base to the foundation in order to prevent sliding. This need can arise with platforms in the Arctic where very high lateral ice forces must be resisted yet skirts are not practicable due to draft restrictions for transport, or where there is inadequate weight of the caisson structure, even when fully ballasted, to drive the skirts into the soil.

Spud piles were proposed for the Sohio Arctic Mobile System (SAMS). These would be driven after founding of the platform. They would be jacked or driven through steel sleeves to engage firmer soils such as partially ice-bonded dense sands at some depth below the seafloor. A typical spud might be 2–2.5 m in diameter, with wall thicknesses up to 100 mm, designed to penetrate 12 m. It will extend a further 20 m up into the spud well; multiple such spuds might be used on a platform. The spuds could, of course, be jacked in, in the same manner as the legs of a jack-up rig. The internal plug can be broken up by jets, which are permanently attached to the sides of the spud. Alternatively, the spuds can be driven in with a large vibratory hammer or even a diesel or steam hammer. Penetrations are very short as compared to piles, and hence installation time should be minimal.

If subsequent removal is required due to the desire to relocate the platform, the spuds can be pulled up using the following procedures (see Figure 12.35):

1. If there is concern that the spuds have been frozen in by permafrost, jet and airlift out any remaining plug inside, then use steam to heat the water.
2. Remove top spud supports.

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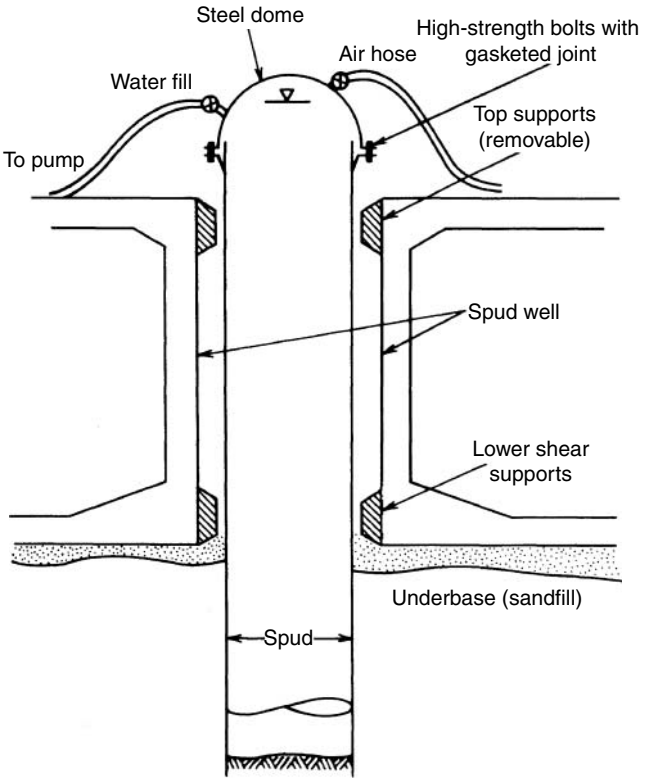

FIGURE 12.35
Removal of steel cylinder piles.

3. Raise the water level inside the spud as high as possible, well above sea level, to raise the pore pressure in the adjoining soil.
4. Activate jacks to remove spuds, or use a vibratory hammer to break them loose for lifting.
5. An alternative method of removal, used on similar-sized but longer piles or spuds, is to cap the piles filled with water, and then apply air pressure to jack the spud out. Since platform relocation is a planned event, the caps can be welded on long ahead of time. Alternatively, attachment of the caps can be by high-strength bolts.

If a spud has become buckled or distorted so badly by overload that it cannot be extracted, then the pile can be cleaned out by grab and airlift or by a drill. Then, a diver can always go down in the water inside the spud and cut the pile off. Similar-sized damaged piles have been readily cut off in this fashion at offshore terminals in Cook Inlet, Alaska, and Kharg Island, Iran.

Spud piles can also be used effectively in seismic areas where shear transfer to the foundation soils is required but there may be insufficient friction between the base and soil. The spuds can then be used to supply the needed shear resistance.

Spuds do not transfer vertical load. Hence, the caisson is free to settle. The weight of the caisson on the soil increases the passive resistance of the soil acting against the spud and hence the efficiency of the spuds. Thus, the full benefits of direct founding are maintained.

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Uplift-resistant piles were drilled and grouted into the foundation soils in order to hold down the three large underwater oil storage tanks off Dubai, in the Arabian Gulf. Skirt piles have similarly been proposed for concrete oil storage and drilling structures where the soils are inadequate to provide support for overturning.

When piles are used, provision must be made to transfer the pile resistances into the structure, which will usually be done by grouting within sleeves. This typically requires a relatively long sleeve length, say, 20–30 m, although use of mechanical shear transfer lugs or ribs will reduce the necessary length.

Skirts may also be used to develop supporting capacity for the structure. For example, 35-m-long skirts were used on the Troll platform, which is located in over 300 m of water depth, with extremely soft surficial soils. The skirts were designed to develop both skin friction (cohesion) and end bearing in the soils, thus minimizing long-term settlements due to compressibility.

The penetration is often able to be attained by ballast weight alone, provided provision is made to vent the trapped water and semifluid mud. This may be augmented by reducing the water pressure within the skirts, so that the ambient hydrostatic head, acting on the horizontal surfaces, forces the structure down. In effect, this is the principle of the suction anchor.

Alternatively, techniques previously used on the caissons for deep bridge piers may be adopted, namely, jetting and removal of the soil from under the base while increasing the driving weight by controlled ballasting and underpressure. To remove the material from the skirt compartments, a combination jet-airlift or jet-eductor system may be built into the structure. The peripheral jets slurry the material which is then pumped up and out over the base for disposal. Vents must be provided to the skirt compartment to prevent piping in the case severe under- or overpressure develops as a result of imbalance in the flows. In weak soils, such as clays, the increase in weight must be carefully monitored to prevent sudden shear failures of the soils. Removal of material from underneath is best accomplished within skirted compartments or open-bottom cells extending below the base slab. Multiple or rotating jets are used to break up the soil.

As an alternative to jets and eductors, an open sleeve may be run to the top of the base structure through the domed roof and a portable dredge pump lowered down—for example, a Pneuma Pump, a Marconaflo System, or a hydraulic dredge pump—to remove material successively from each of the many skirted compartments underlying the base. External jets around the periphery may be installed to permit lubrication with water or bentonite slurry (drilling mud) if dense sands must be penetrated.

A number of other approaches to the matter of structure-foundation interaction have been proposed, especially where lateral forces and, in deeper water, overturning moments, are very high (see Figure 12.36). Many of these have been described in Chapter 7.

1. In relatively shallow water, with only a moderate depth of weak soils, these may be removed by suction dredge prior to arrival of the structure. It should be noted that this increases the depth of the structure and hence increases its cost, weight, and draft.
2. Dredging, as in item 1, above, may be followed by filling the excavation with sand or gravel. Such refilled soils must have adequate consolidation to minimize settlements and to give adequate strength in bearing and shear. If necessary they may be densified by dynamic compaction (repeated dropping of a very heavy weight) or by vibratory consolidation.
3. Wick drains may be installed under the base area, to enable weak, claylike soils to drain as the structure weight is applied on the surface. A blanket of free-draining

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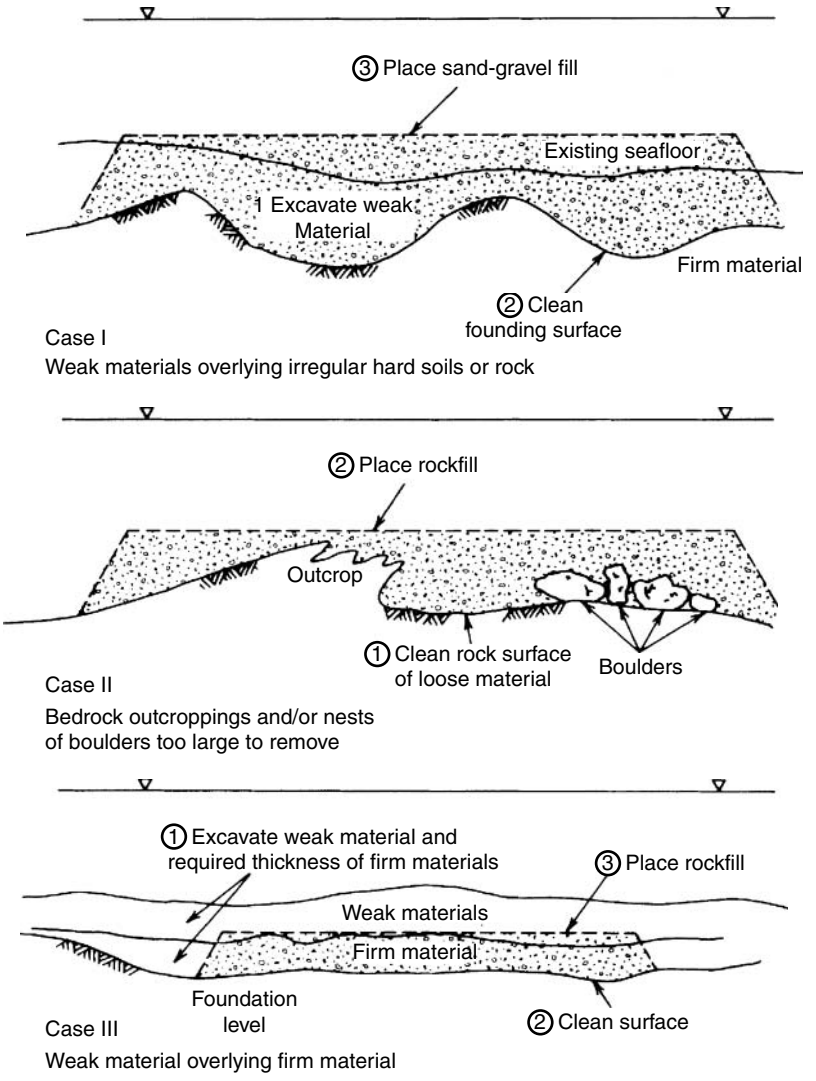

FIGURE 12.36
Prepared stone-fill bases for gravity base platforms.

material should first be placed to provide lateral escape for the water pushed out of the soil. Installation of wick drains may be carried out by barge-mounted equipment ahead of the arrival of the caisson or by working through the caisson itself immediately after founding. Drainage as a means of consolidation is time dependent; several months are usually required for full strengthening to resist design loads. This may be acceptable in some regions of the Arctic and sub-Arctic where the design ice load is not expected until three to five months after installation.

4. Embankments may be placed at the site in the season preceding installation, while the GBS is being built. Especially if these are extended over an area larger than the structure's footprint, then they will help to consolidate the clays and increase their shear resistance, as well as provide external berm resistance to shear failure in the soil. Of course, external berms of stone can also be installed after the caisson has

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been founded in order to increase the effective resistance of clays to bearing failure and sliding.

5. Multiple sand piles or stone columns may be driven in a closely spaced pattern under the area on which the structure is to sit. Such sand piles, if closely spaced, are effective in providing direct bearing and shear resistance. If a blanket of permeable material has been placed over the seafloor, then these sand piles will also act as drains when the structure's load is imposed.
6. Loose sands can be predensified by vibratory compaction. A major operation of this type was carried out to strengthen the foundation for the sixty-six piers of the Oosterschelde Storm Surge Barrier in The Netherlands. Multiple spud piles 1 m in diameter were jetted and vibrated to 20 m depth in the sands. Then intense horizontal and vertical vibrations were applied as the spuds were withdrawn.
7. "Jet grouting" has been proposed for relatively impermeable soils. A high-pressure injection of grout fractures the soil formation, allowing lenses of grout to penetrate.
8. Multiple piles, concrete, steel, or even timber, may be driven on close spacing to consolidate the soil and strengthen it in shear. The tops of the piles should be at or below the seabed and covered with 1 m or so of stone. This is essentially the "piled-raft" concept employed on the Rion-Antirion Bridge (see Figure 9.27 and Figure 9.28) and planned for the Venice Storm Surge Barrier (see Figure 9.55).
9. To cover rock outcrops or irregular rock seafloor areas, stone fill may be placed as a blanket, compacted, and roughly leveled so that the gravity-based structure may have a uniform bearing. This procedure can also be applied to an irregular rock surface that has been purposely cleaned off by removal of soft and compressible material.

12.4 Sub-Base Construction

In some installations planned for gravity-base platforms (often called "offshore caissons"), it becomes desirable to place the structure in two halves, a so-called sub-base being placed first, followed by placement of the platform proper on top of the sub-base. This situation occurs where deployment is constrained by draft as in the Arctic.

The sub-base will typically be large in plan, but will be submerged well below the sea level. Considerations of stability during installation arise. It may be tipped down onto the seafloor as previously described; however, this develops severe stresses in the structure and may deform the seafloor. Therefore, temporary buoyancy "shafts" extending above the sea level are preferred. They permit visual control of position and orientation and give stability and draft control during installation. Once the sub-base is founded on the seafloor, the platform proper can be ballasted down onto the sub-base.

The temporary buoyancy shafts, if properly located, can serve as guides for the positioning of the platform. The platform can, therefore, be accurately positioned as it is ballasted down onto the sub-base.

To provide cushioning and equalize bearing between the sub-base and structure, a number of concepts can be employed. If this is a permanent installation, then polyurethane pads can be located at three or four spots around the periphery and underbase grout injected for permanent bearing. Another material that can be used to fill the horizontal

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joint between units is rubber asphalt; this was used on the Super CIDS platforms between the three segments of the structure. The asphalt was semiplastic when applied, but became stiff at the lower temperatures of the Arctic. Sand or sand and gravel have been proposed as a founding material on top of the sub-base; in this case it should be carefully screeded and compacted before submersion or else screeded after the sub-base is in place. If screeded before submersion, means must be found to ensure that the sand or gravel fill is not displaced by wave wash during submergence, for example, by use of an asphaltic binder.

Once the sub-base is guided down by the shafts, vertical mating cones can be engaged to result in exact mating. For shear transfer, tubular steel dowels or spuds can be entered through sleeves in both segments and fixed in place by compacted sand or grout, depending on whether the installation is temporary or permanent.

12.5 Platform Relocation

Some gravity-base platforms are designed for use as exploratory drilling structures, taking advantage of the relative ease with which bottom-founded structures may be broken loose from the soil, deballasted to towing draft, and then reinstalled at a new location. This is especially attractive for exploratory structures in the shallow to moderate depth waters of the Arctic. Both the CIDS and the SSDC structures have been relocated several times.

For relocation, consumables and operating supplies should be removed. Suction on the base adds to skin friction on the skirts to restrain flotation. Injection of water at moderate pressure over a period of many hours will break the suction. Then deballasting at one end, then the other, will usually overcome the skin friction. The total deballasting must not exceed that required to float with only a few meters of bottom clearance. Otherwise, the structure may suddenly rise to above the draft required for stability.

The process needs to be thoroughly engineered and monitored to ensure that the deballasting does not cause excessive pressure on internal cell walls and that full control of the structure is maintained during all stages.

"Permanent" offshore platforms are required to be removed at the end of their useful life. A detailed description of removal of gravity-base platforms is given in Chapter 17.

12.6 Hybrid Concrete-Steel Platforms

Several offshore platforms have been constructed with a concrete gravity base, set independently, followed by the installation of a conventional steel jacket. In other cases, the steel jacket is erected on the concrete caisson base.

As previously noted, once the base structure passes below water, its stability is controlled by the distance by which the center of buoyancy is above the center of gravity, the so-called "submarine" stability, less the free-surface effect of ballast water in the compartments. Hence, to the extent practicable, compartments should either be pressed full or completely empty.

Some additional stability can be gained by support from a floating vessel above, in which case $\overline{GM}$ can be increased by $\overline{Pl}$ where P is the lift, and l the unrestrained length of the line. Buoys may also be attached.

However, in the actual sea environment, the lifting vessel, typically a derrick barge or drill rig, rises and falls in heave, augmented by pitch, while the large base is essentially fixed in position, unable to respond because of the huge added mass of water above, plus inertial

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effects. There is, of course, some flexibility in the boom and stretch in the wire rope. Lifting over the side will permit additional softening due to heel. However, these are generally inadequate when large base caissons are being emplaced. Hence, some form of heave compensator has to be employed. In practice, stability should be achieved from the basic relationship of center of buoyancy to center of gravity.

Free-surface can be controlled by subdividing the interior of the base so that it still has a small positive buoyancy when a number of compartments are full, the rest empty. Then small control tanks can be used to ballast to the slight negative buoyancy needed for sinking. These small compartments have minimal free-surface effects.

The requirement for some large compartments to be completely empty while others are completely full obviously places high structural demands on the intervening bulkheads. To minimize the additional structural weight, consideration should be given to tubular steel struts. Air pressurization is, of course, another means but must be planned and controlled to take care of the complexity and practical problems associated with the changes in hydrostatic pressure during descent.

As the base submerges below water, there will be hydrodynamic effects from the waves. The so-called "beach" effect will give a lift force, which may necessitate adding additional ballast in the control tanks. Then once the deck of the base submerges further, below water, there will be a tendency to plunge.

Once the base is founded and fully ballasted, the steel jacket or tower is installed. Typically, four legs will be inserted into mating sleeves. Less risk is involved if the dowels are preattached to the base and the sleeves are in the jacket legs; then there is no risk of the legs punching through the roof of the caisson.

The Harding Gravity-Base Tank was placed by the use of three strings of multiple buoys. Then, the steel jacket was placed on top.

Obviously, sophisticated positioning devices and instrumentation will be used to control the final jointing. ROVs with video can be employed. Separation distance can be monitored by short distance sonic transducers. Orientation can be shown by gyro compass.

Mechanical guides, such as conical funnels, should be used for final centering. Consideration can be given to making one dowel and one sleeve combination longer than the others, to engage early. The contact should be cushioned. Hydraulic jacks can be extended prior to contact to control the impact forces.

The second case, in which the jacket or tower is already mounted on the base, presents fewer problems in installation. However, the basic stability problems remain, since the jacket contributes little to the water plane stability. The jacket may allow control of the descent, provided its legs are large enough in diameter to have an effect. However, it will probably be necessary to also employ one or two derrick barges for control.

Another form of hybrid steel concrete has been developed in which structural concrete is placed between two shells of steels with which it works in composite action. Transfer of horizontal shear is provided by studs or ribs welded on the steel plates. Provision for through-thickness shear is provided by overlapping headed bars.

The composite element embodies the stiffness and compressive strength of the concrete with the membrane tightness, tensile strength, and membrane shear strength of the steel shell. Extensive tests in England, Japan, and the U.S. have confirmed its applicability to resist the impact of Arctic ice and of boats and barges, and its favorable performance in deep water. Fabrication methods for the double steel shell with shear connectors have been developed in England, and these shells are now commercially available.

The CIDS Arctic Offshore Platform was constructed in a shipyard in the southern end of Honshu. A heavy steel barge was fabricated to serve as the base. This was in order to reduce draft for the deployment into the Beaufort Sea. Then, a prestressed concrete main body was to be attached, on top of which another steel barge was to be mounted as a topside.

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The three barges were fabricated independently. Then, the base barge was ballasted down to the seafloor under the control of two heavy crane barges. The prestressed concrete mud section was floated over and the base raft deballasted and pulled up underneath.

To ensure full contact, a rubber bitumastic layer was placed on the top of the steel base. This was designed to be plastic in the warm waters of southern Japan but rigid in the Arctic waters at the site.

A fourth form of hybrid, for use in areas where a construction basin cannot be constructed due to shallow water or environmental constructions, is to first launch a steel barge whose side walls are extended upward to serve as a cofferdam for the initial concrete construction.

The SSDC was a converted tanker hull of typical steel ship construction. Using studs and welded shear transfer lugs, a composite concrete hull was constructed. Similarly, the interior of the hull was made composite with concrete infill. Steel struts supplemented the ice load transfer from the sides to the hull bottom.

Till my soul is full of longing
For the secret of the sea,
And the heart of the great ocean,
Sends a thrilling pulse through me.
"Wouldst thou," so the helsman answered,
"Learn the secrets of the sea?
Only those who brave its dangers
Comprehend its mystery."

Henry Wadsworth Longfellow, "The Secret of the Sea"

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13

Permanently Floating Structures

13.1 General

These are structures that are intended to remain floating during their service life. They may be moored. They may be relocated during service, or they may be self-propelled.

This is a concept of great potential for the future as coastal land becomes increasingly unavailable, forcing us to utilize ocean space, despite the difficulties imposed by the environment.

Since many of the structures are oil storage vessels, which float with varying freeboard, even when partial ballasting is carried out, their mooring must give thorough consideration to transverse wind forces. This is especially true of steel vessels because of their inherent small draft when empty. Concrete vessels, on the other hand, have greater inherent draft but more limited capacity, especially in shallow water.

Typically, these permanently moored vessels employ single-point moorings, but in some areas of shallow water, fixed moorings are used. Storage vessels for oil are usually equipped with both a ballasting system and a transfer system.

Although the classification societies require dry docking for bottom inspection every year, they will usually waive that requirement for five years if an annual underwater inspection reveals no defects, especially for concrete vessels. This means that cleaning of the hull also has to be carried out, by high-pressure water jet, supplemented by wire brush.

The great majority of these oil storage vessels are steel ships whose construction are in accordance with International Rules of IMPCO and Ship Classification Societies such as ABS, DNV, Bureau Veritas, and Lloyds. Many of these steel vessels now are required to have double hulls. The design, construction, and maintenance is well covered by the rules of the classification agencies, so will not be further covered here.

Concrete floating structures have a history of equal longevity, the first use of reinforced concrete being a small boat. However, they have had only sporadic development since, due to their inherent weight. However, during World Wars I and II, a number of small reinforced concrete tankers and oil storage barges were built, due to the shortage of steel.

Floating bridges of concrete, floating piers, ferry slip docks, floating guide walls for navigation locks, and large floating storage and production vessels have been constructed in recent years and, utilizing the improved technologies of prestressing and high-performance lightweight concrete, appear increasingly attractive. Prestressed concrete is proving its value by the successful performance of such structures as the ARCO Floating Production Storage and Offloading (FPSO) for LPG, in service in the Java Sea since 1975, the N'Kossa FPSO, moored off West Africa; the floating breakwater and pier in Monaco; the Heidron TLP (Tension Leg Platform) and the Troll West Semi-Submersible, both in the

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North Sea. While all of the above are permanently moored, several in-depth studies have shown concrete's viability for self-powered vessels. See Figure 13.1 through Figure 13.4.

There are many potential uses for permanently floating structures of prestressed concrete, including floating FPSOs (spars, TLPs and semisubmersibles, and ship-shape hulls), floating Liquefied Natural Gas (LNG) production and gasification terminals, floating heliports and airports and ice-resistant vessels. Permanently floating bridge piers and marine terminals, anchored by driven piles have been employed (e.g., The Tappan Zee Bridge across the Hudson River in New York.)

The properties of significance for these uses are the ability of prestressed and reinforced concrete to resist high local import forces, durability, resistance to fatigue, fire resistance, and overall safety. Unit weight, the density of the concrete, is a generally limiting property that affects draft and inertia, the latter requiring greater power to achieve the same speed. In some structures, the greater draft is an asset, reducing the requirement for ballasting.

The bold concept by the U.S. Navy of a Mobile Offshore Basing System has been temporarily shelved. However, a great deal has been learnt about the almost insurmountable problems of mating two massive floating structures in the open sea, each reacting to different phases of the six degrees of freedom. The most difficult appears to be differential

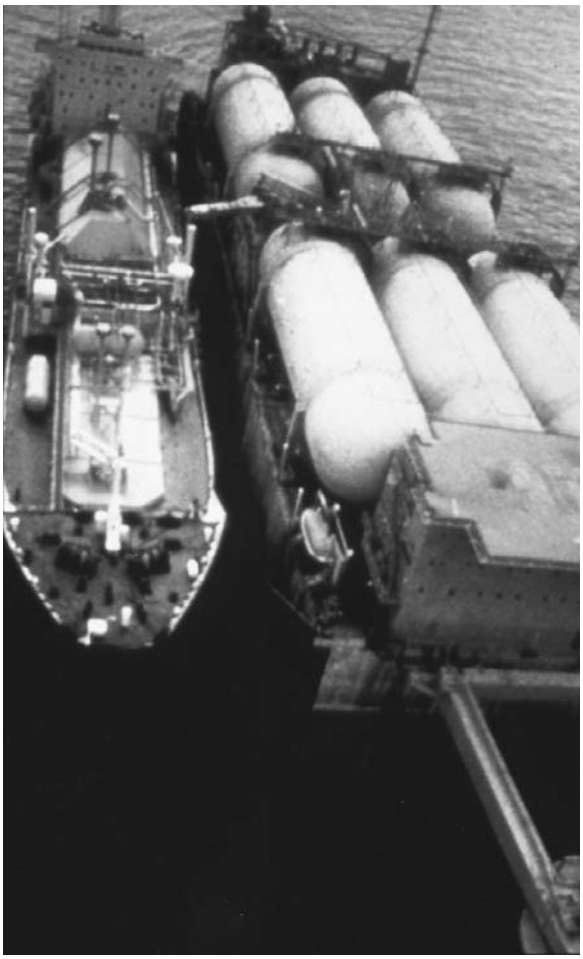

FIGURE 13.1
Sakti Ardjuna floating FPSO vessel for refrigeration and storage of Liquefied Petroleum Gas (LPG). Shown on station in Java Sea, Indonesia. Prestressed concrete hull. (Courtesy of Berger-ABAM.)

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FIGURE 13.2
Mating of steel hulls for floating offshore base. Test of prototype.

heave combined with pitch. Currently, the navy is testing a floating modular pier of prestressed concrete for the mooring of combatant ships.

To attain the desirable qualities listed above and the best performance of concrete structures, prestressing is essential, as is well-distributed reinforcement. Concrete strength and quality must be as high as possible in order to reduce draft and to obtain the impermeability essential to long-term durability. Because of their intended long-term service in a dynamic environment, the levels of prestressing and the quantity of passive reinforcement must be adequate to ensure that after cracking of concrete, the steel is

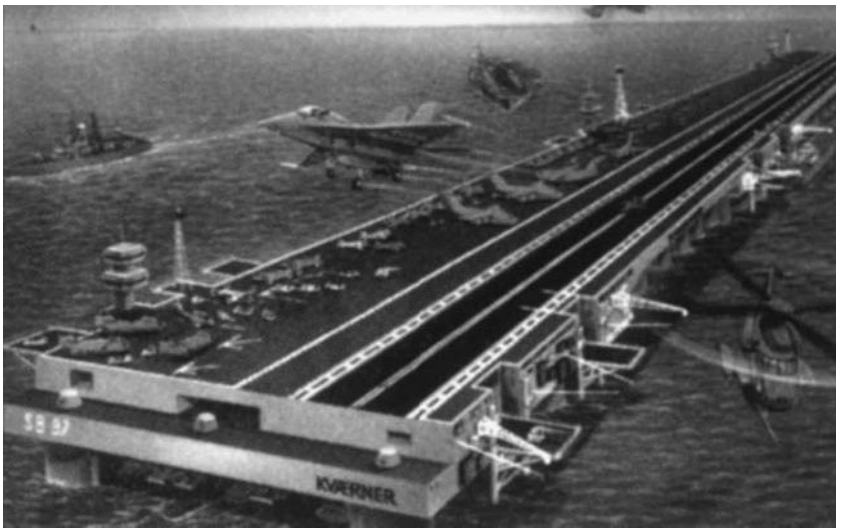

FIGURE 13.3
Conceptual design of mobile offshore basing system (MOBS).

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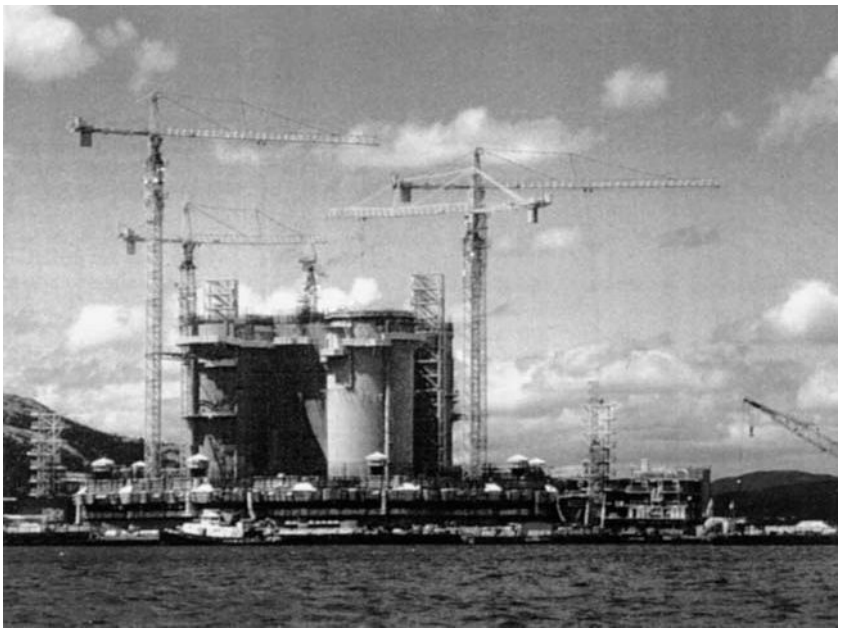

FIGURE 13.4
Heidrun concrete semisubmersible being fabricated while afloat in Bergen Fjord. (Courtesy of Norwegian Contractor.)

below yield. The sections are typically thin as compared with offshore concrete platforms. Therefore, tolerances become of increased importance, both for the concrete sections and for the placement of reinforcing steel and prestressing ducts.

Large floating structures are usually fabricated in a construction basin or graving dock but some have been fabricated on ways. Smaller vessels may be fabricated on a large barge. In all cases, draft at launching may well be critical, so very accurate control of as-built weight and external dimensions must be maintained. Weight is affected by the concrete density, and the weight of reinforcing steel actually placed, including splices.

Hull forms for concrete vessels are typically rectangular with a flared bow and tapered stern, since the towing and service wave responses are primarily inertial. Being deep in draft, due to their mass, they typically respond less to waves and swells than their steel counterparts. For TLPs and semisubmersible hulls, spars, Ocean Thermal Energy Conversion (OTEC), and other deep draft structures, the cylindrical or oval cross sections are much more efficient, due to the ability to use the entire cross section in compression to resist hydrostatic loads. Concrete, being readily molded to any desired shape, is also excellent for such complex shapes as the elbow of TLPs and the torus. This suggests that bow and stern segments can be separately precast and joined by cast-in-place concrete with the hull. By keeping the prestress concentric, it is technically feasible to prestress curved sections, but shear reinforcement is required to resist radial shear on curves. Use of steel fibers in the concrete or ultra high strength concrete is beneficial.

Both standard-weight concrete and lightweight concrete are employed. The lightweight concrete, while saving 25% or more in unit weight and hence draft, may require additional reinforcement in both in-plane and out-of-plane shear. However, where the plate thickness is determined by cover and the need to properly space the inner and outer reinforcements, as well as to accommodate ducts for prestressing, the lighter unit weight usually results in a more efficient and practicable vessel. Lightweight concrete further possesses better

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thermal properties and, when combined with concrete containing silica fume, increases fatigue endurance. It behaves especially well at cryogenic temperatures.

13.2 Fabrication of Concrete Floating Structures

Maximum utilization should be made of prefabrication using precast slabs and shells. Typically, joints are cast-in-place, to ensure full continuity of reinforcing steel and to permit splicing of ducts. Precasting permits the attainment of dimensional accuracy, while dispersing construction activities and increasing production (see Figure 13.5 and Figure 13.6). Segmental construction methods, similar to bridges, can be utilized.

A base slab is laid or cast on a carefully screeded crushed rock underbase, to permit later flooding to penetrate entirely under the rock. This also permits attachment of any fittings that must protrude below the hull proper. In the case of a cast-in-place bottom slab, the base is covered with polyethylene or similar. Drainpipe is placed in the rock underbase.

Base slabs have also been successfully cast on a finished concrete slab, which has been coated with bond breaker. The base slab is typically haunched upward, 100–150 mm or so at the wall panel intersections, so that the eccentricity of the prestressing tendons will eliminate the negative moment tension cracking. Haunches also provide increased shear resistance.

Interior wall segments are usually precast concrete slabs, tilted up using tilt-up building technology, and joined, usually at the juncture of four wall slabs. While this is a very congested area, it may be enlarged by 45° fillets. The external side walls should be haunched over the frames.

What appears to be the optimum system for joining the precast wall slab to the base is to leave a space in the base slabs and design the joint in the base slabs at the same location as the joint with the wall. The wall slab now is constructed with its reinforcing steel projecting into the joint, as are the bars from the two slabs. Thus the bars need to be set in exact

FIGURE 13.5
Hood canal floating replacement bridge is fabricated of precast concrete segments. These will be post-tensioned to form hull.

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FIGURE 13.6
Assembling precast concrete segments for Hood Canal Bridge.

location so that the mating bars can pass without interference. The wall slab is supported by struts at its sides which rest on the precast slabs. Prestressing ducts are spliced, and then the concrete joint is placed, integrating the haunches previously described.

Cast-in-place walls and slip forms minimize the number of joints. Both slip forms and jump forms have been used. Flowing concrete should be used, due to the thin, congested sections.

Mechanically headed reinforcing bars are especially useful in regions of high shear, since they can be placed more readily and effectively in the congested areas. Mechanical rebar splices are also not only better from the point of view of congestion but reduce weight.

The upper deck may similarly be constructed of precast slabs, resting on ledges bolted to the walls. The joints then are made over the walls.

The prestressed concrete modular floating pier now being tested at half-scale by the U.S. Navy and the designer, ABAM, uses all precast slabs of high performance lightweight concrete, for both exterior and interior of the hulls. Since the pier is designed for a hundred-year life, several types of corrosion-resistant steel reinforcement were tested, including stainless steel, fusion-bonded epoxy-coated and corrosion-resistant steel MMFX-2, with minimum concrete cover. Based on these tests and a consideration of costs, MMFX-2 was selected for the test module. Cover was 44 mm.

Since the post-tensioning profile was straight, plastic ducts, with plastic anchorage encapsulation, were selected for the post-tensioning tendons. Self-consolidating high performance lightweight concrete was employed, using lightweight coarse aggregate and natural sand. The unit weight was 1954 Kg/m^3 (122 pcf) and the design strength was 48 MPa at fifty-six days. Haunched precast concrete panels were joined by cast-in-place fine concrete, a scheme initially employed on the Hay Point Terminals in Queensland in 1975.

In many cases, prefabrication is used only for a portion of the hull (Figure 13.7). For example, on the N'Kossa FPSO the interior walls were precast but all exterior walls cast in

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FIGURE 13.7
The curved bottom of the Sakti Ardjuna floating vessel for LPG is formed by match-cast segments of precast concrete. (Courtesy of Berger-ABAM.)

place. Precasting is especially useful with double-curved portions of the hull, such as those for the bow. Slip forms have been used for cast-in-place walls. Stay-in-place forms have been used for horizontal members such as the deck.

Construction joints should be properly prepared by water jet blasting to expose the coarse aggregate. Typically, this will cut back the joint surface about 6 mm. Then, just before the concrete is placed, bonding epoxy may be sprayed on the joint surface. It must still be wet as the concrete covers it. Some experienced fabricators prefer to brush on a heavy coat of latex mortar.

Decks should be sloped to drain so that saltwater spray does not pond. Decks should be treated with silane at the time of construction and at two- to five-year intervals thereafter. In vessels scheduled for service in cold regions, air entrainment should be used in the concrete to prevent freeze-thaw attack.

Thermal and setting shrinkage may produce cracks, especially at construction joints. Proper concrete mix and curing procedures can minimize these. Epoxy coating of the joint, just in advance of concreting, and early partial prestressing will minimize cracking.

Anchorage zones need to be well reinforced to distribute and confine the transverse, longitudinal, and radial strains that occur. Although this is a matter of detailed design, these details need to be determined in cooperation with the contractor or the prestressing subcontractor. Further, since many of these large floating structures are constructed on a design-build basis, the constructor is vitally involved. The highest stresses around the anchorage typically occur at the time of prestressing, not under service or extreme loads, and it is then when cracking occurs if proper details are not employed.

Post-tensioning ducts should be of plastic, fused at their splice joints, or galvanized steel with heat shrink joints. Plastic ducts should not be used where the tendons are sharply curved, since the strands may cut deep grooves in them. In any event, the sharp curves should be watertight steel tubes, pre-curved to the proper profile. Grout should contain an antiwashout and thixotropic admixture to prevent the formation of bleed pockets. Plastic caps should be used to seal over the anchorages. Prestressing anchorages should be

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protected by being located in a pocket, into which reinforcing bars protrude from the surrounding concrete structure. This is filled with concrete after all prestressing activities are finished. The joint surfaces should be coated with bonding epoxy just before placement of the concrete. In regions subjected to prolonged freezing, latex concrete should be used instead and the epoxy coating eliminated.

13.3 Concrete Properties of Special Importance to Floating Structures

Although somewhat repetitive of the prior presentations, such as Chapter 4, a discussion of those properties of concrete of principal concern to floating structures is presented in more detail in this section.

1. *Fatigue*. The vessels are exposed to a large number of cycles of loading from waves through their service life. Being prestressed prevents the hog and sag moments from exceeding more than half the concrete tensile strength, even in cases of severe storm. Under such high-cycle low-amplitude cycles, there appears to be no endurance limit. If the stress were to be allowed to exceed the full tensile strength of the concrete so that cracks develop, then water would be sucked in and create a hydraulic ram effect as the crack closes under high compression. Provided there is enough total steel, both mild and prestressed, across the crack so that the steel stays below yield stress at cracking, then many tests demonstrate that fatigue endurance is still highly satisfactory.

   Prestressed lightweight concrete has been shown in tests to have both good insulating and excellent fatigue resistance even when intentionally cracked. For the concrete ships of WWI and II, which had simple passive reinforcement although no prestressing, no fatigue failures have ever been reported. The fully satisfactory endurance of prestressed concrete has been demonstrated by the shafts of the offshore platforms in the North Sea, several of which have been in service for over thirty years. Research has shown that the combination of lightweight coarse aggregate and microsilica is especially resistant to fatigue.
2. *Fire Resistance* of concrete is dependent on the cover thickness over the steel and the insulating and spalling characteristics of the concrete.

   In an intense fire, spalling of the cover could occur in the very impermeable concrete expected in seagoing vessels. The phenomenon is caused by steam generation in the pores of the concrete. To relieve the excessive pore pressures, polypropylene fibers are incorporated. In a fire, these melt, leaving a tiny void for the expulsion of steam. This could be considered for interior bulkheads in especially vulnerable areas such as the engine room.
3. *Durability*. The concrete envisioned for seagoing structures has been developed and improved over the last fifteen years by the addition of various admixtures to the mix. The principle admixtures now available and utilized on important structures such as the Channel Tunnel, offshore concrete platforms, and floating structures in the seawater environment are:
   a. High-range water reducing admixture—to reduce the water required for workability during construction and thus enhance the strength and impermeability.
   b. Pozzolanic replacements such as fly ash and blast furnace slag, to increase the sulfate resistance, the resistance to alkali-aggregate reactivity, and enhance the impermeability. It replaces an approximately equal amount of cement.

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c. Air entrainment—to prevent freeze-thaw attack.
d. Corrosion inhibitors, such as DCI.
e. Viscosity admixtures such as anti-washout admixture (AWA) or
f. Microsilica (silica fume) to give higher strengths and greater impermeability, as well as viscosity and fatigue resistance.
g. Drainage of decks—to prevent ponding of spray.

The proper use of the above admixtures, together with small size aggregate (10–16 mm max) will produce "flowing concrete" of high compressive and tensile strength and excellent durability.

In the early days of offshore oil storage, considerable concern was expressed over sulphate attack as a result of the anaerobic bacteria in the crude oil. However, this has turned out to be of low probability for the high quality concretes used in modern sea structures, especially when fly ash and/or silica fume has been included in the mix.

- Corrosion of the reinforcing and prestressing steel is prevented by an adequate concrete cover and the impermeability of the concrete. A corrosion-inhibitor such as calcium nitrite will delay the onset of corrosion as will the use of corrosion-resistant steel reinforcement such as MMFX-2. Stainless steel reinforcement is expensive but non-corrosive.
- Leak tightness, both for seawater in leakage and stored hydrocarbon leakage outward, is provided by the impermeability of the concrete and adequate reinforcement and prestressing to ensure that any cracks that do form are closed tightly after the high load has passed.
- Certain refined petroleum products require coatings or liners of the concrete tank surfaces.
- Impact resistance. Concrete slabs or shells, reinforced with closely spaced reinforcement and prestressing tendons, are very ductile and can thus absorb a large amount of energy. If more is required in certain locations, steel fibers can be added or three-dimensional mesh. T-headed bars are effective.
- Abrasion resistance, especially around mooring attachments, and on decks.

13.4 Construction and Launching

The most common method of construction of large and heavy prefabricated structures is by construction in a basin, excavated on the bank of a harbor or river. In effect, this construction basin is a temporary dry dock. Actual fabrication takes place in the bottom of the basin. Such a basin was employed in which to build and launch the Arco Sakti Ardjuna. This vessel was towed from Takoma, Washington, to the Jara Sea where it has served as an LPG terminal for over thirty years (Figure 13.8).

Alternatively, a structural shipyard dry dock may be leased. This was the case with the N'Kossa barge, the largest concrete vessel yet constructed.

For structures built of concrete, the base of the structure is separated from the floor of the basin by such means as a layer of pervious material (e.g., sand), covered by a layer of plywood and polyethylene plastic. When it comes time to launch, the dock is flooded and water injected in the pervious underlayer. By waiting a few hours with pressure on, the

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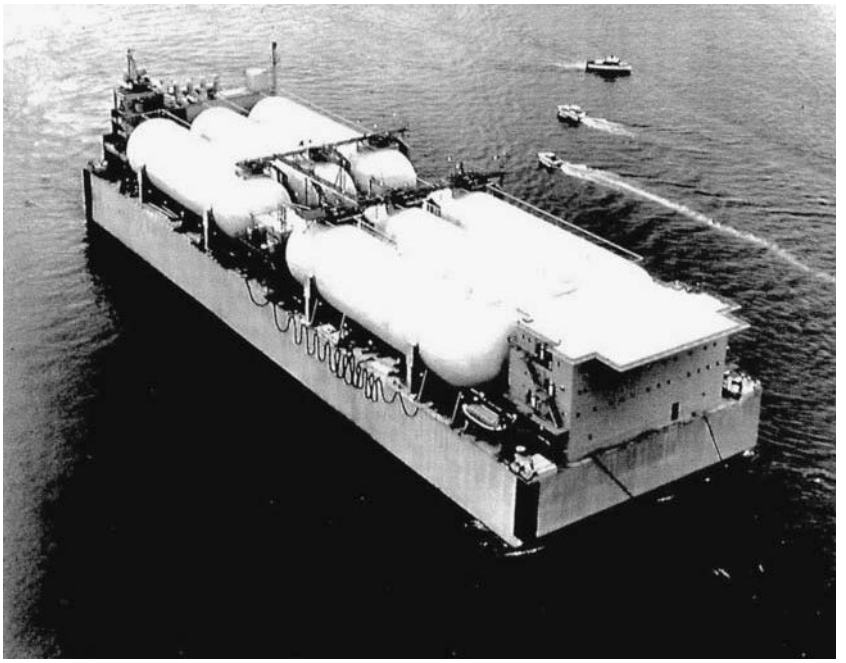

FIGURE 13.8
Arco "Sakti Ardjuna" enroute to Java Sea. (Courtesy of Berger-ABAM.)

vessel will float up. Some of the plywood and polyethylene may stick to the bottom and have to be removed by diver.

Two basins, side by side, one deep, one shallow, may be beneficial, especially where multiple elements are to be built. Then, construction of the lower hull may take place in the shallow basin, floating it sidewise into the deeper basin for completion and final launching, while construction of another vessel is started in the shallow basin.

Exit into the river or harbor must be controlled by tugs and shore lines, especially where there is a wind or current in the channel.

If the structure is constructed at grade level, transfer of the heavy prefabricated vessel, up to 20,000 tn. or more, is carried out by skidding on finely leveled and firmly supported grade beams, using Teflon bearing pads and horizontal jacks. On other projects such as the Øresund Bridge, Hillman Rollers were employed. Steel jackets for offshore platforms have been slid on heavy timber or steel beams using Teflon-stainless launch pads. Coefficients of friction as low as 0.03 have been experienced. The huge 100 m long prefabricated concrete girders for the Great Belt Bridge were slid on concrete beams, using grease. Regular ratchet recesses in the beams allowed jacks to grip and move the girders progressively. For the Confederation Bridge in eastern Canada, low profile rubber-tired carts lifted the 8000 tn. units by hydraulic jacks and rolled them forward on concrete rails.

Use of gaskets on the sides and injection underneath large flat-bottom steel or concrete vessels with pressurized water reduces friction to near zero, allowing easy re-positioning.

Side-launching is a second method used to launch barges and similar vessels. Side-launching reduces the bending moment in the vessel during launching. It also distributes the concentrated bearing loads which travel down the ways as the vessel is launched.

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A concrete barge for a floating phosphate plant was built at grade, on the edge of a harbor, and then slid sidewise, jacked up to an angle, and slid downways into the water. As the barge left the ways due to buoyancy, it temporarily imposed a concentrated load under its upper edge, while causing a transverse sag bending moment in the barge. Both were properly designed to resist this loading.

It is planned that the prefabricated navigation dam units for the Olmsted Dam on the Ohio River will be constructed at grade and slid sidewise on pile-supported beams onto a cradle. The cradle will then be slid down the ways, allowing the dam unit (3500 T) to be floated off at any stage of the river. This lift will be partly assisted by lift from a catamaran crane barge which will support it during transport and later lower it into position.

A vessel or prefabricated element constructed at grade level may also be moved onto a ship lift, such as a Syncrolift, and lowered by hoists into the water. If a permanent installation is not available, it can be constructed as an overhead gallows frame and multiple hoists. The element being lowered must be structurally able to be lifted from the ends, or else a structural barge or frame must be placed under the element. Lifting by a high capacity gantry crane, which runs on trestles over the water, is an alternative scheme being considered for the Olmsted Dam segments.

A vessel or segment can be slid onto a floating dry dock for launching or onto a launch barge, that is, a barge capable of deep submergence, which can stand the external head and still structurally support the structure or vessel. To allow it to submerge deeply enough without losing stability, two or more columns are erected at the corners or one end, thus giving it water-plane stability and residual buoyancy.

Of course, if only one element or vessel is involved, it may be built initially on the launch barge. Float-off from a launch barge is normally attained at an even hull. However, the float-off may be assisted by tipping and sliding off the end of the barge. Then the launching is carried out in known depth of water, so that the end of the barge grounds at about 30° angle, where the lower end of the element begins to lift off. As the load rotates, a tug pulls the launch barge forward, allowing the load to float off.

Many other ingenious methods have been proposed, taking advantage of tides or ice. Sand jacking to lower the vessel through sand has been proposed. Structural integrity must be assured at all stages.

Mooring fittings, such as posts and bollards, should be post tensioned to a thickened and heavily reinforced seat in the hull. Usually, prestressing bars will be used instead of strands, since seating losses of prestress can be eliminated by the use of bars.

Fairleads and/or hawse pipes are either post tensioned on or embedded in the concrete with proper anchorage. Steel plating should be placed alongside to eliminate the wear from the rubbing of anchor chains. Piping and mechanical systems should, as far as is practicable, be integrated into the hull design, with embedments preinstalled. Corrosion protection should be provided with sacrificial anodes, especially at seawater intakes and discharges. There are many other design and construction details applicable to concrete vessels. Reference is made to *Gerwick Construction of Prestressed Concrete,* 2nd edition (Gerwick 1996).

A very interesting hull design has been developed as a result of testing at both the University of Manchester, U.K., the University of California at Berkeley and in Japanese construction laboratories. It is a double-steel shell, made to work compositely with concrete infill. Thus, it can be designed for shell action to resist high hydrostatic pressures and, at the same time, benefit from the ductile behavior of the steel. Additional testing of the concept for ice-resisting structures has been carried on in Japan. As noted in Section 4.3.2, a prefabricated shell using shear stud connectors is commercially available. Among the potential applications are submerged tunnels (tubes) and suction anchors.

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Constructibility of large vessels requires consideration of access and crane reach, since work will need to be pursued concurrently at many locations. Construction at grade is thus more efficient and hence less costly than construction in a basin.

13.5 Floating Concrete Bridges

Floating bridges go back to their origins in about 1500 B.C. It was the floating bridge that Darius built in 400 B.C. across the Bosporus that failed due to dynamic excitation by storm waves, yet engineers have not heeded its lesson. The Hood Canal Bridge and later, the I-40 Lacey P. Morrow Bridge, both sank under dynamic wave action and the floating bridges at Hobart, Tasmania, and Evergreen Point, Washington, were both damaged by storms and had to have extensive retrofit or replacement.

Most concrete bridges have been designed and constructed as a series of long, rectangular barges, with interior egg-crate bulkheads, both longitudinally and transversely. The early ones were conventionally reinforced but later floating bridges have had increasing amounts of longitudinal prestress. Joints have generally been large mild steel bolts but the last two replacement bridges have been post-tensioned to about 7 MPA precompression for their full length. In the case of the Hood Canal Bridge, both original and replacement, the deck has been elevated on short columns.

Moorings have been spread moorings to large concrete block weights. The mooring lines have been led through glands in the sides just below the deck and are jacked for equalization of load. They are cathodically protected.

The dynamic excitation referred to earlier is due to the waves traveling obliquely to the axis of the bridge, thus lengthening the effective span between crests to much longer than the wave length. This in turn leads to harmonic response by the bridge. Any cracking becomes a through crack. The alternate opening and closing sucks in water, then closes, leading to hydraulic fracture. Therefore there must be sufficient steel area crossing all potential cracks to stay below yield at the ultimate tensile strength of the concrete.

Flowing concrete is especially well-suited for application to floating bridges because of the thin sections and congestion of reinforcement.

A curved floating bridge in Norway is supported on transverse pontoons of prestressed concrete.

13.6 Floating Tunnels

A submerged floating tunnel has been proposed for the crossing of the deep Lysefjord, also in Norway, near Stavanger. It would combine the technology of immersed tunnels (tubes), as presented in Section 9.5, with that of floating bridges and offshore platforms.

The concept is to design and construct a concrete tunnel of cylindrical external cross-section, which would be pulled down by tethers to the seafloor. The tethers were selected in the case of the Lysefjord. While dynamic interaction with currents remains the principal design consideration, assembly of segments and deployment are the dominant construction problems.

Because the walls will necessarily be thick, heat of hydration must be minimized. Joints between segments will probably follow the pattern developed for immersed tubes. Assembly may be made in an adjacent bay, and the entire string towed and moored to

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the final location. Jointing and sealing at the tunnel entrances on either side must accommodate peak bending moments and shears.

13.7 Semi-Submersibles

The Troll Olje is a concrete semisubmersible installed on the Troll field offshore Norway to serve as an FPSO. It is moored by sixteen catenary mooring lines in 325 m water depth and supports a topside operating weight of 32,500 tn. and thirty-three risers and tubes. The hull consists of four rectangular pontoons connected to four columns by four corner nodes. These nodes are the most complex zones of the platform. It is here that the external fairleads of the catenary moorings are attached, thus subjecting the elements to high local loading. The columns consist of two concentric concrete columns, thus giving damage stability. The columns are designed to resist ship and beat impact and to have ballasting and pumping equipment. At the top of the columns, a steel corbel ring is post tensioned. It supports the module support frame with a moment-free pin connection. The entire construction was performed in a graving dock near Bergen.

13.8 Barges

The N'Kossa barge is a large spread-moored barge that supports an offshore processing, storage, and shipping facility. It was installed offshore the Congo in water depth of about 200 m. It is 220 m long, 46 m beam, and 16 m deep, and supports 33,000 tn. of equipment and structures. It was constructed in a graving dock in Marseilles and fully outfitted with the topside modules. Unusual was the use of reactive powder concrete to achieve a dense,

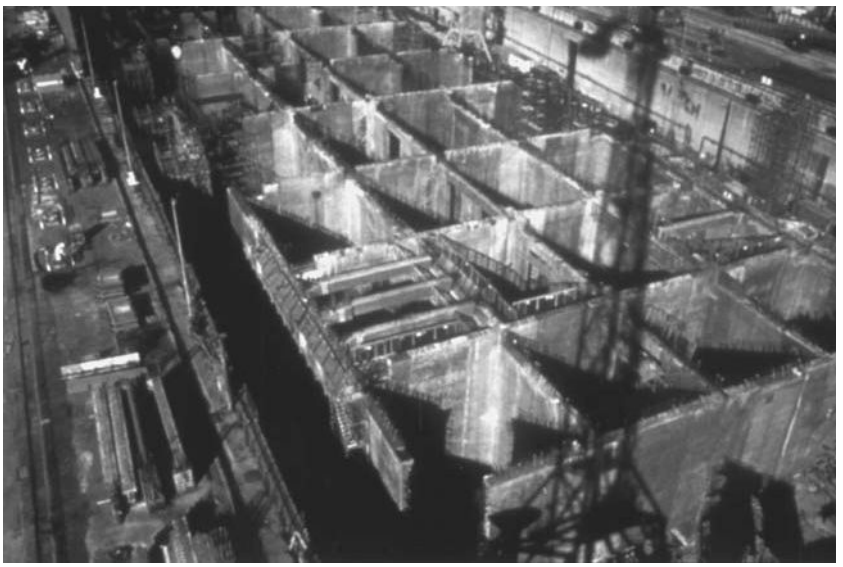

FIGURE 13.9
Assembly of precast concrete segments for inner walls of N'Kossa floating barge. (Courtesy of Buoygues Offshore.)

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FIGURE 13.10
Completing construction of N'Kossa barge. (Courtesy of Buoygues Offshore.)

FIGURE 13.11
N'Kossa barge being towed out of dry dock. (Courtesy of Buoygues Offshore.)

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FIGURE 13.12
Installing superstructure and outfitting of N'Kossa barge. (Courtesy of Buoygues Offshore.)

high strength, concrete hull. Internal bulkheads were made of precast slabs while the exterior hull was cast-in-place, using slip forms.

Concrete barges have been employed to support floating concrete batching and mixing plants, because their mass provides stability and minimum motion (Figure 13.9 through Figure 13.12).

13.9 Floating Airfields

Floating airfields, both runways and taxiways, have been proposed for a number of projects, including one in the Thames, another in San Diego, California, and one at Osaka, Japan.

When the San Francisco Airport was planning the expansion of its runways, one of the alternatives studied in detail was a floating structure. Its 50×50 m sections would be constructed in an offsite construction yard and towed to the site, where they would be joined by prestressing, both longitudinally and transversely. The sections would be designed as semisubmersibles, i.e., columns through the tidal zone. The completed structure would be held in fixed position and tied down against tidal changes by ground anchors.

Although this concept requires three horizontal slabs, the bottom and the top of the hull and the airfield deck itself, it met the important environmental requirement that the water would be free to flow through.

The U.S. Navy has intensely studied concepts for construction of its Mobile Offshore Basin System, a floating airfield runway, plus extensive repair and servicing facilities, the objective being a mobile support base for naval operations at a distance, thus minimizing the need for land bases. This would have to be able to be deployed and moored so as to survive a major storm in the open sea. The semisubmersible concept is favored because of its low response to waves and swells. Since the length of several thousand meters is beyond reasonable shipyard production capabilities, deployment has assumed fabrication

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and deployment in steel segments of less than 1000 m each, with mating to be carried out afloat.

Similarly, preliminary engineering studies were carried out in Japan for a floating airfield, both for generic application and for specific application for the new Kansai Airport in Osaka. These were based on the semisubmersible concept, with pairs of tubular columns formed into underwater Us by tubular pontoons.

13.10 Structures for Permanently Floating Service

Floating oil storage vessels of steel have been used for many years. Design and construction follows shipbuilding practice, except that, being stationary vessels, they are not faced with the necessity to reduce drag; hence in protected seas such as the Persian Gulf, they can be very simple large boxes, with vertical sides and ends.

FPSOs of steel are permanently moved in the North Sea and elsewhere. Designed to weathervane to minimize wave forces, they are molded like typical seagoing vessels. Most recently, turret moorings have been employed, to weathervane about their center of rotation and to permit oil and product lines to be swiveled in the turret.

Recently, the SPAR has re-emerged as a potentially optimum vessel for offshore oil drilling, production, storage, and offloading in the very deep ocean, subject to severe storms. These are much larger than previous spars and have drafts of 150 m or more. They are built in shipyards in the horizontal position and launched like a ship. Concrete versions of the spar have been proposed, utilizing the favorable ability of concrete to resist hydrostatic loads (see Chapter 22).

The Red Hawk Oil Drilling and Production Platform is a relatively small steel SPAR built up of seven steel cylindrical cells, each 26 ft. in diameter. Four of these cells are 280 ft. in length; three extend a full 560 ft. The upper four are open for 7 ft. at their bottom and compressed air will be injected to regulate trim and adjust ballast. To prevent vortex

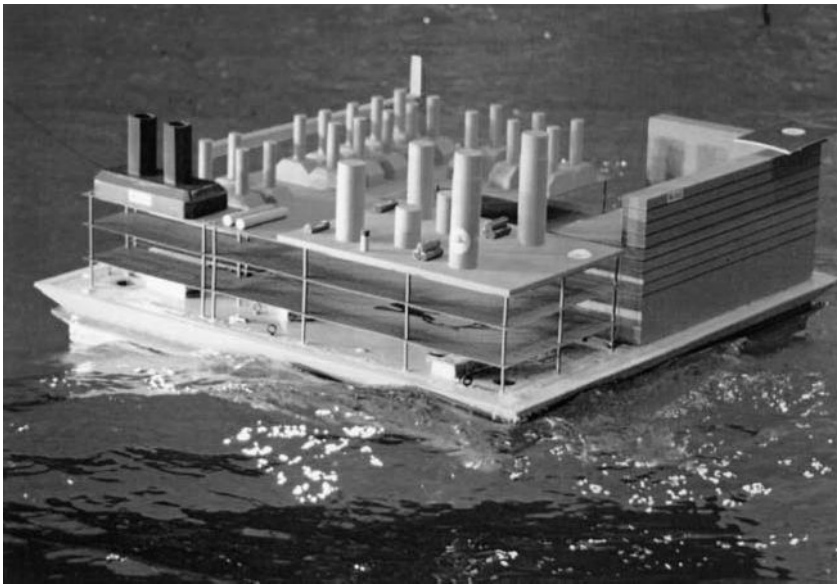

FIGURE 13.13
Floating LNG export terminal concept. (Courtesy of Mobile Technology.)

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shedding, spiral strakes are affixed to each of the cells. In addition, metal fins encircle the extended legs at sharp angles. Three sets of heave dampers are installed. Polyester lines will be used to moor the SPAR to suction anchors in 5300 ft. water depth (Figure 13.13).

13.11 Marinas

Small floating piers, both steel and concrete, have been used for many years. These include marinas for small boats, boat docks, and seaplane docks.

Marinas have utilized pontoons constructed of lightweight concrete and also composites such as fiberglass. Moorings have been single piles, one at each end, with neoprene and stainless-steel sliding surfaces. Concrete and polyester piles have also been used. Composites suffer from ultraviolet degradation, although progress has been made in incorporation of UV resistance. Lightweight concrete utilizes galvanized mesh reinforcement. Shear resistance in the thin walls may be enhanced by 3-D mesh or by inclusion of fibers.

13.12 Piers for Berthing Large Ships

Large floating piers have been constructed. The Valdez Alaska container terminal has been in service for over ten years. It was constructed in two 100 m long segments so as to reduce moments during towing in the open sea. On arrival at Valdez, it was joined together by post tensioning to form a 200 m long pier. A floating pier and breakwater was built in Monaco and joined to the shore bulkhead by a universal articulated joint. The U.S. Navy is currently testing a Modular Pier segment at San Diego (see Section 13.2).

13.13 Floating Breakwaters

Floating breakwaters of concrete pontoons have been successfully used in partially protected installations to give further protection to small boat harbors and critical shore facilities. Proposals have been made to use larger versions in the open sea but have not so far proven viable, due to the extreme motions that they would undergo in storms and the difficulty in providing long-term moorings.

Floating breakwaters are joined in segments, generally using chains. Thus, the attachment to the pontoons must be designed to prevent fatigue and abrasion. Some of those proposed are configured to divert the incoming energy to vertical jets and dissipate it, rather than reflect the waves. Others have been designed with sufficient mass to avoid resonance with the larger waves. A large floating breakwater has been constructed of prestressed concrete to protect the harbor of Monaco. It is moored by an articulated joint to a fixed abutment.

13.14 Mating Afloat

Many concrete and steel floating structures have been mated afloat in inland waters. These include concrete bridges in Washington and the floating concrete container terminal in

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Valdez. Steel vessels have been jumboized, adding a midbody while the bow and stern are afloat. In Tokyo Bay, the two halves of a test module for the Okinawa Offshore Heliport were mated by a method described similar to that described below.

The process of jointing shallow-draft floating pontoons and barges in inland water has been to pull the structure together with deck winches. Mating spuds extending on the deck of one segment engage the mating cones on the other. The barges are usually ballasted to tilt slightly down at the joint. Timber bumpers and/or rubber fenders are used to cushion the impact. As soon as the top edges are in contact, they are locked there by stressed wire rope or steel members. Then, the ballast is shifted so that the hulls rotate to press the bottom edges together.

Joints for concrete structures have rubber gaskets that are compressed to enable dewatering of the joint. Ducts are then run from holes in connecting blocks in one segment to mating holes in the other segment and prestressing tendons installed. The gap in the joint is now grouted. Full moment and shear transfer is developed by prestressing tendons top and bottom and by the grouted shear keys.

When external access to the below-water outside of the joint is needed (for example, to weld steel plates for connecting two steel barges) then a temporary box cofferdam is placed underneath and on the sides. Larger segments may employ more-sophisticated gaskets like the gaskets used in jointing submerged prefabricated tunnel segments.

When mating large structures in the open sea, the dynamic motions of the two segments become dominant. These huge masses develop very large inertial responses in all six degrees of freedom. Forces are often beyond the capacity of conventional mooring systems to control. Presumably the two segments will be joined in favorable sea and wind condition, so the large segments may be headed into the swells. The barges can be joined by mooring lines, one on each side, leading to constant-tension winches. The two segments should be "pulled apart" at the same time, either by anchor lines in the water of moderate depth or by tugs, thus keeping the joining lines under tension, yet allowing the two barges to be slowly pulled together, overcoming the differential surge. Long-stroke hydraulic jacks and commercially available dock fenders, particularly the buckling-type, could be utilized to cushion the final impact between two vessels.

Recognizing the inherent problem of joining floating structures in the open sea, constructors have adopted two solutions. The first is to join all the floating units in a calm harbor, with articulated joints, and then tow the entire string to the site offshore in the same manner described for the tension legs of tension leg platforms. Tension is maintained on the entire string of floating elements by a tug at the stern of the string. Upon arrival, the individual elements are moored, still maintaining shear capacity at their joints. If the system is inherently a flexible system, similar to that of a tubular steel pile or tension leg, then the moment connections can also be made in the calm harbor. Differential heave and associated pitch are the most difficult to control.

The second system obviates joining in shear and moment by spacing the individual elements with a gap between. The gap is crossed by an articulated steel bridge. Articulated arms and deck allow independent heave. This is the system tentatively adopted for the Mobile Offshore Basing System of the U.S. Navy (MOBS) but not pursued beyond preliminary engineering.

Where continuity of barge-shaped structures is required, long-stroke hydraulic cylinders are engaged when they come within reach. These not only can overcome the surge motions, but the yaw as well. Crossed wires can bring the two large segments together as far as sway is concerned. Next is to match the heave and roll motions. Hydraulic cylinders, acting in a near-vertical orientation, can be brought into play. Shock absorbers, up to 5000 tn. capacity and 1 m stroke, have been developed. Alternatively, heavy walled

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pipe, turned on its side, can be crushed, giving inelastic consumption of energy and momentum.

The moment transfer between large floating structures is very difficult. The differential pitch at the joint is countered by the ballasting of the two segments to rotate to closure at bottom. The moment is developed by very large post-tensioning strands, both top and bottom. Matching cones and spuds are used for final jointing. All jointing systems must have ductility so that if they are overloaded, they will yield while still maintaining a force. Use of a heavy vegetable oil in small amounts, dripped from an upwind tug, will calm the waves (but not the swells) during this jointing operation.

There is an obvious need to develop a system for effectively mating floating structures in the open sea. Not only airfields and piers are involved, but also large floating processing plants. Possible avenues to pursue might include that involved in the mating of a seagoing pusher tug to a large barge; a procedure that is currently state-of-the-art, showing that a smaller module can be mated to a large, massive floating structure in the sea.

The successful float-over installation of the topsides of offshore platforms in the open sea, by using long-stroke hydraulic jacks, shows that it theoretically is possible to develop such a system for mating two floating structures in the open sea under favorable conditions. Much more developmental work and testing remains to be done.

His heart was mailed in oak and triple brass
Who was the first to commit a frail bark to the rough seas.
He was not afraid of the swooping sou'wester
Battling it out with the winds of the north,
Nor the weeping Hyades, nor the madness of the south wind,
The supreme judge of when to raise and when to lay the Adriatic.
He did not fear the approaching step of Death,
But looked with dry eyes on monsters swimming, on ocean boiling,
And on the ill-famed Acroceravnian rocks
In vain in his wise foresight did God sever the lands of the Earth
By means of the dividing seas
If impious ships yet leap across waters
Which they should not touch
Boldly enduring everything, the human
Race rushes to forbidden sin.

Horace, 65–68 BC, Ode I:III (9–25)

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14

Other Applications of Marine and Offshore Construction Technology

14.1 General

In Chapter 11 and Chapter 12, typical offshore platforms employed in drilling and production of offshore oil and gas were used as a basis for describing the construction procedures required. In this chapter, a number of other applications and other types of offshore structures will be evaluated with regard to their special construction requirements. No attempt, however, will be made to repeat in detail the "standard" construction procedures previously described.

Section 9.3.3 and Section 9.3.4 ("Prefabricated Lock" and "Navigational Dam Structures") describe examples of the application of offshore technology to riverine projects.

The several other applications to be addressed are the following:

Single-Point Moorings
Articulated Columns
Seafloor Templates
Underwater Oil Storage Vessels
Cable Arrays, Moored Buoys, and Seafloor Deployment
Ocean Thermal Energy Conversion
Offshore Export and Import Terminals for Cryogenic Gas—LNG and LPG
Offshore Wind-Power Foundations
Wave-Power Structures
Tidal Power Stations
Barrier Walls
Breakwaters

Many of the above are employed in the development of offshore energy resources. However, many of these structures are also used by other industries and for other functions. Single-point moorings have been employed for slurry transfer of iron ore and coal, and tension-leg platforms for a variety of military installations. While concrete structures predominate, many are also constructed of steel.

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14.2 Single-Point Moorings

Single-point moorings have been developed over the last several decades. The initial concepts were designed to enable tankers to moor and offload or unload crude oil. They represented an economical solution for transfer of oil wherever sea conditions permitted a reasonable usage, for example, 65% availability. Single-point moorings have subsequently been improved and extended to enable them to handle several different petroleum products and even to load iron ore slurry into ore carriers (Figure 14.1).

From a construction point of view, they generally consist of a base and anchors, underbuoy risers or hoses, and a floating buoy moored to the base by flexible lines or an articulated strut. The buoy contains swivels, which allow a ship moored to it to weathervane with the wind, current, and waves. The ship may moor to the floating buoy or be yoked to it by a rigid or articulated yoke (see Figure 14.2). Many variants on the basic concept have been developed.

The principal construction operations include the installation of the base structure and the anchors. The connection of hoses requires a support rig with hoisting capabilities and divers. The installation of the buoy is mainly a question of positioning, followed by the more tedious job of equalizing the tension in the several anchor legs.

Bases have been usually constructed of steel and are initially buoyant. They are sunk to the seafloor by ballasting down in calm water, with descent controlled by lines from a barge working over sheaves. The elastic stretch in the lines is counted on to absorb the dynamic loads, which are principally due to wave action on the support barge. Ballast water is confined within small compartments in the base in order to avoid free-surface

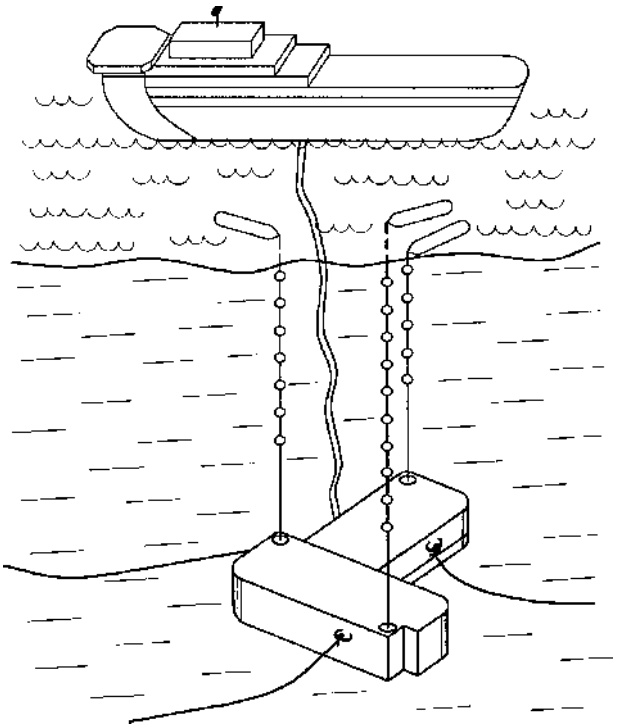

FIGURE 14.1
Harding GBS being submerged by progressive ballasting and submergence of string of buoys.

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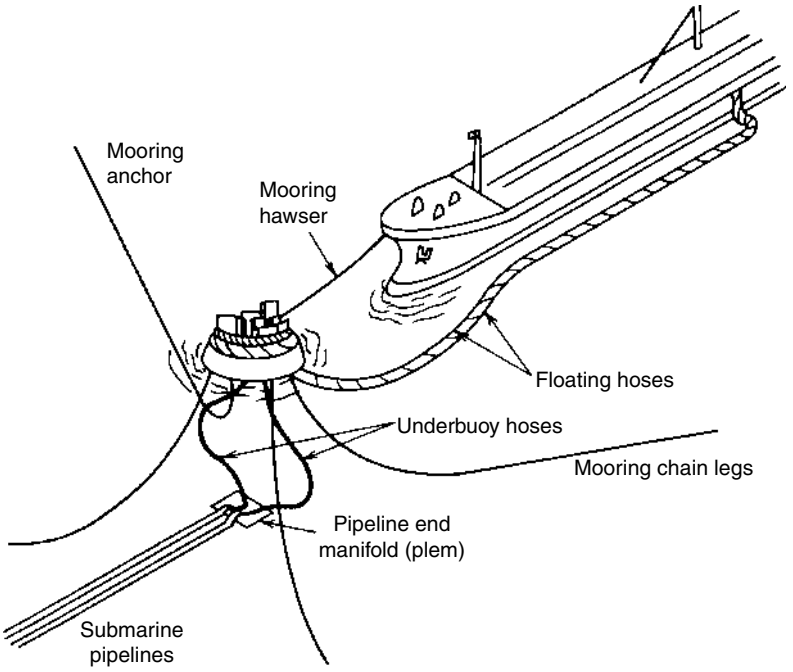

FIGURE 14.2
Typical CALM installation for single point mooring for offshore oil transfer. (Courtesy of Imodco.)

effects. Bases have also been constructed of reinforced concrete and transported to the site afloat.

Bases are usually relatively small, 10–20 m in diameter, 3–5 m in height. Even though relatively small compared to the larger structures previously discussed, their installation requires consideration of dynamic lift forces and stability.

Once the upper surface is below water, there is a large, hydrodynamic added mass in heave. The barge tends to rise with the waves while the base structure acts as a "sea anchor." Elastic stretch in the lines and deflection of the boom can accommodate limited differential wave. At the same time, the loss of water plane on the base makes it necessary to provide a substantial righting moment from the crane hook in order to maintain stability. This added righting moment $\overline{Pl}\ \sin\phi\Delta$ where l is the height from the hook to the attachment points on the base, P is the lift, ϕ the angle of bed and Δ the displacement.

Once on the seafloor, the bases are filled with slurried sand or slurried iron ore or with cement grout pumped through a hose. Grout-intruded aggregate has also been used, even though there may be practical difficulties in placing the aggregate and in the multiple grout injection points required. Grout-injected aggregate was successfully used on the Polaris Missile test pads off San Clemente Island, California, which were of similar size and depth and contained very complex inserts. Tremie concrete may also be employed. With cement-based grout or concrete, the effects of heat of hydration must be considered.

Underbase grout may be placed through a hose to fittings on the base structure, in a manner similar to that of gravity-based structures. Since the skirt length on a base structure is necessarily limited, there being very little net weight to cause penetration, the grout should be highly thixotropic so not to escape out to the open sea. Drop curtains of canvas or sandbags may be used to seal the bottom edges where steel skirts are impracticable.

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Scour protection may be required. The combination of currents plus wave-induced pore pressures can cause serious erosion. Filter fabrics covered by rock are typical, but other clever schemes have been developed, including "artificial seaweed" (closely spaced nylon ropes) hanging from a ring around the structure to slow the water movements. This is intended to cause deposition instead of scour.

The base often is fitted with a manifold on its top, with fixed curved pipes (the PLEM) leading over the edge of the base, through which hoses are connected to the pipelines from shore.

Bases may also be pile supported in soft soils. Piles are driven through sleeves in the base and connected by grout. These piles will be relatively short and lightly loaded; their main function is to prevent excessive settlement, tilting, and sliding. Anchors for the several types of single-point moorings can be large gravity blocks or driven or drilled-in piles. In several cases, in hard seafloors, holes have been drilled and heavy anchor chain run into the holes, after which they are filled with tremie concrete, using very small aggregate, for example, 8 mm, in order to permit placement through a flexible hose guided by a diver. A steel transition flare piece, like a funnel, is set at the top to prevent excessive wear on the chain at that point. Installation of a typical single-point mooring system is shown in the accompanying sequence of drawings: Figure 14.3 through Figure 14.19 inclusive.

Single-point moorings are increasingly being used to moor tankers for storage and especially for floating production systems. In the latter case, they are combined in the field development with subsea templates. When used for semipermanent moorings, the

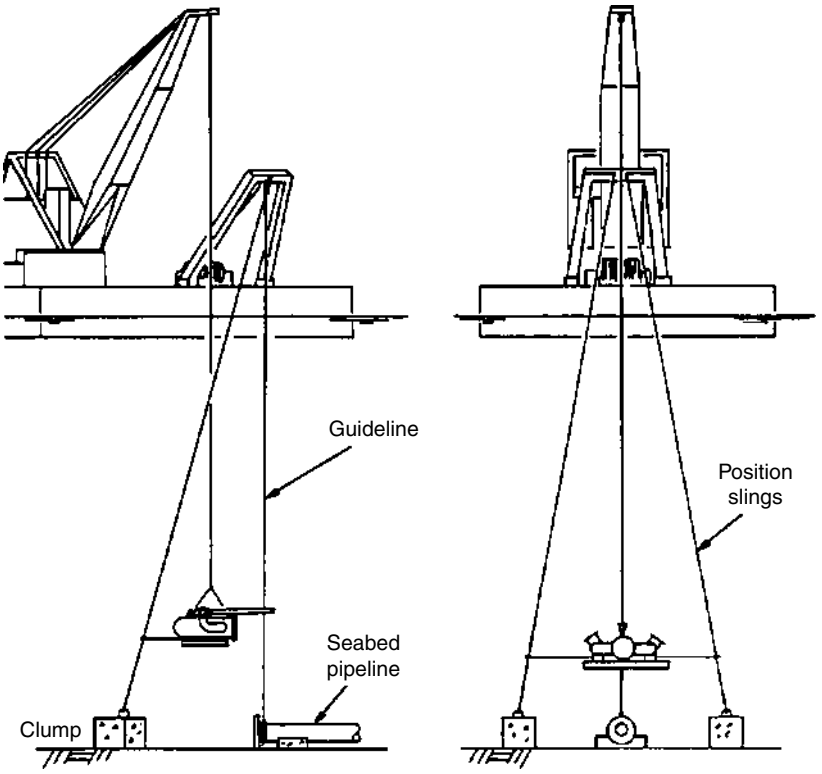

FIGURE 14.3
Installation of CALM single-point mooring for off-loading of oil, using guideline mooring system.

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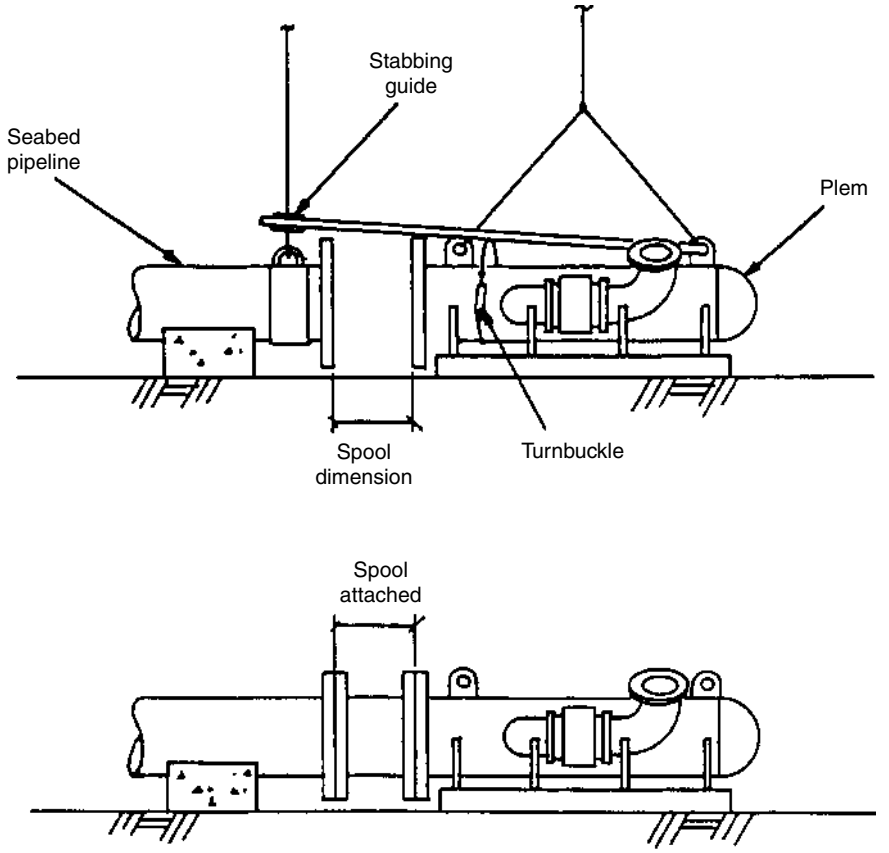

FIGURE 14.4
PLEM installation. (Courtesy of Imodco.)

systems must, of course, be made heavier and more reliable to safely hold the vessel, even during storm conditions. The basic concept is also suitable for offshore processing, storage, and transfer vessels for other commodities as well as for terminals for production and liquefaction of LNG.

Prior to the start of the tow of the single-point mooring to the installation site, an installation barge is moored at the site, using an accurate electronic navigation system. This surface position fix is then transferred to the seabed and an array of transponders is set up to permit positioning of the base and for determining its azimuth. A seabed survey by divers or ROV and a side-scan sonar survey should be carried out prior to installation.

14.3 Articulated Columns

A natural extension of the more-advanced single-point mooring systems has been the development of the articulated columns or articulated loading platforms, such as those installed in the North Sea for the offloading of crude oil. These differ from the single-point mooring (SPM) systems primarily in structural size and offloading rate.

These platforms consist of a base, a column, and a deck structure. The base is anchored to the seafloor by gravity weight or piling. The column is buoyant, and its center of buoyancy is well above its center of gravity. The base and the column are joined by an

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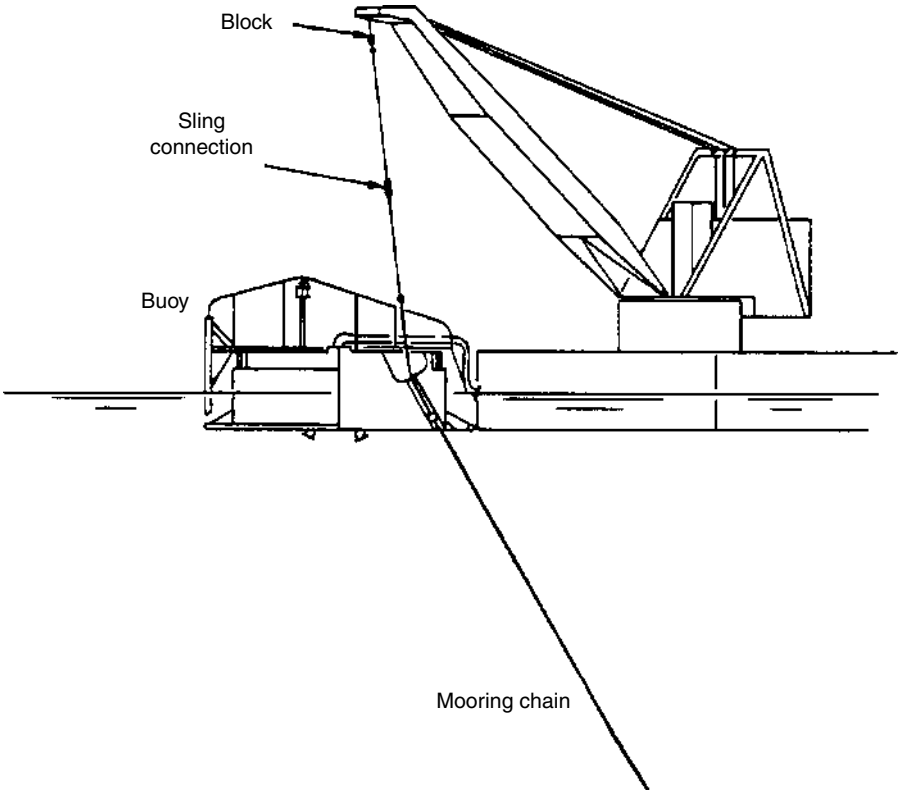

FIGURE 14.5
Mooring line connection. (Courtesy of Imodco.)

articulated hinge, called a cardan, which allows articulation on both axes but is restrained against torsion.

While most of these structures built so far have been of steel construction, some are built of both steel and concrete, employing each material in the zone for which it is best suited. For example, the structure for the Maureen Field in the North Sea had its central column constructed of prestressed concrete, with upper and lower sections of steel.

The articulation ensures that no moment is transferred to the seafloor. The reduction this makes possible in structural dimensions consequently reduces the wave force, and the lateral force tending to produce sliding. The concept has been applied to flare stacks, offloading terminals, and the deep-water mooring of vessels for floating production systems. An extension of this concept has been proposed by C.G. Doris, for example, for deep-water (500–800 m) drilling and production platforms. The installation of these systems can be quite complicated and requires a very sophisticated application of advanced hydrodynamics and construction engineering (see Figure 14.20 and Figure 14.21).

An initial construction concept was to install the base first, ballasting a steel or concrete cellular structure to slight negative buoyancy. As has been seen with other fully submerged structures, the dynamic response during lowering of a moderately large base is very significant, especially when coupled with an offshore derrick barge that is being accelerated in heave, roll, and pitch by the waves. Since these bases are much smaller than those required for support of a platform, the use of a spar buoy as a heave compensator is sometimes adequate. The lowering line will then be horizontal at the surface and be controlled by a winch on the barge or even by a tug.

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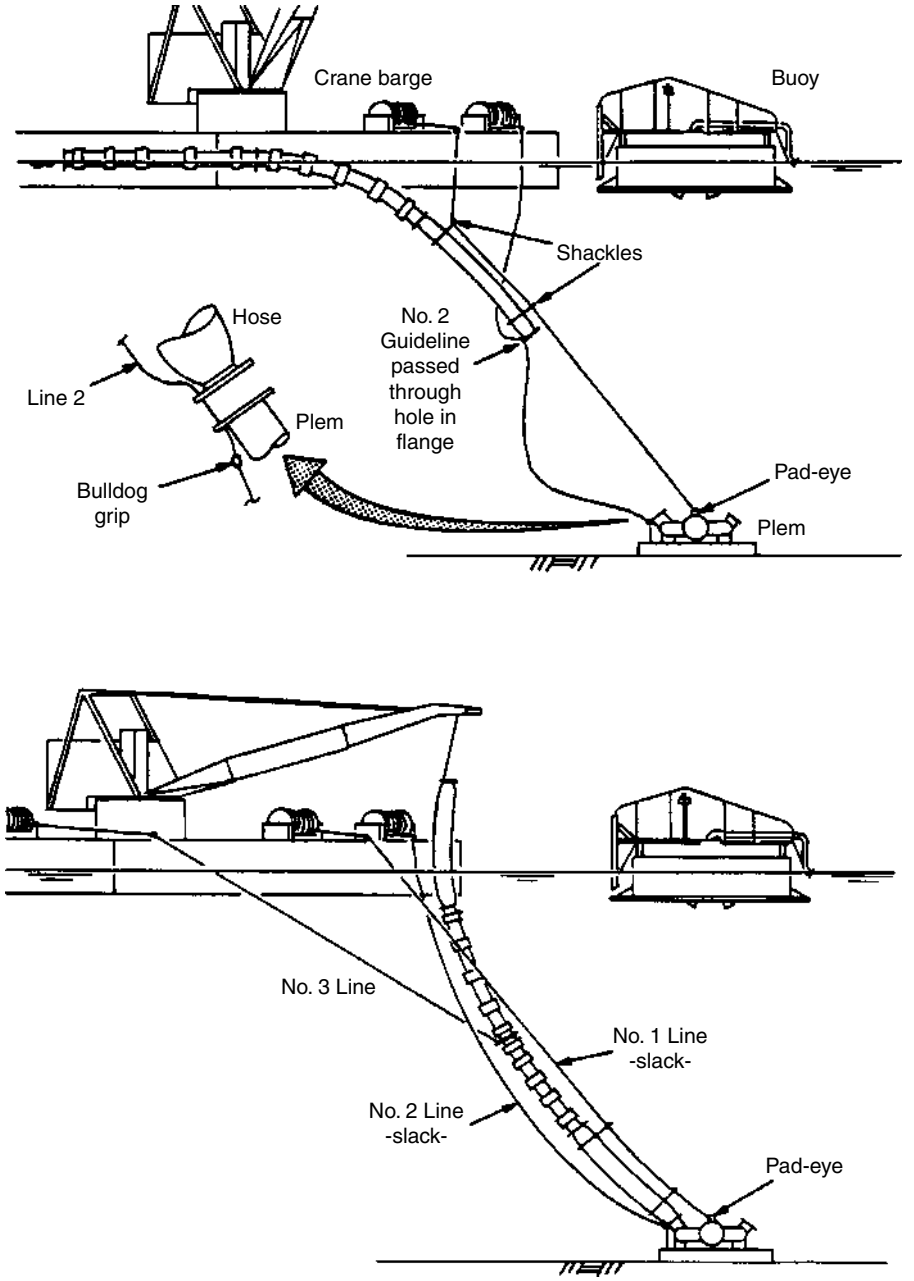

FIGURE 14.6
Underbuoy hose connection to PLEM. (Courtesy of Imodco.)

Once the base is seated, the column is brought to the site floating horizontally, upended by ballasting to the vertical attitude, and then guided and ballasted down to a mating of the hinge. Diver intervention may be required at the mating to drop in the securing pins and activate the hydraulic locks. Tensioned guidelines have been used to control the mating (see Figure 14.5). Often the final connection is made by a pulling line, which directs and pulls the mating cone into position.

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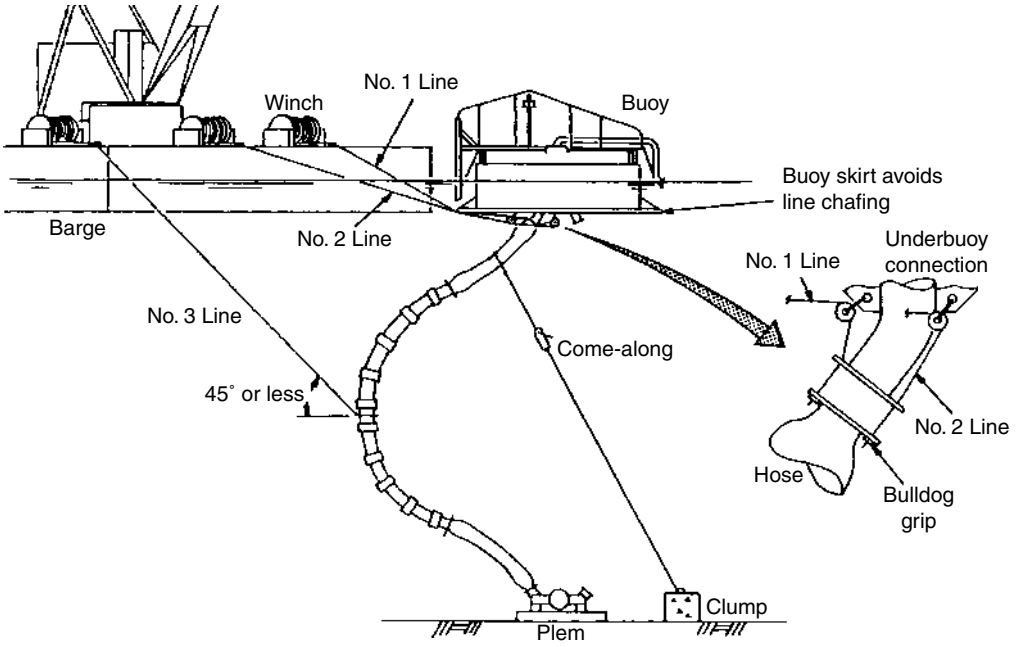

FIGURE 14.7
Underbuoy hose connection to PLEM. (Courtesy of Imodco.)

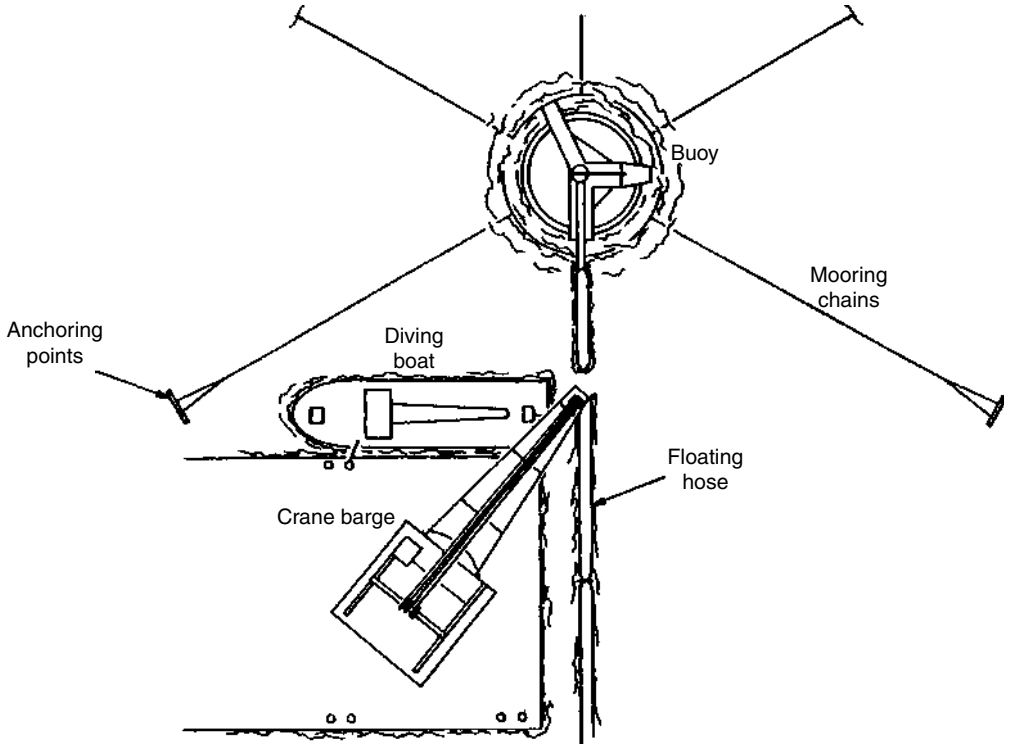

FIGURE 14.8
Plan view of floating hose hook-up for CALM. (Courtesy of Imodco.)

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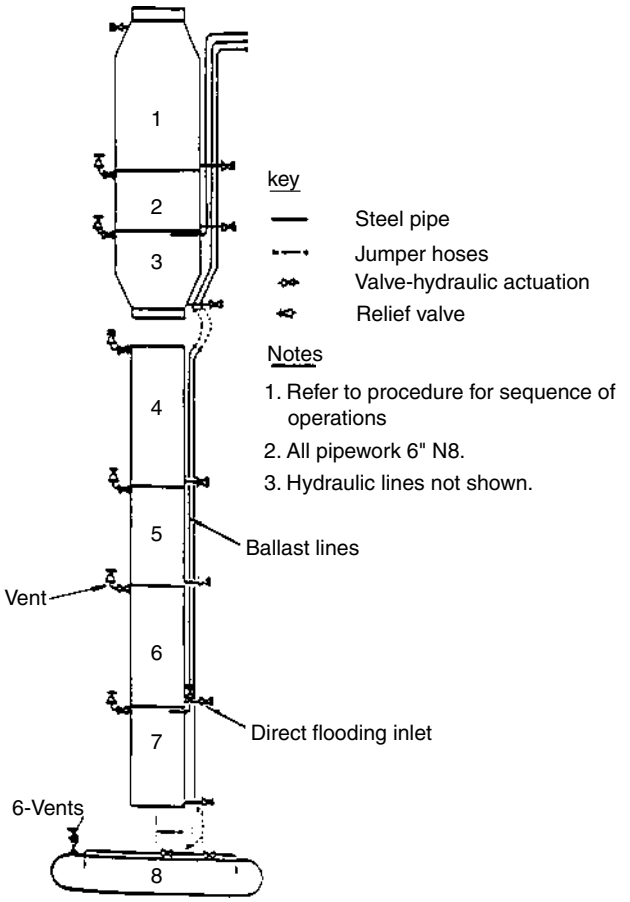

FIGURE 14.9
Installation of SALM. Bilge and ballast system. (Courtesy of Imodco.)

Another method of lowering the base has been to use a controlled free-fall system in which buoys are attached at intervals to lines leading to the barge or boats. As the base structure is progressively ballasted, it descends until restrained by the buoyancy of the next set of buoys. Then, more ballast is added to the base structure so that the base may descend another step. In this way, velocity of descent is kept low, dynamic forces are accommodated, and control is maintained, even to the extent that the process is reversible by ejecting the water by compressed air. However, this conceptually attractive system has encountered some practical problems in actual application. When the prototype underwater oil storage "Seatank" was being installed by this method, the lines joining the buoys become fouled due to waves and wind. Waves washing over the shallowly submerged tank created dynamic instability, leading to eventual total loss of control. The structure plunged to the bottom and imploded. Conversely, as noted earlier, the Harding Gravity-Base Tank was installed successfully by this method (see Figure 14.1).

As water depths have increased and structures have become larger, with greatly increased functional requirements, other methods have been developed in order to give proper control and reduce the amount and complexity of underwater operations, especially that of mating the critical cardan joint.

One such method is to attach the column while the base is floating at the surface. This enables the subsequent upending and descent to be fully controlled with regard to stability and depth. It also ensures that the cardan assembly is carried out properly and

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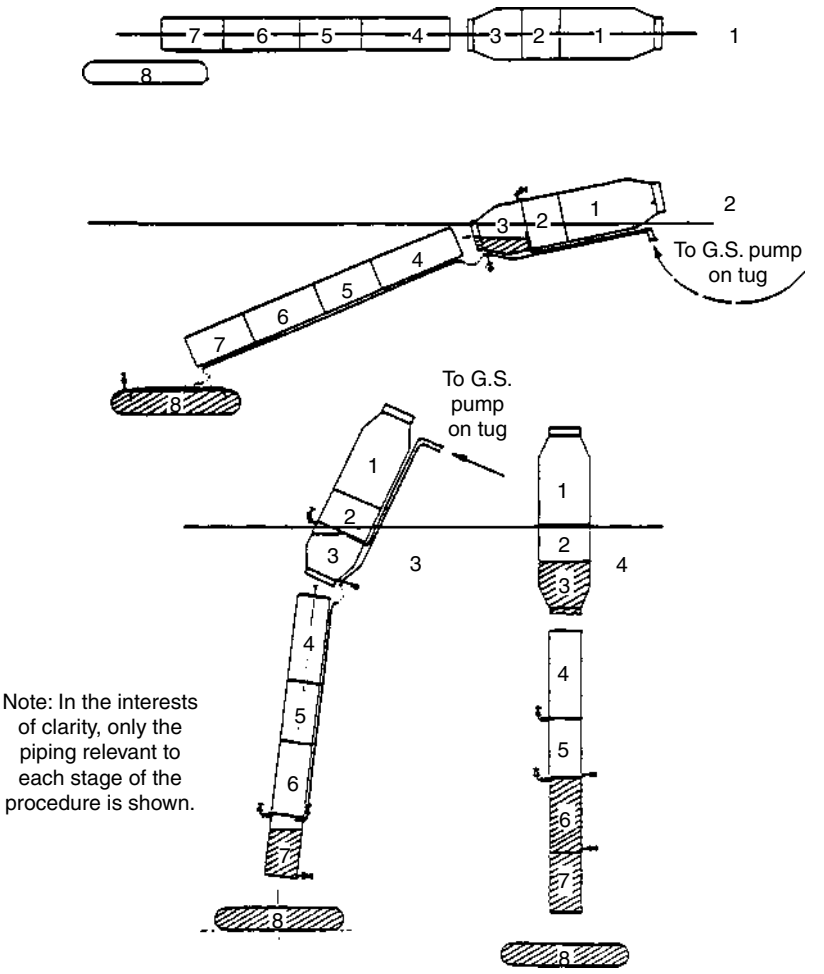

FIGURE 14.10
Installation of SALM, steps 1–4. (Courtesy of Imodco.)

facilitates inspection. The base raft is tilted by ballasting so that the socket for the cardan is just above water. The column, floating horizontally, is ballasted to raise the cardan above water. Now the mating can be carried out in the dry. Control of orientation is, of course, one of the major problems in the initial assembly. Since it can be carried out in shallow, protected water, lines to mooring buoys and to anchors can be employed.

Once the articulated column is mated, it can then be towed to the site in either the horizontal or vertical mode. Smaller structures, with minimal facilities, such as mooring structures for floating production systems, are usually towed in the horizontal mode, whereas structures with extensive topside facilities will be upended to the vertical while still in the harbor, the topsides installed and hooked up, and the structure towed to the site in the vertical mode, to be ballasted down onto the seafloor. To upend the combined base-column structure, iron ore ballast may be slurried into the base.

One of the more demanding of such installations was that of the Beryl A flare stack, where the articulated structure had to be not only upended and seated properly, but had to have an exact distance and orientation from the production platform in order to enable the prefabricated flare stack bridge to be set. Lines were therefore run from the platform to

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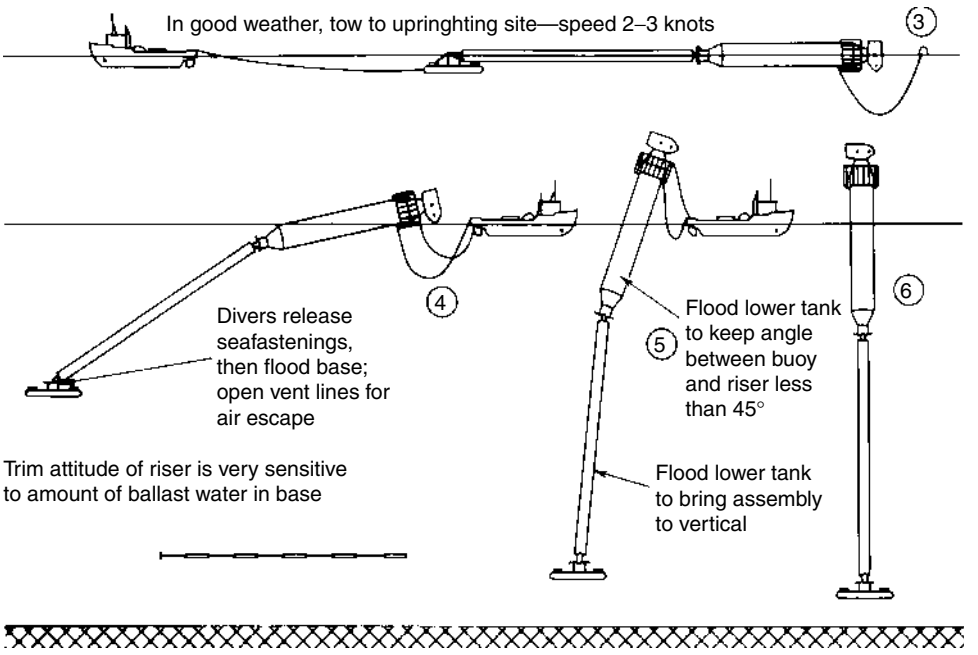

FIGURE 14.11
Installation of SALM, steps 3–6. (Courtesy of Imodco.)

the base structure and a boat with bow thruster used to stretch them as the base reached the seafloor.

Where a floating production vessel (e.g., a converted tanker) is to be moored, a rigid but hinged yoke is usually fitted to the bow of the ship in a shipyard. The mating of the yoke and the single-point mooring riser is then carried out at the site, by use of a pull-in wire system, and connection made by a tension-bolted flange.

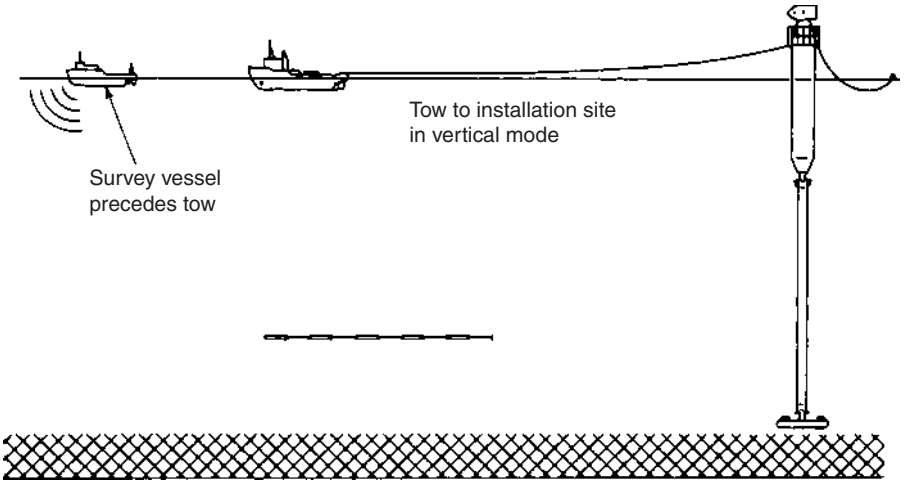

FIGURE 14.12
Installation of SALM, step 7. (Courtesy of Imodco.)

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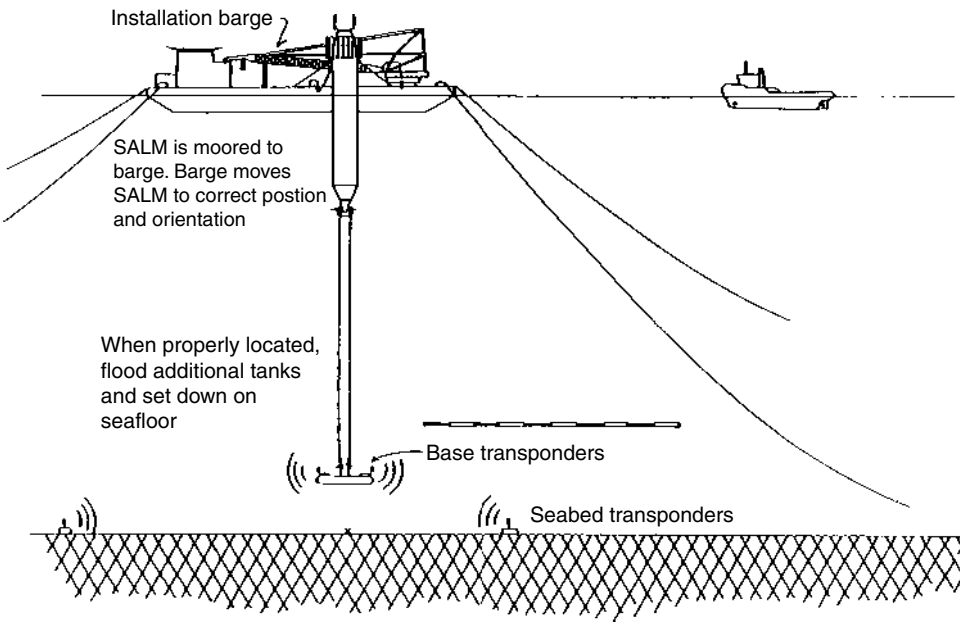

FIGURE 14.13
Installation of SALM, step 8. (Courtesy of Imodco.)

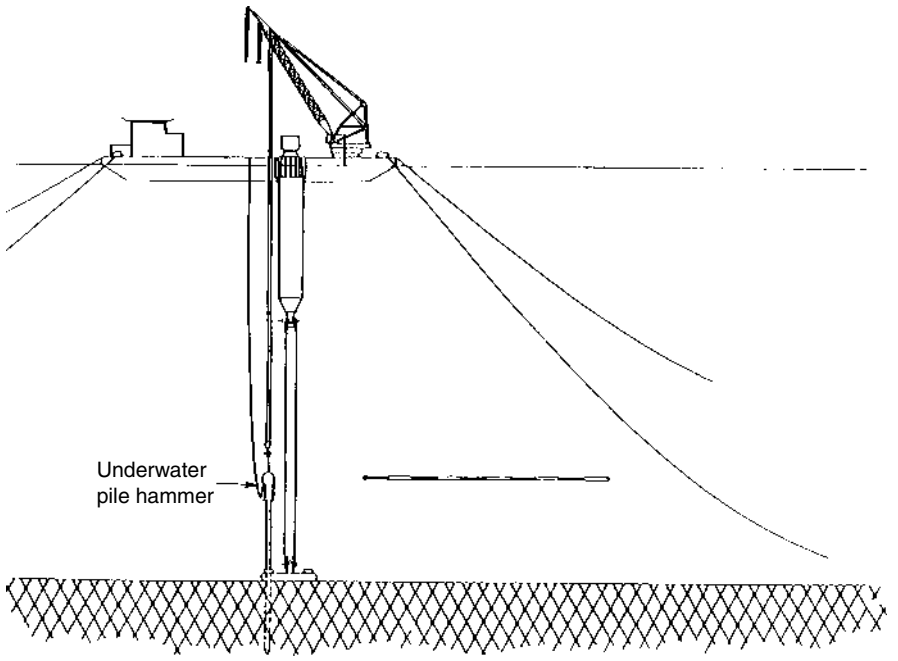

FIGURE 14.14
Installation of SALM, step 9. (Courtesy of Imodco.)

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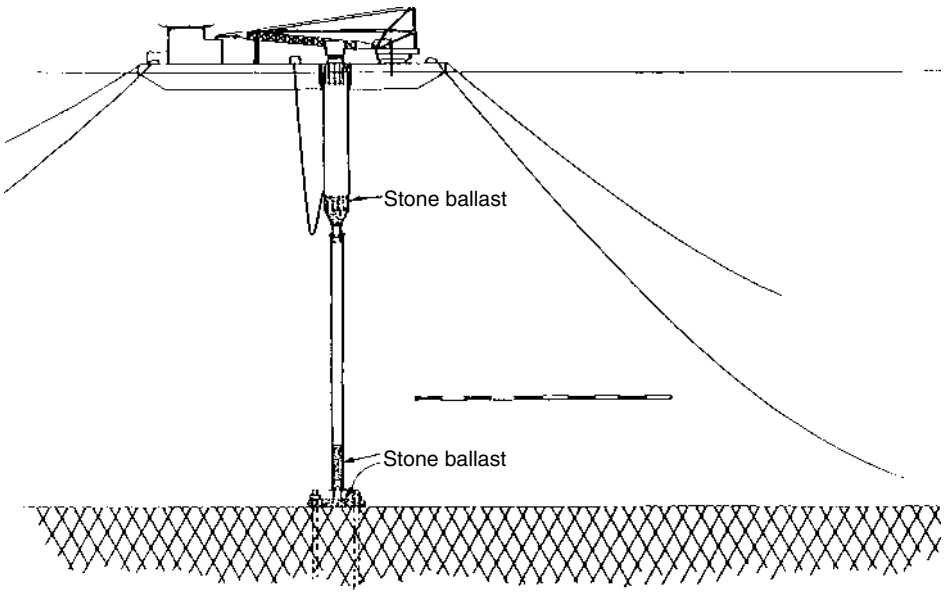

FIGURE 14.15
Installation of SALM, stone and slurry ballasting. (Courtesy of Imodco.)

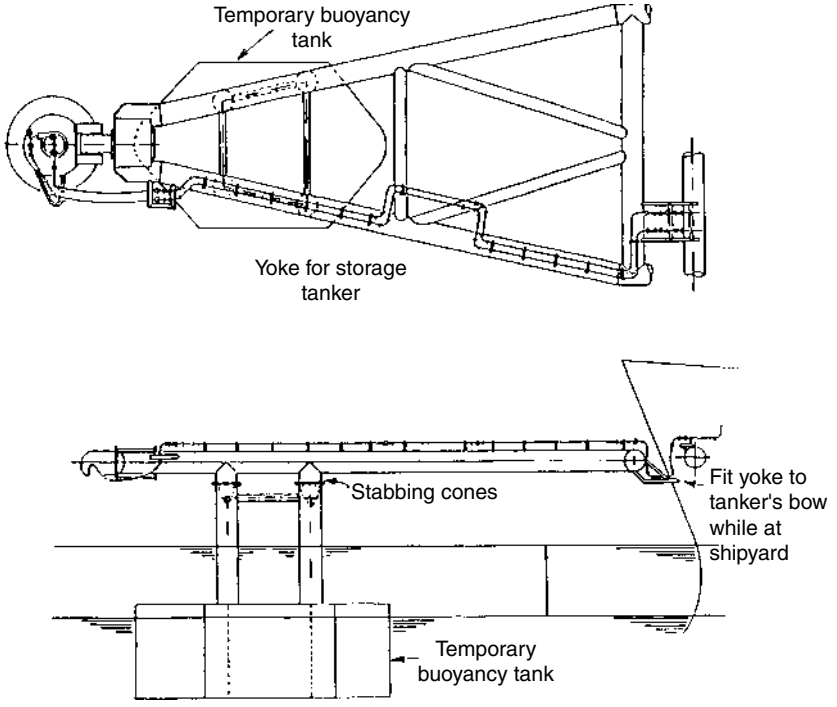

FIGURE 14.16
Installation of SALM; mounting of rigid arm Yoke. (Courtesy of Imodco.)

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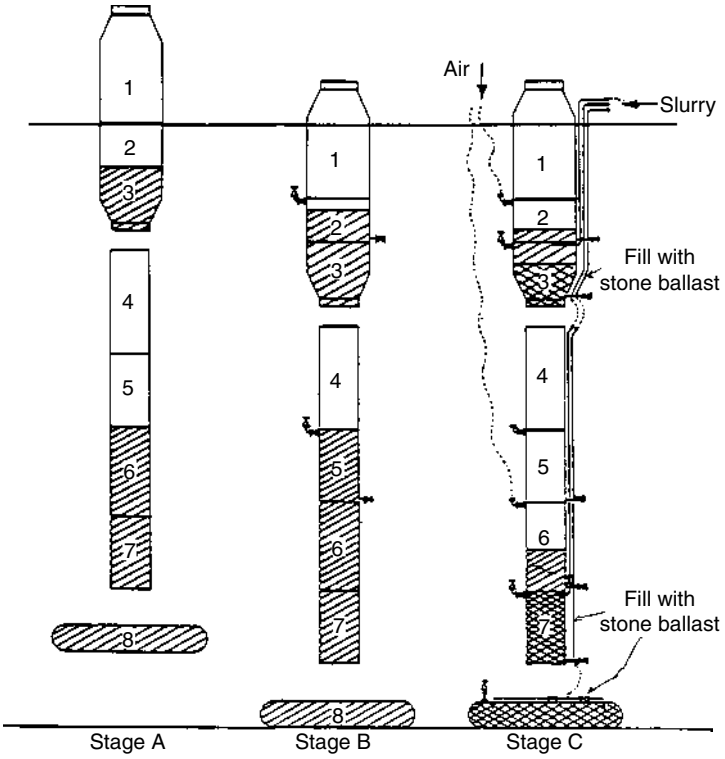

FIGURE 14.17
Installation of SALM; ballasting sequence. (Courtesy of Imodco.)

Another scheme involves vertical construction of the articulated column. The base is constructed first and moored in a deep-water protected site. The halves of the cardan are preassembled, joined with temporary fixing, and then set on the base. Temporary buoyancy tanks are attached to enable the base to be submerged and still remain stable. Then, the column is constructed above the base, allowing the whole slowly to sink as weight is added. Eventually, the structure will have been submerged to the point where the stability and buoyancy can be supplied by the column alone. The temporary buoyancy tanks are then removed and the structure completed afloat. The deck can then be mounted, lifting it on if it is a small structure, or transferring it by floating in over the column and deballasting if it is a large deck. The whole can then be towed to the site in the vertical mode. After seating on the seafloor, the cardan is freed, to permit flexible articulation. Ballasting is added to cause the base to penetrate. In firm soils, underbase grout may be adequate; in other soils, underwater piling will be driven through sleeves in the base and connected by grouting of the annulus.

14.4 Seafloor Templates

The use of subsea production templates, working as part of a subsea production system, is growing rapidly (see Figure 14.22). Even with fixed structures (e.g., the BP platform Magnus) the use of a template through which to predrill the wells while the platform

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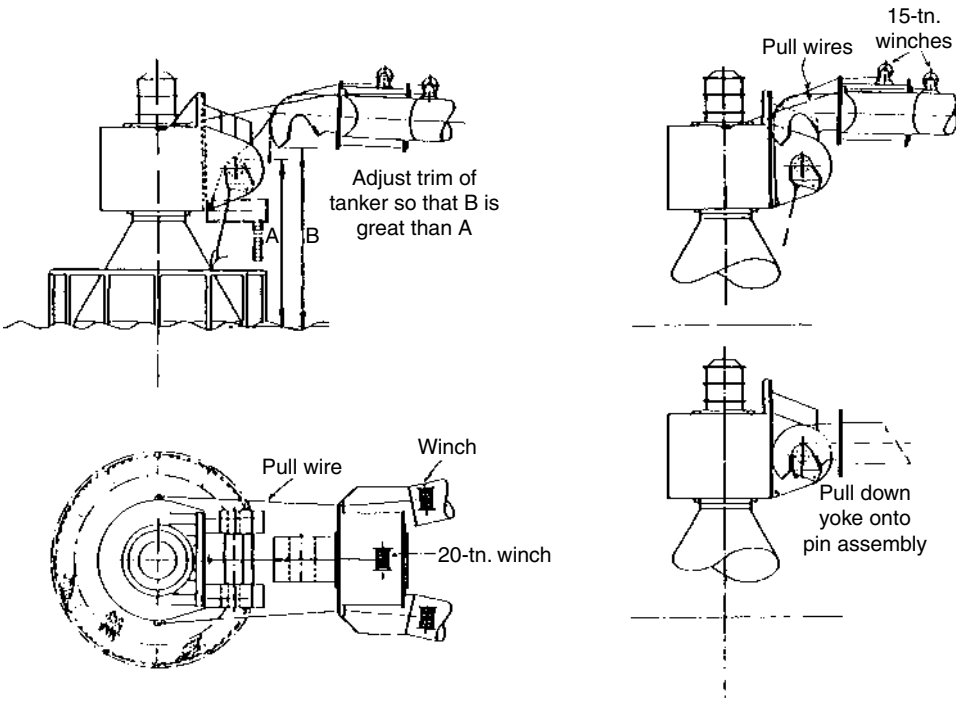

FIGURE 14.18
Details of yoke connections on SALM. (Courtesy of Imodco.)

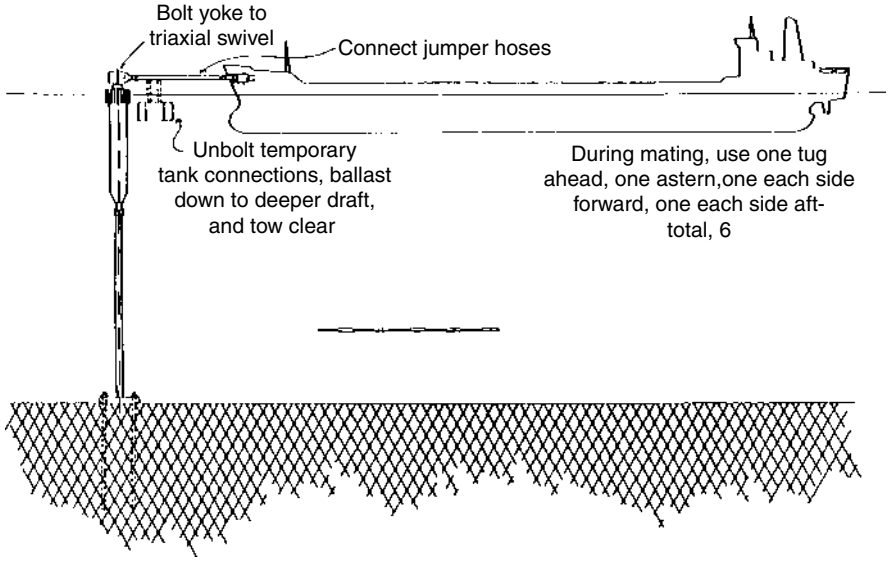

FIGURE 14.19
Mooring of floating storage vessel to SALM. (Courtesy of Imodco.)

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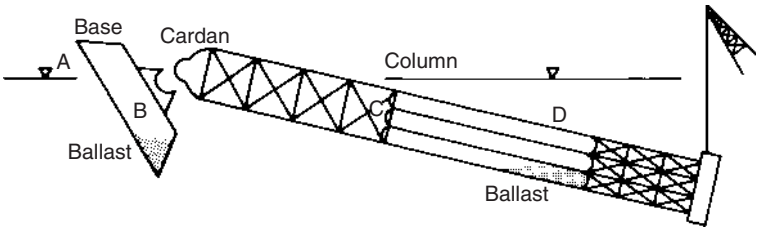

FIGURE 14.20
Assembly of articulated column with base while afloat. (Courtesy of Doris.)

itself is being fabricated shows significant cash-flow advantages by enabling the platform to be brought into production at an earlier date.

Subsea templates have therefore been developed by a number of firms; they have grown in size and weight to as much as 2000 tn. or more. These are typically loaded out on a barge for transport and then launched or lifted off to the self-floating mode.

They then must be lowered to the seafloor, with equipment and rigging adequate to sustain the high dynamic forces involved. Stability during submergence is maintained by designing the structure so that the center of gravity is below the center of buoyancy.

In one system, the lowering is done by a drilling vessel, either a drill ship or semisubmersible. Lines are run from the derrick hoist, down through the moon pool, and then up and attached to the top of the template floating alongside.

A floating derrick barge now hooks onto the template, which is given negative ballast, allowing it to sink below the keel of the drilling vessel. By slacking the floating derrick barge's lines and taking in on those of the drilling rig, the load is transferred to the drilling rig. The barge lines are now disconnected (see Figure 14.23).

This lowering can also be done dynamically without the use of the offshore derrick barge, provided the template has inherent stability, that is, center of gravity (CG) below center of buoyancy (CB). Buoyancy tanks attached to the upper portion of the template can be used to give this stability. For buoyancy tanks that must remain intact during descent into deep water, consideration should be given to offsetting the hydrostatic head by filling with a light fluid such as a solvent or by filling with syntactic foam.

Other means are available for transporting and setting templates, especially the larger ones. A barge may be specially equipped with large cable or casing grip hoists. In an inshore harbor, in relatively shallow water, the template is seated on the seafloor.

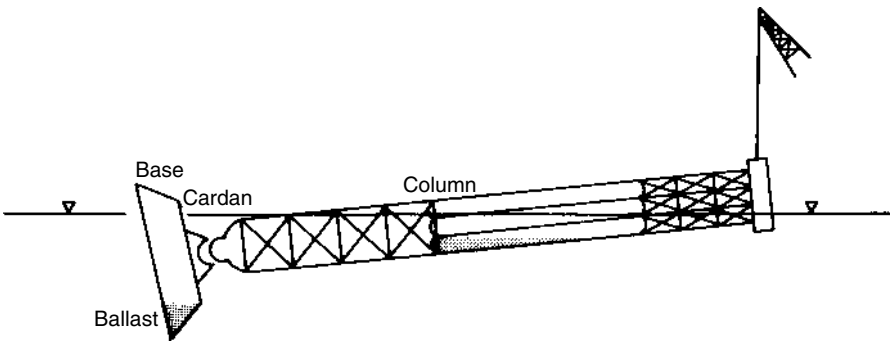

FIGURE 14.21
Alternative method of assembly of articulated column with base. (Courtesy of Doris.)

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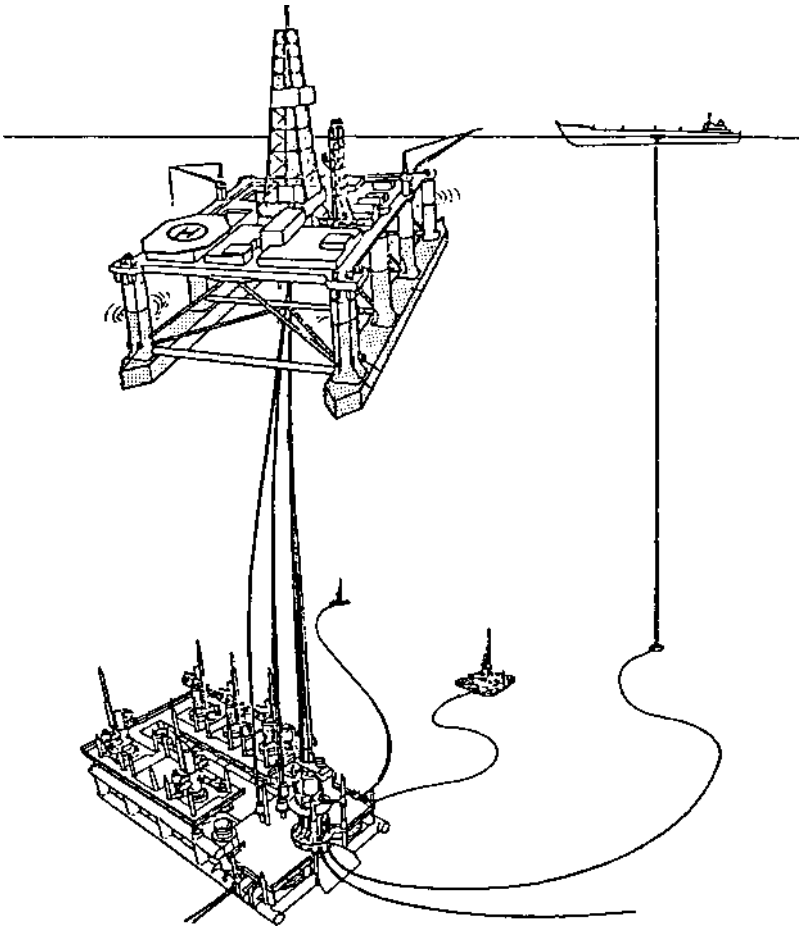

FIGURE 14.22
Multi-cell template for seafloor completions. (Courtesy of Exxon.)

The barge now floats in over the top, attaches the cables or casings, pulls the template up snug under itself, and thus transports it to the site. Once at the site and properly positioned, the template is lowered to the seafloor.

Disconnect is effected by remote-operated devices, operated acoustically or hydraulically or by explosively activated disconnects. One advantage to the use of the drilling vessel to lower the template is that the heave compensator of the drilling vessel can minimize the dynamic loadings due to heave.

For the larger templates of the future, large spars can be used to provide inherent heave compensation as the template is lowered. In this case, the actual lowering can be carried out by linear winches on the spars, while the spars will be able to minimize differential heave because of their small water plane area.

Templates can either be gravity base supported or pile supported. In the first case, grouting under the base, between skirts, may be employed to ensure level support. In the second case, piles are set through sleeves, using sonar and video guidance, driven with an underwater hammer, and the connection made by grouting of the annuli.

If a structure is to be later set over the subsea template, bumper piles (actually seafloor "fender piles") may be installed with the subsea template and driven in, as for the support

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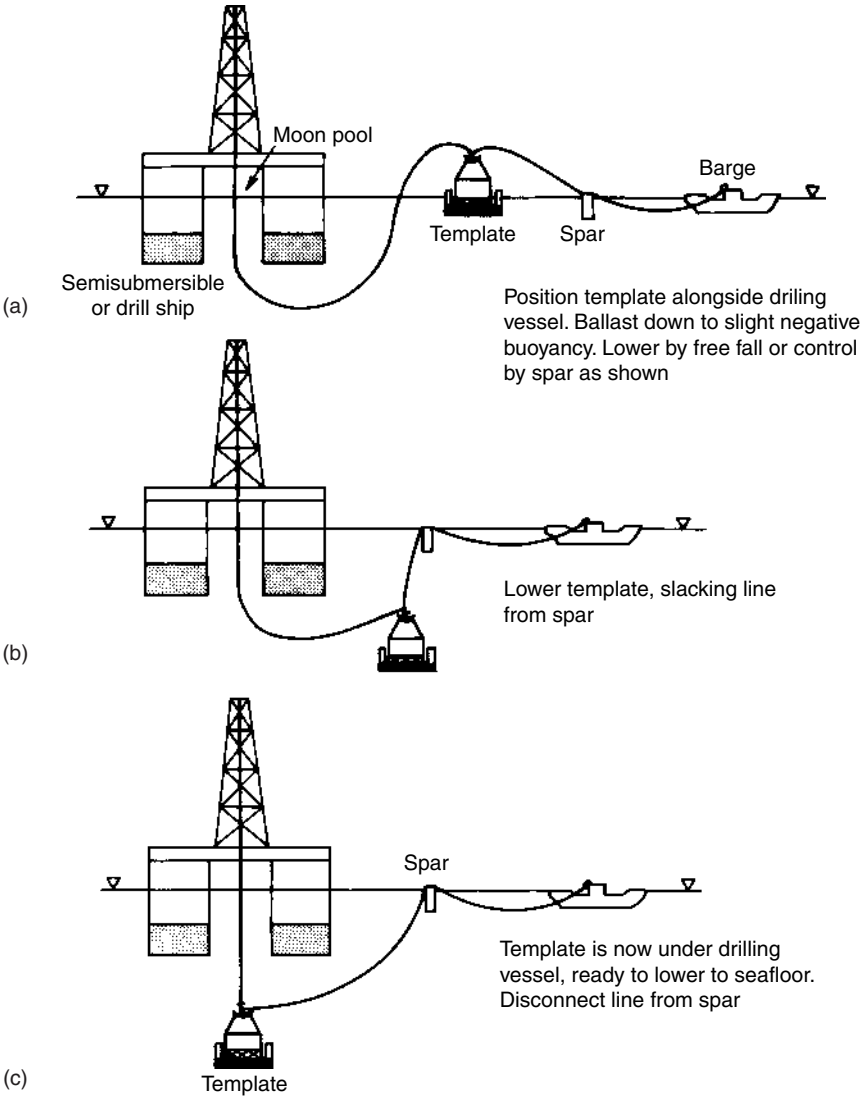

FIGURE 14.23
Keel-hauling transfer of seafloor template to underneath drill vessel. (Courtesy of Exxon.)

piles. Then, they are disconnected from the template so as not to transfer loads into it during the structure's installation (see Figure 14.24).

Flow line installation to subsea templates may be carried out by laying the flow line down beside the connection point. The line may then be pulled in using a wire line through a base plate at the connection point or it may be guided in by an ROV, using video.

Alternatively, one end of the flow line can be lowered vertically using a drill rig and tensioned guideline. The flow line is landed on and inserted into the base, which is on the seafloor. The connection permits a swivel in a vertical plane, using tools run down from the drill rig. The line at the surface is now transferred to a lay barge, which pulls away, keeping tension on the line. Connections of flow lines to the subsea template may also be made using remote-controlled manipulators. An overall subsea production system is shown in Figure 14.25. Also, see Chapter 22.

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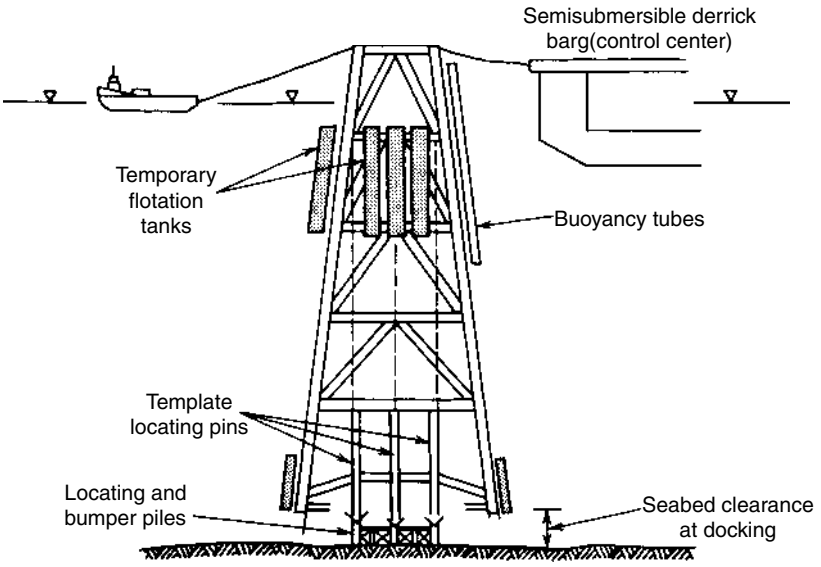

FIGURE 14.24
Docking arrangement for installing jacket over template for pre-drilled subsea wells. Beryl B Platform. (Courtesy of Mobile Offshore Structures.)

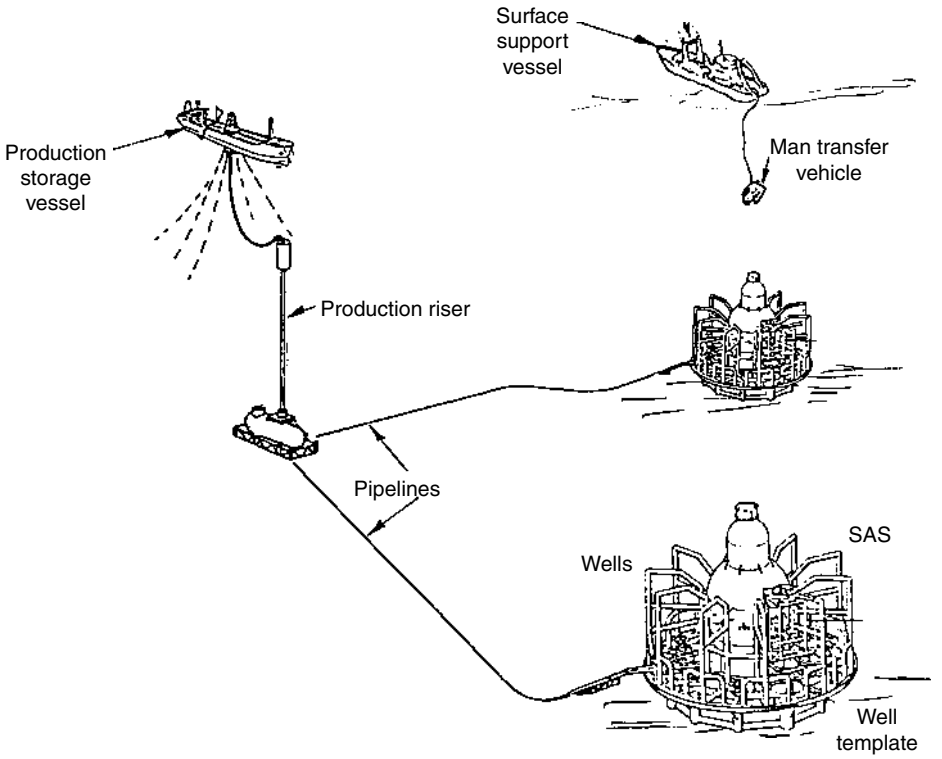

FIGURE 14.25
Total SAS subsea production system.

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14.5 Underwater Oil Storage Vessels

Underwater oil storage vessels differ from other subsea installations primarily because of their very large displaced volumes. A typical storage vessel may be required to store 160,000 m^3 or more of oil. Therefore, very large inertial forces are involved.

Chicago Bridge and Iron successfully installed the Khazzan Dubai offshore oil storage vessels of steel in the Arabian Gulf. Initial construction was in a shallow, dewatered basin. When the tank was sufficiently complete so that it could float as a single unit, using compressed air, the basin was flooded, and the tank, a bottomless hemisphere, was moved laterally into a deeper basin and seated on its floor by release of internal air pressure. The structure was then fully completed. Floated once again by filling of the tank with compressed air, it was towed to the site and positioned by mooring lines, and the air was gradually released. As the air bubble became progressively smaller within the tank, the tank took a significant list (almost 30°!), until its righting moment equaled the dynamic listing moment. It was allowed to slowly sink further, eventually returning to vertical and seating on the seafloor. This initial list, of course, had been shown in model tests and hence was anticipated

Through sleeves in the periphery of the tank, piles were set and seated by hammer. Using the piles as casings, holes were drilled and enlarged in the limestone strata; the piles were lowered to place and grouted to serve both as vertical support and, more important, to give resistance to uplift when the tank was filled with oil.

A number of conceptual studies have been carried out for large tanks, both of concrete and steel, for placement on the seafloor in such areas as the Gulf of Alaska, the Navarin Basin, offshore China, the northern North Sea, and the deep Arctic.

Conceptually, these tanks would be seated on the seafloor, with sufficient ballasted weight or hold-down piles to prevent uplift when filled with oil. The passage of long-period waves can also give significant uplift forces in the shallower water depths but these, of course, diminish with depth. A tank in very shallow water, subject to high waves and swells of long period, will tend to "crawl" along the bottom due to the horizontal and vertical wave effects being in phase. Consideration also has to be given to the very long tsunami wave and its potential for uplift.

The tanks are filled with oil, using the natural pressure from the gas. Discharge is accomplished by the differential water vs. oil pressures at the depths involved, supplemented as necessary by pumping. A large tank of this type is initially manufactured, transported, and submerged in a manner similar to that used for large gravity-based structures. During initial submergence at the site, as the upper surface disappears below water, there is a zone of dynamic instability. This zone has been frequently observed with semisubmersible drilling vessels.

With the sudden loss of water plane, the righting moment depends almost wholly on the vertical distance between the center of buoyancy and the center of gravity. At the same time, the waves no longer move in fully orbital fashion but break and swirl over the top, creating a venturi uplift effect, often denoted as the "beach effect." This problem area is best overcome by the use of columnar buoyancy tanks, temporary or permanent, to give stability until substantial submergence has been achieved (see Figure 14.26).

A large underwater storage tank can be pulled down to the seafloor, under positive buoyancy, using tethers or lines secured to a pre-placed base or else lowered down from buoyant vessels on the surface. Stability and attitude of the unit, once fully submerged, is determined by $\overline{KB}$, $\overline{KG}$, the free surface of ballast water in compartments, and the pull of the lines. A known and definite trim is preferable to an oscillating, indeterminate pitch.

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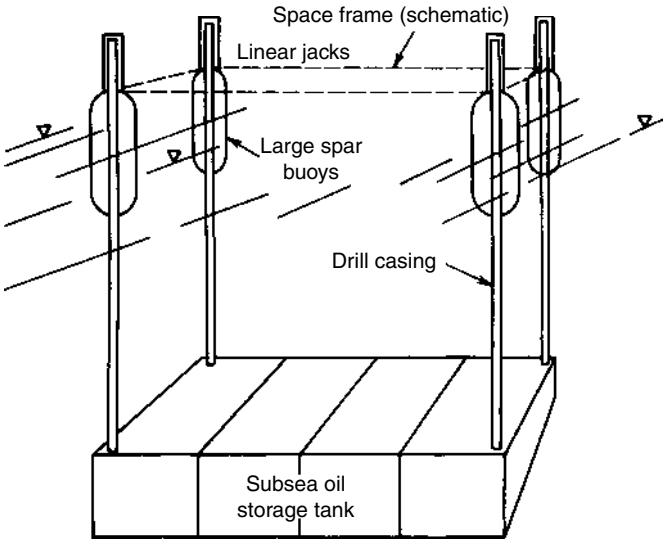

FIGURE 14.26
Using drill casing and large spar buoys to control seafloor oil storage tank during submergence.

The second problem is that of dynamic response to the waves and swells during the lowering phase. Large articulated spars, floating above the storage vessel, have the ability to respond to induced heave with a spring response and minimize the dynamic heave due to passing waves. The spar buoys may either be independent of each other or joined by means of an articulated space frame which maintains the relative positions of the spars in the horizontal plane as well as providing a support for operational control.

In relatively deep water (100–300 m) internal pressurization may be used to prevent implosion of the tanks. Air pressure can be used provided consideration is given to

1. The weight of added air;
2. The temperature rise as air is compressed;
3. The reduction in pressure as the air subsequently cools;
4. The differential pressure gradient between external hydrostatic head and internal air pressure over a vertical distance: the external head is a function of water density, which is not constant but which increases with lower temperatures, greater depths, and, in some cases, greater salinity. The internal head of compressed air is constant over the height; hence there may be a net overpressure at the top with consequent tension in the upper walls and roof.

At greater depths, other methods must be employed. Some potential solutions are described in Chapter 19.

14.6 Cable Arrays, Moored Buoys, and Seafloor Deployment

It is often necessary to deploy anchored buoys in the deep ocean. These generally consist of a clump anchor, a large buoy, and a connecting line of steel wire or, more usually, of fiber such as nylon, polypropylene, or Kevlar. To protect against damage to the line from fish bite, polyurethane or polyethylene coatings may be extruded onto the fiber lines.

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To prevent accidental sinking of the buoy from flooding, polyurethane foam or syntactic foam should be used as fill. Buoys have frequently been lost due to rupture of tanks due to boat collision, fatigue of welds, or gunshots from passing fishermen.

Generally the scope of the buoy line is set at 1.2–2.0 times the depth. Two methods of deployment are used: "anchor-first" and "anchor-last." In the anchor-first case, the anchor is hoisted over the side and lowered to the seafloor, and then the buoy is deployed. This requires considerable time, during which position must be maintained as well as requiring a powered drum to control the lowering. In the anchor-last method, the buoy is launched, the boat travels forward a distance equal to the scope, and the anchor is dropped, free-fall. Both calculations and field measurements indicate that dynamic forces are relatively minimal, that the anchor falls only at 3–5 m/s due to the drag restraint of the line, and that impact forces of the anchor do not lead to excessive dynamic stresses in the line. In general, the maximum force in the line is equal to the weight of the anchor plus line.

When deploying cable arrays, relative position control is very important. In some cases, use of marker buoys (articulated spar buoys) with short scope may be helpful, although today with acoustic positioning systems and DGPS, even more exact control can be attained. In some cases it is necessary to cut a line free—for example, a line used solely for deployment—in which case a hydraulic cutter can be run down the line, either controlled or free-fall, and activated when it reaches the desired depth. Alternatively, an ROV equipped with a cable-cutting device may be employed.

Proving of the mooring capacity of a line may be carried out in relatively shallow water by mooring a barge, such as an offshore derrick barge, between the legs of the moor, and then reeving wire lines to blocks and over the deck to the deck engines, so that two legs react against each other. Structural steel ties must be used to transfer the tension. In deeper water, and with short scopes, the vertical component of force becomes dominant, and can be proven by similar means or by direct pull through the moon pool with the drill rig. In shallow waters, less than 100 m, driven plate or pile anchors are used, while in deeper water, suction anchors are effective.

Deployment of objects to the seafloor may be done by lowering down, as described in the section on seafloor templates, with adequate consideration for the dynamic response of the lowering vessel. Alternatively, controlled free-fall deployment may be used, in which buoys are used to reduce the net weight, increase the drag, and provide vertical stability. The buoys will thus also be progressively descending to deeper water, so they must be filled with either light-density fluid or syntactic foam, so that they do not suffer significant decrease in volume with depth. Once the object impacts the seafloor, the buoy or sphere is released, either automatically on release of load (pelican-hook arrangement) or by an acoustic-actuated release mechanism (see Section 13.2 and Figure 13.5, as well as Section 22.7).

14.7 Ocean Thermal Energy Conversion

Extensive studies and initial test installations for ocean thermal energy conversion (OTEC) have been carried out under contracts with the U.S. Department of Energy as well as by engineering organizations in other countries. In general, all systems employ a cold water pipe of large diameter, raising cold water from a depth of 1000 to 2000 m to a surface plant where the warm waters are used to provide the heat source. The surface plant may be a very large floating structure, in which case it is usually planned to be moored in deep water (2000–4000 m). Alternatively, the plant may be constructed as a fixed structure on the continental shelf in perhaps 200 m water depth, or even on shore (Figure 14.27).

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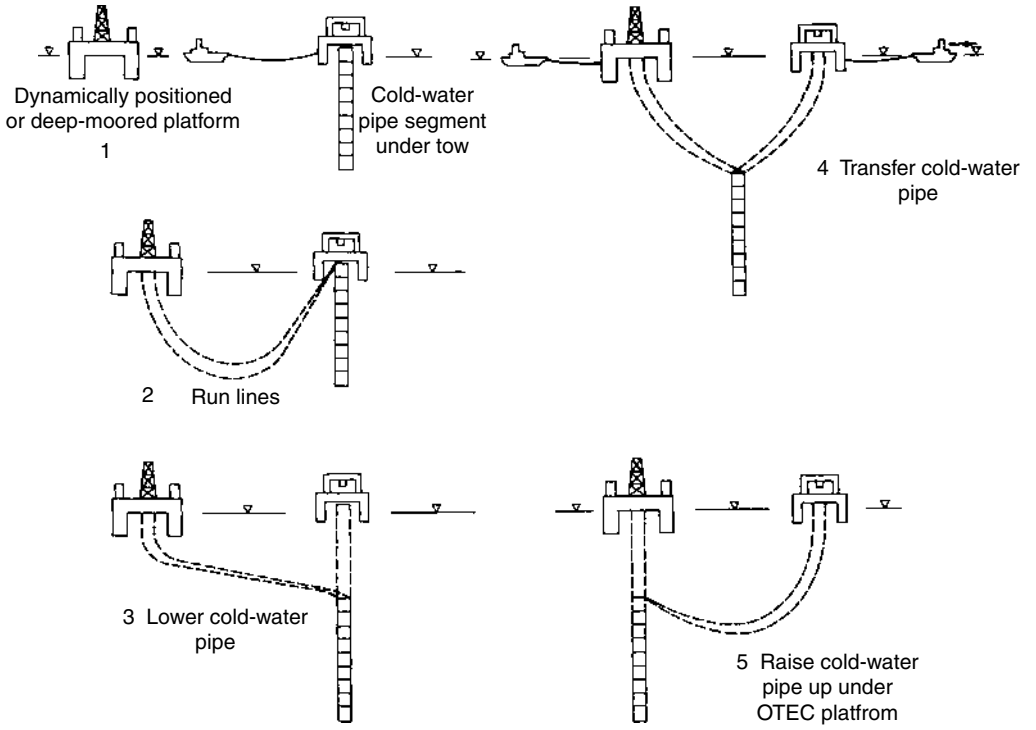

FIGURE 14.27
Concept for deployment of OTEC cold-water pipe.

The principal unique construction aspects to be discussed in this section have to do with the deployment of the cold-water pipe. In test installations of relatively small-diameter pipe, several have been lost during deployment, while two have been successfully installed.

For the floating plant system, the pipe is usually conceived as being about 30 m in diameter, 1000 m in length, weighted at the lower end, and suspended from the floating vessel at the upper end. Pipe materials considered have included lightweight concrete, steel, fiberglass, polyethylene, and hybrid designs. The principal problems both in service and during construction are dynamic ones, resembling those successfully surmounted in the installations of the Cognac platform and Hutton and Heidron tension leg platforms. Here, however, the sizes and weights are an order of magnitude greater. The major construction problem is that of transport, upending, and transfer of the cold-water pipe to its location hanging beneath the floating vessel.

The pipe is usually conceived as being articulated in order to reduce its in-service moments generated by internal waves and currents. During installation, these joints may be of use to the constructor or may require temporary locking. Alternatively, polyethylene pipes have been tried, which are able to be buckled during deployment yet subsequently regain their shape.

In one proposed system for deployment, long segments would be assembled in shallow water and towed to the site in the horizontal mode. Upon arrival, they would be upended by selective ballasting. Temporary bulkheads or compartments within the pipe are required, along with filling and venting lines in order to minimize moments. Alternatively, the pipe may be placed under axial tension during upending, by means of tugs pulling in opposite directions.

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The successful use of high tension for upending the Heidron TLP tethers is perhaps the most-promising solution for the OTEC cold-water pipe (see Section 22.5.4).

In recent years, most attention has been devoted to shelf-mounted plants, located on steeply sloping margins such as those found in Hawaii and on the east coast of Taiwan. For the MINI-OTEC plant on the island of Hawaii, a small polyethylene line ran down the slope to a depth of 600 m. Since it was positively buoyant, it was held down at intervals by tethers to clump weights. The line was assembled in quiet water near shore, towed afloat to the site, and submerged by progressive flooding. As it submerged, it progressively buckled but then resumed its circular shape after it passed the critical bend. Reinforcement and softening pads were provided at each tether to prevent chafing. The pipe was successfully installed.

Deep Oil Technology has proposed a system for pulling a large-diameter line down a slope:

1. Using a drilling vessel, a pile anchor will be drilled and grouted into the rock at the lower end.
2. When the pile is set, it has a terminal gimbal base attached, which contains a large sheave. Now, after the pile is in place, and cemented, a line is paid out from the drill ship, around the underwater sheave, and back to a surface boat. This end is towed to shore.
3. A landing base for the cold-water pipe termination is now lowered onto the pile, using tensioned guidelines.
4. When the line reaches shore, it is attached to the prefabricated pipe. The pipeline is now pulled out to the termination pile. A small vessel is moored vertically above the end of the pipeline, moving out with it. The vessel supports a jetting hose and TV camera and provides buoyancy to support a drag scraper or plow.
5. Finally, a retrievable filter and entry port are lowered down by tensioned guidelines and fitted to the termination structure.

A steel pipeline through the surf zone, through which the cold-water pipe is pulled, can act to protect and stabilize the cold-water pipe. Because of low friction, pulling forces will be reduced. In all shelf-mounted installations, adequate hold-back capacity must be provided at all stages to prevent the pipe from running downslope.

14.8 Offshore Export and Import Terminals for Cryogenic Gas—LNG and LPG

14.8.1 General

While Chapter 11 through Chapter 13 discussed the construction of fixed and floating terminals of steel and concrete, this chapter will present the special considerations applicable to export and import terminals of steel and concrete employed for the gasification, refrigeration, storage, and subsequent regasification of LNG and other cryogenic gasses. There are predictions that LNG will be a principal energy source in the future, and terminals are needed for initial production and later regasification for transport in the land-based pipeline system.

LPG terminals are less demanding than LNG terminals, since LPG is liquefied at about −60°C. However, liquefaction of LNG at −163°C is currently the most efficient and

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economic way for moving large quantities of gas energy across oceans to their market. At the same time, the concentration of so much energy at specific points demands that safety be paramount in design and construction.

GBS platforms for LNG may provide an above-water base for conventional LNG storage and gasification/liquefaction or they may incorporate integral LNG storage within the hull.

For LNG storage, both primary and secondary containments are required and external hazards take on a new order of importance.

1. Extreme low temperature will embrittle carbon steel. Material properties may change.
2. Thermal movements will be far greater: local effects due to rate of temperature change as well as global effects should be considered.
3. Escape of pressurized gas through leaks must be anticipated, resulting in localized spots of intense cold. Controlled release cools the remaining portion and can be utilized to liquefy it.
4. Sudden escape of a large quantity of gas may result in a major explosion.
5. In the liquid state, under high pressure, LNG exhibits all the properties of a liquid, including surge and wave effects (e.g. ovaling pressures).
6. LNG tanks are most dangerous when partially in the liquid state and partially in the gaseous state.
7. Impact, as from a moderate ship collision or external explosion, must neither penetrate to the primary containment nor cause the membrane to leak. No sparks or fire must occur.
8. Insulation delays the transfer of heat but does not prevent it from happening over a long sustained period. Balsa wood is able to provide excellent insulation as well as moderate bearing, whereas soil can freeze and expand irregularly.
9. Interior boundaries require both a gas tight membrane liner and insulation.

All the above is in addition to the requirements for performance in the marine environment. Primary consideration is being given to structures built and substantially completed, in a graving dock and near-shore harbor, and then towed to the offshore site and installed.

The manufacturing process for prestressing wire (cold-drawing) renders the steel not subject to embrittlement under cryogenic. Nickel steel alloys and aluminum are not subject to embrittlement (stainless steel, Invar, 9% nickel steel, or 5083 aluminum). Thus, they can be used as a membrane for lining the primary containment.

Concrete, especially lightweight aggregate, and controlled density concrete, are not subject to degradation under cold—they actually get stronger. They have moderately good insulating properties. If partially or entirely saturated in a portion of their thickness, they are subject to internal freeze-thaw attack, which can be minimized or overcome by heat tracing and/or sealing of surface.

GBS import terminals of prestressed concrete are currently under final engineering and/or construction in the Gulf Coast of the U.S. as well as the West Coast (including Baja, California) and the East Coast. Exxon-Mobile is currently constructing an LNG import terminal for installation in the Adriatic Sea. Two systems are practicable (Figure 14.28).

In one, the GBS is kept independent of the LNG processing and storage. All LNG-related activities are carried on above water, supported on the GBS.

In the second, the space below water is utilized for storage of LNG.

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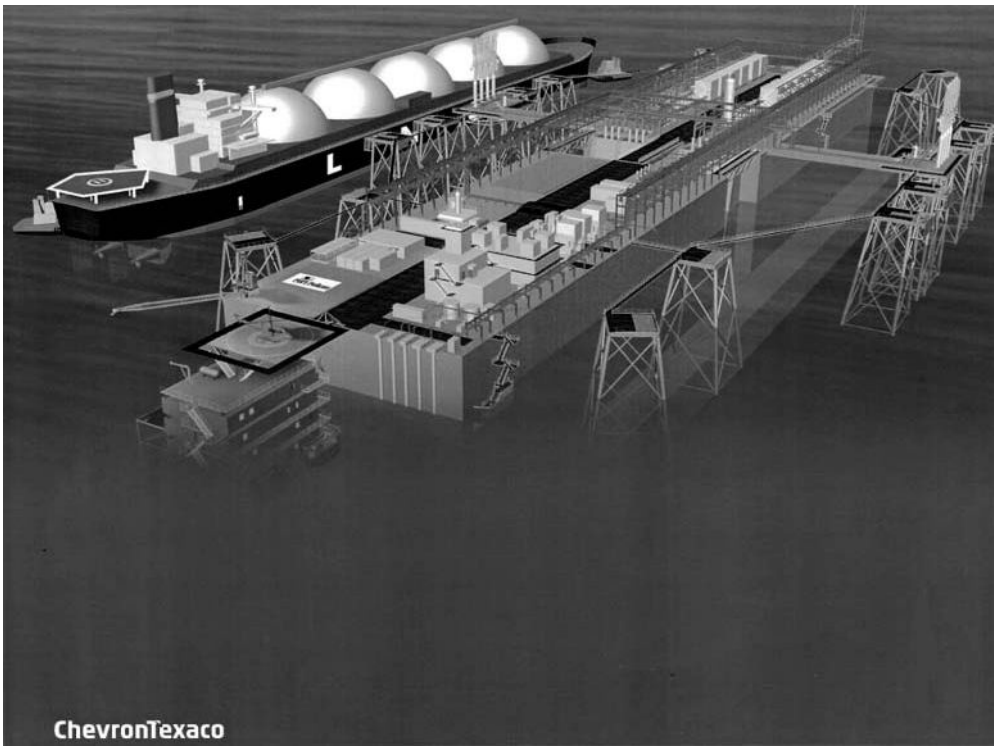

FIGURE 14.28
Proposed GBS offshore terminal for receiving, storage gasification, and shipment of LNG. Port Pelican, Texas. (Courtesy of Chevron.)

For deep water export (production) terminals, floating structures of prestressed concrete are under construction. They also could be integral, with storage within the hull or independent. This latter was adopted for the prestressed concrete LPG barge Sakti Ardjuna, built in 1975 in Tacoma, Washington, and towed 10,000 miles across the Pacific to Java, where it has been on station ever since, with no requirement for interim dry docking. Periodic underwater surveys are made at the site.

Advantages of integral structures, in which the LNG storage is incorporated within the hull, are the longer periods between warm-up and subsequent cool-down for inspection. It is during such drastic changes in temperature that most leaks occur. Further, these periodic inspections require taking that tank out of service, thus reducing the effective storage capacity.

However, to ensure the successful performance of integral structures of prestressed concrete, special construction technology needs to be followed.

1. The multi-axial post-tensioning needs to be applied initially with consideration to differential elastic and time-dependent shortening. Thus, the hull side walls and tank side walls need to be stressed to similar levels.
2. Transverse stressing needs to be applied without excessive distortion of the longitudinal walls.

The above two guidelines generally mean that connections need to be constructed after the walls are stressed. Connections may be conventionally reinforced and may incorporate fibers.

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Right-angle corners, where members join from two or three axes, must not be sharp, otherwise they will crack under thermal expansion or shortening. To the extent practicable, they can be curved on small radii, using small alloy bars (e.g., stainless) or fibers to minimize cracks.

Transverse walls present a special problem in that they must accommodate the thermal shortening. Some steel LNG ships use the double-wall concept, circulating seawater in the space between. This has also been considered for concrete hulls. Heat tracers or similar means should be considered.

Special considerations for fixed GBS import terminals include:

1. Adequate inherent weight when empty to prevent movement in an extreme storm. The shear force may need to be transmitted to the soil by penetrating skirts or spuds rather than solely by friction. Overturning and rocking need to be prevented by permanent ballast or piling.
2. Due to being sited in relatively shallow water, scour prevention must be given full consideration.

ABS has published "Guidelines for Building and Classing Offshore LNG Terminals" (December 2003).

A prestressed concrete LNG receiving and regasification terminal is currently under construction in Spain. It will be floated and installed in the Adriatic Sea offshore Italy (Figure 14.29).

Moored floating plants are especially well-suited for production, liquefaction, storage, and transfer of cryogenic gases because they can be relocated as necessary (e.g., when the gas reservoir is depleted).

The Arco Sakti Ardjuna LPG vessel was constructed of prestressed concrete in 1975 in Tacoma, Washington, and deployed in the Java Sea where it served until 2005, when it was taken out of service due to general corrosion, primarily of the steel equipage and systems.

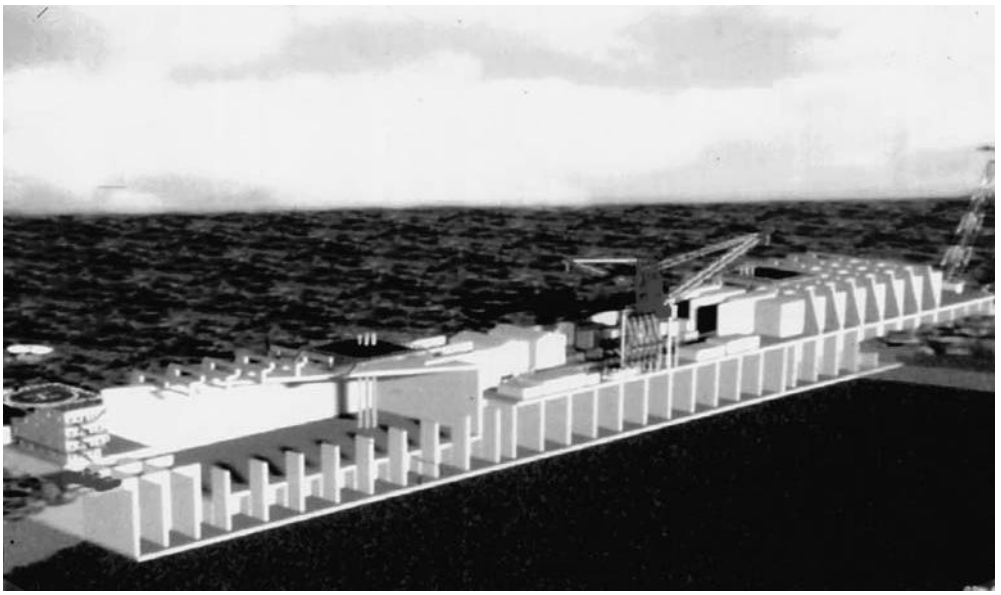

FIGURE 14.29
Offshore LNG Import Terminal (ALT) being constructed as GBS for installation in the Adriatic Sea. (Courtesy of Exxon-Mobile.)

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It employed precast match-cast segments for the curved hull bottom. The sides were cast-in-place, with the concrete pumped up from the bottom.

The vessel has been periodically inspected on station to meet the insurance requirements. Abrasion spalling has been caused by the mooring chains where they were attached. Corrosion of the aluminum anchorages for the post-tensioning tendons has occurred.

Large concrete floating plants for LNG production, liquefaction, storage and off-loading have been studied by international petroleum companies. Most of the studies are proprietary. The advantages of concrete have been the durability and the acceptance of inspection on station, thus eliminating time out of service and the time for taking tanks out of service for inspection. Most leaks in steel hulls have been caused by the warm-up and subsequent cool-down required for inspection of tanks.

Mobile has developed a design for a very large floating LNG production facility, employing a square layout in plan with an opening in the center. This is a deep draft concrete hull, which gives it stability. The orthogonal sides will be used for mooring of the off-loading tanker. The major hurdle is the construction due to the size of the hull. Conceptually, one plan is to construct one-fourth of the hull as a rectangle, and then launch. Four such sections would then be joined afloat in sheltered deep water (see Section 13.15).

14.9 Offshore Wind-Power Foundations

Those constructed as of 2005 are all located in relatively shallow water, with moderately firm soils. Hence, concrete GBS foundations, consisting of a base raft and shaft have been adopted. These relatively simple GBS structures have to go through the same considerations as their larger and deeper counterparts. Especial attention has to be given to scour under storm waves. The rocking action under storm waves may give rise to local liquefaction, increasing the potential for underbase scour. Skirts and articulated concrete block mats on filter fabric are potential solutions.

In deeper water and weak soils, large-diameter steel tubular monopiles may be driven. Steel abrasion–corrosion at the sand line is at a point of high bending stress. Hence, increased wall thickness and durable epoxy coating are indicated. Sacrificial anode cathodic protection should be considered. A reinforced concrete jacket may be slipped over the steel cylinder and jetted down using an O-ring seal to exclude soil, then grout injected into the annulus. Tripod and multiple steel structures spread the base so as to develop overturning resistance. Although they are easier to install, they are subject to greater erosion and corrosion. The zone near the pile head will be subject to high moment and shear under lateral loads.

14.10 Wave-Power Structures

A number of different solutions have been proposed by which to capture the energy available in waves. The problem with wave energy, as it is with many other forms of natural energy, is that it is widely dispersed and hence difficult to concentrate in a package of high volume plus high gradient, which is generally needed for commercial power generation. Locations in the surf zone must withstand the impact of storm waves.

One of the most successful is in Norway where the waves are captured in the throat of a wide V and then channeled by concrete walls to a tube at the focal point. This makes use of the Mach "stem effect" in which waves impacting a wall at an angle, run along the wall

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building up as they capture the energy. An adequate channel has to be provided to get rid of the water after it passes through the tube and generator.

Other concepts involve multiple small fixed or floating structures responding to the heave. Structures in the surf zone have to be able to withstand ware impact as well as abrasion from moving sands. This group of wave power generators, while able to generate a small amount of power in prototype tests, has not so far indicated significant potential.

A proposed wave power "farm" is proposed for installation off the coast of Portugal, in a water depth of 50–60 m. The generators will consist of hinged steel tubes, 150 m long and 3.5 m diameter, moored parallel to the prevalent sea. As the waves travel down the length of the hinged tubes, they will activate hydraulic cylinders which will pump high-pressure hydraulic fluid and in turn generate electric power.

14.11 Tidal Power Stations

There are many estuaries and sounds on the coasts that are subject to extreme tidal rise and fall, of the order of 5–0 m. A number of major projects have been proposed but only a few built: La Ranse in France, Kislogybusk in the Russian Arctic, and Annapolis in Quebec. The Chinese are reportedly undertaking several tidal power plants in the Bohai Gulf. These have all been equipped with low-head reversible turbines. In-depth studies are being carried out in the U.K. and U.S.A.

One of the problems with tidal power is the cyclic nature of the tidal cycle. One answer seems to be to have a number of these at various locations so that a few are always generating power. Other proposals have been put forward to use multiple basins to store the incoming tide surge and release it over a period of time. This has been proposed for tidal current power generation on the Severn. Extraction of energy from tidal currents at depth is under development, using large propellors mounted on vertical cylinder piles, driven in deepwater and extending to the surface.

14.12 Barrier Walls

The Ekofisk oil storage caisson was the first large offshore concrete structure installed in the North Sea. During the ensuing years of oil production from adjacent steel platforms, the seafloor subsided several meters and it became necessary to install a higher barrier outside and around the existing structure.

The barrier wall was fabricated in two semi-circular halves, each half of sufficient width to provide sufficient buoyancy. By careful control of ballasting, the semi-circular rings floated in a level attitude. One was towed to the site. It was positioned just outside the Ekofisk tank and ballasted down to the seafloor.

The second ring was then deployed to the site and ballasted down to an underkeel clearance of 2 m. Positioned slightly outside its final position, it was carefully rotated so that protruding hinges from the first half-ring meshed with those from the second rings. A large-diameter steel tubular pin was then dropped through. Then the other end was winched into corresponding enmeshment and a second pin dropped. This second ring now was ballasted down to the seafloor and the annuli of the pins was grouted.

This was a complex operation in the open sea. The deep draft, of course, minimized heave, and the day selected for operation had minimal swells. The method of hinges was selected partly because it allowed some differential heave prior to final fixing.

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Multiple steel pile dolphins have been proposed to protect Arctic offshore exploration structures in relatively shallow water. They have been closely spaced so as to build up a surrounding rubble pile of ice which will eventually ground.

14.13 Breakwaters

Large rectangular concrete caissons have been built to serve as the core of rubble-mound breakwaters. They usually are simple open-topped boxes, launched and floated to position. Once on site, they are seated adjacent to other similar caissons and filled with stone. Each is subject to six degrees of freedom in relation to its neighbor. The best method of joining is by short arcs of steel sheet piles, with the space between filled with rock. The sheet pile interlocks have sufficient play to accommodate the differences in attitude.

Attempts to provide structural connections using prefabricated members of steel or concrete have usually been unsuccessful because of the variations in attitudes.

Larger concrete caissons have been used as seawalls or breakwaters, with only minimal stone embankments. These have employed various devices to dissipate the energy of the waves. These include multiple holes connected to surge basins within the caisson, and in other versions, deflectors to turn the impact energy upward or back to the sea. However, the outward energy of the negative wave (the trough) must also be dissipated or the caisson will fail outward, the usual mode of failure. Rocking of the caisson will lead to build-up of pore pressures in the sand at the toe, leading to liquefaction and erosion. The Jarlan breakwaters, installed in the Mediterranean, have exit ports near the seafloor designed to cause build-up of sand at the toe rather than erosion.

Such caissons must be fabricated of very dense concrete in order to withstand local cavitation. They must also be constructed with minimum permeability and, where appropriate, air entrainment for freeze–thaw durability.

The Japanese have a number of different configurations and designs that have been installed in prototypes for evaluation. Most of these are designed to dissipate the energy of the breaking wave in a vertical jet, rather than in a reflected wave.

For the Costa Azul LNG plant near Ensenada, Baja California, Mexico, the breakwaters will consist of large concrete caissons, constructed in a basin and deployed to the site, where they will be seated and joined to each other by a shear key. The mating of adjacent caissons will have to contend with the prevailing swells.

For the Atlantic Generating Station, a floating nuclear plant proposed for installation off the coast of New Jersey, rectangular concrete caissons were to be floated out, seated, and filled with stone. Recognizing the difficulty in joining caissons seated on the seafloor, with their six degrees of freedom, a space was to be left between adjoining segments, to be closed on both sides by steel-sheet pile arcs, and then filled with stone.

Floating breakwaters are discussed in Section 13.14.

For I gazed across the ocean
Far as human eye could see
Saw the vision of the future
And all the wonders that would be
Saw the ocean filled with commerce
Fleets of wave-swept argosies
Pilots of the purple twilight
With their treasures of the seas.

Paraphrased from Alfred Lord Tennyson

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15

Installation of Submarine Pipelines

15.1 General

This Chapter will address the installation of submarine pipelines used for the transmission of petroleum products, gas, water, slurries, and effluents.

The diameter of steel submarine pipelines typically runs from 75 mm (3 in.) up to 150 mm (54 in.) with occasional lines running 1800 mm (72 in.). Diameters of steel pipe worldwide are often expressed in inches, even though all other units are metric; this is due to their historical tie to the U.S. petroleum industry.

The steel for these lines is usually of relatively high yield strength, 350–500 MPa (50,000–70,000 psi), and is selected for weldability. Wall thickness will normally run from 10 to 75 mm (3/8–3 in.) with the upper limit again being constrained by weldability.

Almost all steel pipelines have been joined by full-penetration welds, especially in the petroleum industry, where pressures typically run 1500 psi (10 MPa) and leakage of oil or gas is unacceptable. Consideration is being given, however, to the use of mechanical joints, for example, joints similar to those used with well casing. Developmental work continues on explosively and hydraulically expanded connections. In a few cases, flanged connections are used, but these are for lower pressures, which are used in off-loading of tankers.

Since most submarine pipelines are installed empty, they are subjected during installation to high hydrostatic pressure, along with whatever bending may be taking place. They are laid under axial tension. Buckling under combined loading becomes a principal design consideration. Tolerances are consequently of great importance, out-of-roundness and wall thickness being the most critical.

The steel is protected from external corrosion by coatings such as bitumastic or epoxy, supplemented by cathodic protection, usually sacrificial anodes. Internally, the line may be uncoated if it is to be in petroleum service, or it may be internally coated with epoxy, polyurethane, or polyethylene or cement lined when it will carry seawater or corrosive substances. The external coating may be further protected from abrasion by concrete or fiberglass wrapping. To give stability to the line when in service, especially those lines which must be emptied at some stage of their life or which carry a low-density material like gas, the line must have net negative buoyancy. This is usually supplied by concrete weight coating (which can also serve to protect the anticorrosion coatings) or by increasing the wall thickness of steel (see Figure 15.1). A number of pipelines have experienced "floating up" off the seafloor due to spalling and shedding of their concrete coat. This indicates that the reinforcing mesh may have been underdesigned or that the pipelines may have been subjected to excessive overstress during installation.

The latter is within the purview of this book. It appears that most such cases of damage have occurred during the more severe sea states when the pipe laying barge was subjected

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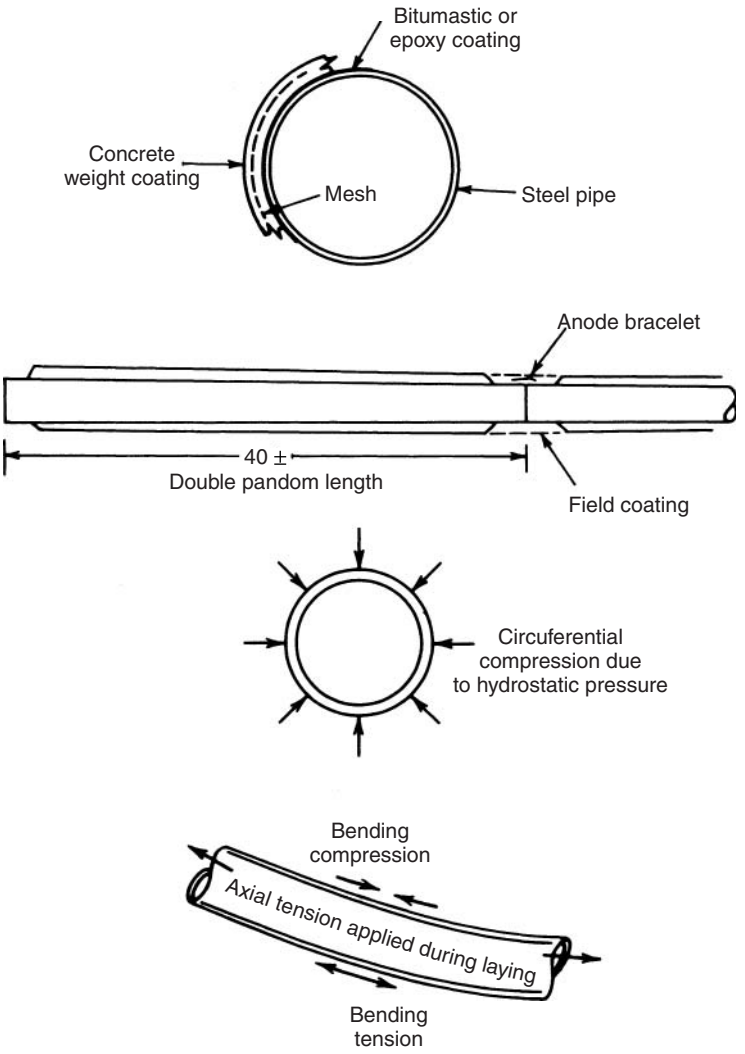

FIGURE 15.1
Typical steel submarine pipeline, showing stresses incurred during installation.

to severe dynamic surge. If the coating was not only cracked but delaminated from the pipe, then transient pore pressures under the storm waves break the coating off in progressive failure. This type of failure has occurred a number of times during pipe-pulling operations. The most obvious solution is to increase the amount of circumferential reinforcing in the coating. Since the coated pipe is usually furnished by the oil company, this obviously presents a contractual problem to pipeline installation contractors. Nevertheless, contractors may often find it in their best interests to verify the amount of circumferential reinforcing and, where necessary, recommend increased reinforcement.

Pipelines are basically designed to lie on the seafloor or in a trench in the seafloor, with more or less continuous support. However, unsupported spans may occur in rough, rocky seafloors or where the sands move under the action of currents and waves. The designer will have set limits on the unsupported span lengths, which the contractor must not exceed; this may require either prior seafloor leveling or post-installation support.

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Lines are buried beneath the seafloor in many areas of the world to protect them from fishing trawl boards, from dragging anchors, and from fatigue due to vortex shedding in a current. Usually, the trenches are backfilled with the excavated soil or covered with rock, but in many cases, natural sedimentation is counted on to fill the trench. Cyclic oscillation of pore pressures due to the passing of long period wave crests and troughs may "pump" the pipeline out of the trench. For this reason, pipelines in the North Sea are laid in trenches that are then filled with crushed rock.

As noted earlier, the pipeline usually sees its most severe stresses during installation; thus, very close integration is required between the designer and the installation contractor. The designer needs to be aware of and to address the needs of the contractor during installation. The contractor, conversely, must be aware of the limitations and constraints imposed by the installation procedures, taking into account the sea state (waves and current), the varying water depths, and the varying seafloor.

In addition, all parties must be cognizant of other pipelines, cables, and facilities in the area, recognizing the tolerances in location both of the previously laid lines and facilities, and the tolerances that are inherent in the contractor's procedure for the new line.

Submarine pipelines are typically laid in a "corridor" whose centerline and width are given by the client and shown on the approved permit. The installation contractor must have an adequate survey system to enable the contractor to comply. This system is usually an electronic positioning system or real-time differential GPS but may include lasers, ranges, and preset spar buoys.

The installer must verify to the satisfaction of the client and the regulatory body that the line has been satisfactorily installed. Externally, this is done by side-scan sonar and ROVs, using video or acoustic imaging. Internally, the line is pigged and then tested with hydrostatic pressure to a pressure in excess of the design pressure.

A pipeline "pig" is a short cylinder, of slightly smaller diameter than the pipeline, with several sets of squeegee wipers. When the pig is entered in the pipeline and excess pressure is applied to one face, it travels along the pipeline. The diameter of the pig and its length verify that there is no dent, crimp, or buckle more than the small annular space. The squeegees restrict the loss of pressure yet allow the pig to move. The pig is usually equipped with an acoustic transponder or radioactive marker so that if it does get stuck, its position can be determined. For short lines, an umbilical line "fishing line" can be unreeled behind it (see Figure 15.2). Guidance for the design and installation of submarine pipelines is given in the DNV Rules for Submarine Pipeline Systems.

Many methods of pipe laying have been employed, selected on the basis of environmental conditions during installation, availability and cost of equipment, length and size of line, depth of water and constraints of adjacent lines and structures. The following are those most commonly employed:

1. Conventional S-lay barge (see Section 15.2)
2. Bottom-pull method (see Section 15.3)
3. Reel barge (see Section 15.4)
4. Surface float (see Section 15.5)
5. Controlled subsurface float (see Section 15.6)
6. Controlled above-bottom pull (see Section 15.7)
7. J-tube from platform (see Section 15.8)
8. J-lay from barge (see Section 15.9)
9. *S*-curve with collapsible buoyancy (see Section 15.10)

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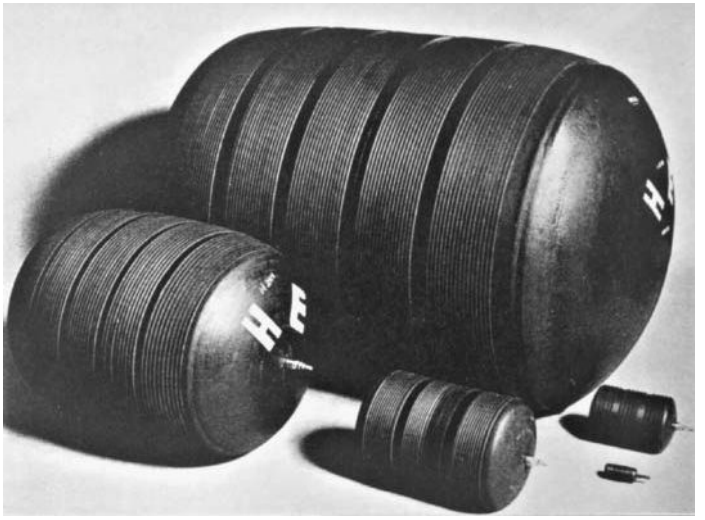

FIGURE 15.2
Pipeline "pig." (Courtesy of Oil States Rubber Co.)

These lines have almost all been installed empty, so as to reduce the weight. The exceptions have been small-diameter lines in relatively shallow water. When laid empty, the pipe must have an adequate wall thickness to withstand the combined stresses, that is, longitudinal installation force, plus the circumferential hydrostatic force, plus bending. These will be described in the following sections.

15.2 Conventional S-Lay Barge

The offshore lay barge has grown up from the specially modified cargo barge of the 1950s to become one of the most sophisticated, efficient, and expensive vessels in the world. Lay barges are often characterized as first-, second-, third-, and fourth-generation to denote major "quantum jumps" that have been made in extending the ability to lay lines in deep water, with current achievements being the successful installation in depths over 1700 m in the Gulf of Mexico and in such adverse environments as the North Sea (see Figure 15.3). Also, See Chapter 22, Section 22.10.

First-generation lay barges have a conventional barge hull, with the pipe-laying assembly mounted on one side. The stinger is hinged but rigid. The inclination of the stinger is controlled by buoyancy tanks at the outer end. Second-generation lay barges have a semisubmersible hull, with the pipe laying assembly on one side and an articulated stinger. Third-generation lay barges lay pipe on the centerline, over a fixed cantilevered stinger. First-, second-, and third-generation barges all have deck engines and mooring lines. Fourth-generation lay barges use dynamic thrusters and a fixed cantilevered stinger. They are usually equipped for both S-lay and J-lay operations. These arbitrary distinctions are descriptive of the rapid advances that have been made in pipe laying technology.

The lay barge is a system that comprises the following principal operations and systems:

1. Seaborne work platform vessel
2. Mooring and positioning systems, either lines or dynamic positioning
3. Pipe delivery, transfer, and storage facilities

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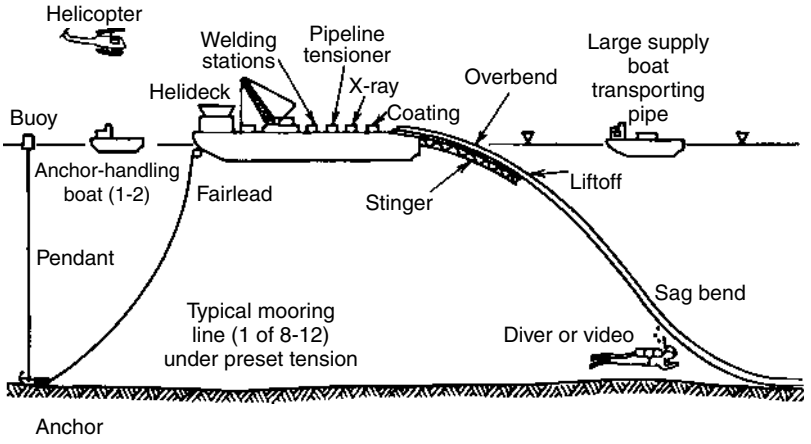

FIGURE 15.3
Typical lay barge operation.

4. Double-ending of pipe, conveying to lineup station, and lineup equipment
5. Welding of joints
6. X-ray
7. Joint coating
8. Tensioning of line during laying
9. Support of line into water either by "stinger" or cantilevered ramp
10. Survey and navigation
11. Anchor-handling boats
12. Communications
13. Personnel transfers—helicopter and crew boat
14. Diver or ROV for underwater inspection
15. Control center
16. Crew housing and feeding
17. Power generation
18. Repair facilities and shops

A typical second-generation lay barge is shown in Figure 15.4. The layout of equipment is shown in Figure 15.5 through Figure 15.7.

The basic operations of the lay barge can be outlined as follows:

1. The lay barge is positioned on its anchors, eight to twelve in number, holding it aligned with the pipeline route, with a "crab" or slight orientation angle as needed to accommodate the effects of the current. Its position is determined by an electronic positioning system or GPS, augmented by laser in some cases. Its orientation is by gyroscope.
2. The anchors will be progressively moved forward as the laying takes place, usually in 500–600 m jumps. One anchor-handling boat on the starboard side will move each anchor ahead in succession; another anchor-handling boat will move each of the port anchors ahead in succession (see Figure 15.8).

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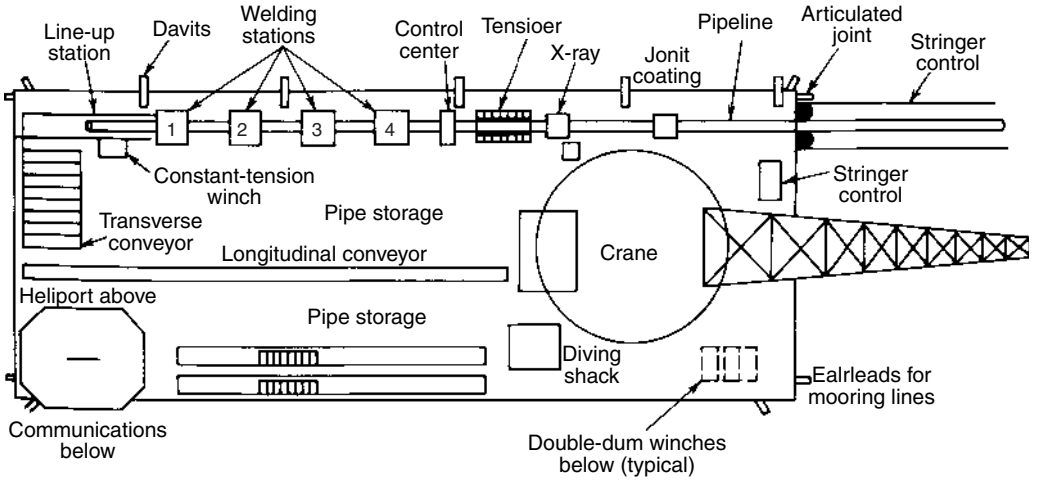

FIGURE 15.4
Second-generation pipe-laying barge.

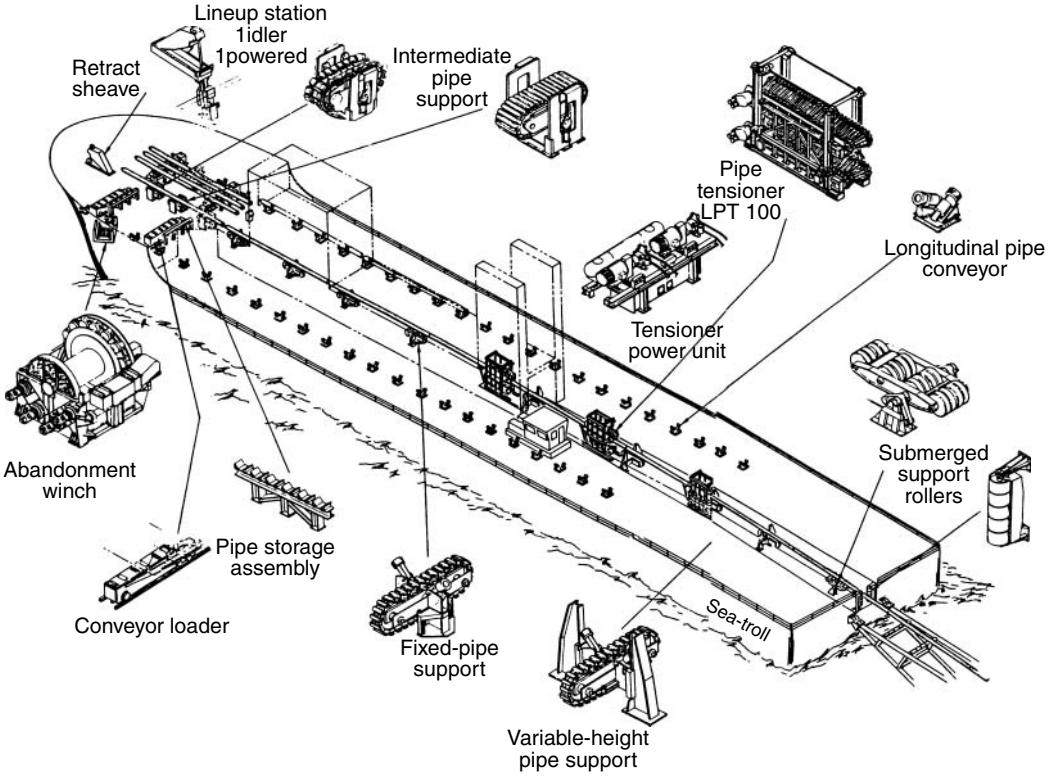

FIGURE 15.5
Layout of equipment on third generation lay barge. (Courtesy of Western Gear Corp.)

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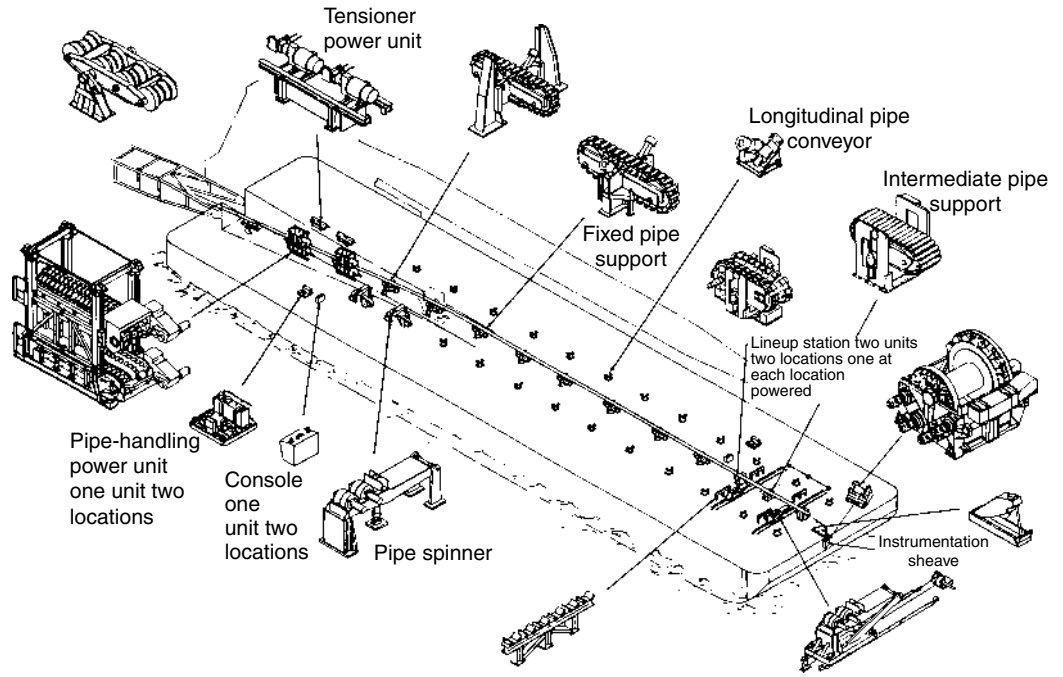

FIGURE 15.6
Equipment on third generation lay barge. (Courtesy of Western Gear Corp.)

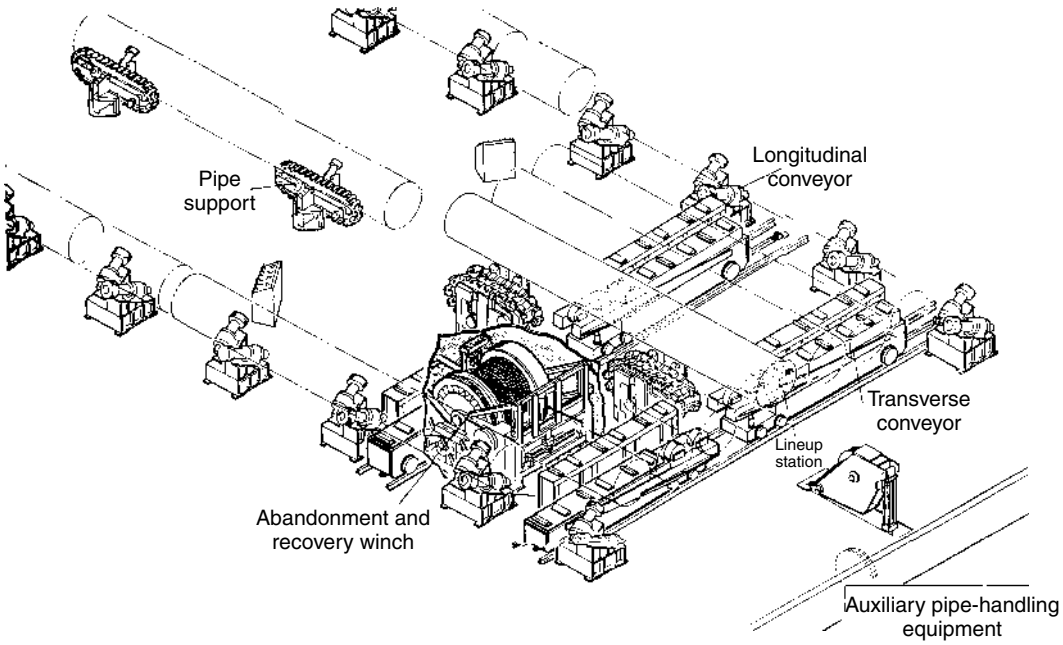

FIGURE 15.7
Arrangement of conveyors and winches at line-up station. (Courtesy of Western Gear Corp.)

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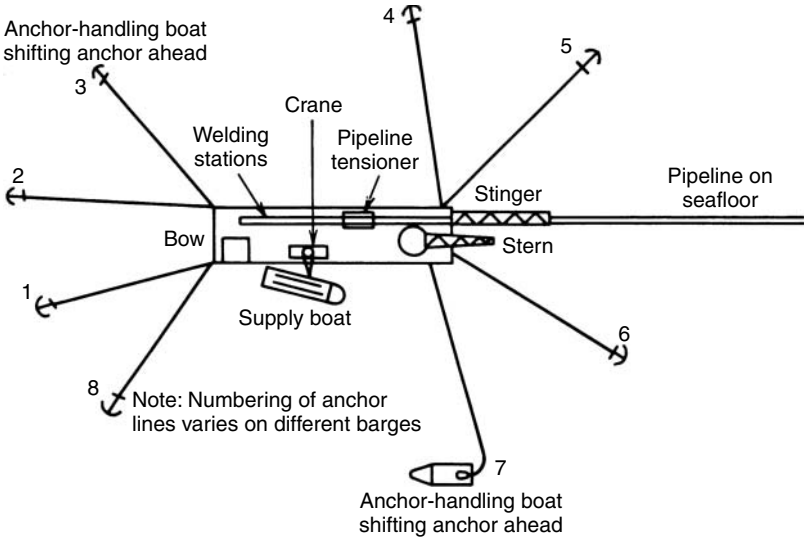

FIGURE 15.8
Typical lay barge operational spread. (Courtesy of Western Gear Corp.)

Typically, the anchor-handling boat maneuvers close to the anchor buoy to enable the deckhand to hook an eye in the end of the pendant. The deckhand attaches a wire line from the deck engine of the tug, which either pulls the buoy aboard or pulls the pendant through the buoy, thus lifting the anchor clear of the bottom 5 m or so. The boat then runs forward, setting the anchor as directed in its new position and releasing the buoy. The boat turns outboard and goes back for the next anchor in the cycle. The new position of the anchor is given by voice radio command from the control house, which is based on radar, gyro, and the reading on the remote mooring line length counters, reading the line length paid out by the winch.

Anchor handling is a very dangerous operation. Hydraulically operated ramps and booms have been developed to enable these operations to be carried out safely.

The proper paying out and taking in of each mooring line on the winch drum is monitored by video in the control house to ensure against crossed lines on the drum or fouling of the line.

3. From a supply boat or barge alongside the port side, the crawler crane on the lay barge snags (picks) one pipe length (12 m) at a time, turns, and sets it in storage. From storage, the crane picks a pipe length and sets it on the end-O conveyor, which moves it to the transverse conveyor at the bow. This conveyor feeds it onto the lineup station, where it is positioned, usually semi-automatically, in correct alignment and then run forward to the end of the preceding segment (see Figure 15.9).
4. The internal lineup clamp positions it in exact spacing and holds it for the hot-pass weld.
5. The hot-pass weld is made and ground or gouged (see Figure 15.10).
6. The segment moves forward successively to weld stations 2, 3, and 4, with one or more passes being applied at each station, and then chipped or gouged.
7. The fully welded line now passes through the tensioner, where it is gripped by polyurethane cleats on caterpillar-like treads. Hydraulic rams push the pads against the coating, adjusting their pressure so as not to deform the pipe or crush

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FIGURE 15.9
Pipe lengths stored on barge.

the coating, while still developing frictional resistance. The tensioners run on torque converters or similar devices to pay out under a set tension. This tension or tensioner typically has a rather wide tolerance on external pipeline diameter (see Figure 15.11).

8. The joint now goes to the x-ray station, where it is x-rayed, and the films are developed and checked. If a flaw is found, it must be cut out, re-welded, and re-x-rayed. For a cutout, the barge must be moved astern and the line brought back up on board one or two lengths so that the cutout is forward, i.e., on the untensioned side, of the tensioner.

FIGURE 15.10
Grinding the welds.

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FIGURE 15.11
Pipeline tensioners.

9. The pipe section now moves astern, where the joint is coated with the special corrosion-protective coating. A bracelet of zinc–aluminum or other anode is affixed. Concrete mortar coating is applied to protect the corrosion-protective coating at the joint. This fresh concrete is protected by a sheet-metal wrap-around (see Figure 15.12).
10. The completed pipeline now passes down the ramp and over the stern of the barge and bends downward. This downward bend is called the "overbend" (see Figure 15.13).
11. The line rides down the stinger or ramp to a point of departure, where it leaves the stinger due to the tension in the line. The stinger has a hinged connection to the barge. It has built-in flotation to support the pipeline while still allowing a downward inclination and some flexibility to accommodate surge. The stinger may be articulated to permit continuous curvature or may have a fixed vertical curve. Load cells on the roller supports, plus depth indicators such as bubble gauges, enable the stinger to be ballasted for optimum support.
12. The line now moves downward through the water and bends back to the horizontal at the seafloor. This bend is called the "sag" bend. At this bend, the pipeline is usually subjected to its maximum stresses and potential buckling due to the combined axial tension, vertical bend, and circumferential hydrostatic pressure.
13. As the line lays out on the seafloor, its integrity is checked either by divers and video or by ROV.

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FIGURE 15.12
Coating the joint.

From the above sequence, it can be seen that the typical lay barge system described at the beginning of this section has the following physical components:

- Lay barge
- Anchor-handling boats (usually two)
- Supply boats (usually three) or supply barges (usually two) with tug
- Helicopter service and/or crew boat
- Shore base
- Pipe storage racks
- Pipe conveyors
- Lineup station
- Internal line up device and clamp
- Welding stations
- Tracked tensioner
- X-ray equipment
- Joint-coating equipment

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FIGURE 15.13
Pipeline in overbend, passing down stinger.

- Constant-tension winch for abandonment and recovery
- Stinger and stinger control
- Winches with mooring lines
- Control room
- Radio circuits to shore and boats
- Voice and indicator circuits to welding stations, stinger control, and x-ray
- Gyrocompass
- Radar
- DGPS and/or electronic positioning
- Tensioner force readout
- Mooring line tension readout and video display
- Mooring line length-out readout
- Diver shack
- Decompression tank

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- Heliport
- Quarters for crew
- Mess hall and kitchen
- Office
- First-aid and medical facilities
- Owner's quarters and office
- Repair shop
- Power plant
- Fuel and water storage
- Store rooms for slings, shackles, etc.

The crew required to operate an offshore pipe-laying vessel may be 150 or more per shift. Normal operations use two twelve-hour shifts. A third shift will be off on leave. Work schedule is usually two weeks on, one week off, or one week on, one week off.

Tension is maintained in the pipeline from the barge to the seafloor in order to reduce the vertical bending and the tendency to buckle. Values of applied tension range from a low of perhaps 100–150 kN (20,000–30,000 lb) in shallow water and calm seas to 300 kN (70,000 lb) or more in deep water and rough seas.

The lay barge is subject to dynamic surge motions, depending on the relationship between wavelength, barge length, and depth of water. This surge is usually too fast for the tensioner and the welder to follow. Thus, under low sea states, the pipe is locked in fixed position in relation to the barge. Therefore, the tension in the pipeline varies cyclically about the steady-state force. Typical ranges of variation are of the order of 100 kN (20,000 lb) each way in a moderate sea. Heave and pitch also have some effect on the tension, but generally to a much lesser degree than surge. This tension must also be introduced and maintained during the startup and lay-down of the pipe. The skill of the welders is critical to the operation. They are working on a rolling and heaving barge, often with a moving joint, yet must produce essentially perfect welds. They must be protected from spray and rain and must have adequate light and ventilation. If the x-ray discovers a flaw in the weld, the resultant cutout repair stops the entire operation until it is completed.

The actual performance of the welds is also of serious concern to the pipeline installation contractor, because of the responsibility to ensure a sound, leak-free pipe on completion. The combination of axial tension and overbend stresses on the weld are very severe, especially since the latter is dynamic. Not only the toughness of the weld itself is involved, but also that of the heat-affected zone (HAZ), which in turn is influenced by the parent steel quality as well as the welding procedures. The constructor may therefore find it prudent to test the pipe steel and welding procedures under dynamic tension loads prior to finalizing procedures.

In a typical offshore operation, the barge will move one pipe length every fifteen minutes. On the most modern third-generation barges, using advanced welding techniques and double- or triple-ended pipe joints, rates of a mile per day are achieved. This means that all the work must be completed at each station within that same time frame. This translates to 100 or more 12 m lengths per twenty-four-hour day. These performances have been exceeded by topnotch crews on good days, even with manual welding.

Stresses in the pipe in the laying operation are controlled not only by axial tension but also by the net submerged weight of the pipe. This latter is the difference between two large numbers, the one being the air weight of the pipe, the other being the buoyancy due

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to the displaced volume. The major variable is the thickness of mortar coating, which affects both air weight and displacement, but not equally.

In a typical case, a pipe may have an air weight of 15 kN/m (1000 lb/ft.) and a displacement of 14.3 kN/m (950 lb/ft.) leaving a net (buoyant) weight of 0.7 kN/m (50 lb/ft.). If the coating increases the weight by 5%, the displacement may increase by only 2%. These numbers may sound small, but they develop an increase in net buoyant weight of 0.45 kN/m or a 60% increase in the force causing the bending. Thus, while weight control is normally not as critical with lay barge operations as it is with bottom pulls, it nevertheless is of great importance and must be monitored.

The pipe is generally furnished in double-random lengths, which are normally 12 m (40 ft.). Most sections will run 11.4–12.6 m. However, generalized pipe procurement specifications allow a few sections which vary widely from the norm, even as short as 3 m and long as 17 m. While this may be accommodated on land pipelines, it is unworkable at sea. Such sections should be cut or spliced to the normative length of 12 m (40 ft.) at the shore base, or else the procurement order should exclude these variances. Experienced pipeline contractors will require that the pipe be furnished in 40 foot (12 m) lengths, plus-or-minus 2 ft. (0.6 m).

As previously noted, rates of progress with third-generation lay barges may reach one mile per day or more. This means that one hundred or more sections must be loaded out each day from the shore base, transported to the site, and then unloaded to the deck of the barge. This last is a critical operation when the seas are running high and may, along with anchor handling, be the controlling operation. The transfer at sea of the pipe is a typical case of operations involving two vessels alongside each other, of different characteristics, each responding in its own way to the seas, in each of six degrees of freedom. The relative positions in plan can usually be maintained in a moderate sea state by tying the transport barge alongside the lay barge, with suitable fendering, so that the major individual responses are limited to heave, roll, and pitch. In heavier sea states, barges can no longer be kept alongside, and so supply boats are used. By running a line from the boat and keeping power on, a good skipper can hold the boat in reasonably close position, although now the boat will develop some relative sway, surge, and yaw motions as well as heave, pitch, and roll (see Figure 15.14).

The typical lay barge is restrained from lateral motion by the mooring lines; it is also moved periodically one pipe length ahead. These lines, while catenary in scope in deep water, are kept under tension by the winches. The line tensions are measured by tension meters on the wire rope or on the winch drum or both. In the typical second-generation lay barge, the tension may be 400 kN (80,000 lb) with a variance in a moderate sea of ± 100 kN. This variance is due to the long-period sway plus surge built up by the waves, storing energy in the wire lines as the barge gradually moves to one extreme of its transverse range. The lines on the far side gradually become more taut, so that eventually the barge changes direction and starts its sway excursion to the other end of the range. The sudden reversal at the end of its excursion causes a shudder effect in the overall system, and translates into a severe horizontal whip of the stinger and of the pipe. The surge excursions cause cyclic bending in the pipe at the overbend and in high sea states can lead to low-cycle fatigue in the pipe.

The mooring lines must provide the horizontal and longitudinal restraint against wave drift, wind drift, and current drift. They also react against one another and especially must counter the tension on the pipe, which in effect is like a mooring line of relatively equal tension, leading directly astern. Balancing out the tensions in eight to twelve mooring lines plus one pipeline is a complex problem, especially when these line forces are not steady but subject to the significant ranges introduced by the long-period excursions.

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FIGURE 15.14
Third generation pipe lay barge. (Courtesy of Exxon.)

Typically, the tensions in the mooring lines are set so that under the maximum design surges, the force will not exceed 50%–60% of the guaranteed minimum breaking strength.

To offset the pipeline tension requires additional mooring line forces in the lines leading forward. The system must be balanced, which is difficult enough with one positioning of the anchors, but which is rendered more complex due to the constant lifting and relocation of anchors. The system can be satisfactorily resolved by preparing calculations of typical and extreme positions for each permutation; it, of course, lends itself to the use of an on-board mini- or microcomputer, which can then solve for intermediate situations. The most modern lay barges (of this generation) use dynamic positioning by means of thrusters to maintain lateral position and heading. These are controlled by computer and connected to the GPS system. However, reaction lines to the pipe tension, usually two forward leading lines, are usually still required because of the large forces involved when laying by the *S*-curve method, especially in deep water for which high tension is required in the pipeline. Dynamic thrusters eliminate the long-period sway and the consequent kick back of the barge, making it practical to continue welding operations in higher sea states.

The cost of pipe laying is related to the progress, since the cost per day is more or less the same whether any pipe is laid or not. The rate of progress has until recently been controlled by the time required for welding. There is a specific amount of weld metal, which must be applied. Only two welders (one on each side) can work at any station. Therefore, the rate of progress depends on the number of stations. Typically, these are placed one pipe length (40 ft.) apart. There is only room enough for a certain number of welding stations on a barge; therefore, the longer the barge, the greater the rate of progress. This explains why prior double-ending of the pipe does not speed the operation. Another means of accelerating the welding is by the use of microwire welding, but this is usually only acceptable in hot climates because of the dangers of cold lap at lower temperatures.

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The biggest jump in pipe-laying progress has come with the introduction of automatic welding of one type or another. Dual-torch automatic welding equipment, riding on a self-propelled carriage, can complete quality welds at a high rate, even in rough seas. The operation is fully computer controlled.

Second-generation lay barges are limited by the sea state. When the significant wave height exceeds about 8 ft. (2.54 m), operations must shut down. The specific limit, of course, depends on the relative direction and the period of the waves, as well as the barge length and width. The limiting item is usually control of surge and the interaction between stinger, pipeline, and barge. The working limits can be increased by using a wider and longer barge, by using more powerful tensioners, and by using a fixed cantilever stinger.

When seas reach 10–12 ft. ($H_s = 3$–3.5 m), other constraints arise. Anchor-handling boats can no longer pick up the anchor buoys, although this limit has been extended by clever arrangements enabling the boat to run past the buoy and snag it rather than having to back down for the deckhand to make fast to the pendant. Pipe transfer from a barge alongside is no longer practicable with an H_s of 2–2.5 m, but a supply boat can be used to extend this operation to the 3- to 4-m range.

The barge motions in roll and long-period sway (snapback) become too severe in higher sea states, especially with a beam or quartering sea, and the welders are unable to produce quality welds. The pipe starts to jump out of the stinger, and there is danger of buckling the pipe. Even with dynamic positioning, the long-period surge causes severe variations in the pipeline tension and profile.

At this stage, a decision must be made whether to hold on or to initiate abandonment procedures. The major factor here is the weather prediction. If improvement is forecast within the next few hours, it may be practicable to hang on, maintaining tension. Another factor is whether or not the anchors will hold in the seafloor soils or are likely to drag; a dragging anchor will almost always lead to a buckle.

When abandonment is decided, a bull plug (cap) is welded onto the pipe. A line from the constant tension winch is attached. A buoy and pendant are also attached to the bull plug. It is a good precaution also to attach an acoustic pinger to the bull plug. The barge then moves ahead, paying out on the line, until the pipe is fully lying on the seafloor. The end of the constant-tension line is buoyed and run off. The barge can now pick up its anchors to move to a sheltered location or decide to ride the storm out at sea, on its anchors, but turned now to head into the sea.

When the storm ends, the barge moves back to location and resets anchors. While one hopes to find the buoys, it is not unusual for them to have been torn away by the storm. That is when the acoustic pinger is needed.

The constant-tension line is now pulled on-board and the tension applied. The barge slowly moves astern, bringing the pipeline back up onto the stinger. A line from the crane may have to be hooked on (by diver) to help guide the line back onto the rolls of the stinger without fouling. Now the pipe is pulled on board, through the tensioner, until the bull plug reaches the lineup station; the bull plug is cut off, the pipe end re-beveled, and the laying operation recommences.

An important consideration is that abandonment procedures are almost always carried out under extreme conditions, at or above working limits, whereas recovery operations will normally be carried out in good sea conditions.

Earlier, it was stated that the most serious problem in pipe laying is a wet buckle. In the case of a dry buckle—that is, where the line does not take on water—the pipe can be just pulled back up on board. In the case of a wet buckle, however, the pipeline has been flooded and cannot be brought back on board without leading to progressive buckling. For this reason, at start-up, at least one pig was placed in a pig chamber at the start-up end, along with air fittings. If a wet buckle occurs, compressed air lines are connected and the

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pig run along the pipe to the point of buckle. This empties the line so that it can be recovered.

Actually, there is one even more serious case, known as a propagating buckle. This is the case where the ovaling of the pipe at the point of initial buckle reduces the collapse strength below the resistance to the external hydrostatic pressure so that the buckle travels back along the pipe. While this case is usually within the province of the designer, the constructor must make sure that this cannot occur; else the entire line could be lost. Where calculations show this to be possible, buckle arrestors in the form of thicker pipe or reinforced pipe are installed at intervals of 1000 m or so. For example, wraparound plates may have been pre-installed. Sleeve-type buckle arrestors may be installed by fusion welding to the ends of a 12 m length of pipe or a thicker walled pipe segment used.

Occasionally, a line may be damaged after it has been successfully laid down. Often, this is due to an anchor dragging into the pipeline. It may even be an anchor from your own spread, that is, the barge or boats, or it may be from another contractor working on the same platform. The line must be repaired. One method is to use a hyperbaric chamber (a "habitat") lowered down over the line and centered on the junction or repair point. Compressed gas is used to expel the water, and divers descend to make the weld in the gas atmosphere. The selection of the appropriate gas mixture is critical in order to ensure the proper weld quality. Such a repair in 100 m of water, for example, may require several days. It may be tended by the lay barge, but if the sea state permits, a smaller support vessel may be used.

The repair procedure consists of accurately cutting the lines and beveling their ends. A template is made to ensure an exact fit of a "pup" (a short, specially cut pipe section), which is fabricated on board and lowered down for welding. After the welds are completed, the joint is coated for corrosion protection. X-ray is usually not practicable, and reliance must then be placed on visual inspection and magnetic-particle or other NDT techniques to verify the quality of the weld.

Wet-welding techniques have been under development for many years; the problem, of course, is ensuring a weld that will be reliable under its working pressures, which typically are 10 MPa or so. At the present time, there is not universal confidence in the quality obtainable by wet-welding techniques, but development continues. Wet-connector devices are now commercially available, permitting some repairs to be made underwater mechanically.

In shallow water, the damaged section can be brought to the surface for a dry weld. The line should be empty, if possible, and a long length brought up so as not to exceed curvature limits in the pipe. For this reason, many shallow water lay barges are fitted with davits along one side, enabling lifting from the entire length of the barge. The derrick crane may also pick from the stern while the pipe transfer crane picks from the bow. Where this curvature is still too great, floats or buoys may be attached to give positive buoyancy along appropriate lengths of pipeline.

As the line is brought up, there are, of course, length-compatibility problems in all but very shallow water. It is sometimes necessary to cut the line, thus flooding the pipe. Once brought to deck level, the ends are beveled, a pup fabricated and installed, and the line laid back. Now the new line is longer than required, so it must be laid back to lie in a horizontal curve on the seafloor. The pig is now run to empty the pipe.

Installation of risers at platforms is another special operation requiring careful preplanning of each stage. There are a number of methods which have been used successfully (see Figure 15.15):

1. The riser is pre-attached to the side of the platform. The end of the line, pre-laid on the seafloor but still empty, is pulled over to that same side of the platform by

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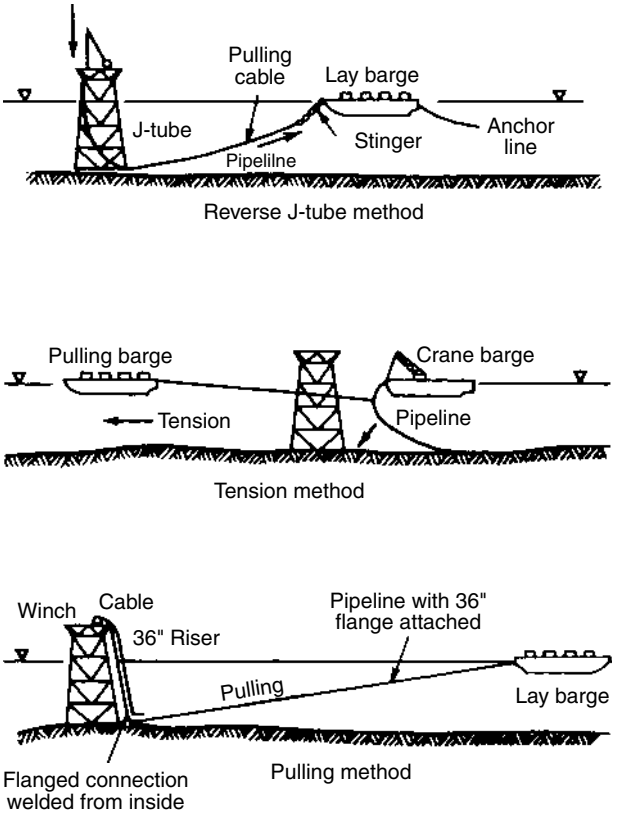

FIGURE 15.15
Riser installation methods.

means of a line rigged to maintain axial tension. Then, divers make a template of the intervening space and a pup is prefabricated and installed using a hyperbaric chamber lowered over the joint so that welds can be made.

Alternatively, with large-diameter pipe (e.g., 42 in. or greater), flanged connections may be used. The line is cleared of water, using a pig if necessary, and a welder descends in the riser to weld the joint from the inside.

Hydrotech has developed a two-piece diagonally flanged coupling which can be sleeved over the two pipe ends and then rotated to accommodate a difference in angle. A three-piece coupling can accommodate up to 15° misalignment. After the joints are bolted up, they are seal-welded, using a dry chamber filled with inert gas.

Vickers has developed an explosive-welding method which is especially suitable for tie-ins between pipelines and risers. The explosion is initiated inside the pipe, forcing it out against the sleeve to give a solid intermolecular bond. The reliability of this method has not yet been fully accepted.

Advanced "pull-in and latch" connectors have been developed for deep-water connections.

2. In shallow water, the line is picked up by the davits along the starboard side. The derrick picks the riser so that it hangs just off vertical, at the proper angle to the pipe. The welded joint is made. The riser and line are then lowered back down to the seafloor, the riser coming into position along the jacket to which it is now

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clamped. In moderately deep water, it may be necessary to add on to the riser from time to time, the so-called "stovepipe" operation. As the riser and pipeline are being lowered, the riser is stopped off from the position and a new length of riser added.

3. For smaller lines, such as flow lines, J-tube risers are built into the platform. The laying is started from the platform; the pipe is pulled off the lay barge and up into the riser tube by a line from the pipe and to a winch on the platform deck. The J-tube bends the pipeline in a permanent but controlled deformation around the 90° bend and then straightens the line with a small reverse bend.
4. For deeper lines, risers are pre-installed on the platform. Alongside is a riser pull-in tube. In some cases, a line may be led out through the riser. This line is then run to the lay barge. As the laying starts, the pipe end is pulled off the platform and to the mating joint, using a winch on the platform to pull in the line. Initial connection is made by bolted flange, followed by internal or external welding as described earlier.

For lines to be run ashore, there are also several alternatives:

1. A line may be separately pulled out from the shore through the surf zone. The lay barge now moves in just seaward of the end of the pulled line. With a line from the barge exerting axial tension, the shore line is pulled on board into the tensioner and the new pipe sections welded on. Now the standard laying can commence.
2. The lay barge moves into as shallow water as is safe. A wire line is run ashore to a winch on shore. As the lay barge makes up pipe, the winch on shore pulls the end to the shore. Then, the lay barge proceeds with its standard pipe-laying procedure.
3. The lay barge lays from the platform toward the shore. When it reaches shallow water, it lays the end of the pipeline down, then turns itself around and resets anchors. It now pulls a line out from shore. Using the davits, it picks up the end of the previous line, joins the two ends by welding, and relays the line on the seafloor in a horizontal curve to accommodate the slightly excess length.

Third-generation and later lay barges are indeed highly sophisticated systems, enabling pipelines to be laid in more severe sea states, up to H_s of 5–6 m, and in deep water, up to 600 m and potentially more. Among the most advanced are the SAIPEM Castoro Sei, which successfully laid the lines from Tunis to Sicily, and the SEAMAC, now renamed the Bar 420, which laid the 36-in. FLAGS lines in the North Sea in record time (see Figure 15.16 and Figure 15.17).

Third-generation lay barges are more advanced:

1. A stable platform is provided, generally being a semisubmersible but in a few cases, a very long (over 200-m) ship-shape vessel.
2. The stinger is now fixed to the stern of the barge and cantilevered out behind in a long curve.
3. The pipeline is laid down the centerline, not down the side.
4. Higher tension is provided.
5. Advanced welding systems are employed to speed the welding process.
6. Dynamic positioning, using computer controls and GPS, is now utilized to control lateral positioning.

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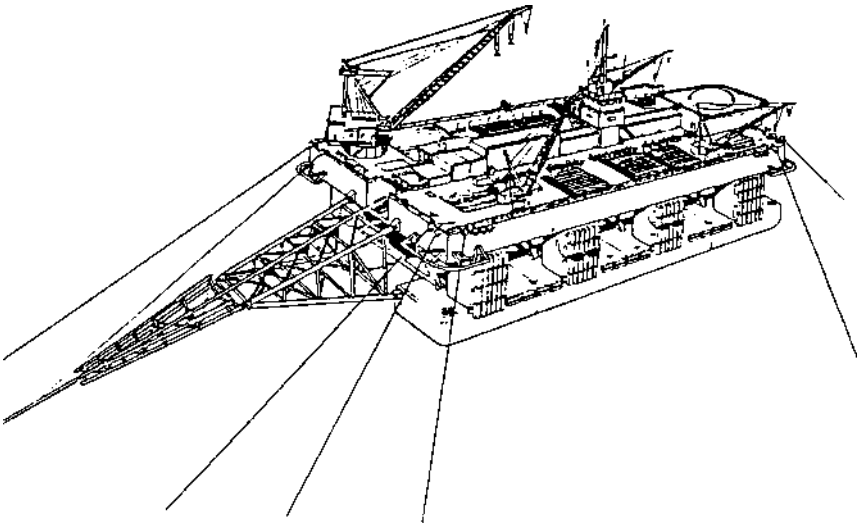

FIGURE 15.16
Third generation lay barge, SEAMAC. (Courtesy of Exxon Exploration & Production.)

Fourth-generation lay barges incorporate the above improvements and in addition, lay in a near-vertical attitude.

This is the J-lay method. The pipe lengths are double-or-tripled-jointed on board, then hoisted into a set of leads inclined to about 75° from the horizontal. The pipe is then

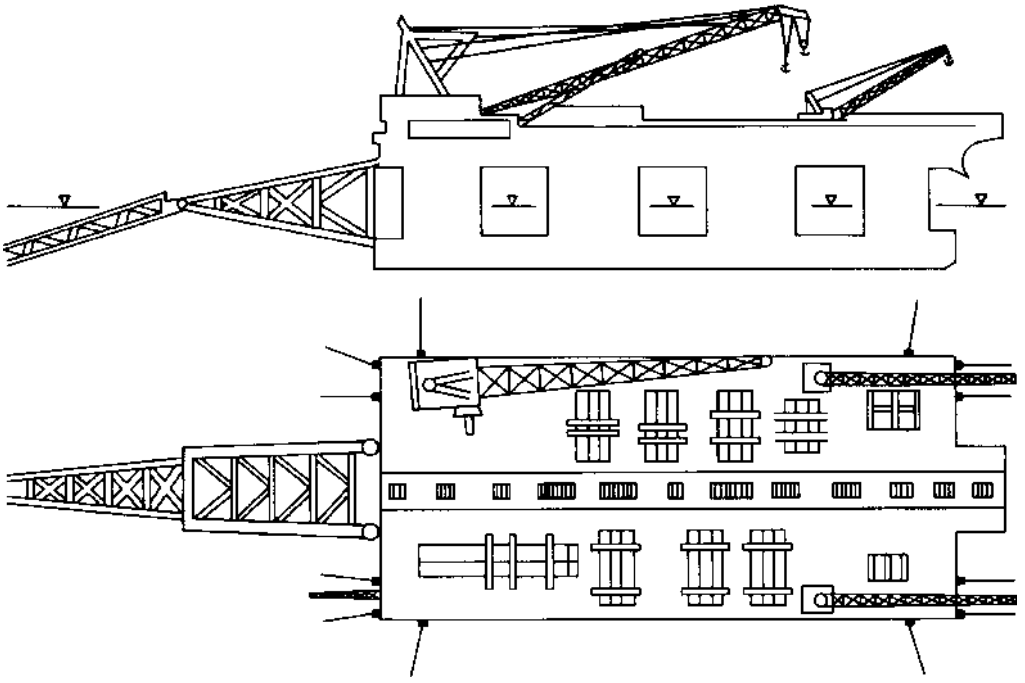

FIGURE 15.17
SEAMAC, third generation lay barge. (Courtesy of Exxon Exploration & Production.)

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lowered to contact at deck level with the previously laid line and the weld is performed by an automatic welder. The line is then lowered through a tensioner into a moon-pool until it reaches near the seafloor where it deflects to the horizontal in a long sag bend (see Section 15.9). Dynamic positioning is employed, thus eliminating the problems of anchor relocation in deep water and the dynamic surge effects of energy stored in the mooring lines.

15.3 Bottom-Pull Method

The bottom-pull method has been developed and extensively used to install pipelines through the coastal zone, to extend out to loading terminals in deep water. It has been further developed in recent years as a means of installing relatively long lines in deep offshore areas.

Initial discussion will be directed to those lines that extend from shore out a distance of several thousand meters. The program is as follows:

1. The pipeline is assembled on shore in parallel segments of 200–300 m in length.
2. A launching ramp with roller supports is constructed, leading out through the inner surf zone.
3. The inner surf zone may be protected by a sheet pile cofferdam so that a trench will stay open.
4. The first 200- to 300-m length of pipe is made up on the launching ramp, with joints welded and coated (see Figure 15.18). Since the ramp is sloped to seaward, the pipe is restrained from longitudinal movement by a holdback winch at the landward end (see Figure 15.19). The seaward end is fitted with a nose section, consisting of pig storage for one or two pigs, a positively buoyant nose, and a swivel. A sheave may be attached seaward of the swivel, with supports or a buoyant tank to keep the sheave from flipping over during the pull (see Figure 15.20).
5. A pulling barge is anchored offshore, on line, at a distance of 1000 m or so.
6. On board, a very large winch is installed, one or two drums, having high pulling capacity, for example, 1350 kN (300,000-lb) line pull on a full drum (see Figure 15.21). This winch is connected by wire lines around equalizing

FIGURE 15.18
Pipeline made up and joined on loading ramp. Note rail cars to reduce friction while pulling out. (Courtesy of H.V. Anderson Engineers.)

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FIGURE 15.19
Attaching the hold-back lines to prevent premature launch. (Courtesy of H.V. Anderson Engineers.)

sheaves to two bow anchor lines, with large anchors set well out to sea (see Figure 15.22). If the deck of the barge is utilized for this transfer of reaction force, it may have to be reinforced or struts installed.

7. The winch line is now run ashore and connected to the nose of the pipeline. If two parts of line are to be used, the line is run around the sheave and back to the barge. Proper fairlead guides are used where the line runs off the edge of the barge in order to prevent chafing and wear.
8. When all is ready and the weather forecast is favorable, the first section of line is pulled out through the surf zone (see Figure 15.23). When its landward end reaches the shoreline, pulling stops and the pipeline is stopped off. The next

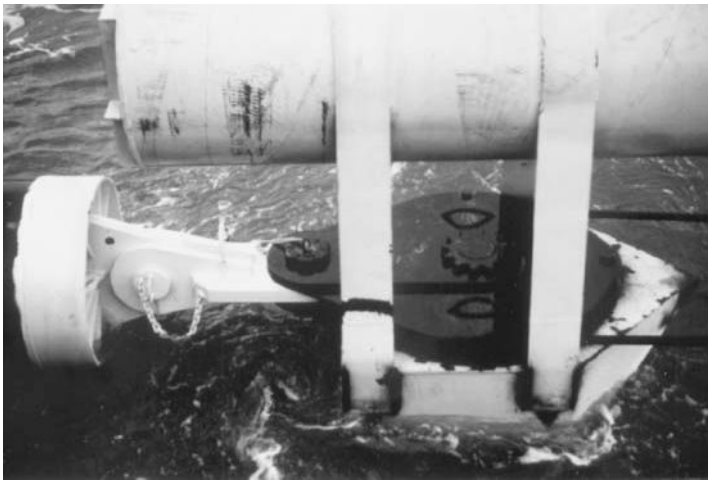

FIGURE 15.20
Pulling nose assembly, with sheave to enable two-parting of pulling line, swivel to prevent twisting, and buoyancy tank to keep sheave upright. (Courtesy of H.V. Anderson Engineers.)

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FIGURE 15.21
Pulling winch on barge has high capacity on each drum concurrently. (Courtesy of H.V. Anderson Engineers.)

200- to 300-m length of pipeline is rolled sideways onto the launching ramp and the joint welded and coated. The next pull is made.

9. Now the barge itself must move seaward. Its anchors are reset. A third section is placed on the ramp, welded, and pulled. The pulling force is that needed to overcome friction.

Friction on the launching ramp can be reduced by the use of rollers or small railcars to support the pipe. The pipe here is in the air, thus having its full weight exerted on the ramp. Movement seaward can be helped by the use of side-boom cats or by an assisting caterpillar-tread tensioner being used in reverse to push the pipe out. As noted earlier, initial sections of pipe may require restraint by use of a holdback winch.

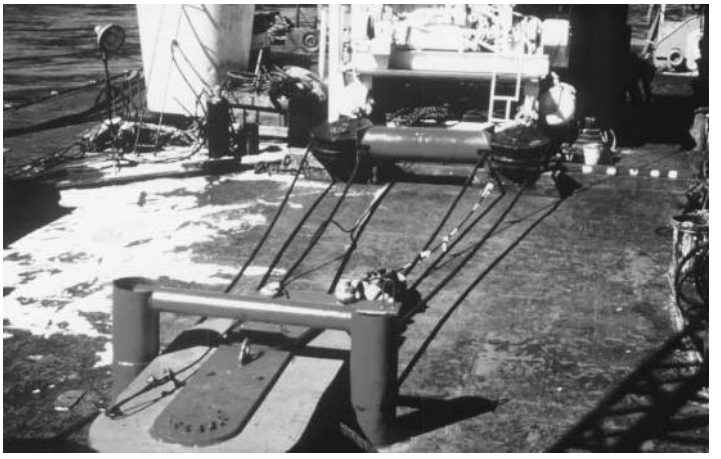

FIGURE 15.22
Equalizing sheaves on pulling barge to engage both seaward anchors equally. (Courtesy of H.V. Anderson Engineers.)

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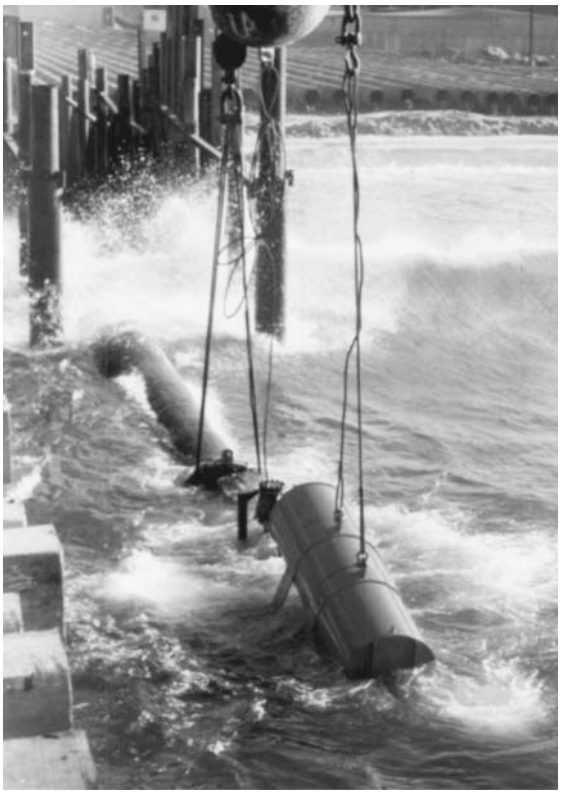

FIGURE 15.23
Starting the pull, the nose, with buoyancy tank attached, is being fed through piles trench at surf-zone.

Once underwater, the empty line has only its buoyant weight. This must be slightly negative. This results in friction on the seafloor. It is this friction which the pulling barge must overcome.

Friction coefficients have been measured in the range of 0.3–0.5 for the dynamic, moving condition, but rise to 0.6–0.8 when the pull is stopped to weld on a new section. Conservative values up to 1.0 are often used in planning since if the line cannot be moved, it will be a total loss.

The pipeline needs enough net weight to be stable on the seafloor and not move laterally. The amount depends on the surf, current, and seafloor conditions, but typical values for coastal lines range from 0.20 – 0.66 kN/m (15–50 lb/ft.). For bottom-pull installations on the seafloor that do not have to have stability in shallow water, net weight can be significantly reduced. It is the total friction force developed when the line is fully laid that limits the length that can be pulled by this method. If we assume a net weight of 0.3 kN/m, a friction factor of 1.0, and a winch having 1500 kN of pull on full drum, then the maximum length that can be pulled with a single line is 4500 m (15,000 ft.). This can be slightly extended by making short pulls at the end to keep the winch drum half-full, since the winch can apply more force under this condition.

By using two parts of pulling line and a sheave at the nose, the potential overall length of line can be doubled. However, the risk of jamming of the line in the sheave makes this solution acceptable only if the required force cannot reasonably be provided with a single line.

The bottom-pull method is extremely sensitive to weight and displacement tolerances, since the net weight, a small value, is the difference between two large numbers. Therefore, great care has to be taken to control and monitor the actual values.

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The principal potential variances are

1. Steel pipe wall thickness (often 3%–5% over);
2. Steel pipe outside diameter (O.D.);
3. Concrete weight coating thickness;
4. Unit weight of concrete;
5. Water absorption into concrete during pull, often 2%–3%.
6. Out-of-roundness.

The weight coating is often applied in such a way that the ends are much thicker than the midsection. This needs to be accounted for. In some cases it can be compensated by an under-tolerance in applying the field coating over the joint itself.

The effect of these tolerances will be illustrated using the example in the following table.

	Nominal	**Actual**
Air weight	16.5 kN/m	16.7 kN/m
Displaced water weight	16.0 kN/m	16.0 kN/m
Net weight	0.5 kN/m	0.7 kN/m
Total force for 4000 m assuming a moving friction of 0.6	1200 kN	1680 kN

If the single line capacity of the pulling winch is 1350 kN, this means that if the tolerance is shown actually realized, it may be inadequate. A double line around a sheave will be required. If due to sanding in, the friction rises to a factor of 0.8, then the total force available of about 2700 kN will be just barely enough.

In order to measure and check tolerances, the following procedure has been found effective. Three random pipe sections of 40-ft. nominal length are selected for weighing. They are placed in seawater for twenty-four hours and then lifted out and accurately weighed. Their steel pipe wall thickness is calipered and the diameter measured. The circumference of the coated sections is also measured at three points along the length. Then all subsequent pipe sections are measured for steel pipe wall thickness and for diameter and measured with a tape for circumference of the coated section. The above will enable net weights to be calculated within 2%–5%, if the pipes are relatively uniform.

Once the pipeline has been pulled, it is flooded for stability. A test plug is fitted on the inner end to permit hydrostatic testing. After testing, one pig is then activated by compressed air to empty the line. The second pig is there for potential problems such as buckling. Buckling of a pulled pipe does not occur in the vertical plane, as it does when laying from a lay barge, but in the horizontal plane, usually due to long-shore currents.

Out-of-roundness was found on steel pipeline sections being assembled for pulling a long line in Singapore. The line had to cross a deep channel enroute. To prevent buckling under hydrostatic pressure, one atmosphere of air pressure was maintained in the line by installing a steel plate closure at the pulling nose and a pig in the end of the first string. This section was pressurized. When the second string was welded on, the pig was progressively pushed by increased air pressure to a location shoreward of the deep trench.

When even longer lengths of pipeline are to be pulled, three options are available:

1. Increase the winch capacity. Conventional winches have upper limits, so a linear cable jack with spooling drum may be used, increasing single-line pulling force

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to 4500 kN (1,000,000 lb). Wire rope of this capacity working strength would be excessively large and moreover might have excessive bottom friction itself. For a bottom-pull pipe project across Spencer Gulf in Australia, the contractor used high-strength, 10-in. pipe, empty, as a pulling line, and a linear jack, in this case fitted with pipe grips, as the pulling winch.

2. Pull one line out its maximum distance. Pull a second line out beyond it, so that the inner end of the second is at the outer end of the first. Make a connection by flanges or by lifting up the two lines above surface for welding. This procedure was employed for an offshore terminal off Antigua. The ends of the two lines were brought as close together as possible, a "pup" section fabricated from template measurements, and flanged connections made since flanges were adequate for the pressure.
3. Decrease net weight and apply very careful monitoring. On a long pull across the Bay of Trieste, Yugoslavia, each length of pre-saturated pipe was weighed in water in a special tank. This enabled the net weight on bottom to be decreased to 0.1 kN/m (7 lb/ft.).

The net weight can also be reduced, and hence the required pulling force lowered, if floats are attached to the line. Oil drums have often been used in the past but prove rather crude and unreliable. Polyurethane floats can be accurately designed and attached by straps. The reduction in buoyancy in deep water must be considered. This increases the risk of problems during installation, since the floats add significantly to the drag force from waves and current. On occasion, they have been torn loose; if numerous floats are torn off, the line may become too heavy to move, and thus end up as a catastrophic loss. This occurred on an early line offshore Libya.

Assuming satisfactory installation, the floats must later be cut off. While divers were used in the past, mechanical equipment has been designed to travel along the completed pipeline, severing the straps. ROVs, equipped with cutters, have also been used. Despite the potential problems, floats are a viable and accepted solution for heavy pipelines.

Gas lines are much more susceptible to hydrodynamic forces due to their buoyancy in service. Thus they require thicker coatings, which in turn require increased reinforcement, so as to prevent loss of coating. During installation, they will have an increased net buoyant weight, which may require attachment of floats.

The reason a swivel is installed at the nose of a pulled line is to prevent the natural twisting of the wire line under tension from imparting twist to the pipeline. The nose is made buoyant and sometimes shaped like a sled to prevent it from digging in as it is pulled. A pendant and buoy is often fitted to the nose to enable its progress to be observed visually and to facilitate recovery in the event of problems. Often a flanged elbow is incorporated in the nose piece to facilitate later connection to a riser or hose.

The highest pulling force occurs when the pull has been stopped temporarily, for example, to weld on another string. Dynamic friction values are usually in the range of 0.3–0.4 but static (break-out) friction can be much greater than that. Because of the high potential loss if a pulled line cannot be pulled to its design length, the pulling gear and anchor system is often based on a factor of 1.0.

A pulled pipeline normally will follow the path of the pulling force, so that it is usually possible to pull around a long radius curve. However, on a hard sand bottom this is not always true, and in such a case the line may drag sideways. One solution is to make the line heavier, that is, increase the net weight to give it more stability. In other cases, lines to anchors have been rigged so that periodically the curve is pulled back to position. Solutions such as this, or trying to pull around a pile, often result in buckling in the

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horizontal plane. A better solution may be to spread crushed rock on the seafloor at the zone where the bend must take place in order to get more lateral stability (i.e., higher local friction). Even better is to pre-trench.

Another solution is to secure several shots of chain inside the pipe, held at the location of the bend or in the heavy surf zone by a line running back to the holdback winch's second drum. This way, the extra weight of the chain stays at the critical location while the line is pulled past.

In one unfortunate project, the owner furnished coated steel line, which was almost in equilibrium as to buoyancy, having only a few pounds of negative net weight. The contractor, faced with an oncoming storm, decided to go ahead with the pull and try to get the line in place and flooded before the storm hit. As indicated in Chapter 2, long-period swells run out ahead of a storm. Therefore, as the pull was in progress, the swells came in at about a 45° angle. While they refracted around to the normal in shallow water, there was still a net volume of water to be displaced to the south, resulting in a strong wave-induced longshore current. This bowed the pipeline out until it buckled. The contractor aggravated the situation by attaching a line at the buckle leading to a tractor on shore; this attempt to pull sidewise broke the pipe, and the entire line had to be abandoned. On the next try, the contractor used an ingenious trick. He filled the line with rock salt to give it weight, pulled the line out properly, and then washed the salt out.

For pulling around a curve, saw cuts have been made in the concrete coating to reduce the stiffness in that zone. This is not recommended as the coating may come off.

Another tale of disaster will be told to illustrate the interaction of the hydrodynamic weight, buoyancy, and structural aspects. This line was in the Bay of Fundy, with 10-m tides twice a day. The concrete coating was reinforced only with a very light mesh resembling chicken wire. The launching ramp terminated at high tide; the line was to be pulled across the long tidal flats at high tide and out to an offloading buoy. As the pulling operation commenced, the offshore anchors of the barge slipped. By the time these were reset, the tide was falling, exposing the line on the mud flat. Now as they pulled, they had the increased friction of the pipe's air weight burying the line in the mud flat and developing excessive friction. They then held on until the tide came in and released the pipe from the mud, but now the winds and sea were kicking up and the line bowed laterally. This caused the concrete weight coating to crack, the light wire mesh to break, and the line to float. Eventually, the line broke and ended up on the beach, where it had to be abandoned due to multiple kinks and buckles.

Unfortunately, variations on this theme have occurred elsewhere in the world; for example, one of the first submarine lines to Kharg Island, Iran, also reportedly ended up as "spaghetti" on the beach. Recently, a number of cases of "float-up" of pipelines have occurred in the North Sea, apparently due to breaking off of large segments of concrete coating due to flexing of the line under vortex-shedding movements and subsequent wave-generated pore pressures within cracks and delaminations.

A gas line pulled across the Strait of Magellan became exposed as it crossed the beach, due to longshore currents and storm waves. Trenching was impracticable in the cobbles and gravel. The pipe was eventually covered with large riprap. Use of heavy-weight aggregate was another solution that was considered.

Pipelines crossing sandy and/or silty beaches in such widespread locations as Bass Straits, Australia, and the landfalls from the North Sea have become uncovered due to the pore pressures generated in the underlying sands combined with the uplift as the top of the pipe became exposed. These wave-generated pressures can reach 3.5 kg/m^2.

The lessons are clear. To all the recommendations regarding weight and buoyancy control must be added the need to ensure adequate reinforcement in the concrete

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weight coating, to backfill over the pipe in areas of strong current, and to special protection in shallow water and the beach zone.

The bottom-pull method has been successfully extended to the installation of relatively short deep-water installations such as interconnecting lines between platforms and flow lines. The pulling force is usually that of a large tug and hence is limited to the bollard pull which the tug can exert, with the maximum force being in the range 80–150 tn. Being laid in deeper water, out of the surf zone, the net weight on the bottom can be reduced to a bare minimum, say 0.2 kN/m.

In 1983, a 4-km (2.4-mile) long bundle consisting of a 12-in. (300-mm) diameter oil line and a 4-in. (100-mm) diameter fuel gas line was pulled off from Ninety-Mile Beach in Bass Straits, Australia, and towed 100 km to connect Fortesque and Halibut platforms. The launch from the beach, which included the jointing of sections, took twenty-one hours, the bottom tow thirty-three hours. To reduce friction force and to prevent digging in, 500-m (1600-ft.) long sections were buoyed at each end with pontoons to raise them above the seabed with slight positive buoyancy. This enabled the new bundle to be pulled over an existing line. As the tow approached the platforms, the end sections were flexed laterally, using winches on the platforms, until mating fittings on the ends of the pipeline were mated with receiving fittings on risers from the pipeline. ROVs were then used to disconnect the 3-in. (75 mm) tow cable and all eighty-eight pontoons.

A similar method was used to install the 36-in. (900-mm) connection line, 2200 m long, between the Statfjord A and B platforms in the North Sea, laying it in a trench that had been previously dug with a plow.

Care has to be exercised with this method to make course changes very gradually and to avoid rock outcrop areas, since if the coating is abraded or spalled, the delicate weight-buoyancy balance can be upset, with disastrous results. One of the earlier attempts to use this method involved a relatively long bottom tow with severe course changes to avoid known minefields in the North Sea. The progressive damage to the coating from the sharp bends led to eventual loss of the line.

Another unfortunate catastrophe occurred in the first deep-water installation in the Gulf of Mexico. Seafloor surveys, obviously inadequate in hindsight, had failed to disclose outcropping reefs. The line hit these and was badly damaged through much of its length.

Thus, the importance of a thorough bathymetric and, in many cases, side-scan sonar survey is emphasized.

15.4 Reel Barge

A significant innovation, originally directed to the installation of small-diameter flow lines, but subsequently extended to pipelines of 300 mm (12 in.) and 400 mm (16 in.), is the concept of spooling a long length of pipe on a huge reel and then laying it in a manner similar to an underwater cable.

The first reel barges had a horizontal reel on which the line was spooled. This meant that the line was laid off one side of the barge, making it difficult to move the barge ahead on line. A subsequent "second-generation" reel barge, the Apache, has a large, vertically mounted reel leading astern at the centerline (see Figure 15.24).

A line designed for laying by reel barge can have no concrete weight coating but must have thick enough pipe walls to give negative buoyancy even when empty. This, of course, is relatively economical for smaller-diameter pipe such as flow lines. The steel quality must be such that it can undergo bending beyond yield during winding, and again during unwinding and straightening. The coating must also be able to be bent without cracking

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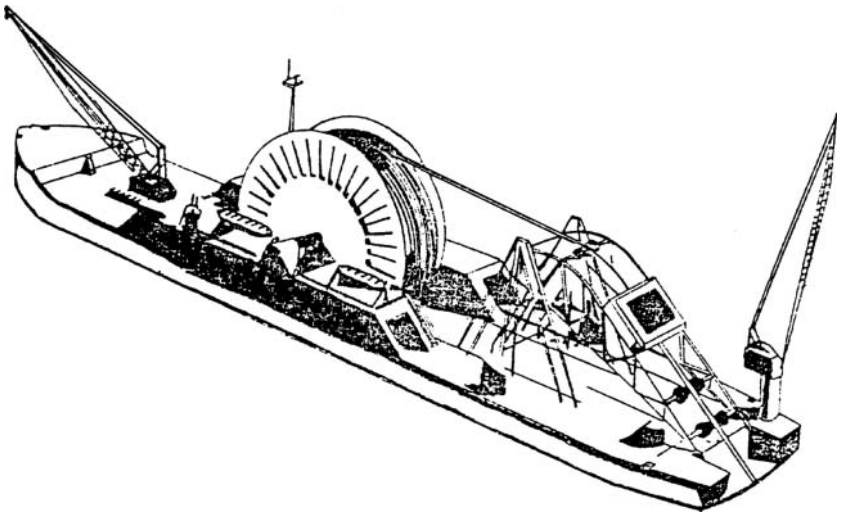

FIGURE 15.24
Apache pipe-laying reel barge. (Courtesy of Santa Fe International.)

or loss of adhesion; epoxy coatings have been developed which will undergo this bending without damage.

The basic procedure is as follows. The line is made up in long lengths at a shore base. The reel barge moors at the dock and pulls the line onto the reel through a spiral J-tube, which bends the pipe beyond yield to the proper curvature. The tube and the spiral are designed so that the pipe bends without significant ovaling and without buckling.

Then the reel barge goes out to location. Start-up generally occurs at the platform, where the end of the pipeline is pulled off the reel, through a straightener and tensioner, over a short ramp or stinger, down to mate with a J-tube riser at the base of the platform, then up to the deck.

The reel barge then lays away from the platform. The straightener is a shallow *S*-curved pipe sleeve with an overcorrecting bulge that brings the pipe back to a straight configuration. This develops significant frictional resistance. Additional tension can be supplied by the powered reel or by a conventional tracked tensioner.

The reel barge now lays out the entire line, letting the end down onto the seafloor by means of a line from a constant-tension winch. The end is buoyed to facilitate recovery for welding to the next reel length. By having suitable onshore spooling facilities, one reel can be wound up at the shore base while another reel is being laid. In many cases, the reel has enough capacity to lay a full-length flow line. As the diameter of line increases, the storage length, of course, decreases. The number of turns that can be placed on a drum is a function of pipe diameter, wall thickness, and tension, to prevent crushing of the pipe. The Apache has the following capacities:

Pipe Diameter	Length on One Reel
8.725 in. (220 mm)	360,000 ft. (110,000 m)
12.75 in. (325 mm)	140,000 ft. (43,000 m)
16.00 in. (400 mm)	92,000 ft. (30,000 m)
24.00 in. (600 mm)	24,000 ft. (7300 m)

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The Apache was used to lay a 10 in. (250 mm) gas line across the Strait of Georgia, which separates British Columbia from Vancouver Island. Depths ranged up to 500 m. Elsewhere, the reel barge has been used for depths up to 1000 m and more.

15.5 Surface Float

The idea of moving long lengths of pipeline while floating and then progressively sinking the line to the seafloor at the site has attracted many contractors over the years. It appears relatively simple to provide the line with the desired positive buoyancy by attachment of floats, which will later be cut free. To keep the pipeline in line and prevent buckling due to waves, wind, and so on, one boat tows while a line to shore, or a sea anchor, or a second boat astern acts as a drag. This keeps the line under tension.

Unfortunately, this method of flotation has a number of serious drawbacks, which can only be overcome by thorough engineering; even then there may be excessive risk. The first problem is that of waves acting on the floating line, causing it to "snake" in response to the short-crested waves. This may damage the coating and cause it to fall off; in turn, the weight balance and stability in service are affected. Keeping the line under tension minimizes the dynamic bending. Even with the line in tension, the lateral and vertical forces will alternate over many thousands of cycles and can eventually lead to coating damage. Such a line must obviously be towed out in calm weather. It is very susceptible to even small storms such as squalls. A number of offshore lines have broken up and been lost in this way.

The second problem has to do with the attachment of temporary floats. For a moderately long line, there will be hundreds of these. Under the wave action, some attachments may fatigue and fail, thus leading to local areas where the line takes on an increasing sag.

The third and most serious problem to be overcome is that of ballasting down to the seafloor. If floats are cut loose at one end, that end will bend downward sharply and may buckle. The same or worse can happen if attempts are made to introduce water ballast into the line. It will run to one end or a low point; this will cause the rest of the ballast to run to that location and the line will take a sharp bend and buckle.

Successful installations of relatively short lines in shallow water have been made by the flotation method, although most of these were in protected or semiprotected waters. In shallow water, such as a river or estuary crossing, the line may be lowered from a series of barges, with multiple control points. Attempts to carry out the surface float method in the open ocean, even in relatively calm seas, have usually failed, an example being an oil import line off Dakaar, West Africa.

The essential point is that the entire sequence must be thoroughly engineered to ensure success. Adequate redundancy must be provided to ensure that the loss of one or a few buoys does not lead to progressive failure.

Floats pulled underwater near the bend may collapse progressively. The French have developed and tested a well-engineered system of air-filled rubber (neoprene) bags, which intentionally collapse progressively with depth. This "*S*-curve" method is described in Section 15.10.

To facilitate connections at the platforms, as the line nears its final location, a line from a J-tube on the platform is affixed to the pipeline end. The buoys at that end are progressively released, while a winch on the platform pulls the end down, either to mate with the J-tube or run on up into the J-tube to deck level.

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15.6 Controlled Underwater Flotation (Controlled Subsurface Float)

The controlled underwater flotation method has been developed to overcome some of the deficiencies of the surface flotation method described in Section 15.5. In this method, the pipeline, having slight net negative buoyancy, is towed at a depth of 5 m or so below the surface, where it is much less affected by local waves and not at all by wind.

Support for the line is by hinged or articulated spar buoys attached at frequent intervals. These provide a relatively constant upward force, giving a "soft" response, that is, not very responsive to the changes in sea level due to short waves. With their small waterline plane, they do not respond significantly to wind-driven waves, and the frequency of response of the system becomes very long (over one minute). This system, therefore, virtually eliminates the first major problem of flotation.

The line is kept under tension, with one boat towing, another acting as a stern drag. Upon arrival at the site, the line must be lowered to the seafloor. This is done by cutting off every second spar, by a diver or ROV, while still keeping the line under tension. Of course, when the line is on the seafloor, the remaining spars are also removed.

This method of controlled subsurface float has been successfully used for flow lines and platform interconnection lines in the North Sea.

15.7 Controlled Above-Bottom Pull

Continued efforts to develop a reliable method for transport and installation of lines have led to the development of a number of ingenious methods, many of them developed by R. J. Brown. One of these is the "controlled above-bottom pull" method in which the line itself is designed for slight positive buoyancy. Short lengths of chain are attached at frequent intervals to give the overall combination negative buoyancy. Thus it is the end of the chain that drags on the seafloor, not the pipe.

The chains automatically control the net underwater weight of the combined system. If the pipeline tends to rise, it lifts more chain off the bottom; if the pipeline tends to sag down, more chain rides on the bottom, reducing the downward pull on the line.

The friction force is now determined by the weight of the "tails" of the chains, which drag on the seafloor. The length of tail is in turn determined by the variations in bathymetry over a short distance and the safety required to offset tolerances in weight and buoyancy. While these can be calculated ahead of time, once the pipeline has been launched in relatively shallow water it can be inspected by divers and adjustments made in chain length made before pulling out to deep water.

The attachments of the chain to the pipeline must be properly detailed to prevent chafing and abrasion. A weak link may be installed to ensure that should a chain snag on an underwater obstruction, it will break free before buckling or crimping the pipe.

The controlled bottom-pull method has proved cost-effective in installations in the North Sea and offshore California, especially for lines of limited length, of the order of 3000 m. During tow, at 3–4 knots, the pipeline may "fly" at a depth of 30–40 m below the surface. As it approaches location and the tow is slowed, it gradually lowers to just above the seafloor, with the chains dragging the bottom. Then it is pulled into final position and ballasted down onto the seafloor.

A larger-diameter pipeline may be employed as a carrier line for several smaller flow lines and cables bundled inside. Obviously, it is essential that the carrier pipe maintain its watertight integrity during the tow. Internal pressurization has been utilized to overcome

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any leakage. Unfortunately, in at least two recent cases, the combined line has prematurely flooded and sunk, damaging the flow lines being carried inside the carrier pipe. This would indicate that consideration needs to be given to methods of "damage control" of in-leakage due to possible overstress during tow. Appropriate steps might include use of foam, compressed air, multiple pipes, or subdivision.

The controlled subsurface-pull and controlled above-bottom-pull methods have been combined by C. G. Doris for bundled lines to constitute what they have named as the "guide rope method" for placing flow line bundles. The bundle assembly includes both floats and chains. When the pipeline is empty, it floats 30 m below the surface; when filled, it floats 10 m above the bottom. Thus, it can "fly" over obstacles, rock outcrops, escarpments, and other pipelines. Filling produces significant bending in the line. This is best controlled by flooding one line in a bundle at a time.

One such bundle of 4- and 2-in. pipes was towed 13 miles just below the surface, and then sunk to the above-bottom mode and the ends pulled into subsea manifold connections, using a guide funnel and pull-in lines through sheaves. Cutting the floats allowed the line to sink to the seafloor.

For installing pipelines between the underwater manifold and the Cormorant A platform in the North Sea, an 8-in. oil line plus 2- to 3-in. TFL well test service lines were made up inside insulated sleeves which in turn were placed within two carrier pipes, one 26-in. line and one 24-in. line. The bundle was then towed in two 3.3-km lengths, kept at mid-depth by the chains for a distance of 490 km using one tug of 75-tn. maximum bollard pull pulling and another of 35-tn. maximum bollard pull restraining, that is, keeping the line in tension. The average pulling forces were 50-tn. pull, and 12-tn. restraint, and the average speed 5.5 knots. Depth below surface was 30–100 m. Slowing down and reducing the tension allowed the line to submerge to near bottom. When in position and mated, the carrier pipe was flooded.

The carrier pipes were pressurized with nitrogen gas to 15 atm to prevent water in-leakage. A full contingency plan was developed to cope with possible accidents or difficulties, such as the snagging of a chain on an obstruction. The position of the pipes as they approached the field installations was monitored by acoustic transponders and side-scan sonar.

Figure 15.25 illustrates four different versions of pulled lines.

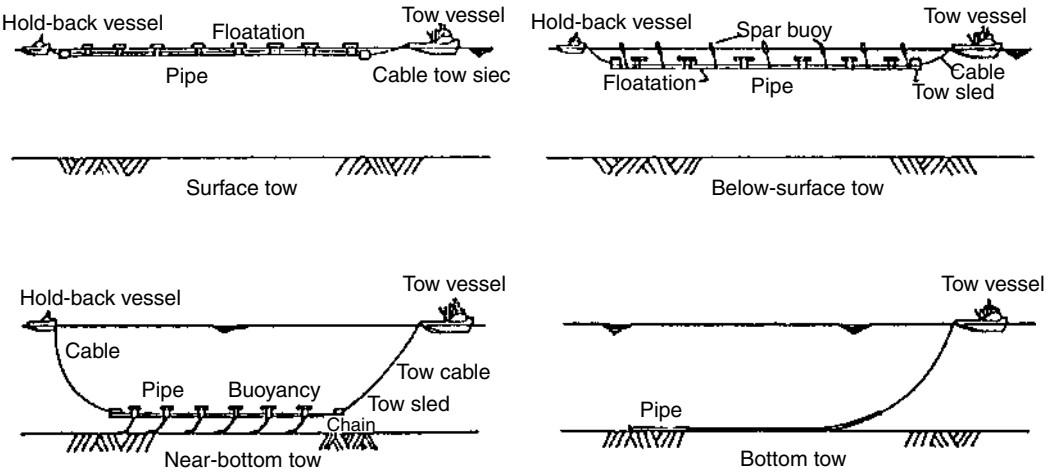

FIGURE 15.25
Tow installation methods for installation of flow lines.

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15.8 J-Tube Method from Platform

Curved J-tubes with a "straightener" curve at the exit can be pre-installed on a jacket. After the platform is in place, a messenger line (also pre-installed in the J-tube and temporarily run up to stop off at the deck) is connected to a long pulling line from shore or a barge. The pipeline is made up vertically on the platform, using steel quality and coating similar to that used for reel barge operations (see Section 15.4). This practice of successive vertical jointing is called stove piping.

The barge or shore winches now pull the end of the line down the J-tube, around the bend, and out over the seafloor as a bottom pull. Thus, the length of line which can be pulled is limited by the friction in the bend and straightener plus the bottom friction.

15.9 J-Lay from Barge

This is the fourth-generation barge, especially designed for deep water. See Chapter 22 for full description of this method. The pipe departs from the pipe-laying vessel almost vertically, and hence has no overbend. This method utilizes a hinged ramp, inclined only slightly from vertical on which a triple- or even quadruple-jointed pipe segment is placed. The joining of the segments to the previously laid pipe takes place at a single station just above deck level. To make acceptable progress, advanced methods for fast automatic welding are employed. These include electron-beam welding, high-frequency induction welding, friction welding, flash-butt welding, high-speed electrical resistance forge welding (HPW), and laser welding. Mechanical connections have been proposed. The aim is to complete the jointing process in two to three minutes, followed by magnetic particle testing (see Figure 15.26).

The axial tension is largely determined by the weight of the pipe hanging below the lay barge, which reduces the forward-leading tension requirements to where this thrust can now be applied by dynamic thrusters, and all mooring lines can be eliminated. This in turn eliminates the slow-drift accelerations, so that work can proceed in more severe sea states, the limiting factor being that of pipe transfer.

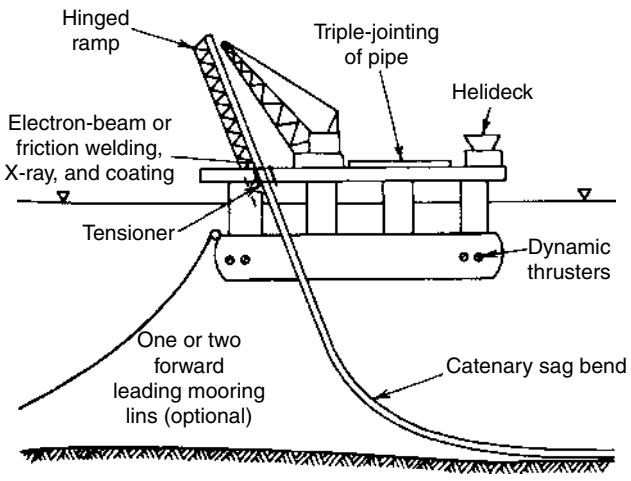

FIGURE 15.26
Catenary or "J-curve" method of pipeline installation. (Courtesy of Exxon Exploration & Production.)

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15.10 *S*-Curve with Collapsible Floats

This system employs inflated bags of neoprene or rubber which exert a buoyant effect on the line. As bags are pulled below water, they partially (and eventually, fully) collapse, thus reducing their uplift force.

This method was proved by a test installation of a small-diameter steel line in water 2500 m deep in the Mediterranean. The short length of line was towed to the workboat and fed up over the bow to the deck where the bags were attached and then out over the stern. The sinking was initiated by pulling the end under water; from then on the line automatically took its S-shape and so went to the seafloor without buckling. It was later successfully retrieved by lifting up on the end, after which the bags progressively expanded and brought the pipe to the surface in a gradual curve. Using this method, a gas line was successfully laid across the Strait of Gibraltar, at a depth of over 300 m, despite strong bottom currents.

15.11 Bundled Pipes

The simultaneous installation of two or more pipes in a bundle is feasible by many of the above methods. The usual reason is to handle several products. The main requirement is to ensure that the attachments are sufficiently rugged to take the stresses imposed during laying without failure and without damaging protective coatings. The use of two or more lines in a bundle enlarges the opportunities for construction engineers to control weights and buoyancies to suit their needs. For example, if an oil line and gas line are pulled together, the oil line, in service, may provide the net negative weight to stabilize the system, even though the gas line may be near neutral buoyancy. Similarly, inclusion of a second or third line in a bundle may reduce the net buoyant weight during installation yet increase stability in place after it is flooded. In this case, the additional line becomes an expensive but effective means of stabilization of the system on the seafloor (see also Section 15.7).

15.12 Directional Drilling (Horizontal Drilling)

Directional drilling is frequently employed as a means of installing cables and pipes under the surf zone and adjacent beach. The slant drill rig is set up on the shore and drills and cases an initial length to a depth where stability can be maintained. A drilling fluid is used. Polymer slurries are preferred because they are biodegradable and do not contaminate the water. This method has been successfully used on a number of important river crossings and coast line crossings.

Using directional drilling techniques, the drill drives on a downward angle to the depth required, pulling the drill string behind it. The drill is then steered to near horizontal and eventually slightly upward to exit on the seafloor. The pipeline has been previously pulled out on the seafloor with its shoreward end near the exit. The drilling bit is now changed to a hole-opener, slightly larger than the permanent pipeline to be installed. It is attached by a swivel to the permanent pipeline and then drill string, hole-opener, and pipeline are

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pulled back to shore. Variations on this procedure enable the pipe to be pushed from shore into the slurry-filled hole, following an oversize bit.

The critical zone for collapse is the seafloor exit, where there is increasingly shallow cover. Hence, the endeavor is to select a point for exit that is in stable material.

A somewhat similar result can be attained by jacking of a pipe tunnel. Usually concrete pipe sections are used. The initial thrust is from a jacking pit on shore. The line is laid dry, like a tunnel, with a cutter head, essentially a small Tunnel Boring Machine (TBM) at the face. Polymer slurry may be injected into the annulus at intervals to reduce friction. When the initial jacking force is not sufficient, an intermediate jacking station is established to jack a further length. The line may be steered into a pre-set seaward terminal box, which is essentially a small box caisson, sunk to design grade below the sea. Pipelines are then pulled through the concrete pipe tunnel.

15.13 Laying Under Ice

To install pipelines below the ice, several variations on the bottom-pull method have been developed. These have assumed that the work would be done in winter and that the ice would be "fast ice," with little movement over the period involved in the pipe-laying operations. Initially it is necessary to run a messenger line. Holes can be cut in the ice at intervals along the line and acoustic transponders installed at each. An ROV can then be lowered through one hole, and programmed to lay out a messenger line to each hole in succession.

The messenger line is then used to pull a wire rope line (the pull line). The pipe is then made up on a launching ways and pulled into place by the conventional bottom-pull method.

Prior installation of a larger-diameter pipe under the shoreline through which the principal pipeline is to be pulled may be required for permafrost insulation. This casing can also be used as a means of reducing friction and separating the near-shore excavation and construction from the main pull.

Installation of pipelines in the Arctic is described in more detail in Section 23.16.

15.14 Protection of Pipelines: Burial and Covering with Rock

The burial of pipelines is often required in order to provide protection to the pipeline against repetitive pounding under wave action, the impact of dropped anchors, snagging by trawl boards, and to prevent loss of fishing gear by bottom fishermen. Burial of the pipe also permits the pipe to be designed with less net weight (less coating) which in turn reduces the bending stresses during pipe laying.

The "pounding" referred to above is especially serious in the surf zone, as well as in shallow water where vortex shedding by wave-induced currents can cause alternate raising and lowering of sections of the line, leading to fatigue. Concrete coating can be ruptured and break off, allowing the line to rise. The same phenomenon can occur due to high currents alone, in locations such as Cook Inlet, Alaska. In the inner portion of the surf zone, the damage may be aggravated by direct wave impact and by abrasion from moving sand and gravel.

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Burial can be accomplished by laying the line in a predredged trench or by subsequent trenching after the line is laid. A similar protection may be given to a surface-laid line by covering it with rock.

Where underwater sand dunes are migrating, as in the southern portion of the English Channel, then predredging by a trailer suction hopper dredge has proved practicable. The line is predredged to a stable elevation, and then the pipeline is laid.

Through the surf zone, a variety of solutions are employed. At beaches where the surf and longshore currents are relatively mild, a hydraulic dredge may be employed to overdredge a channel through the beach. The line can then be pulled ashore from the lay barge, allowing the sands to naturally backfill the trench. However, where the beach is subjected to heavy pounding from storm surf, over a period of time, the iterative raising of the pore pressures in the sand may jack the pipe up and out to exposure. Therefore, the pipe must be sufficiently heavy in this zone so that in service it remains stable. This may require extra jacketing, the use of the double-pipe concept, pipeline anchors, or high-density backfill over filter fabric. Instances of such raising and exposure are reported from such widespread areas as Cook Inlet, Alaska, Ninety-Mile Beach on Bass Strait, Australia, and the Strait of Magellan.

Another method, of course, is the use of a sheet-pile cofferdam through the surf zone, which keeps the trench open while the line is pulled through it. Finally, a tunnel or tube can be preconstructed through the surf zone. This can be concrete or steel pipe prelaid in a cofferdam, a directionally drilled hole with a casing of larger-diameter pipe, or a precast concrete tube jacked in.

A large, precast concrete submerged tunnel or tube was constructed at Cove Point, Maryland, through which an LNG line was run. Similar precast concrete tube segments were installed on the west coast of Norway, on a very exposed rocky coast. The Statpipe Gas Line was then pulled through the tunnel.

For burial of pipelines in deeper water, trenching has most often been carried out by a jet sled, designed to be guided by the pipe and to excavate the soil beneath it so that the line will sink below the seafloor. The jet sled may be designed to run on the pipe, using rubber wheels. The machine must be designed to ensure that its tires cannot damage the coating (see Figure 15.27). In one case in southern California, repeated running of the jet sled over the line broke up the coating to the extent that the line had to be replaced. Other jet sleds are designed to skid, crawl, or run on the adjoining seabed, being centered and guided by the pipe but not supported by it. A problem here arises, of course, if the trench side slopes become too flat due to encountering loose sediments.

Excavation under the pipe may be accomplished by a combination of jetting, airlift, or eductor removal, or mechanical cutting (see Figure 15.28). The emphasis is on powerful equipment (see Figure 15.28 through Figure 15.30). It should be able to cut to the required depth in one pass. Multiple passes not only are costly, almost in proportion to the number required, but may become less effective due to the increasing depth and progressive infill of the trench.

A steel pipeline has significant bending rigidity and strength. Hence, it will not move down into a dredged trench unless the dredged length is sufficient to cause it to deflect to the bottom. Therefore, the trench when cut must stay open long enough to enable the line to feed itself to the bottom of the trench. Fortunately, this is usually not a problem in most deep-water pipeline installations, since bottom currents and sediment infill are usually limited over a short period of time in the relatively calm seas in which the operation will be carried out.

Power requirements for jet sleds and trenching machines are high. As much as 32,000 HP has been used to power the jets and eductors of a large pipeline burial system used to trench boulder clay in the North Sea. For the pipeline bury barge Creek, eight engines

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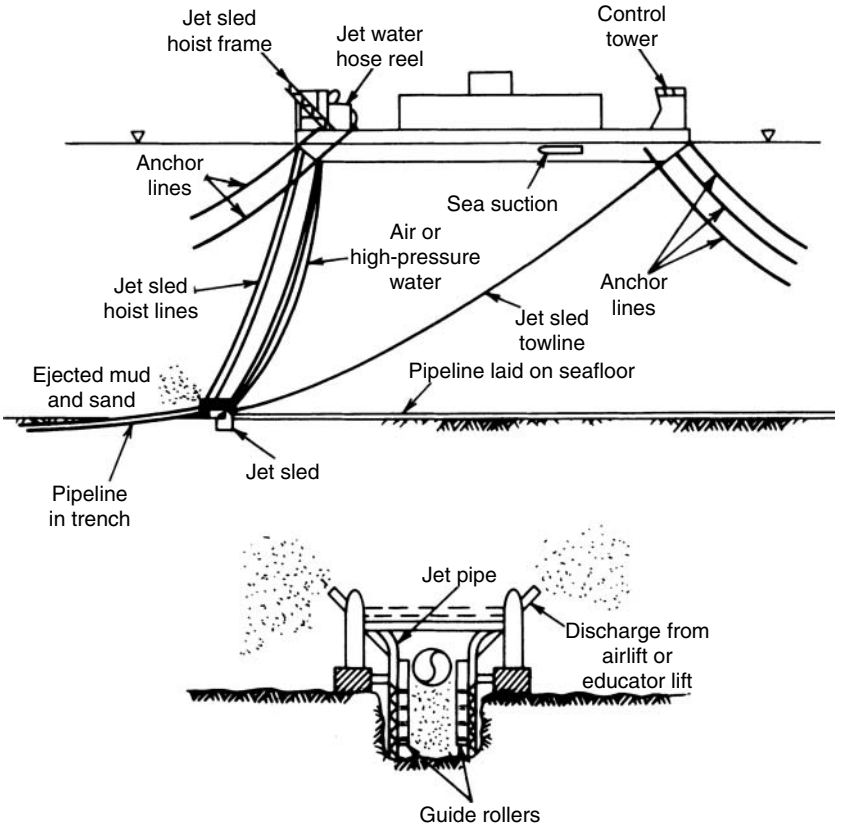

FIGURE 15.27
Jet sled operation for pipeline burial. (Courtesy of Pennwell Publishing Co.)

drive jet pumps to produce 76 m^3/s at 17 MPa (2500 psi) pressure. Other jet trenchers use 21 MPa (3000 psi) pressure. Eductors are more efficient than airlifts in the removal of material.

In most cases, once pipelines have been trenched and lowered to their designed elevation, natural sediment transport has often been counted on to backfill the trench. If a pipeline is to be backfilled by dumping or placing sand, care has to be taken that the flowing sand, which is temporarily a high-density fluid, does not raise the line out of the trench. This has occurred on both small and large pipelines, to the great embarrassment of all concerned. Another method of pipe burial involves the principle of liquefaction. By introducing water and air under the pipeline along a length, the sand is "fluidized," allowing the pipeline to sink of its own weight. Obviously, this works best in easily liquefied materials such as fine sands and silts. Vibration applied to the pipe—as, for example, from inside—aids the process. This method has so far been used, to the author's knowledge, only for relatively short lengths of line (e.g., through a beach zone).

Ever more sophisticated trenching and burial equipment has been developed, such as mechanical trenching machines. Like the jet sleds, these are guided by the prelaid pipeline, but they take their support from sleds on tracks at the sides. Rotating trenchers excavate beneath the pipe and throw the material to the side.

A recently developed form of trenching is that of plowing. A monstrous plow is pulled along the seafloor, stabilized against tipping by widespread outrigger sleds. The plow digs the trench, with the shares forcing the excavated material up on the sides.

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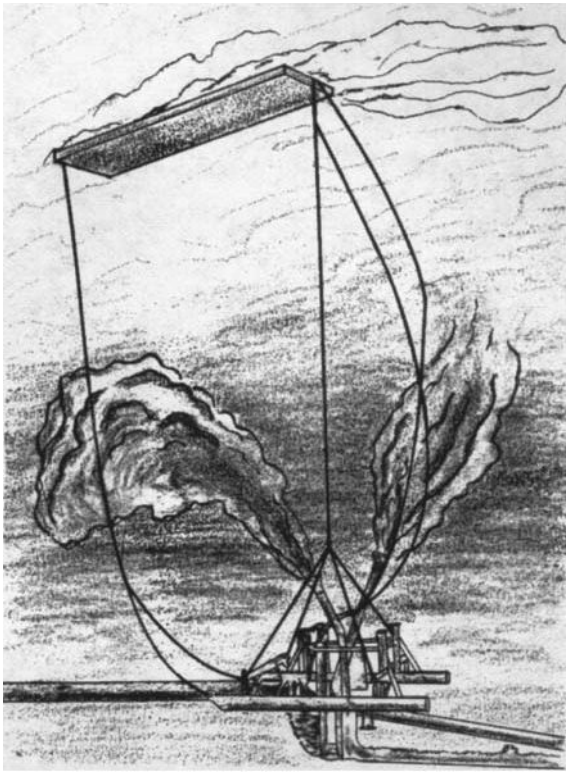

FIGURE 15.28
Deep-water jet sled employs a two-stage seawater eductor system to excavate trench for pipeline burial.

This development was pioneered by R. J. Brown and has proved very successful in the firm clays of the North Sea. The plows have been designed for the soils expected to be encountered, heavy in North Sea clays, lighter in recent sediments in the Beaufort Sea. In the Bass Strait, Australia, an 80-tn. plow was used to post-trench the line, with the plow designed to ride along the pipeline. This pipe dug a furrow up to 1.2 m deep in sand and partially cemented cap rock ledges.

Towing forces generally are one to two times the weight of the plow. Traction is provided by a large, dynamically positioned towboat for relatively light plows or by an offshore derrick barge for heavy plows.

One of the most spectacular uses of the plow to date was on the Northwest Shelf of Australia, where an enlarged version of the plow weighing 380 tn. trenched a 1- to 2.3-m-deep trench for 118 km of 46-in. (1150 mm) O.D. line in only one month (see Figure 15.31 and Figure 15.32). The plow had to dig through limestone and caprock, requiring up to 460 tn. pull, whereas in softer materials, sand plus silt and clay, 250-tn. pulling force was sufficient. Plowing rates reported were 15–45 m/min in sand, 10–20 m/min in sand over rock, and 5–10 m/min in limestone. Difficulties arose principally in soft material, where the plow dug itself in too deeply.

The plow was pulled by a large offshore pipe-laying barge, using a chain-pulling line and developing its reaction force from the barge's anchor lines. The sequence of start-up was as follows. The two mating (positioning) cones were lowered over the pipeline, being spaced apart 40 m by a strut. Divers guided them to seat over the pipe on the seafloor. The plow was now floated to position over the pipe and ballasted down. Removable female sleeves fitted over the pre-placed mating cones. With the plow now accurately in place, the pipe was pulled up into roller guides at each end. The plowshares were then lowered

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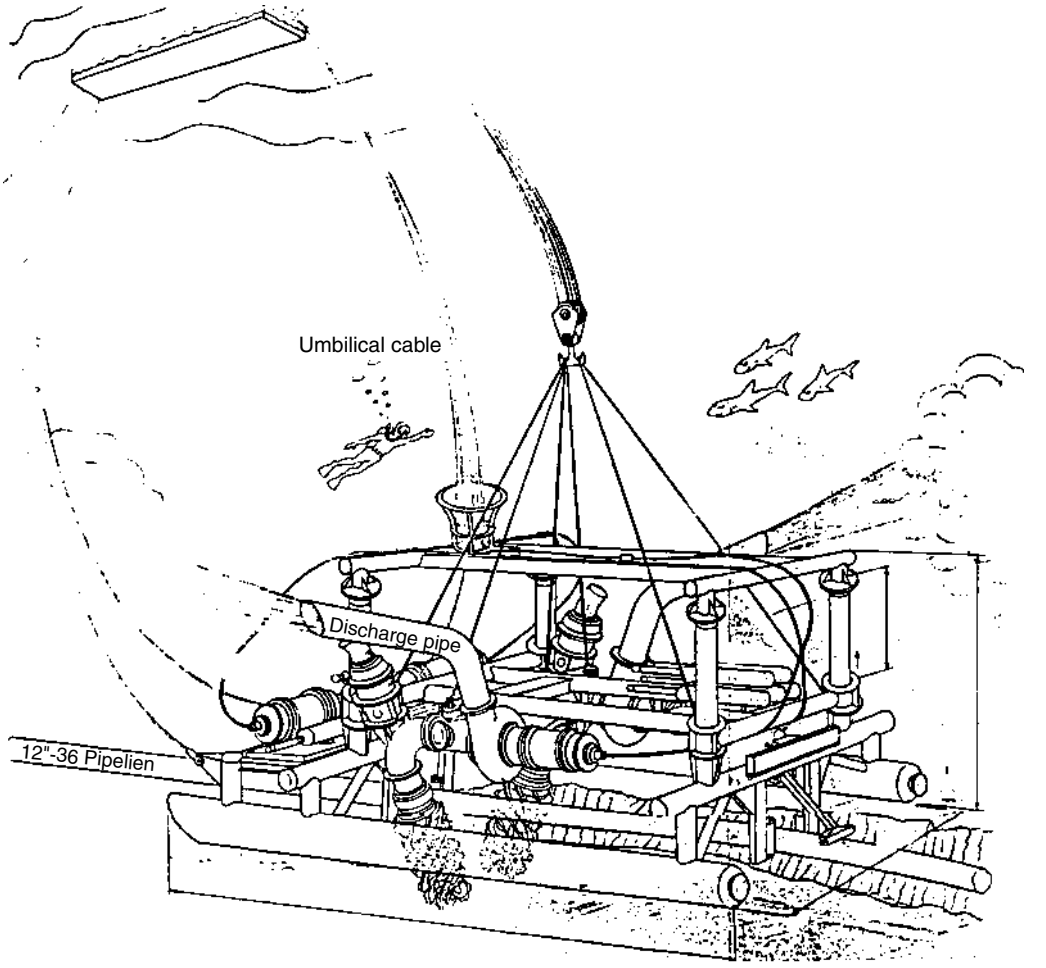

FIGURE 15.29
Jet sled for pipeline burial. (Courtesy of Toyo Pump Co.)

hydraulically to meet underneath the pipe, where they were clamped together. The forward part of the plow rode on two outrigger crawlers, which were Caterpillar D-9 tractor underbodies. The derrick barge had hydraulic controls, which enabled it to raise or lower the plow in relation to the crawlers, thus regulating the depth of cut.

The plow concept uses a long spread between the Cat tractors to automatically even out irregularities on the seafloor. R. J. Brown has suggested that use of computerized controls, reacting to leading sensors, will be developed in the future to handle even rough seafloor profiles. It is believed that a similar system could be employed to scrape the spoil piles back into the trench, should early backfill be required.

Plowing appears especially attractive in Arctic soils where lines will have to be trenched deeply (up to 3 m) in order to protect them from the scour of sea ice pressure ridge keels. For the connection line between the Gullfaks A and Statfjord C platforms, the hard boulder-clay required several passes of the plow. Hydraulically operated grader blades were designed to push the spoil banks aside, leaving a flat level surface for the following passes of the plow. On the Heimdahl pipeline in the North Sea, the 115-km pipeline was post-trenched by plow in only 11.5 days. The plow weighed 145 tn. It was deployed onto the pipeline in 150-m water depth in 19 h. Recovery after completion required 13 h.

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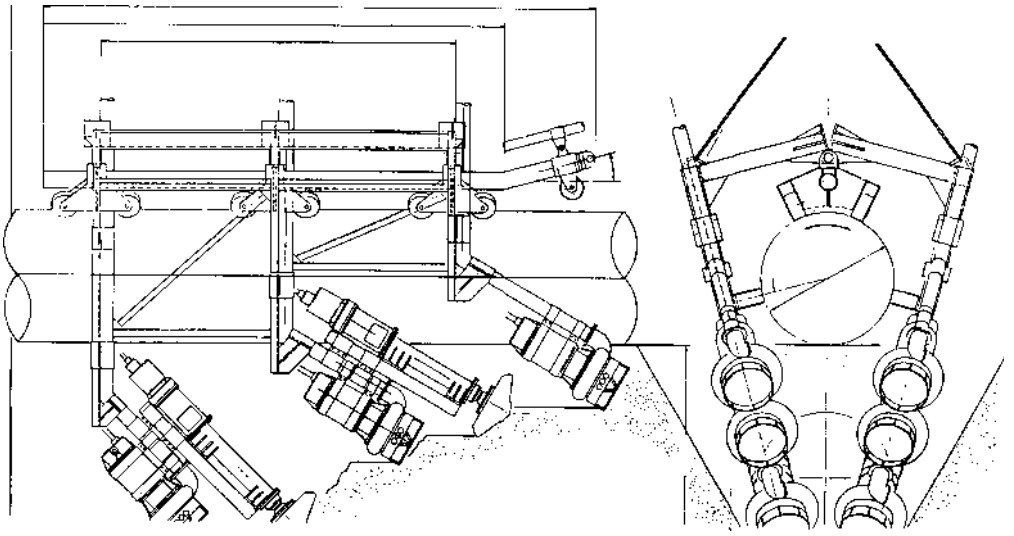

FIGURE 15.30
Jet sled for pipeline burial uses multiple submersible pumps with agitators. (Courtesy of Toyo Pump Co.)

The most recently developed plows have small shares at the leading end, which ride the previously installed pipe under sensor control, to clear the pipe for the principal plowing action of the larger, deeper plow behind.

When caprock or rock outcroppings must be trenched, it is normal first to break them up with explosives. These can be shaped charges laid on the seafloor or drilled in, using high-pressure jet drills or percussion drills. With caprock it is important not to drill through the hard overlying layers, since then the explosion will take place under the caprock, resulting in it being broken into large slabs only, which are extremely difficult to excavate. For overconsolidated silts and in permafrost, high-pressure jet drilling has generally been found more effective than rotary or pneumatic drilling for the placement of explosives.

FIGURE 15.31
Huge plow used to dig trench for pipeline on Northwest Shelf of Australia. (Courtesy of R.J. Brown.)

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FIGURE 15.32
Plow being lowered over pipe. (Courtesy of R.J. Brown.)

Rock breakers (huge chisels repeatedly raised and dropped, or driven with an impact hammer) have been used effectively for caprock in the Arabian Gulf, breaking it downward into the softer soils below.

Covering of pipelines laid on the seafloor with rock has been carried out very effectively by Dutch engineers, using a converted trailer suction hopper dredge, dynamically positioned, fitted with an inclined ladder and conveyor belts, discharging the rock down the ladder to encapsulate the exposed line. Use of the ladder ensured accurate deposition and minimized the impact of falling rock. More recently, this same contractor has used a flexible tremie tube, of steel and polypropylene, hanging vertically under the rock dumping vessel, capable of controlling the deposition of rock. Electronic position indicators are installed at the tip of the tremie.

Sarmac of Italy has developed flexible mattresses of rock-filled fabric which can be placed over submarine pipelines to protect them. They were utilized by Snam-Progetti to cover the gas pipelines from Algeria to Sicily, at depths of 500–600 m. They were lowered to the seafloor, using a structural steel frame that was automatically released once the mattress had been placed on the line. These mattresses are also being used to protect pipelines at locations where they are crossed by another line.

Flexible (articulated) mats of concrete blocks were used to cover portions of the gas pipeline off the Northwest Shelf of Australia, where it was laid on a bare rock seafloor (see Figure 15.33 and Figure 15.34).

Pipeline anchors have been used to hold down the pipe at beach crossings and in areas of high bottom currents, especially when the line lies on a hard, bare seafloor. These are

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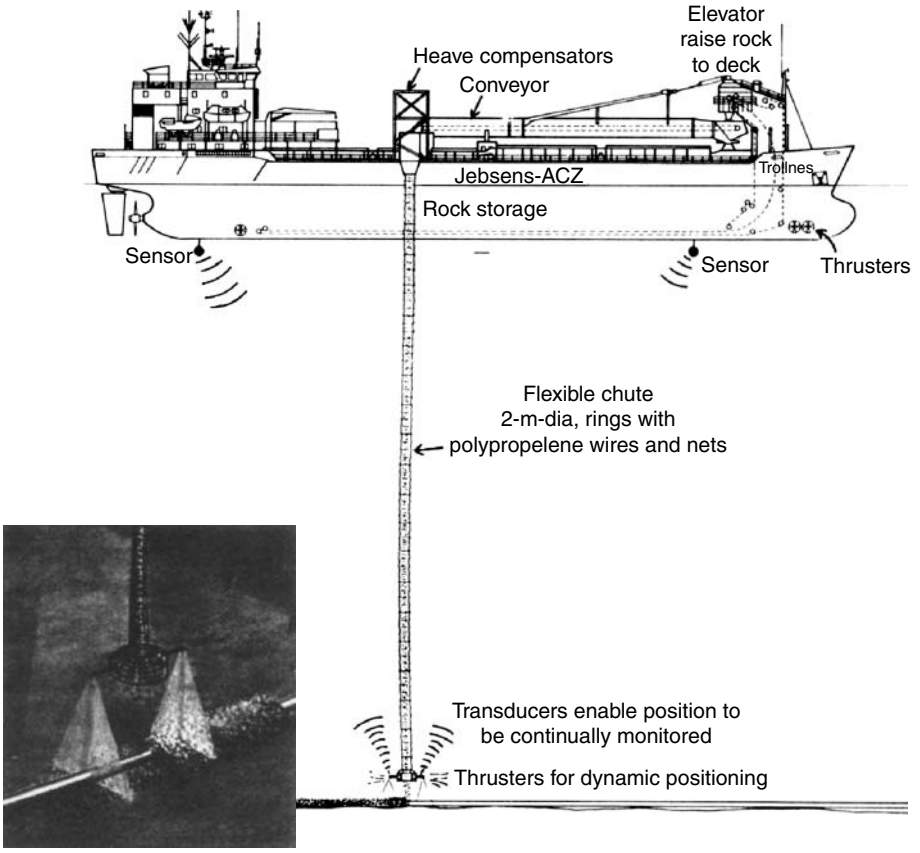

FIGURE 15.33
Stone blanket being placed for pipeline protection and stabilization.

usually screw anchors, which are drilled into the soil to secure an inverted U-clamp over the pipe. While these anchors can be installed by divers, this is slow and expensive. Systems have therefore been developed for installing them from a barge directly, using a frame lowered over the pipeline.

For crossing beach areas where high currents and waves have created a deep layer of cobbles, the protection of the pipeline poses significant difficulties. The use of riprap has been previously described. Heavy-density rock such as iron ore is a possible solution, as well as double-jacketing with concrete pipe. Horizontal drilling should be considered.

15.15 Support of Pipelines

When laying pipelines across an uneven seafloor of hard material, such as an area of rock outcrops, it may become necessary to provide supports to prevent excessive sag moments in the pipeline span. In shallow water, sandbags and grout-filled bags have been stacked in by divers. Burlap bags, half filled with fresh concrete mix of low slump, are best, because the grout exuding through the burlap mesh will knit the adjoining bags together. In deeper and more exposed waters, neoprene and flexible fabric bags can be placed by divers and pumped full of grout.

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FIGURE 15.34
Articulated concrete mat for pipeline protection and stabilization.

The 520-mile Statpipe Ormen-Lange gas-gathering system, crossing the Norwegian Trench, in depths up to 1000 m with steep rock escarpments, resulted in free spans up to 100 m. In addition to grout bags, steel support frames were designed to give intermediate support to the pipe. An underwater bridge was used to support the pipe in waters of moderate depth. To excavate a trench, bottom-crawling "spiders" were used. These were controlled by computer systems utilizing 3D terrain models. These employed small buckets to dig the trench and sidepost the spoil.

15.16 Cryogenic Pipelines for LNG and LPG

For submarine pipelines to transport LNG and LPG, current technology uses INVAR (36% nickel steel) and low-pressure or vacuum systems for insulation. Recently, a new system has been developed using 9% nickel steel product line and high efficiency micro-porous or nano-porous insulating materials, contained within a carbon steel outer jacket and separated by non-metallic bulkheads. This system was used for butane and propane lines to a marine terminal at Pisco, Peru.

Eternal Father, strong to save
Whose hand doth still the mighty wave
Who bids the restless ocean deep
Its own appointed limits keep;
O hear us when we cry to Thee
For those in peril on the Sea.

William Whiting, Hymn of the U.S. Navy

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16

Plastic and Composite Pipelines and Cables

16.1 Submarine Pipelines of Composite Materials and Plastics

Submarine pipelines have been constructed of a wide variety of materials designed to suit the specific environment, operations, and criteria involved. In Chapter 15, steel pipelines were addressed, since these constitute the bulk of deep-water pipelines for the transport of oil and gas. Their selection is based both on the need to accommodate high internal pressures in service without leakage and the bending and combined stresses developed during installation.

Composite pipelines are relatively flexible, light in weight and relatively easy to install. Coflexip has developed and installed a composite pipeline based on multiple layers, wound with wire. Epoxy-bonded carbon fiber has been used for some subsea completion risers, utilizing the high strength of carbon fiber and its corrosion resistance.

Other materials widely used for submarine pipelines are high-density polyethylene (HDPE), glass fiber filled polymers and epoxies, and flexible composites of steel and neoprene. Cables for submarine transmission of electric power are another type of flexible pipeline, involving somewhat similar offshore construction techniques. Concrete pipelines for outfalls and intakes are discussed in Section 10.2.

16.1.1 High Density Polyethylene Pipelines

A highly flexible line, which is resistant to chemical attack and has low friction, can be produced by using HDPE. It has, therefore, been utilized for test installations for ocean thermal energy conversion (OTEC) plants. To date (2005), manufacturing limitations restrict availability to relatively small diameters (1.5 m and less). This material has a density less than seawater and hence has to be weighted or anchored down to stay properly submerged.

One installation for an OTEC test plant off the island of Hawaii utilized polyethylene pipe made up in long floating strings in quiet water and towed to the site, where anchors with short Kevlar lines were used to progressively submerge it to where it floated just above the rough and steeply sloping rock floor. As the weights were attached, the polyethylene line progressively collapsed (buckled) but regained its circular shape as it went underwater, so the buckle traveled along the line as it was laid, without permanent damage.

The line was reinforced by saddles of polyethylene, which were fitted at regular intervals where the mooring lines were attached to hold the line near the seafloor in an inverted catenary. Stress concentrations at moorings and fittings must be avoided, as polyethylene is subject to internal fatigue under sustained high stress. One advantage of the

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polyethylene line over a steel line is that no ferrous ions are picked up by the seawater; thus it is suitable for use in supplying aquaculture facilities.

Another, larger polyethylene line was successfully suspended in a vertical hanging position under an OTEC test facility to a depth of over 1000 m. Later, when this test had been successfully completed, the line had been upended to horizontal and was being towed to a new test site. Reportedly, the heavy end was supported by wire lines; during the tow and under the wave action, these chafed and the concentrated stresses on the polyethylene led to a local failure and loss of the line. (It was later salvaged.)

HDPE joints are made by fusion and hence develop the basic strength and flexibility. Flash-butt welding is usually employed. Mechanical connectors have been developed. Corrosion-resistant steel bolts can be applied.

The most spectacular HDPE line so far was the Montpellier Outfall into the Mediterranean. This 11 km long line, 1600 mm in diameter was manufactured in 550 m lengths in Norway and towed to the French Mediterranean coast where it was assembled by glass reinforced coupling sleeves into 1100 m lengths. Precast concrete collars were slid over the line to serve as continuous weights and to protect the HDPE (see Figure 16.1 and Figure 16.2).

Meanwhile, a trench was dug by a large-mounted backhoe dredge and a hydraulic suction dredge. Then, the 1100 m lengths of pipe were floated into place and gradually sunk into the trench by a combination of water and compressed air, buoyancy pipes and end-tension by tug boats. For joining the next 1100 length, the end was lifted up by a strongback to the surface, where it was coupled to the new length.

Then, the line was covered with fabricated mattresses of articulated precast concrete blocks placed in 50 m long strips. Finally, the trench was backfilled with sand.

A 6000 m long HDPE line of 1.0 m diameter was designed to serve as a conduit for multiple fiber-optical lines across San Francisco Bay. One-thousand-meter lengths would be pulled off from shore and towed to the laying barge, using a tug at each end so as to keep the line under tension. Ends would be brought up onto the lay barge, clamped, and fused. The terminus at San Francisco, being rather abrupt, would be pulled through a pre-installed steel casing, so as to result in a shallow "S" curve. Then, the fiber optical bundle

FIGURE 16.1
HDPE pipe strings ready for towing 2300 km from Norway to Mediterranean. (Courtesy of Van Oord Dredging and Marine Constructor, The Netherlands.)

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FIGURE 16.2
Laying 1100 m long lengths of HDPE by flotation and progressive flooding. (Courtesy of Van Oord Dredging and Marine Constructor, The Netherlands.)

would be pulled through, using a special nose to avoid damage to the HDPE pipe. This line has not yet been built.

HDPE is currently being designed for a discharge line into Lake Mead, Nevada. It will consist of 5–1.5 m diameter lines, bundled.

16.1.2 Fiber-Reinforced Glass Pipes

Fiber-reinforced glass (FRG) has been used for a number of sewer outfalls and seawater intakes, as well as for slurry transport of salt. The material is highly resistant to chemical attack. It can be coated or pigmented to protect it from UV degradation.

It is light in weight, hence even relatively large-diameter (2-m) lines can be readily handled in long lengths, set by a derrick barge, and joined or, alternatively, pulled out from the beach. Because it is of low density, added weight is necessary to enable it to be properly seated on the seafloor. As it is placed, precast concrete saddles are usually lowered over the line to hold it in position under wave action. Because of their light weight, FRG lines are especially hard to lay and hold in place in the surf zone. Weight can temporarily be added internally—for example, a chain wrapped with canvas and/or fiber so as not to abrade the interior. Even sandbags have been used.

As with polyethylene lines, details of saddle bearings must be carefully developed and accurately constructed to prevent local wear and point bearing. Fiber-reinforced glass is susceptible to abrasion and to internal fatigue under sustained stress concentrations.

An FRG line was selected, rather than the original steel line, for an underwater conduit in which to later pull a bundled electric power transmission line across a shallow arm

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of Lake Mead, Nevada. Unfortunately, during the laying, some of the bell-and-spigot joints pulled apart. Unlike the HDPE lines, the joints cannot be fused.

16.1.3 Composite Flexible Pipelines and Risers

In recent years, the technology for the design and manufacture of steel-reinforced plastic hoses has advanced significantly so that reliable lines for crude oil and petroleum product service are now available up to at least 16-in. diameter. These lines consist of several layers of neoprene, double-wrapped with helixes of steel in a manner resembling that of armored power cables. Being flexible, lines of the size normally used for flow lines can be transported on reels. Larger sizes may be faked out on the deck of a barge.

These pipelines are led off the barge over a curved cradle, not unlike a small cantilevered stinger. Rollers are used to prevent abrasion. Tension must be applied to control the lowering. Since such lines are usually laid with their end open to the sea, they become progressively flooded. This prevents collapse due to hydrostatic pressure. The net weight of the line may therefore approach that of a steel line. Such flexible lines are usually so short that they are laid in one day or less; hence weather conditions may be selected to minimize the required tension. In very deep water or where the tension value exceeds the allowable value for that line, a wire rope line may be married to it to give it support and to take the tension force.

Great care must be used to avoid cutting or abrading such flexible lines. Nylon or other fabric must be used for the slings with which the pipe is handled.

Coflexip has developed an integrated system using a large-diameter wheel trencher on bottom crawlers to enable the simultaneous trenching and laying of the line (see Figure 16.3). Such a system was employed for laying flow lines and cables in the caprock seabed of the Zakum Field, in Abu Dhabi. Coflexip has also developed a method to install its flexible pipe in 450 m of water in the Montanazo Field off Barcelona, Spain. It utilized a very sophisticated positioning system, with a reference array of thirty subsea transponders, and an ROV to monitor pipe touchdown.

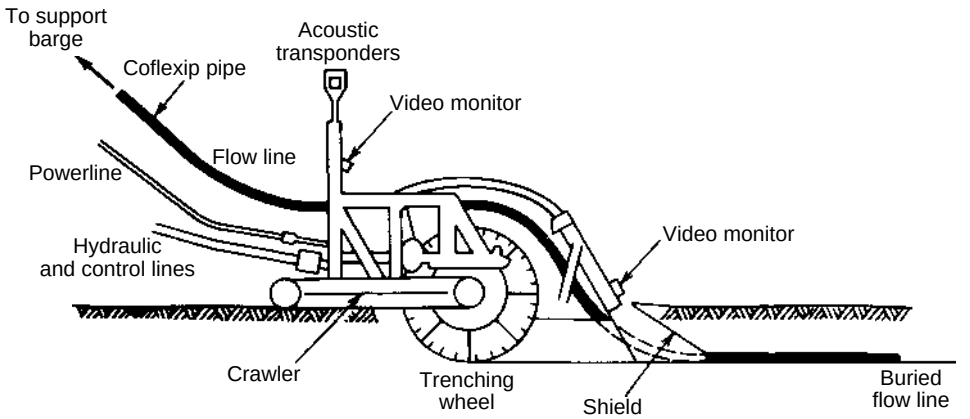

FIGURE 16.3
Trenching and laying machine for buried flowlines. (Courtesy of Coflexip.)

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16.2 Cable Laying

An early and most dramatic application of ocean engineering and construction was the bold laying of communication cables across the oceans, undaunted by the depths and, in most cases, able to span the irregularities of the seafloor by intentionally laying in a loose snake-line pattern as opposed to a straight line. These early cables were not buried. Almost equally remarkable was the development of the ability to lift them up for splicing or repair and to lay them down again.

In recent years, the need has developed to lay electric power cables across open water areas. The currently most advanced application is the construction of four power cables across the English Channel, trenched in the limestone seafloor. Proposed, but not yet built, are the power lines across the Strait of Belle Isle, between Newfoundland and Labrador, an irregular rock seafloor gouged and polished by the keels of icebergs. Another deep power cable has been constructed to link the principal Hawaiian Islands.

Key elements in the technological development of subsea cables have been the perfection of armored cable using double-wound helices of steel over the insulated copper or aluminum cable and the development of rock-trenching machines capable of cutting a narrow trench up to 1 m deep in hard rock while laying a guide cable in the bottom of the trench. These cutting wheels, fitted with diamond or carbide teeth, must be supported by outrigger sleds or tracked carriages, which run on the seafloor. Therefore, the seafloor must have been first cleaned of surficial sediments and debris by one or more of the dredging or jetting schemes discussed in the section on dredging, for example, the use of a trailer suction hopper dredge.

The cutter-trencher, essentially a wheel, cuts a trench 200–300 mm wide and a meter or so deep. As it moves on, it lays a wire rope line into the bottom of the cut trench. This remains there to later serve as a guide for the power cable. When the trench is complete, the power cable is laid. High-pressure water jets operate from a sled, which is guided by the prelaid wire line. These jets clean out any sediments. The sled device resembles a pipeline trenching sled and may incorporate eductors or airlifts in addition to the jets. The power cable is then fed into the trench.

The power cable differs from the pipeline in its much greater flexibility, which enables it to be reeled onto a drum of moderate radius and subsequently unreeled without damage. However, as power cables grow in size and armoring, their flexibility decreases, so that they require a laying barge not too dissimilar from a reel barge for pipe laying.

A review of the above procedure will highlight several critical features. First, the seafloor must be either naturally level or artificially leveled to the extent that the trencher can cut a relatively uniform depth. This may require blasting of some outcrops and ledges, using shaped charges or even drilling. The problem with explosives is that the remaining rock is fractured, which can cause the trenching cutter to bind or jam. Mechanical cutting and leveling, with a cutter-head suction dredge or ladder dredge is, of course, preferable when water depths permit their use and the rock is able to be mechanically excavated.

In many seafloor soils, a heavy plow, such as that described in Section 12.12, may prove suitable for direct trenching or for pre-leveling to enable the wheel to work. The methods needed to clean and level the seafloor prior to trenching will vary widely, depending on the character, depth, and hardness of the seafloor surficial materials. A sled device, similar to the larger pipeline plows, may be applicable, dragging boulders and loose rock aside. If blasting becomes necessary at discontinuities in profile such as ledges, then closely spaced minimal charges should be used in order to avoid deep fracturing and the resultant large blocks of broken rock.

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The power cable crossings of the English Channel consist of four cables, with two being constructed by the British using the methods described above. The seafloor was cleaned by a trailer suction hopper dredge. In some areas, bucket dredges were used to clear the route of boulders. In areas of irregular rock, explosives (shaped charges) have been employed. An underwater crawler has been used to survey the route prior to trenching. The trench cutter is a self-propelled trencher, having a 4.5-m-diameter drum with 180 cutting picks (teeth). It cuts a 600-mm-wide slot to a depth of 1.5 m in the limestone and sandstone seafloor. The trenchers can climb a 1.5 m ledge. In one 600-m section, the sandstone boulders were dredged out by drag line. Unfortunately, prior to the arrival of the trencher, some of the boulders had fallen back in due to the high currents and side-slope erosion. It would appear prudent when pre-dredging such an area to first clear a wider swath. After the trencher had cut one slot across the channel, the cable layer followed, using a sled equipped with high-pressure water jets, enabling the sled to follow the pre-laid wire line and thus install the cable at the bottom of the trench.

The French, on the other hand, used a combined trencher-cable layer, which laid the cable in the bottom of the trench as it was cut. The trencher was preceded by a wheeled submarine. Overall navigation guidance was by the SYLEDIS positioning system.

An in-depth study was carried out by SNC-Lavalin for the power cable crossing the Strait of Belle Isle, between Newfoundland and Labrador. This strait is dominated by the icebergs which scour the bottom. Many such scour marks are grooves, up to 10 m in depth. To protect the cables, it was planned to lay them in trenches cut into the very strong bedrock.

Removal of the overburden would be by either a powerful jet sled such as that employed for pipeline burial in the North Sea, or by a hydraulic suction dredge. Plows were not considered in the study but could be used in the overburden and weaker sediments.

Tests at the Colorado School of Mines showed that both upward cutting disc cutters and conical picks were effective in cutting a trench in the hard rock. The conical picks were much faster but had to be changed more frequently. The trenching equipment would be mounted on a tracked bottom-crawling vehicle, powered by electric hydraulics, and monitored by TV and side scan sonar.

The power cables were planned to be laid in the trench from a cable-laying vessel, by following a steel wire rope laid in the trench in an earlier operation. Specialized equipment for laying and monitoring the cables has been developed by several European contractors.

The sea was rough and stormy
The tempest howled and wailed
And the sea-fog, like a ghost
Haunted that dreary coast,
But onward still I sailed.

Longfellow, "The Discovery Of The North Cape"

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17

Topside Installation

17.1 General

Over the past years, almost all topside facilities have been first fabricated into modules and then transported by barge and set on the platform by an offshore derrick barge. The capacity of offshore derrick barges has steadily grown to where 1200-tn. modules are commonplace and individual lifts of 4000–11,000 tn. and more have been made.

A further extension of this trend to prefabrication has been the innovative development and application of the Float-Over System, in which a complete deck, with all its equipment and facilities installed, has been floated over a jacket and set down. So far, this has been restricted to relatively calm seas, such as the Arabian-Persian Gulf, offshore West Africa and in the Timor and Philippine Seas.

The purpose of using large modules is to enable more of the fit-up and testing to be completed at the shore site. Not only does this allow the work to be done under optimal conditions, it also disperses the work so that it can be accomplished concurrently with other modules and other structural work.

17.2 Module Erection

The modules are set by a crane barge onto the module support frame, which is a skeletonized deck structure (see Figure 17.1). Some will be set onto skid beams and skidded and jacked to final position; others may be set directly. The modules must be structurally adequate in themselves, both for the temporary loads imposed during transport and installation and for the permanent loads due to the operations and environment. The structure of each module must first support the vessels and the piping within it and then transfer the forces developed by dead and live loads and environmental loads to other modules or the module support frame.

Lifting of such extreme loads must follow the general principles of heavy offshore lifts outlined in Section 6.3 and must be thoroughly engineered for all stages of the operation. Picking points and padeyes must transfer the forces to the slings. The slings, with their angles in three-dimensional space, must in turn transfer the loads to the hook. Where more than one crane will be involved in the lift, the interaction of loads between the cranes must be considered, including the effect of tolerances in boom position, sea-induced motion and the change in the derrick barges' water planes as the load comes onto them.

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FIGURE 17.1
Large module being lifted onto dock of platform. (Courtesy of Aker Marine.)

When picking a module with multiple pickup points, deflections must be carefully controlled so as not to distort the equipment and piping. Very elaborate rigging systems often result. The supporting structure may have to be stiffened.

Picking loads are dynamic; adequate allowance must be made for dynamic amplification in lifting force, as well as in lateral swing. This latter can be greatly reduced by power-controlled tag lines. Low-temperature effects, possibly causing embrittlement under impact loads, need to be addressed and suitable steels and welding procedures adopted. Many modern heavy lifts of modules are assisted by onboard computers monitoring the loads, the radii, and the position of the boom. Some crane barges are equipped with boom tip motion sensors and onboard computer systems to determine the best headings and boom angles to minimize boom-tip motion.

Modules are usually loaded onto a barge at a shipyard or shore base by skidding out, much as a jacket is loaded out. Dimensions are smaller and total weight much less, but loads may be more concentrated. Alternatively, they are loaded by transporters. The modules must then be properly tied down for sea.

Engineered slings are pre-attached to each module so that all that remains to be done as the lift commences is to raise each sling up over the hook by means of the crane whip line. Meanwhile, the tie-downs are cut loose.

When sea conditions appear favorable, the module is lifted clear of the barge, slowly moved astern or rotated to position, and set in its place (see Figure 14.3). Auxiliary means, such as powered tag lines on the deck of the platform, tapered guides, and fenders are used to help seat the module in correct position. The module needs to be set

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down smoothly and quickly so as not to expose the operation to higher waves or low-cycle fatigue. It is desirable to incorporate tolerance in initial positioning into the structural design. Once the unit is set, jacks can then move it to its final exact position. Jacking points should be provided in the module frame.

The problem of overhaul during set-down—that is, of getting rid of a load from the hook when there are up to 24 or more parts of line in the hoist blocks—is a difficult one. A free overhaul clutch for the crane hoist is the primary solution. In some cases, ballast may be transferred to the stern of the crane barge as soon as the load touches down. The object is to prevent the load from being inadvertently lifted back up if a subsequent wave raises the derrick barge's stern before there is enough slack in the falls.

Some rather spectacular lifts of modules have been made by the use of two or even three crane barges working in concert. The three Statfjord A quarters modules were too high (40 m) to be lifted by a single crane. The weight of each, about 1000 tn., was not too unusual, but the height and profile required that three crane barges be used. These were moored together with all deck winch controls and dynamic-positioning thruster controls at one control location. The three barges picked up the module at the dock, transported it to the concrete gravity platform moored in the fjord, and repositioned the barges while carrying the load. At the final lifting site, the barges were moored to the structure. Because of the short length of lines, nylon rope was used in order to have some elastic stretch to accommodate surge as the quarters modules were then raised and set on skid beams mounted on the deck frame. Each module was then skidded sideways to its final position. The pick had to be engineered with extreme care, since the load exceeded the capacity of any one of the crane barges.

On a subsequent platform, similarly high and heavy modules were set by a semisubmersible crane barge. Since this work was carried out in a fjord, the semisubmersible was not selected for minimal response to seas but rather for its extreme height when deballasted to ride on its pontoons. Now it was able to lift the module over the deck structure and to set the module directly in place. Up on the platform deck, each module was then welded to the module support frame.

A typical series of modules will include the following:

- Utilities modules
- Control room module
- Quarters modules
- Helideck
- Wellhead module
- Separation module
- Dehydration module
- Pig-launching module
- Generator module
- Switchgear module
- Metering module
- Bulk storage modules
- Pedestal cranes
- Drilling modules
- Drilling derrick
- Flare stack
- Casing and drill string laydown racks.

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17.3 Hookup

The hookup of these modules and their subsequent testing is highly demanding in terms of both manpower and support (see Figure 17.2). It delays the start of production of oil or gas and hence adversely affects cash flow. In recent years, the complexities of hookup have led to overruns in cost and time of 100% or more. To reduce these problems, the first step is to use larger and fewer modules—that is, more self-contained modules. A second step is to space the modules apart by 1 m or so, to allow a crawl space for access for interconnection. A third step is to use flexible connections to the extent permissible for the high operating pressures in the pipeline connections.

Careful control in tolerances of all interconnecting points at the time of module fabrication is essential. Templates and pre-matching may be used to ensure compatibility.

Hook-up offshore is very demanding of personnel and very costly. If the hookup work is supported by a semisubmersible derrick or "floatel," a suitable gangplank or walkway is required. This must have rollers to accommodate surge of the semisubmersible. It must be supported, for example, by cantilevering, so that it will not fall even if the barge drags an anchor or parts a mooring line. Sophisticated articulation and hydraulic compensators are often required to accommodate the barges' relative motions.

FIGURE 17.2
Hook-up of topsides of Condeep Platform. (Courtesy of Aker Maritime.)

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Fire protection during hookup is critical and must take precedence over actual work. This requires the early installation of a fire pump casing, the submersible pump, and headers around the platform decks. Until the platform system is fully established, fire hoses must be led over from the tending semisubmersible or derrick barge. A fire alarm system must be hooked up.

Life safety must be ensured; this means that life jackets and safety lines must be employed initially, that lifesaving capsules must be placed with the initial modules, and that a patrol boat must be on duty to pick up anyone who falls overboard. The technically specialized workers are usually employees of subcontractors and may not normally work offshore. Hence, they will need special instruction in safety offshore.

Other services and systems on board the platform must be activated at an early date. These include the generators (both primary and emergency) with their diesel fuel supply tank, and lighting systems for night work. A freshwater system must be established for potable water supply and wash down. Compressed-air systems are required both for instrument air and for utilities and tools.

Radio communication to shore and boats must be established, as well as public address systems throughout the platform. A smoke-detection system is needed in the quarters modules, as well as sprinkler and fire alarm systems. At the helideck, a foam extinguisher system must be installed. The rope landing nets must be installed and the landing lights activated. Ventilation systems must be activated in the quarters and within the facilities area.

Because during the hookup stage welding is the principal item of work, welding generators must be installed and cables led around the deck. Heated welding rod storage needs to be provided.

In addition to the permanent platform cranes and hoists, temporary hoists and powered winches will be required during hookup. Adequate lighting must be provided for night work. Temporary shops and offices are needed. An x-ray lab is required. There must be an electrical shop, an instrument shop, and a general tool shop and warehouse for bolts and pipe flanges. A paint shop is required, with a separate fire-extinguishing system. Finally, a first-aid room must be available for emergency treatment of personnel who suffer, for example, burns and eye injuries.

It is obvious from this long summary listing that each module should be equipped as far as possible with the items needed to complete its hookup. Beyond that, very thorough and detailed planning is needed by engineers, craft supervisors, and their craft supervisors, to insure that all needed supplies and materials arrive with the modules and do not require separate lifts. However, these separate items cannot be loosely stored in the module but rather must be properly boxed and secured so that they cannot be displaced during the lift. Further, their weight needs to be computed and added to the calculated lift weight. The module lift weight then becomes the sum of the equipment, piping, cables, module frame, lifting gear, including slings, and the tools and supplies stored on board.

Vertical access has proved to be a significant problem, due to the heights involved from waterline to deck, and the large number of personnel. In addition to well-constructed stairways, construction lifts for personnel are usually installed.

17.4 Giant Modules and Transfer of Complete Deck

For very heavy offshore lifts, a large semisubmersible derrick barge with two cranes at the stern can used both of its cranes in concert Combined capacities of 10,000 to 13,000 tn. are thus available. It is vital that such operations be conducted, free from long-period swells. Work is carefully scheduled for the weather window. The derrick barge may sit several

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days awaiting suitable seas. Of course, the large semisubmersible barge does not react to short-period wind waves of moderate height, but it is sensitive to longer-period wave energy, such as that from remote storms. A complete deck or very large section of the deck is constructed on a quay wall or in a prefabrication and assembly yard adjacent to the quay wall. It is then either skidded onto a large transport barge or walked out by multiwheeled transporter, on which each of the many wheels has its load equalized by hydraulic jacks. The transfer onto the barge requires that the barge is either grounded at the quay wall or equipped with a variable ballast system enabling it to adjust to the change in load as the module moves onto the barge. The barge then is towed to the site and moored at the stern of the crane barge. The cranes lift the module above deck level, the barge is towed clear and the barge moves into the platform. It then lowers the module onto the deck, using a guide system.

The complete deck of the Ninian Central Platform was transported by barge to a sheltered bay in Northwest Scotland. There, a catamaran barge was positioned around it and one end closed by a truss span. On each barge, vertical frames were mounted so that the deck could be picked by high capacity screw jacking rods.

When the deck had been raised, the transport barge was removed and the second end of the catamaran closed by a raised truss. Then the catamaran, with suspended span, was towed to the mating site.

Meanwhile, the large concrete offshore caisson was towed to the mating site and ballasted down. The catamaran was positioned around the caisson and lowered the 20,000 tn. deck structure onto it. Loads on the support points were equalized by flat jacks, which were then grouted.

With all such immense deck modules, the deflections and stresses due to dead load during picking need to be carefully computed and special means taken to accommodate the dimensional changes in support locations that occur as the module is picked and then set.

The DNV Rules contain an appendix for Heavy Lift Operations, which is applicable to these operations.

17.5 Float-Over Deck Structures

17.5.1 Delivery and Installation

The delivery and installation of a complete deck, with all systems fully hooked up and tested, has many advantages, from both cost and schedule viewpoints. The complete deck can be prefabricated at a shore base, transported to the site, and set on the jacket legs or gravity-base shafts (see Figure 17.3).

As noted, complete decks have been transferred by barge onto more than 25 concrete gravity-base platforms, but this has been carried out in inland waters where relative motions were very small. Even there, provision has been made to cushion the seating to prevent concentrated loads, and to make final adjustment of relative position by hydraulic jacks, sliding on Teflon pads. See Section 12.2 Stage 12 for a detailed description of the procedure. In this section, the extension of this concept to the open sea is described.

The detailed engineering for the transfer at sea of 20,000 tn. or more must consider the 6 degrees of freedom affecting the barge. It must also include the differing deflections of the deck due to changing support conditions and the consequent changes in dimension at the stabbing points as the load is finally transferred to the substructure. Differential thermal

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FIGURE 17.3
Integrated deck of Statfjord C platform.

expansion must be considered. Finally, means must be developed to equalize the load between the four or more legs after the deck support has been transferred.

The transfer in the open sea, subject to long-period swells, is a much more complex problem than that performed in inland waters. In most cases, the transfer operation first positions the barge between or around the legs (shafts) and dampens out the relative motions. During this phase, the legs of the jacket must be protected against impact due to surge and sway of the barge. Then, the lowering process has to consider the impact in heave, as amplified by pitch and roll. Even more critical is the rapid removal of contact of barge and deck, so that the barge does not strike the structure in the period immediately following transfer. Several methods have been developed and successfully used to carry out a float-over installation in the open sea.

The Float-Over method requires careful consideration of the prevalent waves and swell, not only as to height and period but also as to direction. The range of the tide can be both favorable and unfavorable.

Shock loads are developed as the load is transferred to the jacket legs. Shock loads can also occur once transfer is completed, the swell raising the barge to impact the deck. Rapid removal of potential contact is essential (see Figure 17.4).

High capacity ballast and deballast pumps should be provided.

The barge, with deck loaded, has to have transverse stability but be narrow enough to fit between the jacket legs. Stability and longitudinal strength must be adequate for both transport and at the critical stage when the deck is lifted above the barge.

Lateral impact against the jacket legs is restricted by polyester lines (or a combination of nylon and polyester) attached to winches on the barge. Fenders are fixed to the jacket legs.

For a gas platform in the Eastern Timor Sea, 8 shock-absorbing leg-mating units were installed on each of two platforms. Each had a capacity of 2000 tn., with a deflection of 800 m. Smaller deck-supporting units, to cushion the deck against the heave of the barge, were also employed. The integrated decks weighed up to 13,900 tn. each.

Larger shock-absorbing units, with up to 10,000 tn. vertical capacity, are currently under development for use in mounting the decks on concrete GBS platforms.

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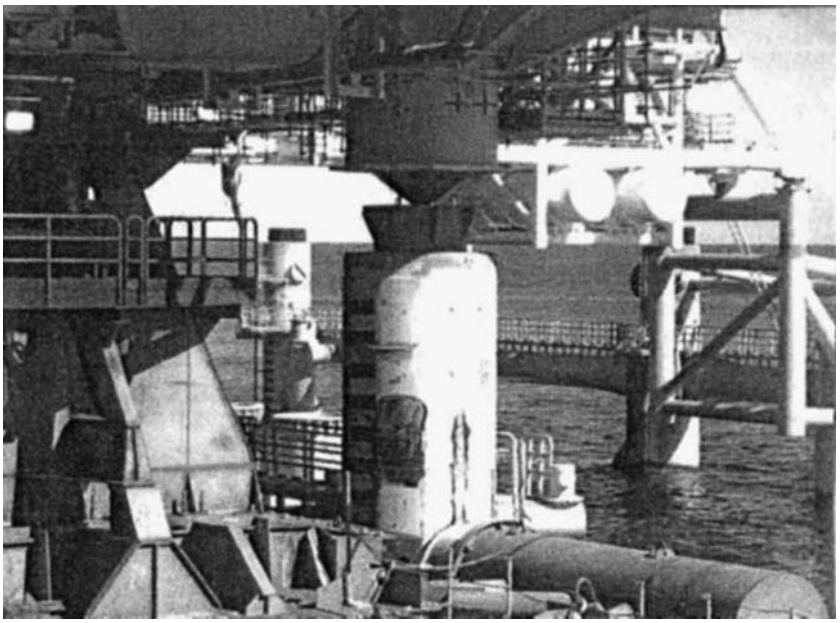

FIGURE 17.4
Deck being lowered onto mating cones of high-capacity shock absorber. (Courtesy of Conoco–Phillips and Clough-Aker.)

17.5.2 Hi-Deck Method

The Hi-Deck method has been used to install several completely outfitted decks in the North Sea, including those for platform Maureen and the Hutton TLP. The deck structure was transported on a steel frame 17 m high, supported on the barge to clear the top of the jacket legs. The barge carrying the deck was maneuvered in between the legs. Composite mooring lines of steel wire and polyester-sheathed aramid fiber (Kevlar) were run to the platform legs, in order to dampen out relative motion in the horizontal plane. Large rubber fenders were secured to the jacket legs. During mating of the Hutton deck, relative motion was limited in design to 200 mm but in the actuality, only 60 mm relative displacement was experienced.

Vertical lowering was by rapid ballasting. Several separate shock-absorber systems were installed to absorb the impact as the hydraulic catch probes engaged the cones in the deck. One system consisted of 1.5 m pillars of polyurethane enclosed in telescoping steel casings. Another used hydraulic jacks, suitably softened by connecting the hydraulic tanks to nitrogen-filled bladders. Model tests of relative motions during transfer proved more accurate than elaborate computational analyses. The operation was successfully carried out in 1.5 m swells.

17.5.3 French "Smart" System

The French "Smart" system has been successfully used for the installation of a deck in the long-period swells offshore Congo. The deck structure is designed with vertical tubulars that match the jacket legs of the platform on which the deck is to be installed. "Smart" legs are pipe columns within the deck tubulars. They are extended by long-stroke hydraulic jacks. Smart struts extend horizontally from the barge, also with hydraulic jacks. These are used to control the relative movement of the barge in relation to the jacket legs in surge,

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sway, and yaw. "Smart" releases are doubly hinged supports which can be collapsed by activating a hydraulic cylinder to rapidly give 3–4 m clearance between the barge and the underside of the deck structure. Rapid removal of the barge is essential to prevent impact on the next heave cycle of the barge.

Operations proceeded by gradually moving the barge-plus-deck combination in between the jacket legs until the "Smart" legs are directly above their top. The "Smart" struts engage the jacket legs and gradually dampen out the surge, sway, and yaw motion. Each strut is hinged to accommodate heave while the other end rolls on a reinforced pad on the jacket leg. Then the "Smart" legs are extended down to engage the top of the jacket legs. The load is gradually transferred. Then the "Smart" releases are tripped so that the barge has clearance beneath the deck and can be extracted from between the jacket legs. These operations were successfully carried out in a 1m swell.

17.5.4 The Wandoo Platform

For the Wandoo platform, on the Northwest Shelf of Australia, a deck structure, fully outfitted, was successfully transferred by the float-over method. A heavily reinforced barge carrying the deck was maneuvered in between the concrete shafts of a GBS. The deck structure was supported high enough to clear the shafts. Composite mooring lines of steel wire and polyester-sheathed Kevlar were used to position the barge accurately. The barge and deck were then ballasted down onto hydraulic jacks set in pairs atop each of the shafts. To enable rapid release of vertical supports, large sand jacks were used. Their openings were sized, on the basis of tests, to empty in about 1 min. Then the barge had ample clearance under the deck underside and could be pulled clear.

17.5.5 Other Methods

Other methods of float-over are under development, including a large catamaran constructed of vertical concrete caissons. The "Versatruss" method, used for the removal of complete decks from decommissioned platforms, involves the use of inclined steel struts. It has been used to raise a deck which had been skidded onto a trestle, then transport it across the Caribbean Sea to Venezuela and lower it down onto a pre-installed jacket.

The development of long-stroke hydraulic jacks will continue to be a major factor in the extension of the Float-Over concept. The Float-Over method was recently (2003) used successfully in the Philippines and has become the current state-of-the-art in the Arabian Gulf. The successful extension of the Float-Over method in the open sea requires increasingly sophisticated engineering.

Or where the Northern Ocean, in vast whirls,
Boils round the naked melancholy isles
Of farthest Thule, and th' Atlantic surge
Pours in among the stormy
Hebrides.

James Thomson, "In Autumn"

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18

Repairs to Marine Structures

18.1 General

Marine structures are subjected to degradation due to environmental, operational, and accidental damage. Of the environmental agents, especially in seawater, corrosion is a principal and universal problem for steel, whether structural steel or reinforcing steel in a concrete structure.

Corrosion is an anodic–cathodic reaction. The cathode is powered by the oxygen from the air, while the anode is the point of discharge of the current. Most commonly, these two points occur in close proximity, but, due to the conductivity of steel, may occur at a considerable distance if there is an insulated zone between.

Normally corrosion is inhibited by concrete encasement, since the cement forms a complex ferrous hydroxide coating on the steel. However, this protective coating is readily dissolved by carbon dioxide from the air in the pressure of moisture and by chloride ion, this latter from the seawater. Corrosion is also caused by the currents set up by dissimilar metals, such as copper or even some alloys of stainless steel. Crevice corrosion can occur at joints and splices.

An especially vulnerable zone is the interface between concrete encasement and bare steel, at the location just inside the concrete. Since concrete has widely been used to cap steel sheet piles above low tide, the junction has become a frequent source of serious corrosion.

Corrosion is most rapid when the water currents flow. It occurs in both cold and warm water—cold water contains more oxygen and warm water, of course, speeds the reaction. Once corrosion has been initiated, it is very difficult to stop.

Among the other environmental attacks is abrasion from silt suspended in the water, which not only removes steel but also acts synergistically with corrosion by removing the earlier rust coating, thus exposing bare steel to corrosion. Moving sand along the seafloor (in the surf zone, for example), sulfate attacks on concrete, as well as thermal and wetting-and drying-cracks and freeze–thaw attack, are other degradation agents for marine structures.

Physical damage is caused by impact from sea ice, driftwood, and by impact from tugs, barges and even colliding ships. These may damage the member struck and also lead to overload of adjacent members.

Whether the problem is physical or environmental damage, it is important to determine the primary and secondary causes. Otherwise, the repaired structure may suffer a repetition of the first problem. While often the cause is clearly obvious, other cases may involve the interaction of two or more phenomena, such as one physical combined with one environmental (chemical).

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Marine structures are exposed not only to the extreme conditions of the environment such as wave slam and fatigue, but also to accidental impact from objects dropped off the platform. The list of such accidental events that have occurred over the past several decades is myriad. It includes ramming by a supply boat that went full ahead rather than full astern, impact from the reinforced corner of a cargo barge, and impact by a derrick barge whose mooring lines had parted. The dropped-objects category includes a number of pedestal cranes pulled off their supports when they attempted to follow the movements of a supply boat and thus exceeded their allowable radii, as well as dropped drill collars, casing, a mud pump, and pile hammer. Anchors have been dragged across a pipeline. Leaks into underwater compartments have developed through corroded and ruptured piping and valves.

In the environmental category, horizontal bracings near the waterline have been excited by vortex action and subjected to vibration beyond that for which they were designed, with consequent failure by fatigue. High currents can also cause vortices, leading to fatigue. Mooring lines and attachments are very vulnerable. Scour has on occasion undermined a pipeline, to develop long, unsupported spans, which then ruptured under the cyclic loading due to vortex shedding. Defective welds and heat-affected zones have led to crack development, and its subsequent propagation has been accelerated by corrosion in the crack. A platform may also be damaged by operational failure, which leads to flooding or overloads. Scour may undermine the legs and lead to excessive lateral response.

18.2 Principles Governing Repairs

When damage occurs, repairs become necessary. Obviously, each such undertaking is highly case-specific and needs to be engineered appropriately to the particular needs of the situation.

One fundamental principle is that carrying out of repair must not increase the risk of failure. This may require auxiliary strengthening before the damaged member is cut out for repair. It may mean that repairs must be delayed until a more favorable season, when the environmental loads are reduced. It may mean that limits are placed on operations until the repairs are completed. Repairs may have to be carried out step by step. A second principle is that the repaired or reconstructed element must not adversely affect the performance of the structure. As an example, if the reconstructed member will be significantly stiffer than the original member, a dynamic re-analysis of the entire platform may be required.

In order to reduce the time of repairs to a minimum (not only to limit the costs but especially to limit the time when the platform is in a weakened condition), the repair procedures must be planned in great detail and all necessary tools, rigging, and fittings provided as one package. For complex repairs, a rehearsal with the crew may be advisable to reduce the problems in communication during the actual repair.

Many repairs are located near the sea–air interface, and hence the repair work will have to be carried out under conditions of wave turbulence due to incident, refracted, and reflected waves. Some local protection might be provided by the derrick barge acting as a floating breakwater.

A major problem with large offshore structures is that of identifying locations so that divers, ROVs, and submersibles may readily return to specific spots. The pre-installation of large, highly visible (yellow or orange) numerals on platform legs and walls is now routine practice. However, for a specific repair, additional local markers must be installed.

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Attachment of a tensioned wire guide line as a first step will facilitate descent and location and help the diver to maintain position, even in a strong current.

Cleaning of marine growth is another operation which must be carried out at an early stage. High-pressure water jets are the most efficient means, although in order to thoroughly inspect or commence repair of a crack in a weld, supplemental wire brushing may be necessary.

Inspection can be carried out by divers, a diving bell, or an ROV with video. The latter two are replacing much of the diver work in order to reduce costs and increase safety. Especially when it is necessary to inspect inside the jacket frame, the ROV will eliminate the potential for divers to be trapped. When divers must go into a frame or under a structure, at least two divers must work as a team, with one tending the lines for the other. As a first operation, a guide line can be carried in and attached, enabling the diver to easily retrace his path.

When corrosion is the cause of damage, the repairs must include a determination of the cause. Steps, such as wrapping or installing cathodic protection, either sacrificial anodes or impressed current, must be instituted to prevent ongoing corrosion.

18.3 Repairs to Steel Structures

When diagonal or horizontal braces have been fractured or badly distorted, the damaged section can be cut out. By use of a template and careful measurement, a "pup" is now cut to exact length and end profile, with the ends beveled for full-penetration welding. External clamps are used to hold the pup in position. Now a habitat is placed around the brace and dewatered, and the full-penetration welds are made (see Figure 18.1). Alternatively, underwater "wet" welding may be employed, provided tests show that satisfactory quality can be attained.

Another solution to this problem is to cut out the damaged portion of the brace and then slip in a longer section of slightly smaller-diameter but thicker-walled pipe to which packers and grout fittings have been attached. Alternatively, the pipe may be of larger diameter and slipped over the two undamaged ends. This insert pipe may be made in sections of threaded casing, to enable a longer length to be inserted or slipped over or through a small cutout. By the use of cementing techniques similar to pile-to-jacket sleeve connections, the strength of the brace may be restored. The packers are inflated at each end and the grout injected and vented into a standpipe at the high end to ensure that all water has been ejected, as in Figure 18.2. A third scheme uses both an internal and external sleeve. Only the external sleeve is structural. The internal sleeve is merely an internal form to facilitate grouting (see Figure 18.3). Special grouts are used, having expansive

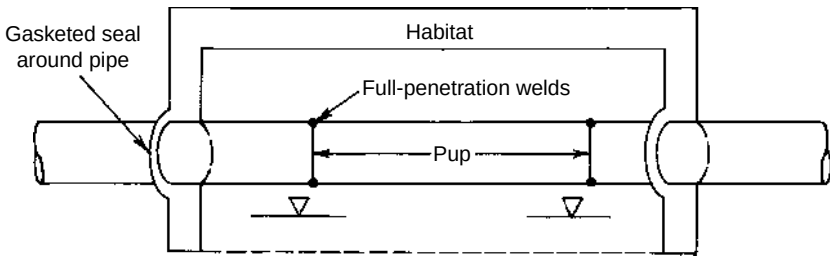

FIGURE 18.1
Repairs to damaged braces, scheme 1.

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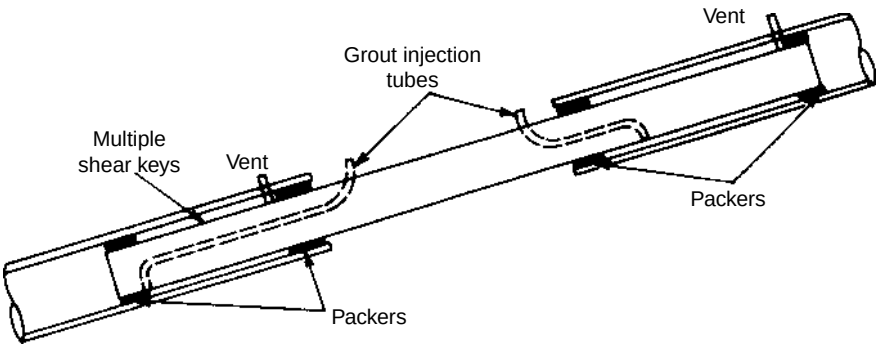

FIGURE 18.2
Repairs to damaged braces, scheme 2.

and high-bond capabilities in order to develop the transfer in as short a length of overlap as possible. The provisions of API-RP2A concerning grout transfer from pile to sleeve are followed as a guide. Multiple weld beads or shear keys are used to enable grout bond-shear transfer in a short length. Epoxy injection is an alternate method.

Another approach involves the insert of a heavy internal structural member of smaller diameter than the original tubular. This new section can be a heavy-walled tubular with shear rings welded on the ends or even a rolled shape (for example, an H pile). At the lower end, a pig or packer is attached. An external sleeve is clamped over the gap, using two half sections, with flanged and gasketed joints. Then the brace is pumped full of concrete. The concrete is actually a fine concrete, using cement plus sand, with the sand graded up to about 6 mm maximum size (see Figure 18.4).

Hydraulic expanders are in the process of development, primarily for use in expanding piles against jacket sleeves to transfer the load by a combination of direct shear on the corrugated surfaces and friction, thus supplementing or replacing load transfer by grout. This "swaging" process appears especially useful for repair of damaged tubular bracing. It is, of course, to be noted that all these methods are but variants on one scheme, with effort directed toward simplicity and reliability.

Other approaches have been used, especially where axial loads are light, involving clamped external sleeves. These may be in two halves and use high-strength bolts to draw them in tight against the arms of the original brace. Load transfer is by friction of

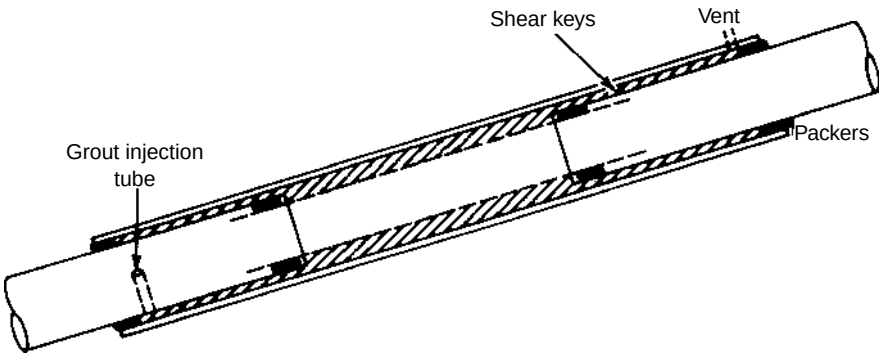

FIGURE 18.3
Repairs to damaged braces, scheme 3.

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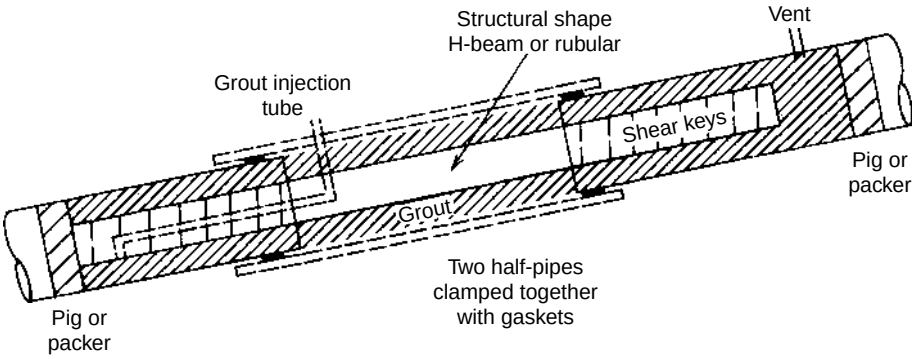

FIGURE 18.4
Repairs to damaged braces, scheme 4.

steel on steel. In the late 1980s, Chevron reported the discovery of a serious fracture in an underwater tubular brace at the Ninian Southern platform. The fracture was discovered by an ROV during routine inspection. The fracture actually was a complete severance of the tubular and is believed to have been due to a faulty weld on a fabrication window used to gain access for a backup weld at a node. It is interesting to note how many serious fractures in tubulars have occurred, not at the critical nodes, but rather at temporary closures or attachments; the Alexander Kjelland tragedy was due to a faulty weld in attaching a sonic transducer.

Repairs were carried out on the Ninian Southern Platform at a depth of 43 m. The tubular was 1200 mm in diameter, fabricated of 30 mm plate. Because the brace would have to take cyclic tension, the decision was made to use a welded repair. It was feared that a bolted and grouted connection might not have the necessary fatigue resistance under the required stress ranges and number of cycles involved.

Oceaneering, Inc. built a specially-designed underwater habitat in which a team of saturation divers would work. The fractured faces were removed and sent for a metallurgical examination. The two ends of the braces were now prepared for the weld. Then a lead-filled template was fitted to each end of the tubular to get an exact impression of the exposed ends. From these impressions, transition pieces of 500 mm plate were fabricated and then welded in place, leaving a gap between them to allow access for a backup weld on the insides. Special collars had been welded around the transition pieces. High-strength bolts were now tensioned to draw the collars together with 400 tn. of tension force.

A 50 mm "pup" (a closure length or spool) was now fitted into the gap and welded from the outside. The cooling of the weld added to the pretension in the brace, so that the final stress was 600 tn. of tension, thus restoring the state of stress that was originally in the member under static load conditions (see Figure 18.5). All welds were inspected and the platform was recertified.

Cracks in welds may be treated in a number of ways, depending on their size and extent, the service required, and the cause of the fracture. In any event, the external surface must be ground smooth. After inspection, if the crack is in the non-severe category, small holes may be drilled at each end to act as crack-propagation arrestors. The holes and crack may now be filled with epoxy, using crack-injection techniques. More serious cracks must be gouged out and re-welded, using either a habitat or wet-welding techniques. Cracked nodes have been repaired by clamping on an external nodal sleeve made up of as many sections as required and injecting the space with epoxy. In some minor cases of dented jacket bracing or legs, it may be sufficient to fill the leg with grout to inhibit compressive buckling.

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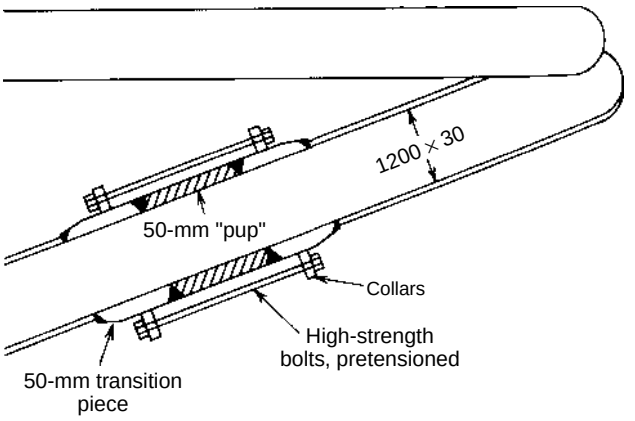

FIGURE 18.5
Underwater repair to tubular brace of Ninian platform.

18.4 Repairs to Corroded Steel Members

Repairs to corroded steel members can be accomplished both above and below water. Where the corrosion is confined to the splash and atmospheric zones, short term protection can be provided by wire brushing or sand/shot blasting, followed by painting by zinc silicate or coating with zinc-enriched epoxy.

Long-term protection of underwater structural and reinforcing steel can be provided by cathodic protection, either sacrificial anodes or impressed current. Sacrificial anodes of zinc, usually alloyed with aluminum or other metals, are suspended in the water, from which electrons flow to the steel. Since these travel only in straight lines, they need to be placed where they can "see" the steel area to be protected. They need to be electrically connected to the steel member and the steel needs to be bonded.

Impressed current cathodic protection involves the provision of a small amount of direct current, obtained by a rectifier, attached to the steel. The amount of current flow must be monitored and modified periodically to ensure against over-protection which would lead to embrittlement of the steel.

Wrapping has also proven effective and, in many cases, more practicable. The wrapping systems seal against further entry of seawater and oxygen and carbon dioxide. The best wrapping system consists of an inner felt layer and a flexible outer jacket. The inner layer of felt is fully saturated with petrolatum liquid and a corrosion inhibitor. The outer layer is a strong and flexible polymer sheet that protects the inner layer. In installation, the pile wrap jacket is tightly wrapped around the steel or concrete member with stainless steel fittings.

18.5 Repairs to Concrete Structures

ACI 546 R-04, Concrete Repair Guide, is an excellent reference for above-water repairs. A companion guide for underwater repair is now in preparation.

The most common problem is corrosion of reinforcing steel in piling in the splash zone and lower part of the atmospheric zones, as well as the lower edge of beams and girders that are intermittingly splashed by sea water, leaving a deposit of salt by evaporation, which thus becomes concentrated.

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By far the greatest number of problems with concrete piling occur in the splash zone, generally corrosion of the reinforcing steel, but occasionally longitudinal cracking and spalling of the concrete cover over the reinforcement from other causes. These other causes include seawater attack (principally sulfate and magnesium ions), alkali–silicate reaction, cyclic wetting and drying, abrasion, and cyclic thermal variations, including freezing and thawing. Inadequate curing may lead to axial cracks or excessive permeability. Bursting strains and tension rebound during driving may occur both above and below water as may alkali–silica reaction. Rock-boring mollusks have bored deep holes in concrete piles made with limestone aggregates in the Arabian–Persian gulf.

Abrasion from moving sands may occur at seafloor and from moving ice at sea level. To insulate the pile from further degradation and provide protection of the concrete and embedded steel reinforcement and prestressing tendons, the pile may be wrapped in a polyurethane impregnated fabric, enclosed within an external composite sheath such as polyethylene (HDPE). Several such systems are commercially available.

Coatings have had only a fair performance, being subject to delamination, disruption by water vapor bubbles and reflection of the concrete cracks below. Metalizing with aluminum–zinc alloy is expensive, but effective. The Japanese have coated the steel tubular piles of the Trans-Tokyo Bay Bridge with titanium.

Coal tar epoxy coatings, such as Inertol, have given excellent performance when applied to new concrete. However, environmental concerns over toxicity may limit their use. Applied over repaired concrete, they render it impermeable but may be subject to pockmarks from escaping water vapor and may reflect active cracks. Polyurethane coatings are more flexible due to reduced bond and will not be subject to reflective cracks.

Encasement of steel and concrete members with reinforced concrete jackets has been extensively applied but has its limitations for some applications. It adds weight, increases stiffness, and increases wave and current forces. Application must include thorough preparation. When placing concrete or grout between the jacket and the pile, it is necessary to consider the net hydraulic head or pressure. Both precast jackets and cast-in-a-form methods have been used.

Wrapping of concrete piling which has been subjected to seawater attack or even corrosion of reinforcement appears to be the best method for protecting the pile from further degradation. It is being applied to all the prestressed concrete piles of the marine piling of the Ford Island Bridge across Pearl Harbor, Oahu, Hawaii, both those showing early signs of seawater attack as well as those still in sound condition (Figure 18.6).

FIGURE 18.6
Wrapping piles for corrosion and seawater protection, Ford Island Bridge, Pearl Harbor, Hawaii. (Courtesy of Manson Construction Company.)

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The wrapping consists of a feet impregnated with anti-corrosion petrolatum grease, encased in an enternal shield of HDPE or polyester. It has reportedly performed well in an earlier application to concrete piling in Norfolk, Virginia and on steel piles in New York.

Cathodic protection is difficult to apply and maintain, due to the many elements of reinforcement. However, it has been successfully carried out, especially when the individual bars are bonded to each other. Sacrificial anodes can be attached in brackets on the face of the concrete, electrically connected to the reinforcement by studs.

On the extensive Seawater Cooling System canals in Jubail, Saudi Arabia, wide-spread corrosion occurred soon after construction. Not only were the limestone aggregates permeable in themselves but the cement selected, ASTM Type V, had zero tricalcium aluminate (C3A) which bonds chemically with chlorides from the seawater to form an impermeable chloro-aluminate.

Sacrificial anodes of zinc–aluminum alloy were installed on the face of the concrete walls, bonded by studs to the reinforcing steel. At Jubail, they proved very effective, protecting the reinforcement up to about 300 mm or more above water line due to partial saturation by capillary action.

A similar system was adopted for protection of a cooling water intake structure on the California coast. Attempts were made to extend the effectiveness of the cathodic protection by covering the face of the wall with a highly absorbent felt, but were only partially successful.

Once a year or so, the anodes need to be wiped clean by a diver. Otherwise they become coated with zinc oxide and are less effective. Anodes are slowly consumed; depending on their size and the alloy selected, they last 5–20 years.

Corrosion occurs only with a relative humidity between 30 and 80%. Hence, keeping the environment completely dry or fully saturated inhibits corrosion. For the portions of the Jubail Seawater Cooling system in concrete pipes of a type for which cathodic protection was impracticable, complete filling with seawater arrested further corrosion.

Corrosion of reinforcement below an elevation a meter or so below extreme low tide is substantially inhibited by the lack of oxygen and the impermeability of fully saturated concrete to oxygen. Thus, there is no contiguous source for cathodic reaction.

In an advanced form of induced current cathodic protection, after cleaning of the surface by high pressure jet, titanium mesh is attached and connected to the reinforcement. The mesh is covered by a thin coat of cement mortar. Then the direct current is applied, causing the reinforcing steel to become cathodic to the titanium anodes. This system will protect concrete in the above-water zones. Cathodic Protection cannot be applied to epoxy coated reinforcement, as it will lead to delamination of the coating.

Patch repairs to specific corroded zones are often are the best answer to repairs of girders, beams and walls, but require temporary enclosures. The steps involved are:

1. Chip out all cracked concrete and deeper, if necessary so as to expose all the rusted steel. Where the corrosion is all around, chip 35 mm behind the bars.
2. Clean the bar completely from all rust.
3. Coat with zinc silicate.
4. Wrap a zinc bracelet or similar at each end of each bar, where it enters the concrete—otherwise the corrosion will just re-occur at the periphery of the patch.
5. Carefully fill the patch with mortar, making sure to fill up under the top and behind the bars.

It is desirable to make the patch mortar chemically like the intact concrete, which is typically partially saturated with chlorides but varying in concentration with depth. The

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aim is to prevent the patch from becoming cathodic to the intact concrete. The French mix their mortar with some salt water. In much of U.S. practice, the patch is coated with epoxy, but this may be counter-productive. At present, it would seem best to attach the zinc bracelet, coat the bars with zinc–silicate and just use a mortar of good cement content and consistency.

While epoxy injection of cracks can be carried out underwater, using it with active cracks may merely cause the crack to be re-established adjacent to the newly-filled crack. It is best to inject a flexible polyurethane.

Beams, girders and slabs, as well as piles, can be structurally strengthened by epoxy-bonded carbon-fiber strips. Both tension and shear capacities may be restored or increased by this means. The concrete surface needs to be relatively smooth and clean. Epoxy glue is applied to the back side of the strip and the strip applied across the critical or damaged areas. In the case of piles, where high shear and moment often are critical just below the pile cap, the pile may be sheathed in carbon fiber applied under tension and grouted.

Steel shells may be welded to encase the pile and the space filled with grout under pressure.

Confinement by either method prevents fracture and spalling of the concrete cover, thus increasing the compression capacity in bending.

Both steel and concrete members may have their bending capacities restored or enhanced by external post-tensioning.

Concrete structures under construction or in service may be severely damaged by impact, collision or dropped objects. They may also be damaged by over-pressurization, in which the hydrostatic pressure on one side of a cell wall is accidentally raised beyond the structural capacity.

The Beryl A platform was badly damaged during construction of the cell walls by ballasting which failed to consider the differential hydrostatic head on the small interstitial cells ("star cells") at the intersection of the circular cells. This led to excessive tension plus shear, resulting in through-wall cracks and displacement of the joining walls. Extensive repairs were required.

These consisted of the following steps:

1. Removal of cracked and crushed concrete by chipping by hand or with a very small air hammer so as not to damage further the adjacent concrete
2. Where major displacement of the walls had occurred (up to 100 mm radially), the addition of an inner concrete wall heavily reinforced with additional steel, about 200 mm thick, tied to the existing wall with drilled-in and grouted studs and expansion bolts
3. Use of grout-intruded aggregate as a means of correcting wide cracks and strengthening walls
4. Use of cement grout injection in moderate-width cracks
5. Epoxy injection of the smaller cracks, carried out by sealing the cracks externally and then injecting a fluid epoxy until material of a similar consistency flowed out the next port above; those cracks were successfully injected against an external water pressure that ranged up to a 40 m head
6. Build up of the shell walls for compressive resistance by shotcrete or flowing concrete

Numerous cores plus acoustic testing verified the soundness of the repaired structure, and it was therefore certified for installation. The platform was installed successfully,

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despite having to sustain differential heads up to 100 m. The repairs have shown no problem in over 25 years of service.

When the Statfjord A platform was ballasted down to mount the deck, a small leak was noted in a juncture wall with another cell. Attempts to epoxy-inject this crack were less successful than with Beryl A. In the case of Beryl A, the cracks were through the wall and hence were crossed by reinforcing steel. In the case of Statfjord A, the crack was laminar and had no reinforcing across it. Even though the injection pressure was limited, the act of injection propagated the crack. A decision was made to limit injections and to fill the adjoining star cells with a long-term corrosion-inhibiting gel.

It would appear from the experience at Statfjord A as well with subsequent long-span bridges, that stitch bolts should be drilled in and grouted before attempting to epoxy-inject a laminar (in-plane) crack.

One of the shafts of a North Sea concrete platform was impacted by an anchor-handling barge whose heavily reinforced corner caused significant cracking at and just below the waterline. Some spray leakage occurred. Repairs were successfully carried out by use of an external work caisson lowered on the exterior of the shaft, clamped to it, and dewatered. The damaged concrete was then cut out and additional reinforcing bars installed. New concrete was then placed as grout-intruded aggregate, using a colloidal mixing process to increase the fluidity of the grout. Epoxy was injected into the construction joint to fill any shrinkage cracks; very little "take" was noted.

The loss of underpressure in a concrete oil storage platform may lead to cracking. Essentially this is an overpressure situation in which the oil inside, being less dense than the water with which it is automatically equalized at the bottom, exerts an upward pressure on the roof of the storage caisson, causing multiple horizontal cracks in the cell walls just below their juncture with the roof. The Draugen platform experienced some leakage during storms, due to the pumping action in the cracks.

In many cases such cracks can be repaired by epoxy injection, applied by divers from the exterior periphery of the base caisson. However, in the case of Draugen, the cracks were active cracks—that is, only open during cycles of wave action, but closed during the passive sea states needed for repairs. Hence, only partial sealing was able to be effected and some cells had to be taken out of service. Access to small cells such as star cells may be very difficult, and, depending on the circumstances, other means of crack-stopping may be preferred. For example, depending on a detailed evaluation of the design, it may be feasible to fill the adjacent star cells with a stabilized or gelled drilling fluid, which will prevent leakage and future corrosion. Where access is feasible, a flexible polyurethane may be injected.

Repair procedures for potential major damage to concrete structures have been developed and tested by Italian engineers in connection with the completion of the Genoa floating concrete dry dock. These included the case of severed prestressing tendons, for which the repair procedure consisted of cutting out the damaged concrete, splicing the severed tendons, and restoring the prestressing force by internal jacks.

More recently, Taylor–Woodrow demonstrated a means of repairing severed tendons underwater. The damaged concrete was removed by high-pressure water jets operated by divers. The damaged and distorted reinforcing bars were cut out by an oxy-arc burner. Broken prestressing tendons were then coupled to prestressing bars which were sheathed in ducts. A prefabricated reinforcing grid was then placed and tied to the existing steel. Prefabricated formwork was secured and gasketed to the existing structure. Then a high-strength concrete mix was pumped in from the bottom. After the concrete was cured, the formwork was stripped and the bars stressed and grouted. Tests showed that the structural strength was fully restored.

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External prestressing has been successfully used to repair damaged bridge girders and would appear practicable for use in underwater structural repairs, as well. Holes drilled into structural elements could be used to anchor the ends. Polyethylene-encased prestressing strands would have long-term durability.

Leakage has occurred in the base of the Beryl A platform, with water penetrating along nests of small-diameter grout piping. Cement grout injection in stages, using techniques developed in tunneling, has proved successful.

The recent development of carbon fiber "patch" and "strip" repair systems appears of particular importance for concrete marine structures. The patch is "glued on," using epoxy. Provided the fibers are properly oriented, the patch adds stiff reinforcement on the face. This has become the state-of-the-art method for repairing cracked beams and girders above water, and probably could be applied below water by using hydrophobic epoxy.

Repairs to the face of the dams and floors of stilling basins frequently become necessary due to erosion. Where deep grooves and holes occur, the floors of the stilling basins may be filled with self-leveling tremie concrete. Some cavities become very large, requiring hundreds of cubic meters of concrete. The repair material is essentially a flowing concrete mix, containing silica fume to promote viscosity and early strength, and to bond with the existing concrete. Both autogenous and thermal shrinkage can be minimized by using coarse sand and small rounded aggregate, such as pea gravel. It is often placed through an inclined tremie pipe which can be dragged along the floor. Fly ash or blast furnace slag can be used to replace a portion of the cement.

On inclined and vertical surfaces, especially where deteriorated or damaged concrete must be removed, a portable cofferdam is used. This is essentially a watertight box, strong enough to resist the maximum hydrostatic pressure and connected to the atmosphere above water by one or more access tubes. Either squeezing or inflatable gaskets are used to seal to the existing surface. The contact area must be cleaned and, in some cases, ground in order to provide a proper seat for the gaskets. The box should be locked to the face of the dam by stressed anchor bars top and bottom in order to prevent displacement in the event of accidental application of shear forces. These bars are drilled and grouted by a diver.

If inspection only is required, the box is not dewatered. Instead, after seating, fresh water or clean seawater is used to displace silt contaminated water, enabling clear underwater video.

Freeze–Thaw Attack is especially severe in northern and sub-Arctic zones where freezing weather combines with high tidal ranges and seawater. Thus, two cycles per day of freezing and thawing occur, along with saturation. The results are rapid and progressive disintegration of the concrete surfaces.

Epoxies containing tiny glass spheres were used to insulate concrete piling in the LNG Terminal at Cove Point, Maryland. Timber sheathing has been used on the coast of Sweden. These measures do not repair the damage, but only stop further disintegration. For above-water concrete, removal and reconstruction of the surface by concrete containing air-entrainment is a suitable repair.

18.6 Repairs to Foundations

The foundations of offshore structures are an integral part of the system that resists the environmental forces and support the operating loads. Hence, they must be maintained in their design condition.

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Scour around jacket-type structures are of two types. One is an areal scour, which lowers the seafloor in a dish-like shape. The remedy here is to dump or chute in rock which will have two characteristics:

1. It is small enough so that it will not work its way down into the soil.
2. It is large enough and dense enough so that it will not scour under currents and wave-induced forces.

A permeable material is required in order to prevent trapping wave-induced pore pressures, which can promote scour.

The above requirements can be met with a two-layered system, one consisting of small rock or gravel, the other of large rock, but this is very difficult and costly to place around and between the legs of a jacket at great depths. A single blended placement of rock is more effective, requiring more quantity of material but enabling placement in one operation. Another effective scheme for scour prevention is to replace the small rock by a filter fabric and then cover it with larger rock. The filter fabric can best be assembled in two layers, sewn together. One layer is a fine mesh to prevent sand migration; the other is a coarse mesh of heavy polypropylene, reinforced with stainless steel wire.

Mattresses can be made up of reinforced filter fabric with concrete blocks attached. They can then be lowered into place, using a steel frame. Mattresses filled with fine and coarse gravel have also been used. Still another system employs an integral filter fabric to which multiple closely-spaced neoprene bags are affixed. After placement, the bags are pumped full of grout.

The other type of scour involves a localized erosion around individual legs. This is often accompanied by gaps forming around the head of the pile as it deflects under lateral loads. Here the placement of a mound of small rock (e.g., pea gravel) has proved to be very effective in the clays of the Gulf of Mexico.

A gas blowout under a platform can lead not only to cratering but also to a general loss of support for the piles. To restore the capacity and safety of the foundation after such an event usually involves a major effort. First, the crater can be filled with small rock or coarse sand placed through a tremie tube. Then both the newly placed material and the adjoining sediments must be consolidated, since the entire zone will probably have been significantly loosened by the escaping gas. The consolidation has to be approached with great care, since the resultant settlement of the upper soils may lead to distortion of the structure. For that reason, densification by shock (dynamic compaction) or vibration may not be suitable. Placement of an extensive blanket of rock may be safer, with careful monitoring of both soils and structure at frequent intervals.

When, the pile capacity has been reduced, insert piles can be installed through the primary piles and joined to them by grouting. These insert piles are extended beyond the tips of the primary piles by drilling to obtain additional bearing and friction load transfer for axial loads. Insert piles, properly grouted, will also stiffen the piles and hence improve their capacity to resist lateral loads. As an alternative to a drilled and grouted insert pile, a belled footing may be constructed using a belling tool. This solution may be preferable in stratified soils or where skin friction transfer is uncertain.

The overhang of the deck will probably limit accessibility, requiring the insert pile to be assembled in short lengths and preventing the use of a hammer. A casing can be installed and reverse circulation drilling, along with a slurry, is employed. Once the hole is drilled, the insert pile can be lowered to position, using either welded joints or mechanical connectors. Then grout can be injected into the annulus, bonding the insert pile to the soil and to the primary pile. This assumes that geotechnical investigations show adequate soil at the

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greater depths. See Chapter 8 for more details on drilled and grouted insert piles and belled piles.

On two of the early concrete structures in the North Sea, after the platform was installed, drilling was commenced in order to install the conductors. As the drills penetrated the temporary closures of the concrete base slab, with the water level in the drilling shaft being lower than sea level, substantial quantities of foundation sand rushed up into the drilling shaft, leaving a void of several hundred cubic meters below. Some "piping" even led in from the outside perimeter due to the difference in heads. To repair and fill this void, an underbase grout mix having a low heat of hydration was flowed in under the slab, working in several stages to ensure complete fill. The zone on the periphery which had been disturbed by the "piping" was filled with small rock. Subsequent behavior of the platform, over a period of 25 years, has been entirely satisfactory.

To prevent this adverse phenomenon on future platforms, water levels in the drilling shaft must be equalized, especially during conductor installation. Steel skirts, surrounding and isolating the drilling compartments, are built into the base slab design.

18.7 Fire Damage

Fire is one of the most dreaded events on an offshore platform, and extensive measures are taken to prevent it. On some more recent structures, fire-protective insulation is placed around principal structural members of the module support frame, and tubular members of this frame are sometimes filled with water. Provision must be made for release of steam pressure. Of course, extensive spray and deluge fire systems are also provided on deck and in utility shafts.

When structural members have been subject to overheating and distortion due to fire, a complete evaluation and analysis must be made. While in extreme cases a member may have to be cut out and replaced, in others it may be practicable to reinforce it internally or externally to restore its capacity. Fire in one of the shafts of a drilling platform, in addition to damaging the exposed steel members inside, may cause spalling of the inside of the shaft walls. In most cases, heavy sandblasting, then placement of wire mesh properly anchored to the existing concrete, followed by shotcrete, will prove adequate.

18.8 Pipeline Repairs

In Chapter 15, repairs to pipelines damaged during installation were described. These included the use of a hyperbaric chamber lowered down over the line, enabling cutout and replacement in the dry.

Wet welding as a means of repair is still under development (see Figure 18.7). Satisfactory welds have been made in many cases, but their overall reliability is still not fully proved. Friction welding appears to be a promising method. Since rotating existing pipes is not possible, a small pup is placed between two cut ends. Friction sleeves are affixed to the ends of the existing pipes so that jacks may draw them together. The pup is then rotated at high speed. The disadvantage of friction welds is that they develop internal ridges which may prevent passage of a pig. Wet-welding in hyperbaric chambers has been successfully carried out offshore Brazil at 300 m depth. Mild steel-welding electrodes were used on carbon–manganese steel pipe, while nickel-welding electrodes were used on high-strength, high-carbon equivalent steel.

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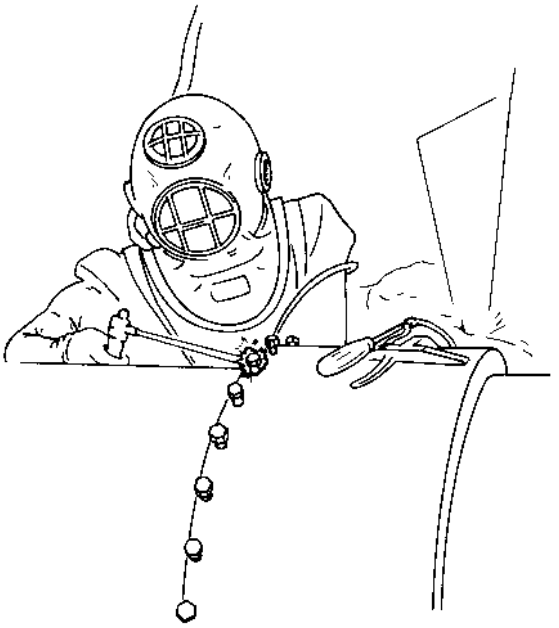

FIGURE 18.7
Wet welding repairs to damaged pipeline. (Courtesy of Plidco.)

In relatively shallow water, pipelines may be raised to the surface for repair. The methods and precautions required are discussed in Chapter 15.

Pipelines in service are often damaged by dropped or dragged anchors, sometimes by commercial or naval shipping, but most often from derrick barges and workboats in the same area. Trawl boards from fishing vessels may damage the coating or even the pipe, although usually it is the lines to the trawl board which break, resulting in its loss and a claim by the fishing boat.

The damage to pipelines may range from concrete weight coating being broken off, to dents, to holing, and even to the line being ruptured and the ends dragged apart. Leakage will usually be detected by the very sensitive pressure differentials employed in monitoring pipeline operation or by sheens of oil appearing on the surface. Exact locations can usually be found by acoustic means, by hydrocarbon sensors, or by internal instrumented pigs.

Side-scan sonar can be employed to determine gross positions of the line and segments. Then divers, manned submersibles, or ROVs equipped with video can be employed to give detailed information. Pigs may be run to isolate the damaged site. Hydrotech Systems, aided by Control Data, Inc., has developed a remotely controlled isolation plug capable of isolating high differential pressures. It is bi-directional and can be run for long distances, being remotely controlled and monitored from a surface vessel by external communication through the pipe wall. This was successfully employed on the 34 in. (800 mm) Teeside oil pipeline, enabling retention of the products in the pipeline and continued operation of the remaining downstream portion of the pipeline.

The coatings must first be removed from the damaged area, using a high-pressure water jet, an underwater concrete saw or grinder, and cutters for the mesh. Once cleaned, the damaged area must be carefully profiled, and a determination made regarding "cutout." Sometimes the damaged section can be cut out and an external sleeve slipped over the gap. In other cases, a pup will be more suitable.

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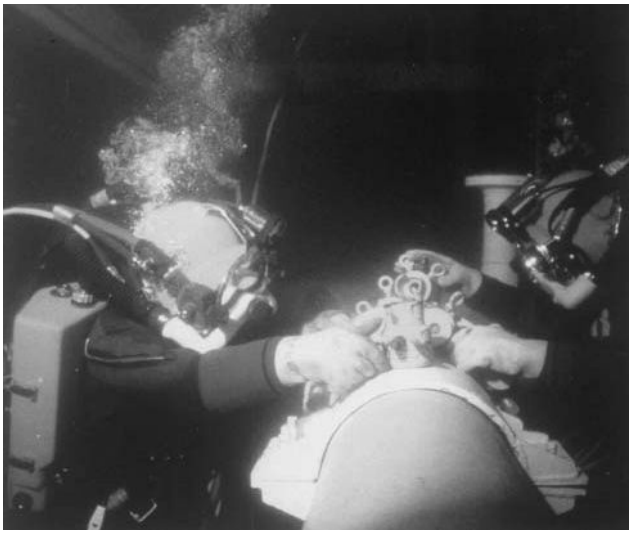

FIGURE 18.8
Repairs to damaged pipeline. (Courtesy of Plidco.)

When the rupture is a case of cracking or splitting, external split sleeves may be adequate. Tightly torqued together and with seal welding at the ends, sometimes augmented by epoxy injection, the line may be restored to full service (see Figure 18.8).

Using a sleeve secured to the pipeline with thrust screws, wet-welding techniques were employed to repair a 36-in. submarine line in the Mediterranean. A number of other systems applicable to shallow water have been described in Chapter 15. These include the use of a hyperbaric chamber ("dry habitat") pressurized to permit welds to be made in the dry. Special gasses must be used so that the quality of the weld is not affected adversely (see Figure 18.9). Continued advances in the development of hyperbaric chambers have extended their effective depth.

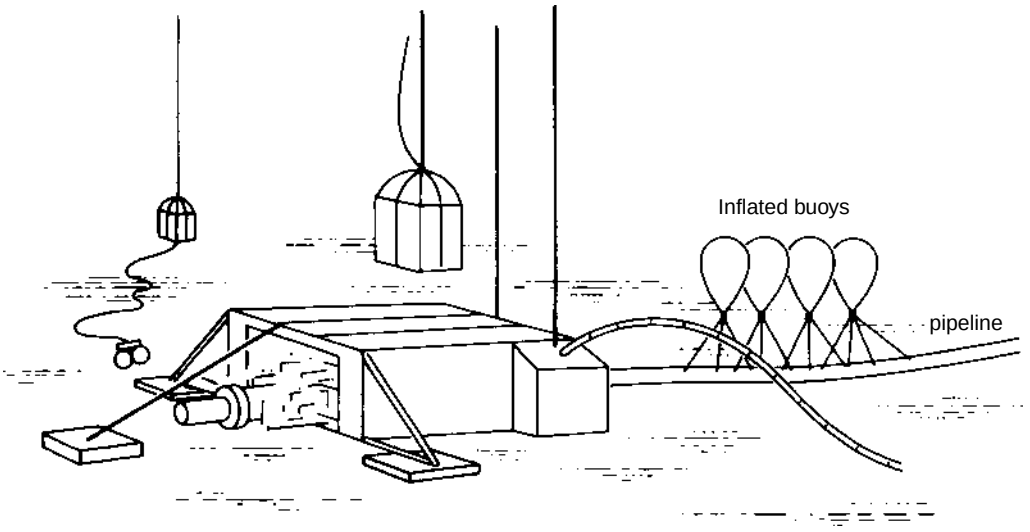

FIGURE 18.9
Hyperbaric repair of submarine pipeline. (Courtesy of Gomex.)

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A number of systems have been developed in recent years to enable pipeline repairs to be made at greater depths and, in many cases, without the need to waterflood the line. In one system, the line is re-pressurized and a small hole is drilled just beyond each end of the damaged section. Inflatable plugs are inserted through the holes and inflated. The gap is now tested to verify that the seals are tight; then the damaged section is cut out. A new pup is inserted, using divers trained for hyperbaric welding. Once completed, the plugs are removed by pigging.

One-atmosphere welds, using a diving bell, have been developed for deep water repairs.

Deep-water flow lines have been cut by shaped charges placed by ROV, then lifted by frames attached by the ROV for repair and relaying.

SNAM is developing a system known as SAS for deep-water submarine pipeline repairs. It consists of five modules: a thruster module which contains power, sensing, and transportation components; a dredge module; a pipe preparation module; a spool-cutter module; and a pantographer module. This last measures the exact length and configuration for the new spool piece. Various types of mechanical connectors are under development. These include cold-forging tools such as those being developed by Cameron Iron Works and Big Inch Marine.

Break, break, break,
On thy cold grey stones, O sea!
And I would that my tongue could utter
The thoughts that arise in me.

Alfred, Lord Tennyson, "Break, Break, Break"

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19

Strengthening Existing Structures

19.1 General

There appears to be a growing need for the strengthening of existing platforms in order to accomplish one or more of the following objectives.

1. Carry increased loads from additional equipment needed to support gas and/or water injection and other secondary recovery operations.
2. Extend the life of an older platform.
3. Upgrade the platform to withstand greater environmental forces, such as seismic or hydrodynamic.
4. Improve the structural system to overcome deficiencies that have been discovered since initial installation.
5. Change function or process.

Four categories of strengthening have been recognized:

1. Increasing the strength and rigidity of individual structural members or assemblies.
2. Increasing pile capacity to withstand greater axial loads.
3. Increasing structure-soil interactive capacity to resist lateral loads.
4. Increasing shear and bearing capacity of gravity-based platforms.

Strengthening of existing bridge piers and other marine structures has recently been required to enable them to resist earthquakes.

19.2 Strengthening of Offshore Platforms, Terminals, Members and Assemblies

As noted earlier in Chapter 18 on repairs, horizontal and diagonal bracing near and just below the waterline is often subjected to vortex shedding from the orbital velocity of the waves, leading to a failure from fatigue. One means of overcoming this problem has been to install underwater K-braces between a lower node and the midspan of the member in question. Tubular members, with split half-rings at the ends are carefully templated and fabricated to the as-built dimensions. The new member is then placed, and the other half of

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the rings or clamps installed and drawn up with torqued high-strength bolts. Epoxy grout is then injected.

The installation of additional bracing such as this obviously changes the response of the structural members, as well as introducing new loads into the nodes. A re-analysis of the platform is therefore required before such strengthening is carried out.

Another and simpler method to prevent fatigue due to vortex shedding is to install spoilers on the brace in question. These can be clamped on. While they slightly increase the load on that brace, the effect is usually limited to the member in question, with minimal effect on the remainder of the structure.

On the Ninian central platform, a number of deck chord members were reinforced to carry heavier axial and bearing loads by filling them with concrete. In order to achieve complete filling, the grout-intruded aggregate method was used; that is, a grout pipe was placed inside the tubular, and then the tubular was filled with coarse aggregate. A sand–cement grout of high fluidity was then pumped through the pipe, which was gradually withdrawn as the voids between the aggregate particles were filled. In this author's opinion, a fine concrete, with 8–10 m coarse aggregate and containing an anti-washout and anti-bleed admixture would have been simpler and perhaps more reliable.

It would, of course, also be possible to insert a steel shape into the tubular before placing the aggregate. The steel shape should be configured to provide for proper flow of the grout and escape of entrapped air and bleed water. For tubular members, a heavy-walled perforated tubular will probably prove to be the optimum insert.

Brown and Root have developed a method by which bracing members of existing platforms can be strengthened by procedures similar to that described above, with the additional inclusion of an ungrouted sleeve through which a post-tensioning tendon is run. The tendon is then anchored and stressed, using a hydraulic jack specially adapted for underwater use. Then the tendon is grouted (see Figure 19.1). This method is especially useful when it is desired to increase the tension capacity of that member. Depending on details of the node, the tendon may be run through the nodes and stressed externally to them.

When the upper chord of deck trusses or the upper flange of deck girders is required to carry heavier bearing loads from the modules, heavy bearing plates can be fitted on, with stiffeners to spread the concentrated load to the tubulars. In some cases, it may be desirable to fill the tubular member with concrete at the location of the bearings to prevent local deformations and to distribute the concentrated loads.

The nodes at the intersection of the legs and braces of jackets may be given increased resistance to buckling and ovaling by filling with fine concrete (typically 10 m aggregate and silica fume–cement mix). Where a pin pile is located within the jacket leg, the annular space may be grouted.

Another method used to strengthen both nodes and braces is to clamp on an external sleeve, in two halves. These can be squeezed around the brace by the torquing of high-strength bolts, and the annular space injected by epoxy grout.

19.3 Increasing Capacity of Existing Piles for Axial Loads

Increasing the capacity of existing piles to sustain increased axial loads is a problem which has been encountered a number of times in cases where calcareous sands have been encountered and where a re-evaluation of the piles' performance under dynamic uplift has led to the decision to increase their axial capacity. It also occurs where a platform must carry heavier loads than those for which it was originally designed.

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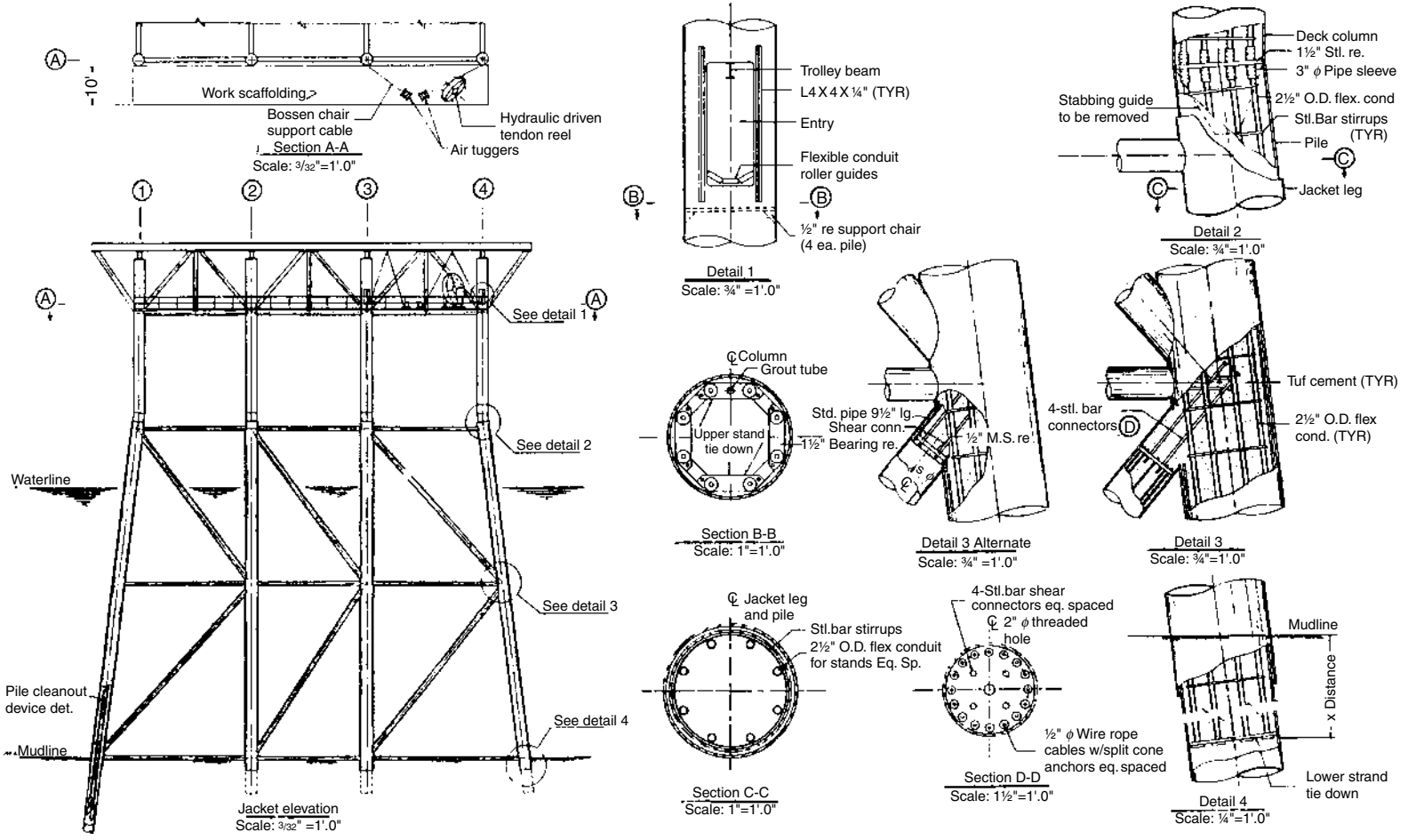

FIGURE 19.1
Platform strengthening. (Courtesy of Brown and Root Energy Services.)

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The most common decision is to install insert piles. Since the overhang of the platform usually limits the clear working space to 10 m or less, the insert piles will have to be installed by drilling, rather than driving. Therefore, a work deck is installed, upon which a short drill rig can operate. Then a riser pipe must be run down to the top of the existing primary pile and sealed to it, sufficiently tight to enable an 8–10 m positive head to be maintained within the riser. Flange or coupling joints are used for the riser. Then the drill string is assembled and a hole drilled for the insert pile to the required depth.

The insert pile must now be made up. Welded connections are standard, but a great many are required, due to the short length of each segment that can be fitted in under the deck. The welding has to be carried out under conditions of spray and wind; hence, protection is required. Nondestructive testing, usually by x-ray, takes time and clear access. The alternative is to use a threaded connection or high-strength tapered-screw mechanical coupling (drill string coupling). Such connections should be qualified by tests under dynamic and cyclic loading.

A grout pipe is run with the insert pile. An inflatable packer is provided at the tip to prevent grout from filling the insert pile. Alternatively, a float shoe may be installed. Cement grout, sand–cement grout, or fine concrete is then placed, until instrumentation shows that the annulus is full.

Shear transfer to the insert pile can be enhanced by welding on shear rings (see Figure 19.2). Means must be provided to prevent the insert pile from floating in the denser grout, since the insert pile itself will usually not be filled with concrete, at least not concurrently. It may be wedged to lock it in place or weighted by filling with barite drilling mud.

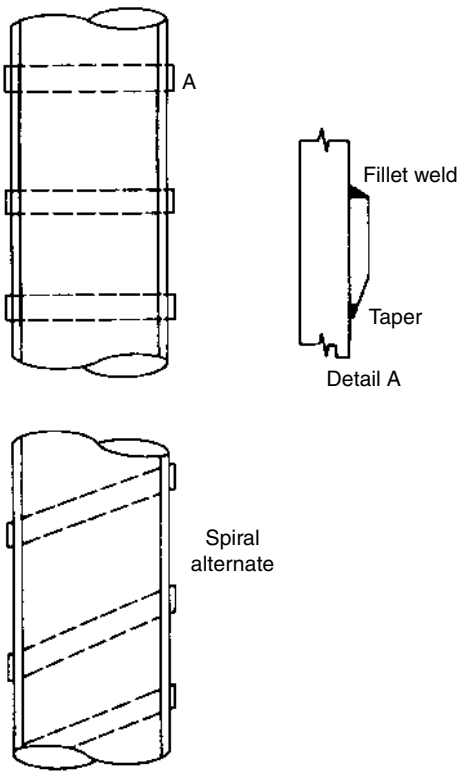

FIGURE 19.2
Shear keys on insert piles.

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To save the time and cost of coming out with the drill string and then setting the insert pile, it may sometimes be expedient and adequate just to grout the drill string in place in the drilled hole using an expendable bit. This was done, for example, on several piles for the strengthening of the Kingfish platforms in Bass Strait, Australia.

Another solution is to utilize the end-bearing capacity of an existing pile, since this will have often been disregarded in the initial calculations. To do this, the material inside the existing pile must first be removed. However, a plug of soil about 3–5 diameters long must be left so as to not unduly loosen the soil around the tip. On the same Kingfish platform referred to above, elongated holes were cut into each jacket leg, above water, for access to the pin pile. A concentric group of pipes was made up, using segments only 4 m long (see Figure 19.3 and Figure 19.4). These sections were progressively made up and lowered down the existing pile. Obstructions such as neoprene pile closures had to be fished out. Using the jet plus airlift, the plug was cleaned to within a few meters of the bottom. Then a sand–cement grout was injected to form a 10 m-long plug. Then an illmenite (iron ore) slurry was fed into the pile to give it added weight to resist uplift.

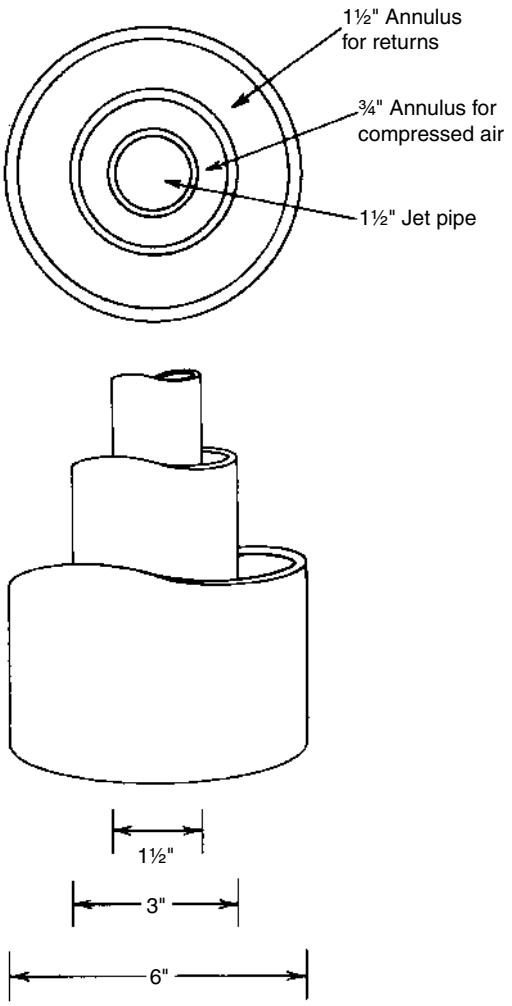

FIGURE 19.3
Multiple pipe arrangement for cleaning out plug in existing pile.

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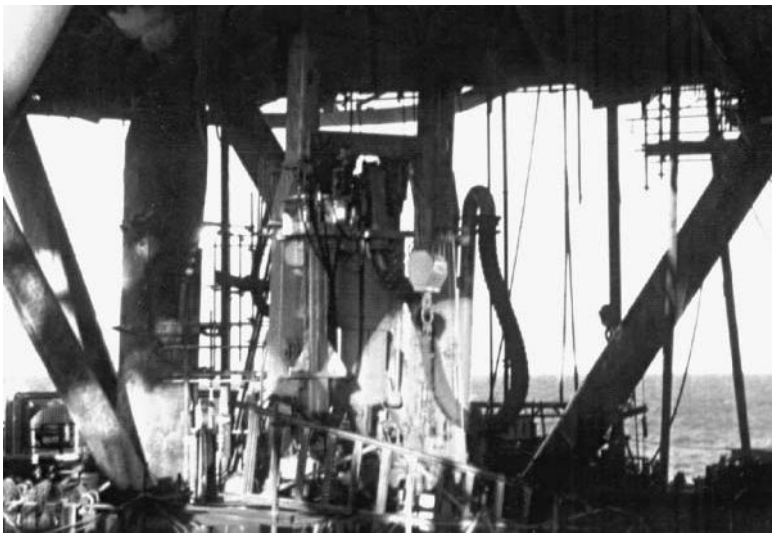

FIGURE 19.4
Inserting airlift and jet assembly into legs of existing Kingfish Platform in Bass Strait, Australia.

Pressure grouting around the tip and walls of an existing pile have been often proposed. Holes can be perforated in the walls of the pile by controlled explosive casing-perforator devices. Grout of very high fluidity and low surface tension can then be flowed into the pile, to exit from the ports and bind with the soils. Special admixtures are available to help penetrate sandy soils. The problems are many, however. The sands, being saturated, are relatively impermeable, since any grout flowing into them must push water out. Grout always follows the path of least resistance, trying to find an exit to the sea. Thus while on paper we can draw symmetrical grouted bulbs, in practice they are rarely achieved. If too high a pressure is developed, the grout may fracture the formation and escape into the sea. Two- or three-stage grouting may help to achieve an effective bulb. Use of packers above and below the grout injection points has been utilized by Halliburton as a means of controlling the vertical length of grouting at any one stage.

Other types of grouts, such as polymers and chemical grouts, penetrate dense soils more readily but also suffer from the problems of inconsistent performance. Shell Chemical Co. has developed a low-viscosity epoxy material, Eposand, which penetrates sands and even coarse silts of low permeability. This requires initial injection of fresh water, followed by injection of glycol or alcohol, after which the epoxy can be injected. Although time-consuming and expensive, it has proved effective in calcareous sands of low permeability.

The ballasting or weighting of a pile to give greater uplift capacity is similar to the weighting of a table leg. Earlier, use of illmenite slurry was reported. Barites added to drilling mud are expensive but effective and can utilize the drilling and mud facilities already on site. Finally, magnetite iron ore graded for maximum density can be used.

Depending on the character of the soil, drilled and grouted insert piles may not be suitable or may be excessive in length. Where good bearing strata exist near the tip of the existing pile, a belled pile footing may be constructed in much the same manner as the drilled and grouted insert pile (see also Section 8.12 and Section 8.14).

Belled footings were constructed under the main pin piles of the Ekofisk drilling platform in the North Sea. A riser was run, and then the drill string. After drilling to the desired depth (T.D.) the belling tool was operated, gradually belling out a truncated cone. Reverse circulation was used, with airlift assist. The air injection was

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well above the bell, up in the primary pile, so as to maintain fluid density in the bell and resist caving. The internal head in the riser, being a meter or two higher than sea level, helped to stabilize the bell by causing all flow to be outward. Either a bentonite mud (pre-converted to calcium bentonite) or a polymer slurry should be used to be compatible with the cement.

Next the bell is concreted. A concrete containing small aggregate, up to 10 m maximum, is preferred to straight cement grout because of higher tensile strength, lower heat of hydration, and less tendency for a brittle mode of failure. In any case, the cementitious materials should have low heat of hydration: a 70-30 blast furnace slag-Portland cement mix is suggested. Alternatively, a cement with 50% pozzolan replacement may be used. The mix may be pre-cooled.

The tremie process using gravity flow is the preferred method. Pumping has been employed but may develop excessive laitance due to disturbance under the high rates of flow and the surge of the stinger. The tremie method appears to give better control, especially if the formation surrounding the bell is sensitive to hydraulic fracture.

Shear transfer from the insert pile to the bell has to take place over a relatively short length, and hence special shear transfer devices are usually required, such as reinforcing bars grouped in bundles or shear keys on both inside and outside of the insert pile. End bearing plates may also be used, providing direct load transfer from the insert pile to the bell. A float shoe or end plate may be provided at the tip of the pile to prevent concrete from filling the insert pile. The insert pile must be weighted or secured to the main pile to prevent it from floating. A secondary system may be necessary for grouting the annulus between insert and primary pile, in order to ensure complete filling of this critical zone.

On the North Rankin platform, off the Northwest Shelf of Australia, it had been discovered during construction that the piles had little if any frictional resistance in the calcareous sands, despite a penetration of 120 m. The platform had already been completed and was operating, so all remedial operations had to be carried out from underneath the deck. The decision was made to construct belled footings under each of four primary piles in each of the corners, for a total of 16 piles. Since these were skirt piles, with heads some 80 m below water, a casing was run, through which the drill string could operate. The drill then made a straight-sided hole extending about 10 m below the tip of the driven pile and 1 m below the design elevation of the bottom of the bell. The proprietary epoxy Epoxand was injected to stabilize the surrounding sands during belling.

The belling tool now expanded slowly to enlarge the bell to a bottom width of 4.5 m. Then the insert pile was placed. Based on tests, the end plate on the insert pile, with a slope of 7°, was found to be sufficiently sloped to ensure full filling on the underside, with no trapped air or bleed water. Then a 3½ in. (88 m) tremie pipe was installed. Concrete mix was 8 m pea gravel, sand, and blast furnace slag-cement, with fly ash and silica fume. Admixtures used were plasticizing and retarding (SIKA Plastiment) and a superplasticizer. Concrete was precooled to 5°C by injection of liquid nitrogen. The ambient temperature of air, water, and soil was 38°C. The tremie pipe was pre-cooled by cold water. Then, using a pig, the tremie concrete mix was placed.

The main pile and consequently the casing and insert pile all sloped at 7° from vertical. This turned out to be advantageous in the concrete placement, in that entrapped air could escape quite readily from the tremie pipe. Concrete was transported from the mixer to a hopper through an insulated pipeline, thence by gravity flow down the tremie pipe.

Cores taken subsequently indicated that the design strengths of 45 MPa compression, 7 MPa tension were met and exceeded.

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19.4 Increasing Lateral Capacity of Piles and Structures in Soil–Structure Interaction

In many soils, the cyclic lateral deflections of the structure cause a gap around the pile near its head, thereby increasing the amplitude of lateral displacements under storm waves and progressively weakening the resistance. The placement of small rock (e.g., pea gravel) on the surface in order to feed down into any gap and plug it has previously been noted. On a more extensive scale, a thick layer of graded rock placed around the jacket legs may help to confine the soil beneath and prevent its local liquefaction under the pumping action of the pile movement.

For a small platform in the Bombay High Field of India, where excessive lateral displacements were occurring due to crushing of calcareous sands just below the surface, insert piles were installed and grouted. These significantly stiffened the piles, reducing lateral strains (see also Section 8.14).

The typical gravity-based platform depends for its stability on the near-surface soils. Where these are known to be weak prior to initial installation, various methods of improving the capacity of the existing soils may be implemented. These are described in detail in Chapter 7 and include dredging, placement of a rock or sand layer on the existing soil, installing of sand piles or sand drains, or wick drains plus a surcharge. All of these can be performed before installation of the platform.

For the Rion–Antirion Bridge, 2 m diameter. steel tubular piles were driven on 3 m spacing. These strengthened the weak clay soils against shear failure. Similar installations are being planned for the Venice Storm Surge Barrier, and also for the strengthening of a clay slope on the existing Transbay Tube (tunnel) in San Francisco Bay, California.

When the soils require strengthening after the platform has been installed, the following methods may be used.

Where sliding shear resistance of a gravity platform needs to be improved, the placement of an embankment around the periphery can lengthen and strengthen the potential shear surface. To prevent shear failure upward through the embankment, it will preferably be constructed of fractured rock so as to interlock. When gravel or sand is used, a thicker layer will be required.

Depending on the seafloor soils, it may be necessary to first place a filter fabric or a layer of smaller rock. Depending on the depth and the wave and current environment, it may be necessary to cover the embankment with heavier rock that will remain stable. Such an external embankment or berm, placed after installation, will give direct passive resistance to the structure as well as lengthen the shear path in the native soils below and, depending on the soils and the length of time, increase the shear strength of the native soils around the perimeter. External berms, with either a graded rock filter or filter fabric, are also an excellent way to prevent liquefaction and pumping of sand from under the edges of the structure. This was one method considered on both Statfjord B and the Hibernia platforms. In both cases, however, they turned out to be unnecessary.

In Arctic and sub-Arctic areas, a berm may also serve to intercept a deep-keeled ice feature and ground it, slowing or stopping it by the passive resistance of the material along the sides and ahead of the keel. Placement of such an external berm or embankment is a relatively cost-effective way for upgrading an exploratory drilling platform to a production platform, since the difference in ice features that must be withstood is predominantly (but not entirely) reflected in their draft and hence keel depth.

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Another system which has been considered, but rejected because of cost, has been to drive a wall of large diameter (2–3 m) steel tubular piles around the periphery.

To enhance the shear resistance of the soils underlying an existing platform, shallow wells may be drilled into permeable strata and drained into the interior of the platform, thus consolidating them over a period of time. In the meantime, these drains are effective in preventing pore pressure buildup and the possibility of liquefaction. Since this may cause settlement of the platform, the effect of downdrag on the conductors must be considered.

To increase the dead weight of a gravity-based structure—as, for example, when it is desired to increase frictional resistance to sliding—additional ballast may be added. Illmenite and similarly fine-ground iron ores have been placed as a slurry and then decanted. These can achieve specific gravities of 3 and higher, which gives a net gain of 2–3 tn./m^3 (20–30 kN/m^3) when placed in a cell which previously was filled with saltwater ballast only. Native sand can similarly be used, but the net gain is limited to about 0.9 T/m^3 (9 kN/m^3). Concrete ballast will give a net gain of 1.5 T/m^3 (15 kN/m^3). Weight can also be added on top of the roof of the base caisson in the form of concrete placed underwater. There is usually reserve capacity for such heavier external loads, since they are often under tension and since the design of the roofs is usually determined by the installation condition. Such additional concrete on the roof may also improve the resistance to the impact of dropped objects.

The Molikpaq platform in the Canadian Beaufort Sea was a steel gravity-base caisson, filled with sand for stability. When an ice pressure ridge was pushed against the caisson by the Arctic ice pack, the ice failed progressively by continuous crushing. This caused the sand ballast to momentarily liquefy. After the event, the sand ballast was densified by the use of multiple small explosive charges.

Experience appears to indicate that collision from boats and barges is a more common event than initially thought. In some older platforms, it may be desired to increase the impact resistance of the shaft walls near the waterline by increasing the thickness of the wall. Work will normally be done from the inside, by lowering the water level inside below the zone in question. Shear dowels may be drilled and grouted in. The existing concrete surface should be heavily sandblasted. Additional circumferential and vertical reinforcing steel is secured to the dowels. As the new concrete is placed, a bonding epoxy may be progressively sprayed on the existing wall, just above the level of the fresh concrete. To enhance the bending capacity of a shaft, the wall of the shaft may be thickened through the region of high moments. These measures may cause a change in the response of the platform and the deck, since the stiffness will also be increased. Hence, this work should only be under taken after a thorough dynamic analysis of the revised structure.

19.5 Penetrations Through Concrete Walls

Occasionally, it becomes necessary to construct a new penetration through a shaft which is below water level, so that there will be a head of water on the outside. Work will usually be scheduled for the summer weather window. The shaft is dewatered inside to below the location and staging placed. On the exterior, a portable steel cofferdam with gaskets is affixed to the structure wall by drilled-in anchors (see Section 18.5). These can be supplemented by taut wraps of wire rope taken up by turnbuckles. The drilled anchors should not extend as deep as the existing vertical post-tensioning ducts (see Figure 19.5). Now, working on the

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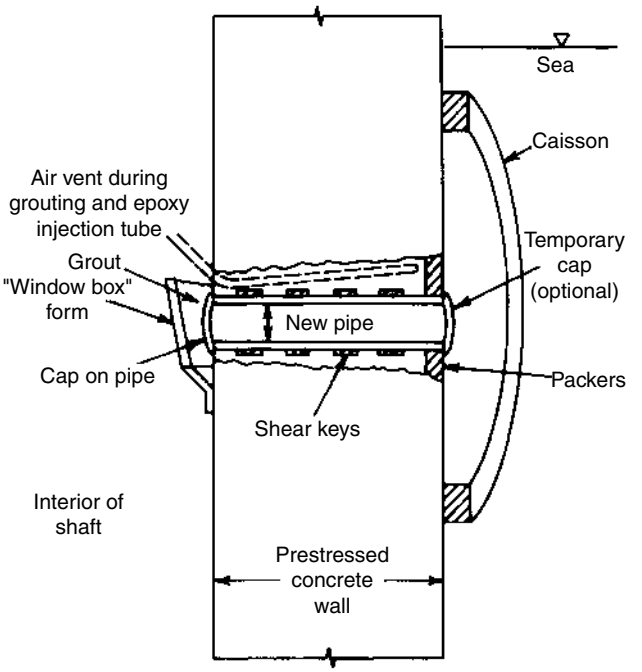

FIGURE 19.5
Installing new penetration through existing concrete shaft wall.

inside, the location of the ducts is carefully determined. As-built drawings will give the approximate location, but this should be verified by careful, slow drilling of a small hole, say 10 m in diameter, since often minor displacement of ducts will have occurred during slip-forming and concreting. It may be possible to locate the ducts by ground radar.

Next a central hole is drilled through to drain the water from the space between the external caisson and the wall; this will squeeze the cofferdam tighter against the wall. Next, using the central hole as a pilot, the hole for the new pipe sleeve is drilled through the wall. It will have a diameter 10–40 m larger than the outer diameter of the pipe sleeve which is to be installed. While it is desirable from all counts to miss the conventional reinforcing steel insofar as practicable, the critical thing is to miss the post-tensioning ducts. An occasional cut reinforcing bar can normally be accepted.

The hole is now enlarged on the outer side so as to be tapered. The walls are roughened. The pipe sleeve, with shear rings attached, is fitted with an external packer or expanding ring. The pipe sleeve is inserted, the packer or ring activated, and the sleeve fixed tight in the center of the tapered hole.

Two small-diameter (3 m) plastic tubes are fitted, one just below the pipe sleeve, the other just under the top of the hole. A nonshrink cement grout or epoxy is now pumped in. Using a window box on the inside, a slight positive head of grout is maintained until final set. Now epoxy is injected through the two small tubes, which are slowly withdrawn. The intent is to fill any small bleed water gap under the pipe or at the top of the hole. The new pipe is capped on the inside.

Next, a valve in the caisson is opened, so that full water pressure acts on the new penetration. If all is satisfactory, with no leakage, the caisson is removed. A diver can

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then place an underwater-setting epoxy sealant around the external edges of both pipe and hole, where the packers were.

If leakage is noted, small holes are drilled along the leaking seam and epoxy injected. This may be done from the outside by diver or from inside even against an external water head.

19.6 Seismic Retrofit

Many existing bridges, offshore terminals, and wharves were not originally designed to resist the earthquake forces to which they may be subjected during their life. The disastrous failures of harbor structures in the Kobe (Hanshin) earthquake as well as the Loma Prieta earthquake near San Francisco, illustrated the need to upgrade these existing structures for dynamic forces. Similar collapses of bridges have led to intensive research and development in the seismic resistance field.

The seismic retrofit of the Richmond-San Rafael was one of the most extensive such projects ever undertaken, involving foundations, towers and superstructure.

To stiffen the foundations, 3–4 m diameter thick-walled steel tubular piles were installed. They were joined by precast concrete elements, connected under the diaphragms by overlapping reinforcement and high performance tremie concrete (see Figure 19.6). These bore against the concrete bells with neoprene and Teflon pads, allowing rotation and vertical displacement but restricting lateral movement. Steel and precast concrete jockets were then installed to increase the stiffness and ductility of the bell and shafts.

Many of these existing structures have adequate vertical carrying capacity but require stiffening to prevent excessive drift, with its amplification of moments by the P-Delta effect. Incorporation of large-diameter vertical steel cylinder piles or shear spuds (see Chapter 8), tied into the existing structure, has proved an effective retrofit in many cases.

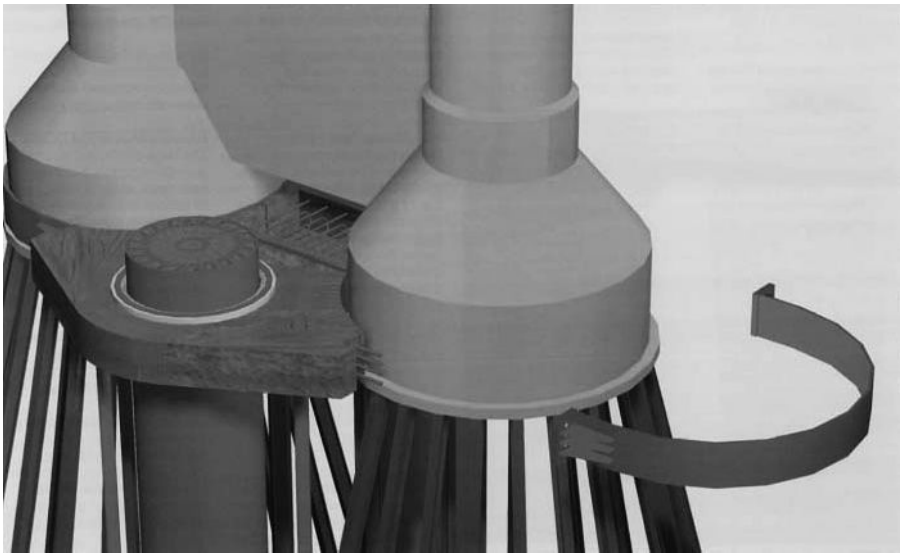

FIGURE 19.6
3.5 m diameter steel tubular pile provides lateral stiffness to existing pier, Richmond–San Rafael Bridge, California. (Courtesy of Ben C. Gerwick Inc.)

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Shear resistance of existing pier footings and shafts can be improved by banding with heavy steel sleeves. These are installed around existing elements, joined by bolting, and grouted to fill the annulus. Where they can be joined above water, and then slipped down, welding is employed.

Wharves and terminals with intersecting batter piles or batter and vertical piles have suffered severe damage in earthquakes. Examples include the wharf at Anchorage, Alaska, in which the steel tubular batter piles were severely distorted into S-shapes and punched through the deck, as well as the many wharves in San Francisco Bay where the concrete batter piles were crushed and sheared during the Loma Prieta earthquake. Similar but lesser effects occurred during the Northridge earthquake.

Retrofit has consisted of cutting loose the batter piles, replacing them with large heavily reinforced vertical piles, and then extending the wharf landward so that new vertical anchor piles (shear piles) could be incorporated. These are typically large-diameter piles with special reinforcement and confinement to take the concentrated moments and shears at their heads.

Many existing shafts and piles have been sleeved with steel or wrapped with carbon fiber to enhance their ability to confine the concrete at the plastic hinge. Steel tubular piles have been filled with concrete over the zones of potential local buckling.

Another approach to the problem of batter piles is to create a "fuse" mechanism by which, when the force exceeds the elastic capacity, a transfer plate is deformed beyond yield. Steel H piles, turned onto their weak axis, can be substituted for the existing tubular or square batter piles so as to buckle globally during earthquake. Small diameter steel tubular pups may be inserted at the head of existing piles, designed to buckle locally under high lateral force.

For reinforced concrete, many of the methods presented in the previous chapter on Repairs also are applicable to strengthening of existing members for increased loads. This applies to carbon fiber plates and strips applied on the surface and to external post-tensioning applied within concrete enclosures. Long-term tests on the durability of carbon fiber fabrics in seawater indicate good performance initially but the potential exists for long-term reductions of up to 33%, due primarily to delamination at the glued joint.

External post-tensioning can also be applied to steel structures provided curved steel deviators and anchorages are affixed.

Concrete barges used in the Viet Nam War suffered damage of their hulls due to explosives. These were repaired by ballasting each barge so that the damage was above water, then removing all fractured concrete, placing new reinforcing steel, and restoring the original section with fine concrete.

Gaze down at the black satin waves,
Stars reflected in tracks of silver light.
Look, see, where the waters are foaming,
Sparkling sea-jewels illumine the night.

Author, "Shadows of Gold"

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20

Removal and Salvage

20.1 Removal of Offshore Platforms

Current regulations of many countries require the removal of offshore platforms and other structures when they have finished serving their purpose and are no longer in use. In most cases, the requirement is that they be removed to a point 2–5 m below the mudline. Increasingly, the constructor is required to develop a full procedure for the eventual removal at the time of initial design and to set it forth in a manual for approval by the authorities before being granted a permit for construction.

In most early cases, there has been little prior planning for removal and hence no fittings or details have been built into the structure to facilitate removal. Even where these have been built in, over the years they may have become fouled by corrosion and marine growth and are no longer operable. Indeed, in many cases, miscellaneous construction during installation or changes during service may have blocked easy access to carry out the work. Pile-stabbing guides and lifting cones are some examples; drill cuttings and antiscour riprap are others.

A number of studies are currently under way in both Europe and the United States to re-appraise the regulations and to re-evaluate the need for removal to below the mudline. In some cases, a platform may have continued utility as a lighthouse, radar station, offshore scientific or educational laboratory, or be suitable for conversion to a platform for purposes unrelated to oil and gas production. Structures extending above the seafloor become habitats for fish and provide their young protection from predators. Offshore platforms become natural habitats for marine life, furnishing protection and breeding places for a wide variety of organisms. A study of Rincon Island off the coast of California has been very revealing in this regard. Prior to construction of this offshore drilling island, the area had been well documented as a "marine desert" with its featureless bathymetry swept clean and bare by waves and currents. After construction of the island, with its slopes covered by riprap and concrete armor units, a host of organisms, over 2500 varieties, has found shelter, so that the island is now an official marine preserve.

It is well known that the best sport fishing occurs around offshore steel jacket platforms; indeed, some smaller jackets, which were abandoned, have been purposefully re-installed as artificial reefs. Finally, some structures may be reused as breakwaters or the support for breakwaters and is the approved disposal solution for the Shell SPAR floating steel oil storage structure, the original disposal plans for which aroused considerable criticism from environmental groups.

Disposal of offshore structures must comply with the International Maritime Commission (IMCO) international agreement on ocean dumping and with national regulations. Obviously, this means that disposal will not be permitted in waters where present or

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future navigation might be impeded or where it would interfere with bottom trawl fishing. Disposal on-shore is, of course, made complex and costly by the draft of these huge platforms and the limited number of areas where structures can be beached for cutting up and disposal. Oily residue must be cleaned from the structure before disposal at sea; hence, on-land disposal as scrap may be more economical in some instances.

Three types of structures will be discussed in order to illustrate general principles and possible solutions for removal. Obviously, any particular platform will have to be addressed in specific detail as to requirements, methods, and control of operations.

Removals are in many ways as complex or more complex than the initial installation. The existing conditions must be fully investigated and considered since structures may be corroded, damaged, or even have missing braces. Marine growth and seafloor changes have to be considered. Planning of each stage must be carried out with thorough attention to detail. Whereas in initial installation there are economic incentives for the owner, the dismantling of a platform is a net economic cost.

Perhaps even more than during initial installation, risks are involved in the removal, such as the risk that during salvage an accident could occur or the structure could become unstable, presenting the constructor with a more difficult and costly or even a nearly impossible operation. Therefore, just as in installation, risks must be enumerated and evaluated and contingency plans prepared.

20.2 Removal of Piled Structures (Terminals, Trestles, Shallow-Water Platforms)

The first step is to remove all superstructure facilities and equipment. Care must be taken when cutting abandoned pipelines by burning to ensure that they are gas-free. A decision must be made as to whether to work from on top of the structure or afloat. Working on top means a smaller crew, few limitations due to weather or sea state, and inexpensive equipment. On the other hand, lift capacities (weight and radius) are severely limited. Working afloat offers greater maneuverability, the opportunity to make heavy lifts of large sections, and immediate availability of auxiliary and supporting facilities. Daily costs are high, and the rig will be subject to the effect of the seas.

In practice, a combination of the two methods will often be used. Removal of the deck structure, either before or during pile removal, is generally straightforward but requires a careful study of material handling—such as to what the sequence and the route for off-delivery of salvaged elements will be.

Piles may be cut with casing cutters, working from a drill rig. The drilling equipment may be supported on the pile itself, from a derrick boom, or from the mast of a drill rig on deck. The casing cutter uses an expanding bit. Explosives may also be used to cut off piles. These are shaped charges, lowered down the pile. Positive means must be taken to ensure that they are not tilted or jammed in the pile; otherwise, the resultant distorted pile may become much more difficult to cut by alternate methods. A third method is to dredge alongside the row of piles, so that they are exposed to the required depth, and then to burn them off using divers and a jet lance. Abrasive jet cutters have been newly developed and have proved fast and accurate.

In some cases, piles may simply be cut free from the structure and pulled. In sands large vibratory hammer with a steady pull may be effective, especially if combined with a jetting operation. Impact extractors are available, but generally, they are of a size suitable only for relatively small piles such as those used in harbor construction.

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In silts and fine sands, overfilling of a pipe pile with water increases the pore pressures and decreases the effective lateral pressure and skin friction. The water may be filled to the top of the pile. It may then feed to the soil through the open tip, but the pile may first need to be perforated by controlled explosives to facilitate the flow of water into the soil. If the situation permits, a jet may also be run along the outside of the pile. Barges can be moored alongside and then deballasted to break the pile free.

Mechanical jacking, especially in conjunction with jetting, has been used. The main problem is finding a support for the jack reaction. Usually this must be the adjacent structure, which then must be checked to ensure adequate capacity and stability.

To break piles loose for subsequent removal by one of the above methods, the pile may initially be driven down a half-meter or so to break any setup adhesion that may have developed. This is especially effective in clay soils. Extraction may then take place.

An ingenious method was developed for removal of the 2-m diameter by 80-m long dolphin and temporary trestle piles installed to support the construction of the Oosterschelde Storm Surge Barrier in The Netherlands. The piles were capped with a steel dome welded to the top of the pile. Water was then pumped in and the pressure raised until the pile jacked itself out. When high pressures were required, there was danger of piping developing around the tip of the pile. Therefore, a blinding course of very fine sand of low permeability was placed in the tip, hence, reducing pressure as the water penetrated below the tip. This blinding course was typically a 1-m deep fill in the pile and often had to be re-installed several times as the pile moved up.

As the water permeates through the blinding course or through small holes near the tip, it raises the pore pressures in the soil and thus reduces the skin friction.

Air pressure alone is not used in such cases as it could conceivably lead to the catastrophic explosion of the pile cap. However, if the pile is first filled with water, then air can be used to apply pressure on top to the water; the volume is now small enough to reduce the potential hazard.

If a pile has been frozen into permafrost, it may best be freed by first breaking up the plug with a high-pressure jet (even if frozen), removing the plug by airlift, and then injecting steam into the water in the pile until adfreeze is broken.

These techniques are all useful when the removal of the entire pile is required; however, these techniques are usually far more expensive than cutting.

20.3 Removal of Pile-Supported Steel Platforms

One of the early steps in removal of offshore platforms is the removal of the deck. To remove it in pieces is costly and requires many days of work offshore. Use of a very large and costly offshore crane barge is seldom economical, even though it can dismantle the deck in large modules.

- One concept, used successfully in the Gulf of Mexico, is the *Versa-Truss* to be described in detail later.
- The second concept is the *GM Heavy-Lift*. It extends an existing semi-submersible barge as an inverted U-shape, which straddles the deck and lifts from the extended structure above.
- The *Pieter Schelte* vessel consists of two tankers joined together at the stern to form a stable platform. It will also straddle over the deck and lifts it by deballasting.

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- The *Marine Shuttle* is a specially built vessel of large diameter tubular steel members, which extend on both sides of the platform. It uses the deballasting procedure to lift the deck.
- The final current (2004) candidate is the *MPU*, a U-shaped concrete vessel with 4 columns. It can lift the topsides as a single unit.

The next step after deck removal is the removal of the jacket.

For large steel jackets with pin piles, the piles are first cut off below the jacket and below the mudline as required. The preferred method when fishing and navigation rights are judged suitable by the regulatory authorities is to topple the jacket over and then drag it to an acceptable location where it can serve as an artificial reef to support marine life.

When this procedure is not permitted, the jacket must be cut into sections and lifted off for transport to the shore side. Strand jacks and buoyancy bags can be used to assist.

The Controlled Variable Buoyancy System consists of groups of buoyancy shells, clamped to appropriate locations on the jacket legs. A sophisticated control system, operated by remote radio and the use of electronics, controls the buoyancy.

Before and during removal and disposal operations, the authorities generally require that oil-contaminated piping and vessels be disposed of on-shore.

The removal of jackets from structures with pin and skirt piles is currently of greatest interest. Many of the platforms in the Gulf of Mexico have been in place for more than 25 years; the reservoirs are essentially exhausted, and the regulations currently in effect require removal to below the mudline.

The costs of removal are high. This by itself often makes the concept of secondary recovery and continued production attractive, even at low flow rates. The following is a typical scenario for removal, which of course, must be adapted to the specific jacket.

1. Cement and plug wells. Purge and disconnect all risers and flow lines.
2. Remove equipment above deck and all facilities. Equipment below deck may then be removed by progressive dismantling of deck frames.
3. Remove module support structures above cellar deck. Remove cellar deck.
4. Use water jets as necessary to clear 15 ft. below mudline externally or internally.
5. Cut well casings and conductors with drill string. Alternatively, use explosives or a cutter externally.
6. Enter skirt piles with drill string equipped with hydraulically expanded casing cutters or abrasive jet cutters. Cut all piles except those on one side. Cut these half through.
7. The jacket is now supported only on mud mats and a few half-cut through piles.
8. The jacket may now be toppled to serve as an artificial reef, using winches, pulling from the crane barge, which is reacting against its anchors. Any crane lines attached must be able to pay out freely as the jacket topples. Portions of mud mats that extend too high can be cut by the remote operated vehicle (ROV) or divers.
9. If disposal on the adjacent seafloor is not allowed, the jacket must be raised to enable it to be set on a barge. A heavy-lift crane barge can lift a moderate-sized jacket but a large jacket will require supplemental buoyancy. In this latter case, the jacket may then be towed underwater to shallow water where it can be grounded and cut up by divers.
10. When employing the use of buoyancy bags or tanks, the rapid expansion of air as they rise must be considered. British Standards warn that using compressed air

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for buoyancy requires attention to detail and the provision of adequate control systems. Free-water surface in tanks must be considered as to its effect on stability.

In the case of large jackets in deep water, the jacket may be transversely pre-cut into two sections. The top section may then be lifted off directly, leaving the bottom to be handled as described in step 9 above.

Obviously, the ability to lift off the entire deck for transport to shallow inland waters is very attractive. One such scheme, successfully used in the Gulf of Mexico, employs the Versa-Truss. Two barges are brought to the platform and positioned one on each side, spaced out from the legs of the platform. The barges are connected underneath the jacket with a combination of heavy struts and multiple part lines, enabling deck engines to pull the two together with high force. Lines at the ends are crossed to prevent relative displacement end-0, in yaw. Inclined, hinged struts are raised from each barge to be secured under the deck girders, which are reinforced and strengthened as necessary. The jacket legs are then cut, and the two barges are pulled together, which causes the struts to move vertically, raising the deck. The deck is now supported as a catamaran and can be transported to shore, where it can be lowered intact on a trestle or lowered to the bottom in shallow water. Thus, the deck removal is accomplished without the use of a crane barge.

The Norwegian Petroleum Directorate has contracted the conceptual design of a vessel for the removal of decommissioned platforms in the North Sea. It envisages the construction of a large, deep-draft vessel, consisting of two lines of 25-m diameter vertical concrete cylinders, joined rigidly. These are spaced about 80 m apart to permit them to straddle the typical offshore jacket. They are rigidly joined at one end by transverse concrete cylinders with reduced height, enabling the vessel to move under the deck.

Over the top are two sets of very heavy girders designed to pick up the deck at the same locations as the four jacket legs. When the salvage vessel moves underneath the deck, the heavy girders are connected to the deck girders by high-strength rods.

Many shallow-water platforms have jackets that weigh only a few hundred tons. Therefore, after cutting all of the piles, a large crane barge can rig slings to the jacket and pick the jacket up from the seafloor. Tag lines are attached to snug the jacket in against the stern of the barge, and the jacket is transported to shallow water or to shore for disposal.

If desired, a short jacket can be placed on a barge for transport provided that suitable cribbing is placed to distribute the load on the barge deck so that the protruding stubs of piles will not punch through the deck. The combined barge-jacket must, of course, be checked for stability.

Removal of the Frigg Drilling Platform required a different system to be employed for each of the components. Lightweight deck segments were lifted off piece by piece using the platform cranes.

- The heavy modules are removed by a heavy lift crane. The modules and module support frame will be loaded onto a cargo barge and tied down to be towed to shore.
- For the jacket, buoyancy tanks are attached to the four corners. Piles are cut off 1 m below the seabed. The jacket is raised 10 m and towed to shore in its upright attitude.
- Each tank is 50 m high, 6 mm in diameter, and provides 1150 tn. of buoyancy. The tanks will be tied at the top and bottom to fittings welded onto the jacket legs and clamped in place by brackets. On reaching the shore, the jackets will be supported between two barges and cut up progressively as they are raised.

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In the case of the Northwest Hutton piled jacket platform in the North Sea, where the water depth is 140 m and the platform is very heavy, the planned procedure is "reverse installation." The 22,000 tn. deck will be separated into 2000 tn. modular components to be lifted by a crane barge for transport ashore and recycling. Then the module support frame and the jacket will be removed in segments down to the pile sleeves by progressive cutting and removal with large offshore crane barges. This will require 20 lifts of up to 3000 tn. The footings, consisting of the sleeves and grouted piles, will be left in place if approved by regulatory bodies.

- Leaving the drill cuttings in place on the seabed to allow gradual natural recovery is also proposed, since removal might cause more environmental damage.
- The proposal also includes trenching and burial of the export oil and gas pipelines.
- As noted earlier, gravity-base platforms have primarily been built of reinforced and pre-stressed concrete, although several are of steel and some recently constructed gravity-base structure (GBS) platforms are of hybrid steel-concrete construction.
- These platforms are characterized by large base caissons; the latter having originally provided flotation during transport and installation. A major concern in removal is that of excessive breakout resistance due to increases in soil shear strength, cohesion, and adhesion of soil with the base as well as with skirts and dowels.
- The basic concept in salvage and removal is to refloat the structure, and then tow it to a disposal site in deep water. A typical procedure is described below (see Figure 20.1).
- Removal of gravity-base structures that have stored oil and oil storage vessels presents a major problem due to concern over contamination of the sea. Norwegian authorities have required the entire structure of a steel storage vessel to be transported to shore, where it is cut into sections and cleaned before being recycled as scrap steel.

The steel GBS platform *Maureen* was subjected to the following steps:

1. Precharge the storage tanks with nitrogen to 9 barg
2. Deballast the secondary spaces
3. Inject water and mud slurry under the base, through grout overflow pipes, in order to hydraulically jack the platform by raising the pore pressure in the sands
4. Continue deballasting. Lower internal pressure of nitrogen to 4.5 barg
5. Maintain stability during the raising of the structure by the use of tugs and slow deballasting. Keep tight control until the tank tops break the water line
6. Lower nitrogen pressure to 3 barg
7. Tow to shore base at 64 m draft

20.4 Removal of Concrete Gravity: Base Offshore Platforms

Preliminary proposals have been either to leave concrete gravity base platforms in place with all of the equipment removed or to tow them out to the deep ocean and jettison them.

Removal of GBS platforms may follow the steps outlined below:

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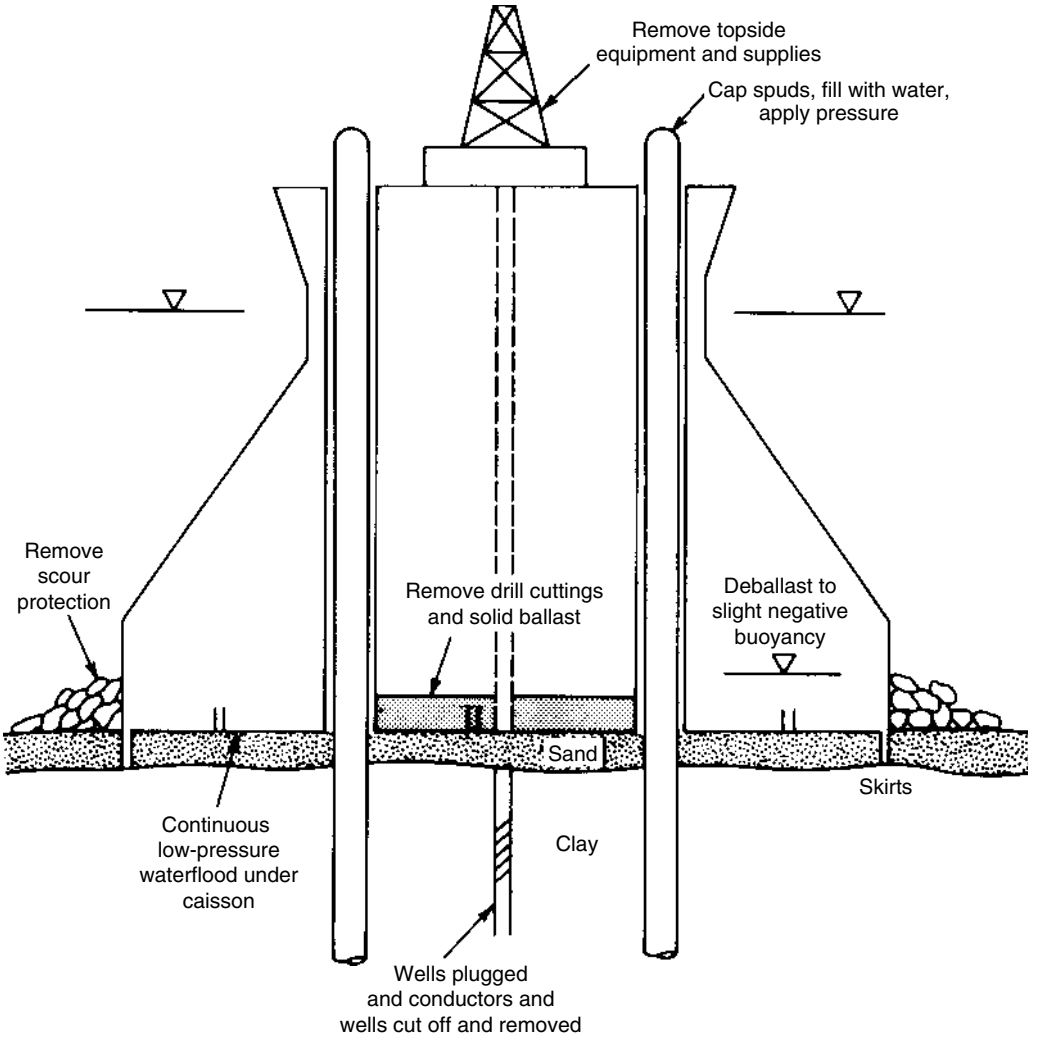

FIGURE 20.1
Removal of gravity-base concrete platform.

1. Cement and plug wells. Purge and disconnect connecting flow lines and risers.
2. Remove conductors and attached well casing. Plug conductor holes in base with concrete. Cut piping connections loose and plug all penetrations by concreting.
3. Remove equipment and facilities piecemeal from the deck, cutting the deck frame into sections as required for removal by crane barge. Alternatively, the deck structure can be removed as a whole. Preferably, its weight has been reduced first by the removal of selected items. The deck is cut free from the shafts. Then the same barge arrangement used to install the deck, typically three barges, can be moved in under the deck so that by deballasting, the deck can be removed and transported to an inshore site for further dismantling.

   However, the initial installation was performed under quiet, protected conditions whereas now the work is being carried out in the open sea. Typically, clearance between barges and jacket legs was of the order of 100–150 mm, which is impracticable in the open sea. Further, as the barges are deballasted to make

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contact, they will be heaving under the swells, no matter how calm the surface. Hence, special hydraulic jacks and compressible contact devices such as those used for "float-over" installations will be needed to cushion the impact.

4. Salvage equipment from riser and utility shafts.
5. Remove exterior ballast walls near base (cut post-tensioning ties), allowing ballast to spill out over the seafloor. Remove remaining ballast by jet or drag. Plug any openings, which may have been formed or cut for piping penetrations.
6. Remove solid ballast in the interior that may have been placed after installation on site. This ballast will probably have been either sand or slurried iron ore. This can be removed by airlift or eductor or specialized equipment such as Marconaflo pumps, which have jets incorporated to agitate the material and slurry it, facilitating its removal by airlift or eductor. The extent of removal depends on the computation of weights and the availability of access to compartments. It would, of course, be desirable to install access sleeves or manholes in the platform at the time of original construction to facilitate this operation.
7. At this stage, the ballast compartments are fully flooded. Using pipes leading to the underbase, inject water underneath at a low, steadily maintained pressure, slightly above ambient at the base elevation. The pressure must be low enough that it cannot cause piping under the skirts. Once piping occurs, little additional benefit can be attained by underbase waterflood. Maintain pressure for at least 24 h. Verify increase in pore pressure in soils beneath base of structure.
8. Deballast caisson to a slight positive buoyancy. If the structure does not break free, deballast one side more than the other to tip caisson off. Once caisson breaks free at one edge, water will be sucked in underneath and break all suction. Of course, if there are no skirts, water can flow under the base freely once the edge lifts off.

   Limit deballasting to the point where, if the structure breaks free, it will not rise above the level at which it is still fully stable. This can be a very critical stage because some excess positive buoyancy must be provided to extract the dowels. Once the structure breaks free, it will rise until equilibrium is reached. A special check must be completed to ensure that at this stage the structure is still stable.
9. Tow structure to disposal site.

Deballasting should normally be done by pumping to reduce hydrostatic pressure inside. The use of compressed air is generally not desirable and may be dangerous. First, if used under the caisson to help overcome suction, the high-pressure air will tend to escape laterally, leading to piping. Second, if compressed air is used internally to expel the water, as the structure rises, the external head decreases. The air then expands further. Thus, the structure tends to rise farther and faster than planned. Third, this expanding bubble of air creates a free-surface effect, traveling to the high side, where it exerts even greater upward force, developing an overturning moment.

In ship salvage, the use of compressed air has led to disastrous results when excess air is pumped in to overcome the suction: the ship breaks loose and rises up; the air bubble expands; the ship's rise accelerates. It rises to the surface, but now the air bubble is entirely on one side and the ship turns over. The air escapes, and the ship plunges back to the bottom!

It is obvious that extremely careful calculations are needed, taking into account not only the weight of the original structure and its displaced volumes but also changes that have occurred since, such as the following:

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1. Marine growth
2. Drill cuttings or sediments stored inside
3. Weight of underbase grout that sticks to the base
4. Setup of soil on skirts and dowels, increasing extraction force
5. Ballast or dropped material on caisson roof

In addition, the structural adequacy of various critical members must be looked at carefully because the uplift forces exerted by buoyancy are extremely great. Physical damage may have occurred during the intervening years.

20.5 New Developments in Salvage Techniques

New techniques are rapidly being developed for salvage, many of which are applicable to the removal of offshore platforms. Guidance can be obtained from ship salvage experts. One of these new techniques is the use of foam instead of compressed air. Foam has the advantage of displacing a fixed volume of water, with no change as the structure rises from the seafloor. Polyurethane foams are relatively low in cost but have depth limitations of about 100 m. Syntactic foams can be designed for performance at great depths but become increasingly expensive.

Inflatable neoprene balls can be used to seal tubular members, facilitating emptying or injection of foam. Inflatable neoprene buoys are also available which can be attached at several locations to reduce the net underwater weight. An attempt was made to salvage the damaged Frigg DPI jacket this way, attaching inflatable buoys. Unfortunately, during salvage operations, a summer storm tore many of the buoys loose. Working conditions at the site became very difficult, so the salvage effort was abandoned. The jacket was finally half-floated and half-dragged to a deep-water disposal site.

20.6 Removal of Harbor Structures

These may have been constructed with any of the materials and components used over the past 100 years: timber, steel, or concrete. They will usually be badly deteriorated due to their long exposure in the marine environment. Depending on the soils in which piles have been embedded, use of a vibrator may be effective in extracting the pile.

Timber piles in salt water may have been reduced to hollow shells by marine borers such as teredo or necked down in the tidal zone by limnoria. Since pulling is often not a viable option due to this structural deterioration, the best way is to "bite" them off in successive pieces with a heavy-duty clamshell dredging bucket. Alternatively, they can be broken off below the mudline by sidewise pulling using a wire rope line. Timber deck structures may be lifted off in sections or in large clamshell bites. Disposal is generally accomplished by burning at an approved site in favorable weather. Due to air-quality regulations, this combination of circumstances is becoming difficult to find. Use of large hydraulic chippers may be more practicable than burning.

Treated timber may be considered as hazardous waste. Alternatives include disposal on site, e.g., as fill, properly contained so as not to pollute the groundwater. When the timber piles are sufficiently sound structurally, they may be pulled by vibratory hammer. Timber piles do not deteriorate in axial strength in freshwater or below the mudline.

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Massive concrete structures, such as old bridge piers, are most effectively broken up by explosives, using delays. However, this practice may not be permitted owing to fish kill. Restricting the size of the explosion may be a solution, especially if accompanied by an air-bubble screen. Hydraulic fracturing may be employed. Alternatively, diamond saws (long lengths of wire rope to which diamonds or similar cutting particles are affixed) can cut the concrete structure into large blocks.

Steel sheet piles can be cut off by a diver using a welding rod, although interlocks are especially difficult to cut. Sheet piles may be pulled complete by vibratory hammer, in some cases augmented by jacks reacting against adjacent piles. Straps should be attached to the web with large diameter pins or bolts. Under high force, the web will fail in double shear. This risk can be minimized by drilling or reaming the holes and by using two or more holes, carefully located, so as to act together. Salvaged steel can of course be disposed of as scrap.

Reinforced concrete decks can be fragmented by hoe-ram, followed by burning the connecting reinforcing steel. Large segments can be cut by diamond wire saws. The large fragments can then be used as riprap or crushed for recycling.

20.7 Removal of Coastal Structures

Reinforced concrete structures will probably have suffered extensive corrosion of the reinforcing steel. For concrete piling, this will be predominantly in the tidal and splash zones; below water the piles may be relatively sound. Therefore, the piles may require extensive jetting. Pulling attachments may have to be affixed below low water.

If permitted, concrete piles may be broken by a sidewise pull, followed by a diver burning the reinforcing bars and pre-stressing strands.

Steel piles will probably have been necked down at the sand line by erosive scour from the moving sands and gravels. This, combined with corrosion, typically attacks the flanges of H piles. In the case of the Richmond-San Rafael Bridge, over a period of 40 years, the 75 mm and larger stone placed as scour protection eroded the tips of the flanges to paper thinness, reducing the pile cross-sectional area by about 15%.

In coastal waters, sands are almost always moving under waves and currents. Removal of piles may be by vibration or the piles may be cut by a ring of explosives jetted down into the seafloor, typically with the use of a casing. It is important that the piles be removed 2 m or more below the lowest seasonal sand line, so as to avoid puncturing the hull of any boat or injury to a surfer or swimmer, since the seabed level in shallow water migrates seasonally.

Who can say of the sea that it is old?
Distilled by the sun, kneaded by the moon,
it is renewed in a year, in a day, or in an hour.

Thomas Hardy, The Return of the Native

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21

Constructibility

21.1 General

Due to the size and complexity of most marine and offshore structures, as well as the environment in which they are deployed and installed, the actual construction—translating the design into a physical reality—requires very sophisticated planning, engineering, management, and verification. These are embodied under the overall term "constructibility."

The construction of offshore platforms has been a heavily cyclic industry, responding almost frantically to the discovery of new oil provinces such as the North Sea or significant changes in price level, such as those which followed the organization of petroleum exporting countries (OPEC) oil embargo of 1973–1974 and the Iraq War in 2003. The discoveries on the Alaskan North Slope and in the Canadian Beaufort Sea triggered a major effort in the Arctic, with its new environmental loadings from sea ice and icebergs. Then there was a hiatus of almost 10 years, during which the market stabilized and the industry matured and became more orderly in terms of construction and cost. By the time the boom of 1997–1998 arrived, technology had also changed, requiring a learning curve before the industry as a whole was geared to the new demands.

Recently there has been a widespread recognition that new sources of oil and gas must be found and developed. Crude oil prices have soared, making money available for exploration and development. The technology to construct in new environments must be developed.

Offshore structures and pipelines are very capital-intensive. They constitute an early expenditure which increases the need to control costs. They are often on the critical path so that schedule becomes of equal or greater importance. They comprise one area where thorough planning and competent construction management can achieve meaningful savings.

Constructibility is involved in the concept development and the integration of design with construction. It determines the selection of construction methods, facilities and stages, the procurement and assembly of materials and fabricated components, the organization and supervision of the work, and the training of workers. It includes analysis and planning, quality control (QC) and assurance, safety engineering and cost estimating, along with schedule and budget control.

It also includes an item of special concern to offshore structures: weight control. It addresses personnel and material transport and access, craneage and the planning of heavy lifts and the installation of mechanical and piping system within the structure. Constructibility employs work simplifications and standardization techniques in order to overcome the difficulties inherent in complex and sophisticated construction in an

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offshore environment. Finally, its scope includes deployment, installation, and subsequent removal, relocation, or salvage.

The actual construction process has been presented in preceding chapters. Because there are special additional requirements for constructability of offshore structures, this chapter will focus on them. Many tasks will also be applicable in some degree to harbor, river, and particularly coastal structures.

21.2 Construction Stages for Offshore Structures

An offshore structure goes through a series of very distinct stages as it moves from fabrication to offloading (or float-out), to completion afloat, to transport, to installation, and to module erection and hookup. The stages pertinent to each of the various types of structures have been described in some detail in the previous chapters. In constructibility planning, it is essential to formally set these stages forth by title, description, and schematic drawing (see Figure 21.1 and Figure 21.2).

Obviously, the first cut will deal with major stages of construction. Each of these major stages can then be subdivided into the detailed stages required. The stages should be further portrayed by a series of appropriate drawings or sketches. Isometric drawings

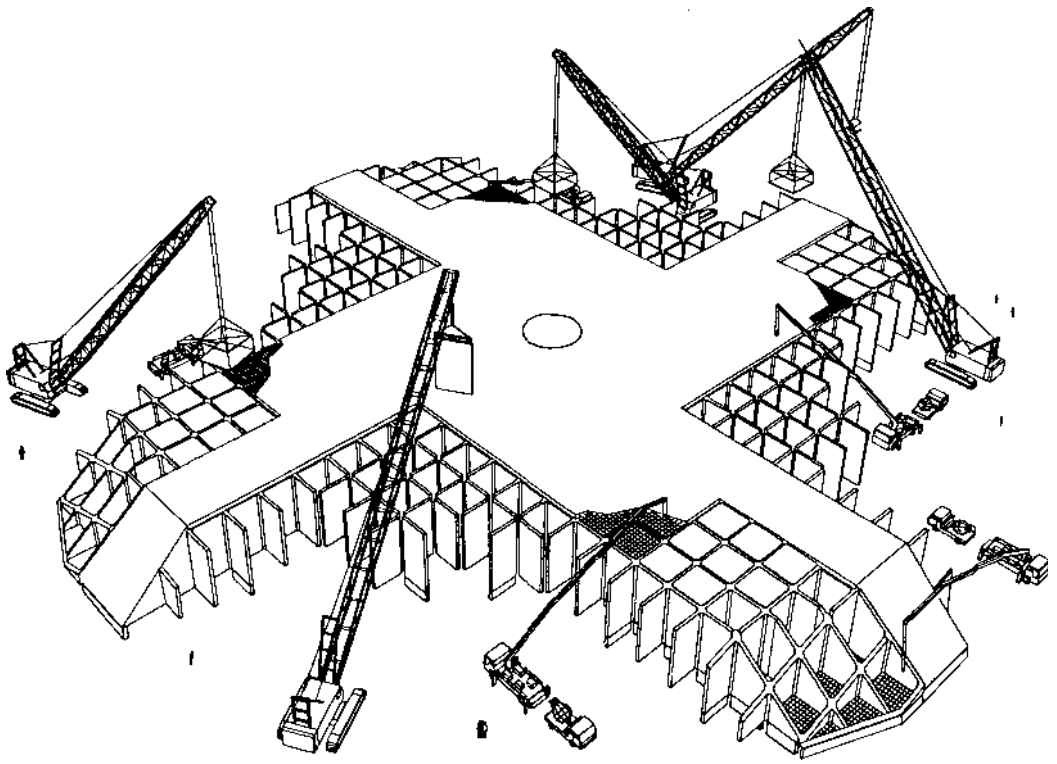

FIGURE 21.1
Construction stage planning diagrams. (Courtesy of Kajima Engineering & Construction Co.)

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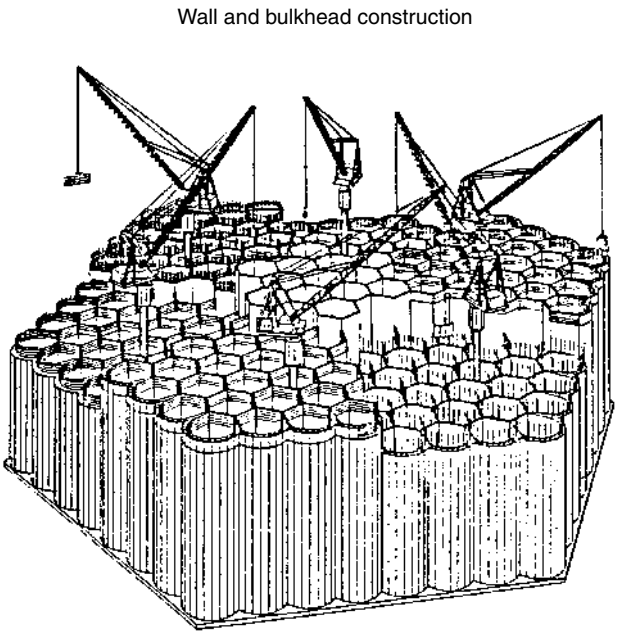

FIGURE 21.2
Computer-aided design (CAD) rendering for constructability planning. (Courtesy of Kajima Engineering & Construction Co.)

have been found extremely useful. The drawings should be essentially outline in character, with key items pertinent to that stage shown in heavy lines. The purpose is to eliminate aspects not essential to that stage so that the key elements can be clearly recognized. Thus, while they are based on engineering design drawings, they differ from them in emphasis, clarity, and use. Computer-aided design (CAD) is especially effective in enabling three-dimensional portrayal of the successive stages (see Figure 21.3).

Experience in the preparation of such descriptions and drawings has shown that serious errors have occurred due to "jumping past" intermediate stages, which have been incorrectly assumed to be unimportant or self-evident. The entire purpose of constructibility planning is negated when this happens, because it is just these skipped stages that so often turn out to be critical.

Once the constructor is satisfied that all the stages have been set forth, then engineering evaluations can be made of each such stage to ensure proper structural, geotechnical, mechanical, and hydrodynamic performance. As was noted in the chapters on steel and concrete structures and embankments, many elements are subjected to higher forces and stresses during these construction stages than under the design environmental loads.

Examples are:

Steel piles during driving
Pipeline bending and radial compression during installation
Legs and bracing of steel jackets during launching
Base raft of gravity-based structures during float-out
Cell walls of gravity-based structures during deck mating

For many of the stages, the key issue will involve the interaction of two or more disciplines. For example, ballasting by means of mechanical systems is intimately connected

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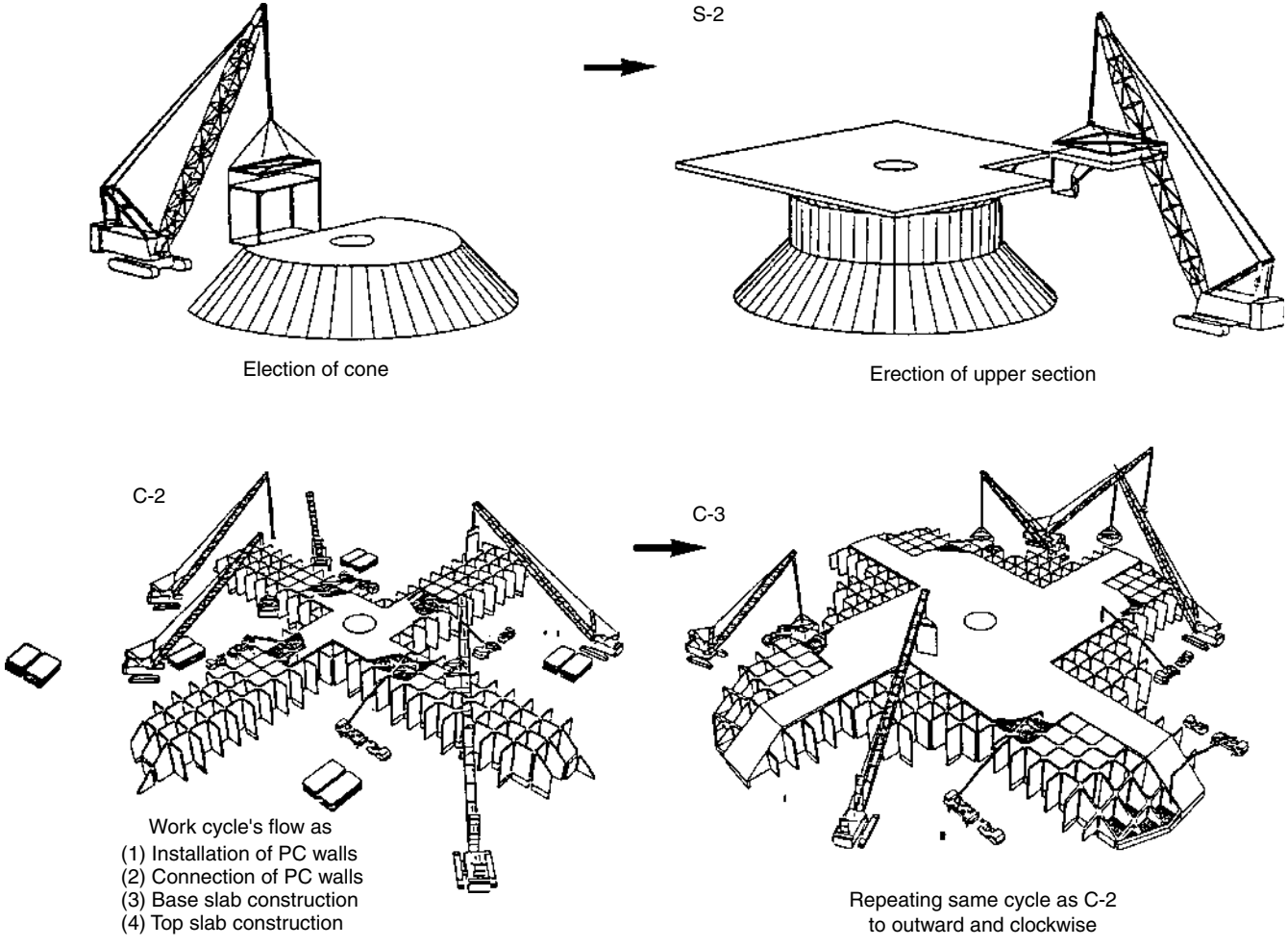

FIGURE 21.3
Construction stages laid out by CAD drawings. (Courtesy of Kajima Engineering & Construction Co.)

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to the structural capacity under differential heads, the stability performance afloat, and the instrumentation with its real-time readout.

Key considerations which have been inadequately addressed in the planning of previous structures include

1. Draft, with relation to available water depth during initial stages of construction or launching; freeboard
2. Stability during all stages of installation; effects of free surfaces
3. Tie-down of jackets and deck structures during transport
4. Hydrodynamic response of structure during tow, especially acceleration forces acting on mechanical installations; cumulative stresses (fatigue implications)
5. Effect of pressure and temperature changes on function of instrumentation, valves, and minicomputers
6. Wave and current forces during construction period. Initial contact with seafloor and the interactive effects of trapped water trying to escape
7. "Snap" loads of mooring lines due to stored energy from long-period excursions; use of fairleads and sheaves of proper diameter
8. Effect of shallow water and minimal underkeel clearance on wave characteristics, structure or vessel response; squat, yaw, wind heel, and seafloor scour
9. Control of draft and stability in event of ruptured ballast line, jamming of valve, or carrying away of bulkhead, allowing internal flooding
10. Human error in ballasting control—adoption of controls, training, and system isolation as needed
11. Arrangement of lines and umbilical control cables to prevent fouling during critical operations
12. Inadequate weight and tolerance control during fabrication, leading to launching mishaps
13. Inadequate consideration of tolerance in differential heads of ballast water in compartments
14. Inadequate securing of piles in jacket legs during tow
15. Inability to attain required penetration of piling with equipment on hand
16. Unintentional flooding of legs of jacket due to stuck valves or unclosed openings
17. Vortex shedding, vibration and fatigue
18. Welding temporary attachments and closures without following prescribed procedures
19. Changes in ballasting and reinforcement details during fabrication without approval of engineer

Item 19 above almost caused the loss of Beryl A when the ballasting sequence during construction at the deep-water mooring site was altered to facilitate accessibility and material handling. It did cause the loss of the Sleipner platform, when T-headed bars across the throats of the star cells were shortened in order to facilitate installation.

The division of the project into stages and the subdivision of each stage into actual steps is a procedure by which the most efficient method can be selected for each step.

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Sound judgment and experience will tend to integrate closely related steps within each stage. However, the limitation of such an approach is that the "forest may be obscured by the trees." Therefore, a conscious overall evaluation must also be made from the holistic point of view to ensure coordination and integration of all steps and stages. In the hands of an experienced constructor, such an overview may result in incisive decisions regarding the program and direction of the work. Review by an independent engineer or technical advisory board is recognized as a very effective way to minimize oversights.

In offshore construction, however, with its revolutionary developments in equipment, tools, and instrumentation, with its new structures and systems and environments, specific experience may not exist. Instead of relying solely on intuition, therefore, the conscious use of constructibility planning and evaluation of stages should lead to a more rational and effective program.

21.3 Principles of Constructibility

Some of the principles which can be beneficially applied to reduce the time and cost of construction are:

1. Subdivision into as large components and modules as is possible for fabrication and assembly
2. Concurrent fabrication of major components in the most favorable location and under the most favorable conditions applicable to each component
3. Planning the flow of components to their assembly site
4. Providing adequate facilities and equipment for assembly—the fabrication site must have adequate space for subassembly, storage, and access; the special facilities may include such items as synchrolifts, both land-based and barge-mounted, heavy-lift cranes, dry docks, and construction basins
5. Simplification of configurations
6. Standardization of details, grades, and sizes insofar as practicable
7. Avoidance of excessively tight tolerances; provision for flexibility and adjustment in connections, especially in mechanical system piping
8. Selection of structural systems that will utilize skills and trades on a relatively continuous and uniform basis
9. Avoidance of intermittent peaks in the demand for the labor force; selection of construction methods that involve relatively uniform demand
10. Avoidance of procedures that are overly sensitive to weather conditions; scheduling shop prefabrication and painting of elements which are very sensitive to the environment
11. Modularization of mechanical systems to be incorporated in or on the structure into the largest possible components, even if this requires additional structural support or interruption of the construction of the structure proper
12. Selection of construction methods which are appropriate to the specific structure, avoiding fixation on only one method, such as, concrete pumping, slip-forming, welding or barge launching; versatility in choice of methods

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21.4 Facilities and Methods for Fabrication

For offshore structures, the early stages of construction are carried out at a shore base. This base may be purpose-built for this one project or may be a relatively permanent facility. The area for such a facility must be adequate to accommodate not only the structure and/or components themselves but also storage of materials, access roads, support buildings, and infrastructure facilities.

Offshore structures are typically large in scope and will require a large number of personnel over a substantial period. Therefore, it will usually prove economical to expend the effort and money to build first-class facilities—with proper surfacing, roads, structures, utilities, and, where applicable, housing, to enable people and equipment to work efficiently.

The work will almost always go on around the clock; therefore, adequate lighting is required. The work will almost always continue even in inclement weather; therefore, adequate enclosures must be provided as appropriate to the work, especially for welding and painting, with adequate change rooms for the workers.

A common mistake is to make the facility too small in area, allowing inadequate room for storage of materials, prefabrication, cranes and trucks, etc. Adequate roads must be constructed, e.g., of gravel, around the platform, and adequate drainage installed.

The construction yard must be stable and firm enough to support the new structure and the construction equipment. Since yards are almost always located near the water, the original soils may require stabilization and fill such as compacted shell or crushed rock on which to operate. In weak sediments, filter fabric or pile supports may be required, over which rock may be placed, or a reinforced concrete slab.

Particularly punishing and critical loadings occur with large crawler cranes, since when they pick their maximum loads, almost the full load of the crane itself plus the lifted load are concentrated on one crawler or the toes.

Cleanliness of the workplace is important, not only for maintaining efficient access and safety, but to prevent blocking essential ports and ducts.

By definition, the structure will move from the onshore yard to afloat, either self-floating or on a barge. This requires bulkheads, dredging, and dolphins adequate to ensure the safe transfer of the structure to the waterborne mode.

A number of ingenious methods have been developed to facilitate this movement from onshore to offshore. Some of these are briefly described below.

21.5 Launching

21.5.1 Launch Barges

This is the method widely used for launching steel jackets. The structure is fabricated at grade. It is then slid forward, either on greased hardwood timbers affixed to steel beams or it is moved progressively ahead by jacks. Meanwhile, a heavy bulkhead has been constructed. The launch barge, a very large steel barge, with multiple ballasting compartments, is moored tightly against the bulkhead. Where conditions permit, as at Inverness, Scotland it is ballasted down onto a prepared sand bed. Where this is impractical, the barge deck's grade and trim during loading is maintained at yard level by ballasting.

The steel jacket is then slid forward onto the barge using winches on the barge. The barge, with jacket on board, is towed to the site. After ballasting the stern down, the jacket

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is pulled off by winches or a tug. To accommodate the concentrated loading at the stern, a hinged section is installed, which rotates as the jacket slides off (see Section 11.4 and Section 11.5).

Jackets of 30,000 tn. and more have been launched in this manner.

21.5.2 Lifting for Transport

Concrete box caisson, up to several thousand tn. in weight, have been fabricated at grade in a prepared yard. They are then moved to a bulkhead by skidding, or similar means. The box caissons for the Great Belt Western Bridge were jacked forward, sliding on concrete beams which had spaced teeth, so that the jacking could be simultaneously carried out in increments. This enabled the segments to be moved to the bulkhead, where they were lifted off by the large crane barge, which then transported them to their site and lowered them in place. When the water was rough or the segment was near the capacity of the crane barge, the segment was partially lowered into the water so as to gain additional lift by buoyancy.

For the Confederation Bridge in Eastern Canada, the box caissons were transported by wheeled carriers, which lifted them up off their fabrication foundations and rolled them forward on heavy concrete beams.

21.5.3 Construction in a Graving Dock or Drydock

This is the method used on most large box caissons which are built in a shipyard. In this case, the structure is floated by simply allowing the dock to fill. To avoid suction effects on the flat bottom, the base slab should be cast on polyethylene or plywood sheets. Flooding water into this joint at low pressure, continued for several hours, will gradually break any suction bond.

The advantage of this relatively simple launching is offset not only by the rental costs of the drydock but by the higher labor costs of fabrication in a deep pit, often with inadequate side clearance for efficient operations. However, in the case of the main piers for the Oresund Bridge between Denmark and Sweden, a very wide graving dock was made available at Malmo. After the two concrete box caisson piers had been fabricated, two large steel barges were floated in, one on each side. The dock was again dewatered and the two barges joined as a catamaran, straddling the pier. Multiple jacking assemblies were affixed to the barge so as to give several thousand tn. of lift. The dock was flooded and the gates opened. The buoyancy of the caisson, plus the lift from the catamaran barges, enabled the structures to be floated to their sites and installed.

21.5.4 Construction in a Basin

Relatively permanent construction basins usually use caisson gates as the closure, since they enable rapid removal and re-installation. This system, incorporating a prestressed concrete gate was used for the construction of the caissons for the offshore terminal at Hay Point, Queensland. The steel jackets for the 40's field in the North Sea used steel gates, while for the offshore platform at Loch Kishorn, Scotland, prestressed concrete gates were employed (see Figure 21.4).

Steel sheet piles in a rock dike have been used for the production of the Condeep gravity-base platforms at Stavanger, Norway. Sand dikes with a steel sheet pile cut off and well points were used to construct the 66 piers of the Oosterschelde Storm Surge Barrier in The Netherlands and the offshore terminal caissons in Queensland, Australia. An ingenious system of successive basins was developed and successfully used on

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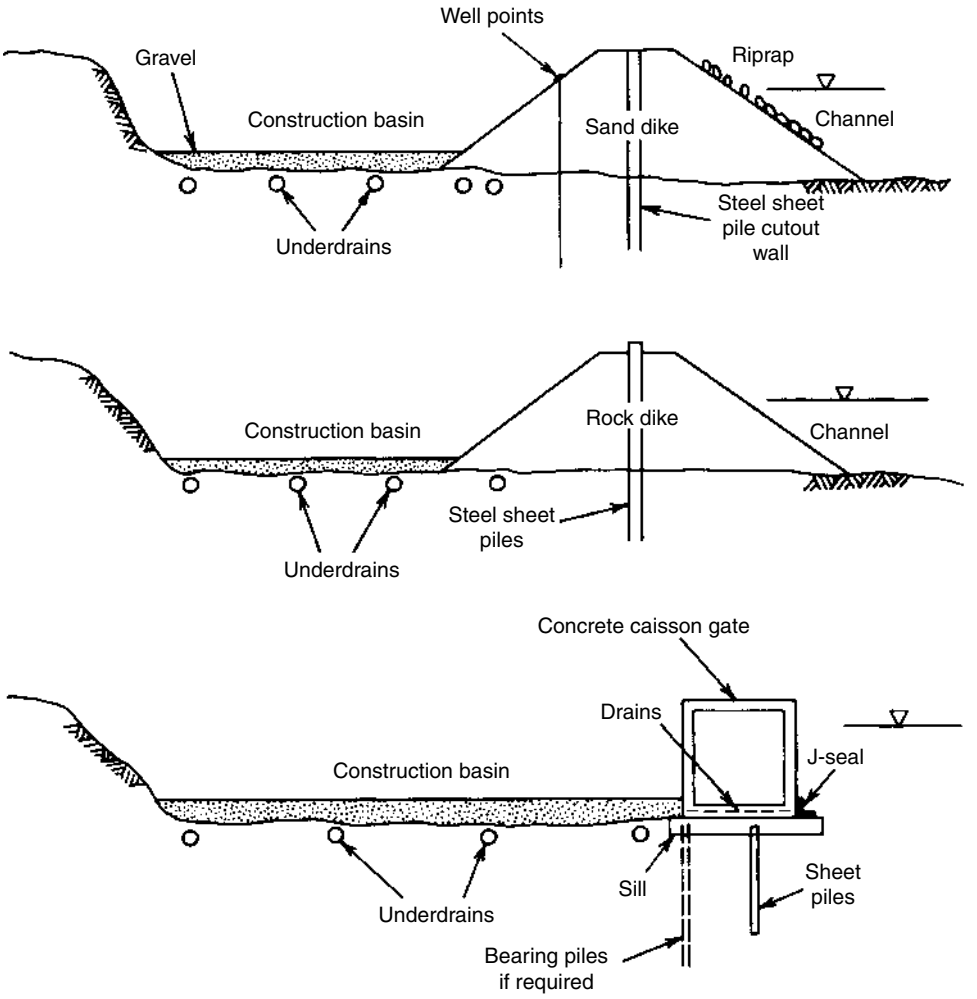

FIGURE 21.4
Three schemes for construction of basins for fabrication of offshore-type structures.

the Kish Bank Lighthouse of Dublin, Ireland, and later in Dubai for construction and launching of the three offshore oil storage tanks. Most recently, the principle has been successfully used for launching the Øresund Tunnel segments in Denmark and the segments of Braddock Dam across the Monongahela River at Pittsburgh, Pennsylvania. It is especially well suited for multiple segments. Two basins are constructed—one shallow, the other deep. The structure is constructed in the shallow basin. When complete, it is floated into the deep draft basin. Then when the river or tide is sufficiently high, the gates are opened and the structure floated out. This concept enables the fabrication to be performed at or near grade, with optimum access. The deep basin is of limited size and never dewatered; hence its cost is reduced.

21.5.5 Launching from a Ways or a Launch Barge

Very large and heavy structures have been launched from building ways—for example, tankers and subaqueous tube segments. Side launching usually results in much lower structural stresses than end-0 launching. It is essential that the launch be uniform and

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that one end not hang up or lag behind the other. End-0 launching produces high bending moments as the leading end is picked up by the buoyancy of the water. The other end, meantime, is transferring very concentrated loads onto the ways, and in turn is itself experiencing very heavy concentrated forces.

Jackets are usually launched from the launch barge in the end-0 direction because of their tapered configuration. However, the Lena guyed tower, with its rectangular cross section, was successfully side-launched, and, as noted in Chapter 11, recent Japanese studies indicate that it may be applicable for tapered configurations as well. Often it will be found desirable to employ a cradle, sliding on the ways, to enable launching in the correct attitude for floating. The prestressed concrete floating phosphate plant Rogamex was constructed on ways in Singapore and side-launched. Side-launching from a ways is planned for the large concrete shells of the Olmsted Dam on the Ohio River.

21.5.6 Sand Jacking

In the rarely used but historically proven method of sand jacking, a basin is excavated by dredging, keeping the basin full of water. The basin is then filled with sand up to a working grade. A temporary rock surfacing is placed. The structure is now constructed at normal yard grade, with full access (Figure 21.5).

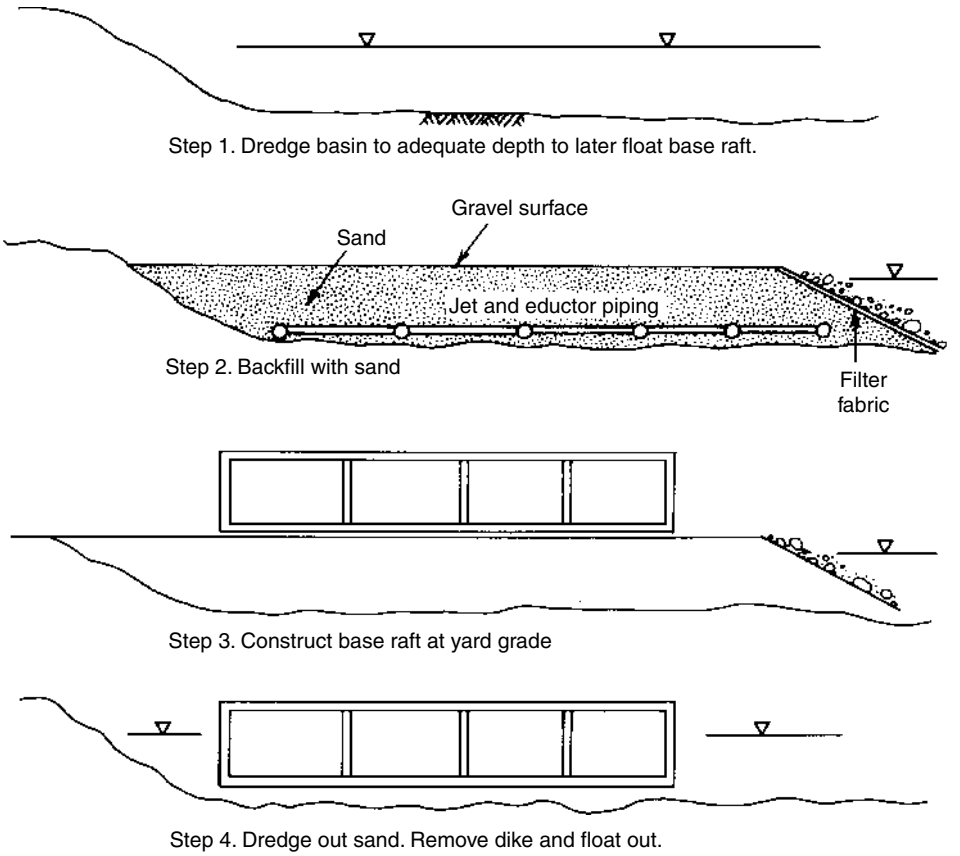

FIGURE 21.5
Sand-jack method for construction and launching of offshore-type structures.

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When it comes time to launch, the sand is excavated by suction from under the structure, using jetting as necessary to promote horizontal flow of the sand so that relatively uniform load distribution occurs. Stresses in the structure are continuously monitored, as are excavated depths along the structure's sides and underneath. Appropriate adjustments are made in the sand removal operations. When fully excavated, the structure floats free and is towed out. The sand fill can now be replaced.

This method eliminates the problems involved in dewatering a basin, while enabling all work to be carried out at yard grade. Jetting and eductor piping may be pre-installed in the sand fill to facilitate dredging and promote flow of the sand.

21.5.7 Rolling-In

Large-diameter piles, cylinders, and tubes may be launched by rolling down a ways. As with side-launching, it is essential that the cylinder move down parallel to the shore and that one end does not hang up. While this method is theoretically applicable to such extremely large cylinders as the SPAR, local bearing and buckling would probably be excessive and thus prevent its use.

21.5.8 Jacking Down

The advent of modern hydraulic ship lift systems enables modules, for example, to be skidded out over the barge slip on girders and then lowered down onto barges by hydraulic lift jacks. Such a facility is especially well fitted for repetitive load-out operations.

21.5.9 Barge Launching by Ballasting

Many sizable offshore structures have been constructed in segments on a large barge or in a floating dry dock. The segment is then launched (or floated off). This system is especially well adapted to the launching of a subsea template. The template may be assembled on the barge. Alternatively, it may be fabricated onshore and skidded or transported onto the barge for transport and launching. During the launching, the barge is usually submerged by flooding. In many cases, the main body of the barge is completely submerged so that the structure can float directly off. Stability during and after the submergence and launching becomes a major concern.

As the deck of the barge goes underwater, the water plane area is now reduced to that provided by the structure on deck. At this critical stage, the center of buoyancy is essentially the geometric center of the barge. The center of gravity of the combined system (barge plus ballast plus structure) typically is still quite high, so the righting moment furnished by the water plane is very important. It no longer is that of the barge but now is limited to that of the structure. The free-surface effects of the water ballast used for submergence must also be taken into account. To overcome these, it is usual practice to have some compartments topped up, others empty, and only a few with a free surface. The structural effects of such unequal loading must then be considered.

At deeper submergence the structure starts to lift off. Now its water plane no longer assists the barge in maintaining stability. Accidents have occurred in which the barge rotated uncontrollably during this stage. To provide stability control, the barge is usually fitted with columns at one or both ends, which give enough water plane moment of inertia to provide stability. These columns also allow the draft of the barge

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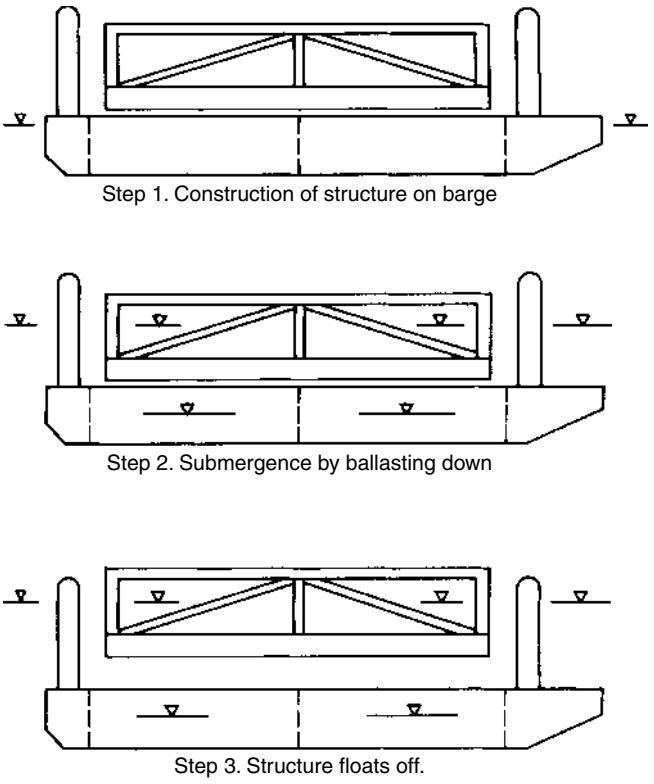

FIGURE 21.6
Barge launching of subsea template. Note buoyancy tanks for maintaining stability.

to be accurately controlled. See Figure 21.6. Alternatively, at this stage, one end of the barge may be tipped down to rest on a prepared seafloor at the proper depth, thus gaining stability from the end of the barge.

The barge is, of course, subjected to an external hydrostatic head in excess of that normal to conventional barges. Obviously, a specially designed barge is called for, or else a standard barge must be modified by internal strengthening and by sealing of the vents and other deck fittings. Heavy-duty submersible barges used for the ocean tow of jack-up rigs and dredges may be available for the construction. These were used to construct, transport, and launch the concrete caissons for the Ma Wan tower of the Tsing Ma Bridge in Hong Kong, as well as for the Tarsiut caissons for the Beaufort Sea in Arctic Canada.

Barge launching has been successfully used for the construction of hundreds of compressor and pump-station concrete barges, which were designed to be towed to locations in the shallow waters of the Gulf of Mexico and permanently ballasted down onto the seafloor. Launching takes place above a carefully screeded seafloor, which allows the barge to ground before lift off. It is then ballasted to stay in place while the new barge under construction floats off. Columns at the corners maintain stability during recovery.

21.6 Assembly and Jointing Afloat

Jointing afloat of large structural units in calm, protected bodies of water is now a well-established art. The Hondo steel jacket, over 280 m in total length, was constructed in two

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segments, which were joined while afloat in partially protected waters. Mating cones, hydraulic ram locking devices, and internal welding all were used to provide full structural continuity.

A large concrete floating drydock was constructed in northern Spain by first fabricating barge-size segments on a barge, launching them, and then joining them by prestressing and tremie concrete. The hulls of large tankers have been constructed in two halves and then launched and joined together while afloat. A temporary cofferdam was used to enable dewatering. Heavy rods and bolts held the sections in alignment for welding. Subaqueous tunnel sections and large outfall and intake tunnel sections have been joined underwater by various combinations of bolting, prestressing, and underwater concreting and grouting.

The Valdez Floating Container Terminal was built in two 100-m-long sections, then towed 1000 miles to the site and joined by concreting and prestressing the joint. The Hood Canal Floating Bridge was similarly constructed of large concrete sections which were joined at the site by grouting and prestressing. The Japanese have joined prototype barge-type units together in Tokyo Bay, in preparation for their construction of an offshore floating helicopter base off Okinawa. Floating Bridge segments have been joined to form a number of long bridges in the state of Washington and in British Colombia, Tasmania, and Norway.

In general, the principles applied were the following:

1. Use of large mating cones and sockets plus winch lines for initial positioning; these should be sequential, so that each degree of freedom is snubbed and then locked prior to another fixation
2. Sealing the joint zone to make it watertight; temporary locking by external bolts and girders
3. Dewatering the joint zone
4. Permanent jointing by bolting, welding, or prestressing plus concreting, grouting, or epoxy injection
5. Permanent sealing against water in-leakage

Chapter 13, Section 13.7 examines the joining of large structures in the open sea.

21.7 Material Selection and Procedures

The design will, of course, have determined the specifications for the materials based on their performance in service. Constructibility considerations will now go further, as the constructor addresses the practicability of building the structure to meet the specifications.

With steels, for example, welding procedures and materials are intimately related to the ambient temperature and moisture conditions under which the work will be carried out. The constructor has an opportunity to optimize these by one or more of the following steps:

1. The constructor may elect to carry out the majority of the welding within protected, heated, and dry enclosures.

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2. The constructor may elect to use preheat and/or postweld treatment to attain the required results.
3. The constructor may elect to purchase specially processed steels which are less sensitive to the conditions, provided the design engineer has approved the change.

With concrete structures, the constructor has even more alternatives from which to select the optimum combination. The constructor may increase the cement content in order to gain workability and early strength. The constructor may use a superplasticizer admixture (i.e., "flowing concrete") to improve workability and strength and lessen the need for vibration. The constructor may add air entrainment to improve workability and prevent segregation. The constructor may include condensed silica fumes in the mix to increase early strength and cohesiveness. The constructor may use anti-washout admixture to eliminate bleed and laitance of underwater concrete.

The timing and sequence of addition of the various components of a concrete mix have a decided effect on its properties. For example, air entrainment should usually be added at the end of the mixing cycle. Aggregate selection and gradation may be modified. Surface characteristics, absorption, strength, and thermal properties are all-important parameters. To control heat of hydration and thermal gradients (and hence cracking), aggregates may be pre-cooled, ice may be used in the mix instead of water, and cement type may be changed. Fly ash or blast furnace slag may be used to replace a portion of the cement.

The method of delivery and placement of the concrete affects its quality. Pumping of concrete, for example, compresses the air entrainment bubbles. It forces water into absorptive aggregates, thus gradually stiffening the concrete mix. The curing of fresh concrete and the insulation provided to the forms and freshly exposed surfaces are of great importance in preventing shrinkage and thermal cracking and ensuring durable concrete. Concrete with superplasticizer admixture is subject to sudden slump loss, especially in hot weather. Retarding admixtures may be needed. Grouting of prestressing ducts in cold weather may require the use of antifreeze admixtures.

With embankment materials, the in-place density and side slopes are very sensitive to the gradation (fines) and the method of deposition. Overflowing of dredged materials may be effective in reducing fines. Sometimes materials must be blended from two or more sources in order to obtain an optimum mix.

Soil and rock materials may be deposited underwater in a variety of methods: dumped as a mass, placed through a tremie tube or with a skip, discharged hydraulically at the surface, or discharged at the seafloor through a specially designed separator. The constructor may have to decide between greater care (and cost) in placement and a supplemental operation of densification.

Dredging of slopes underwater can be controlled to a significant degree by the method of operating. For example, in hydraulic dredging, cutting "up" at the slope may allow the bank to collapse and initiate a slide whereas cutting "down" may prevent it. The creation of a deep vertical cut in the slope by any dredging means is undesirable and may initiate a slope failure.

Deposition of material during dredging is especially sensitive. If deposited at the top of the slope, the weight surcharges the bank and may produce failure. Dropping the materials, e.g., from a clamshell bucket, produces an impact which may cause shear failure. Lowering the bucket to the surface and then discharging may be required. Ponding of water adjacent to the slope during hydraulic dredging increases the likelihood of slope failure.

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21.8 Construction Procedures

Within the context of each construction stage, suitable procedures have to be developed to meet the following criteria:

1. Strict compliance with the specifications and drawings
2. Assurance of meeting quality requirements
3. Ability to meet schedule requirements
4. Adaptability to equipment, facilities, and skills available
5. Economy in overall performance: lowest possible costs consistent with items 1, 2, and 3
6. Minimum risk of accident or delay

Each major operation within each stage is analyzed with regard to the most efficient method of construction.

Since the two largest expenditures within the control of the constructor are the fabrication of the structure and the hookup of mechanical facilities, the principal attention insofar as efficiency is concerned is directed to these two phases. However, the phases involving heavy lifting, load-out, launching, delivery, and site installation, while not heavily labor-intensive, are controlling from a technical and equipment viewpoint, so attention must be directed to them also, to ensure technical performance and safety. Thus the focus in the procedures for the different stages of the work differs, in one case being directed to efficiency, in the other to equipment selection and technical performance. Evaluating the procedures and selecting methods is essentially a series of sub-optimizations. The constructor temporarily isolates each, placing boundaries at each end of the stage, and develops the most efficient methods for that stage.

Due to the immense amount of work in fabrication, whether steel or concrete, the approach should follow the same logic and patterns as those used in the Japanese shipbuilding industry. There the work is broken down into as many sub-units as practicable. Each is then fabricated in the most favorable attitude (often upside down) and under the most favorable conditions. Advantage is then taken of the great advances in heavy transport and lifting gear to move large components to the assembly site.

Since offshore structures are assembled on or near the water, this opens the opportunity for wide dispersion of the fabrication site for the components and their subsequent transport by water. The assembly proper can then be carried out afloat in a sheltered location, or in a dry dock, graving dock, or basin or on land at a launching facility. Use can be made of heavy-lift transporters of several thousand tn. capacity, sheer-legs and hammer-head crane barges up to 8000 tn. and more, synchrolifts capable of handling 50,000-tn. components, overhead gantry (bridge) cranes of 1000–4000 tn. and catamarans 20,000 tn. capacity.

Large crawler cranes and crane barges can be used together in parallel, to raise the complete sides of steel jackets or to lift huge modules. Obviously, very close coordination and control will be required. The planning must consider the changing distribution of loads and radii as the lift takes place. Final assembly is facilitated by having detailed the fit-up so that the connecting pieces are automatically guided to exact location. Obviously, accuracy is essential. The detailed engineering must consider the effects of thermal differences and of deflections due to deadweight in each of the different attitudes.

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With regard to the fabrication of steel tubular components, the decision must be made where to place the junctures, whether at nodes or in midlength. Since the nodes are three dimensional, fit up is usually much more difficult there. If the juncture is in the mid length of a tubular, the nodes can be first erected to their correct position, the main tubular cut to exact length as measured in the field, and the girth welds readily made. However, this then involves an additional joint. Another system is to pre-cut and contour one end, allowing the other to run long. After the first has been welded in place, the other end is field-cut to length.

Most modern yards now have computer-controlled cutting and beveling of the members, which ensures exact fit at the nodes.

With regard to a jacket or large module frame, how should subassemblies be selected? Should the jacket be split into its several panels for component fabrication, as is extensively practiced at the McDermott and Brown and Root yards on the Gulf of Mexico, or should it be split into three-dimensional space frames, as used by NKK for the North Rankin platform?

With concrete structures, several decisions have to be made. Will all elements be cast in place or will some or any be precast? For the Ninian central platform, several hundred concrete shell units were precast in southern England and then transported to northwestern Scotland and erected by a sheer-legs crane barge. For the CIDS Arctic platform, precast cellular internals were combined with cast-in-place external walls. Precast stay-in-place shell forms have been used on a number of platforms for the North Sea. The opportunity for making them composite with the cast-in-place concrete should be considered. Precasting offers many opportunities for dispersion and sub-optimization, but also requires consideration of lifting capabilities and joint details (see Figure 21.7).

The next decision is in regard to cast-in-place concrete. Should slip-forming be used or panel forms? Slip forms have been very successfully used on the Condeeps, for example, but require a large surge in manpower during short periods of time. Will the climbing rods and yokes interfere with embedment and reinforcement placement? Panel forms enable the reinforcing installation to be carried out at different locations from the concrete

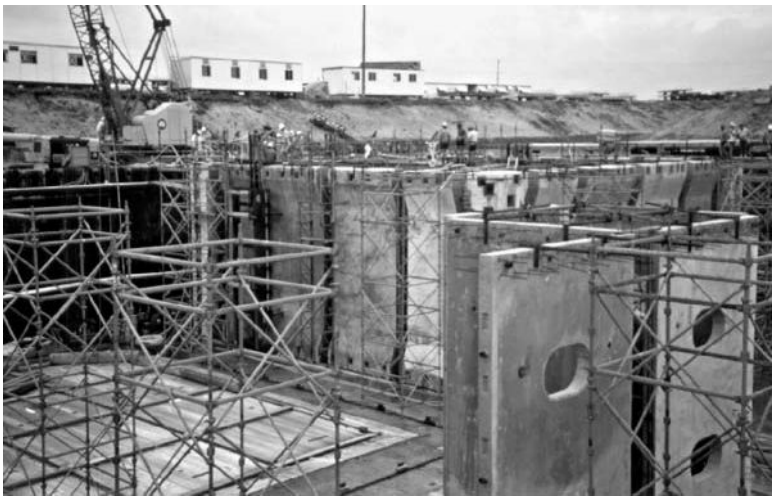

FIGURE 21.7
Precast concrete panels being assembled in construction basin. Joints will be cast in place and assembly will be post tensioned.

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placement, facilitating dispersion and equalization of manpower requirements, but these require more construction joints.

A third major decision is the method to be adopted for concrete delivery and placement. Will it be by pumping, or by bucket or wheelbarrow? All have been used effectively. The required rate of placement may be a determining factor. If "flowing concrete" is used, this affects both the method of forming and that of placing.

Most of the critical manpower requirements on a concrete offshore structure relate to the reinforcing steel installation. Should the bars be handled individually, as is practiced on slip-forming operations, or should pre-assembled cages be used? How should splices be made: by lap, weld, or mechanical connectors? Will color coding of reinforcing and prepackaging of the reinforcing bars for individual zones save time for the placing crew? In particular, should the stirrups, of which so many are required in the typical offshore structure, be bent bars, pre-welded loops, or mechanically headed T-bars?

Mockups can play a very important role in the decision making for critical fabrication operations. Full-size sections of the structure, whether intersecting tubular nodes or the juncture of concrete shells, are selected. They are then fully fabricated, with all inserts, post-tensioning ducts, reinforcing steel, and stiffener plates. This mockup enables the visualization of the interaction of the many details and the practicability of welding, steel placement, and concreting. Such mockups have invariably proved their worth, especially if carried out by the same individuals who will be responsible for their subsequent construction in the field.

During the sub-optimization process, many operations during the fabrication and erection will have to be carried out at high elevations, 50 m or 58 m above the base and land. Since the workers will require staging, can this be pre-attached before erection? If precast concrete or steel components are to be erected, what can be done to facilitate their initial setting quickly and accurately? On the Ninian central platform, bearing plates with screw adjusting nuts were set in the previous concrete pour. A survey crew then surveyed each plate accurately, adjusting it to the proper level and scribing the exact location on the bearing plate where the bearing of the shell should sit. Before each shell element (100–300 tn.) was lifted, tag lines were attached. The workers on top could then catch the tag lines and guide the shell unit to the proper location without waiting for further survey checks.

Mockups also serve a valuable purpose in training workers, especially if they are shown the results of their work. For example, if placing concrete among congested reinforcement results in honeycomb and rock pockets, the workers can visually see the need for vibration. If welding studs with excessive heat results in plate warping, they will understand the reason for the requirements for frequent shifting of location and imposed time lags (see Figure 21.8).

One lesson from Japanese shipbuilding practice that has been demonstrated repeatedly in their construction of offshore equipment and structures (see Figure 21.9 through Figure 21.12) is that teams of workers, comprising several trades and assigned to a specific task group with the objective of completing all work within a specific zone, are much more productive than a highly centralized organization by trades alone, with each trade then responsible for all work within its classification throughout the structure. Their analysis of worker productivity has shown that the most important elements are

1. Good access and adequate work room
2. Favorable position for working
3. Ability to pace one's own work without excessive dependence on the progress of fellow workers

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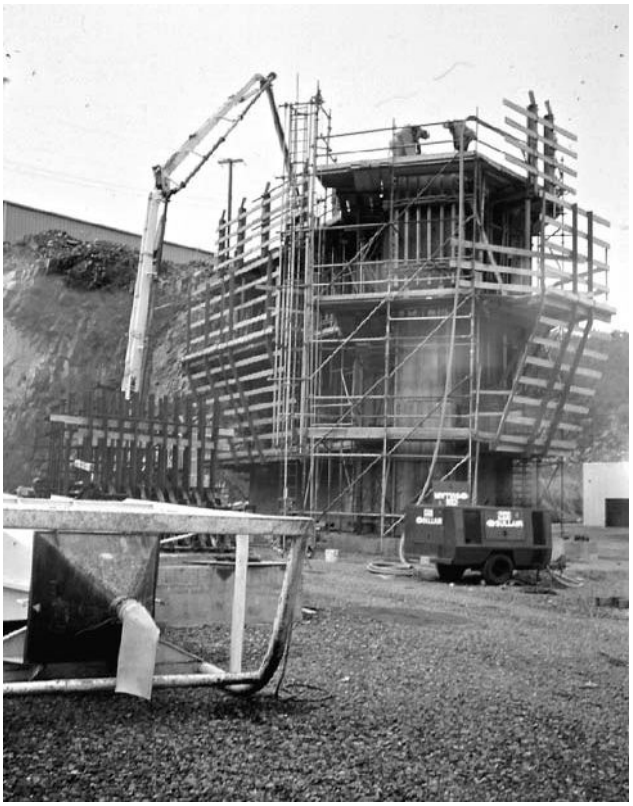

FIGURE 21.8
Mock-up of wall for Hibernia platform, complete with reinforcing and prestress in steel.

4. Immediate availability, close to hand, of tools and materials
5. Clearly defined work program and procedures
6. Identification of the individual worker as part of a team.

The adoptions of the "zone" concept or organization, as opposed to the "trade" concept, is a return to decentralization. Since basic skills and techniques continue to be implemented by trade, the new approach resembles the matrix system of organization which has been adopted by some large engineering organizations. The task group teams will, of course, be reassigned and reconstituted as the needs demand. Even where specialist subcontractors are involved (e.g., for post-tensioning), the task group organization appears to give greater overall efficiency and reliability.

Construction procedures offshore are planned with primary consideration being given to the sea states and weather conditions under which they will be performed. Chen and Rawstron, in their paper "Systems Approach to Offshore Construction Project Planning and Scheduling," *Marine Technology Journal*, October 1983, make use of advanced simulation techniques in the planning of offshore construction operations. Limiting sea states are determined for various operations such as module lifting, pile driving, pipe laying, and saturation diving, and the effects of vessel motions on operations are evaluated. From such analytical techniques, the duration of operations, adequacy of equipment, sequence of work and risk of delay or cost overrun can be evaluated.

Construction offshore must similarly be planned stage-by-stage in order to ensure the most efficient operation. This is most effectively done by a series of sketches, showing, for

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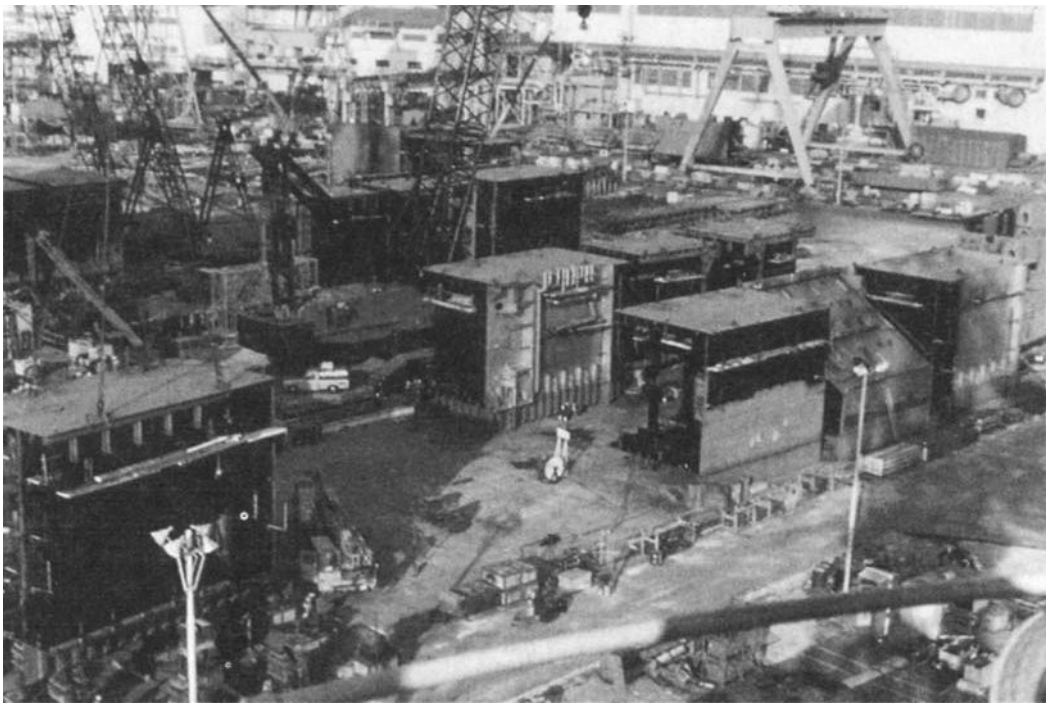

FIGURE 21.9
Japanese shipyard constructing steel base for Arctic offshore platform. Modular segments were shop-fabricated and joined in graving dock.

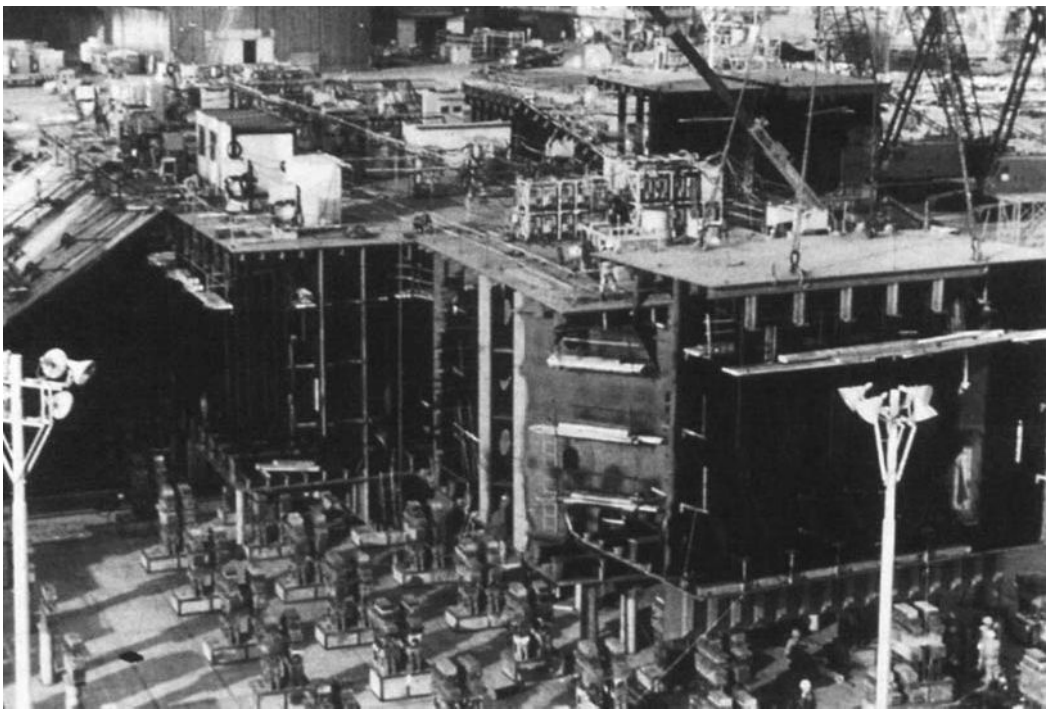

FIGURE 21.10
Assembling steel modular segments in dock.

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FIGURE 21.11
Rehearsing the lifting and insertion of prefabricated shiploader arm, Hay Point Terminal #2, Queensland, Australia.

each stage, the arrangements of the equipment, structure, and support vessels in plan, along with the location of anchors and mooring buoys and the lead of mooring lines. It is essential that each substage be shown so that, as equipment moves, the new leads of lines and the new locations for support craft are clearly apparent. Crane radii can then be

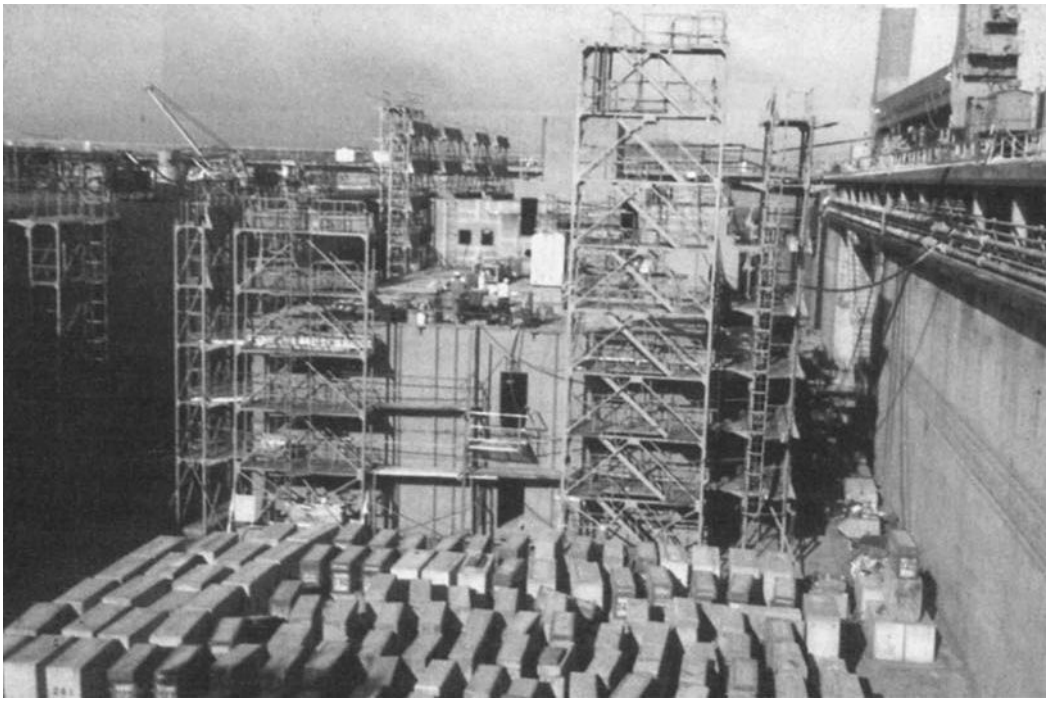

FIGURE 21.12
After module are assembly, all welding is performed semi-automatically.

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plotted. Isometric or vertical elevations will ensure that there will be no interference between boom and structure during critical lifts and that tag lines can control the lift. These stage drawings should be on waterproof paper. Kajima Engineering and Constructors prepared a set of these for their construction of an offshore terminal off Hokkaido. For the Statfjord platforms, Norwegian Contractors prepared detailed drawings of horizontal slices through the shafts which showed all the reinforcing bars and inserts at each elevation.

21.9 Access

A much-overlooked aspect of constructibility is that of providing access for personnel and equipment to the areas where they must work. Workers need safe and convenient access. Studies have shown that up to 50% of a worker's day is associated with moving. It is inefficient and expensive to have workers climb ladders, thread or force their way through congested reinforcing steel, climb across scaffolds, and walk planks. Proper and safe access needs to be engineered (see Figure 21.13).

How will personnel be transported to the various sites afloat? How will they be transferred from transport to platform or structure—by hydraulically operated walkways able to accommodate swell and heave or by Billy Pugh net? How will vertical access be provided—by elevator, hoist, or ladder?

An offshore structure during fabrication is usually a repetition of many similar cells or frames. Statfjord B Condeep, for example, had 90 identical skirts and 24 almost identical cells. It is very easy for workers to get temporarily lost. Markers easily read at night or in rain are needed, identifying locations both in plan and elevation.

Crane location, reach, and swing need to be carefully laid out to ensure that the boom will not hit the side of the structure as it reaches out to set the load and that loads can be picked and set within the allowable radius. This requires a three-dimensional study. Internal communication must be planned for general supervision and for guidance

FIGURE 21.13
Precast breakwater segments for Ninian central platform are erected with walkways already attached.

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of lifts, control of concrete slump, and supply of materials. Lighting for night work must consider shadows cast by the structure and cranes.

21.10 Tolerances

Offshore structures are not only among the largest structures built by humans but they are structures which must be moved, floated, and rotated to attitudes differing from those under which they were initially built. They are subject to a wide variety of external loads during construction. They must then interconnect with other systems. Tolerances, therefore, become of far more than normal importance. Weight control is critical. Steel jackets typically are launched and are then required to return dynamically to float on an even trim with only a minimal freeboard. Concrete structures, when ballasted down to receive the deck, have typically only 1% or less of reserve displacement at this stage. Weight control procedures must therefore be instituted and an organization set up to control the weights during the entire construction period.

For steel structures, items subject to weight variances include:

1. Variations in thickness (steel plates usually run to the plus side)
2. Variations in diameter
3. Stiffener plates
4. Lifting attachments
5. Weld material (usually overruns)
6. Erection bolts
7. Slings
8. Closure plates
9. Scaffolding
10. Instrumentation
11. Grout piping and vent piping
12. Coatings and paint
13. Anodes and anode supports

For concrete structures, items subject to weight variances typically include:

1. Variations in wall thickness (usually over)
2. Variation in geometry
3. Reinforcing bar length and allowance for splices (usually over)
4. Spacer bars, chairs, and supports
5. Embedments
6. Post-tensioning anchorages
7. Ducts (empty at some stages)
8. Unit weight of concrete
9. Water absorption into concrete
10. Scaffolding
11. Bearing plates

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12. Ballast quantities
13. Ballast unit weight in place (density, water, etc.)

The other group of factors affecting weight control comes under geometry control, which also affect fit-up, buoyancy, and structural performance. For a cylindrical structure, whether a pipeline or a concrete structure, the tolerance controls may include:

1. Out-of-roundness
   a. Two diameters at 90°
   b. Best-fit circle
   c. True circle
   d. Local variations from circle
2. Diameter
3. Wall thickness
4. Displacement of centerline
   a. From true position
   b. From relative position
5. For pipelines which are coated with weight coating:
   a. Thickness of coating
   b. Density of coating
   c. Water absorption of coating
   d. Bulging of coating at joints

Geometry control is important to ensure against buckling under external hydrostatic head and for subsequent fitting of other components—for example, embedments, piping, and precast units.

Pipeline segments are typically difficult to accurately evaluate for displacement and hence buoyancy, because of variations in coating thickness along the length, especially bulging of the coating at the end of each double-jointed segment.

With underwater embankments, elevation and slope tolerances must be realistic and related to the prevalent sea state under which the work will be undertaken, the equipment available, and the surveying facilities installed. For some of the earlier steel caissons (hulls) installed in the Alaskan and Canadian Beaufort Seas, very tight tolerances were required for the screeded surface on which they were to be seated, in order to prevent local damage to the bottom of the hulls. Adoption of more readily achievable tolerances at an early stage might have then been fed back into the structural design of the caisson bottom, with a significant reduction in site work under a critically limited schedule and extremely costly conditions.

21.11 Survey Control

Survey control is, of course, intimately associated with geometry control but extends beyond it to guide the fabrication and erection process before and during construction. Where components must fit to others, it is often difficult to establish the proper reference line. One must be selected—for example, a line connecting the center of two best-fit circles.

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The other points must be properly related in all three planes. Templates will often be found to be the best method of transferring complex interactive dimensions.

Match-casting of precast concrete members has been highly successful in assuring later fit. It was utilized effectively for the breakwater segments of the Ninian central platform. Care has to be taken in such match-casting to avoid distortions due to thermal effects, for example, warping during steam-curing. Match-casting, followed by erection with epoxy-glued joints and post-tensioning has been widely employed in bridge construction.

Similar match-fitting and templating can be used with steel fabrications, again recognizing potential distortions due to welding. Pre-assembly of the complex steel work for the Shasta Dam Temperature Control Device assured that all bolt holes would match. Templating has been effectively used by the Japanese module fabricators to ensure proper fit between adjacent modules and thus facilitate connection. Elastic deflections in the assembly position must also be considered.

Proper survey control procedures must also be set up for erection of space frames, of which the steel jacket is the most common example. Distances are large, 50–100 m or more, and points are high in the air and hence of limited accessibility. The sun's heat may cause significant elongation of upper members during the afternoon, while lower members on the ground are partially restrained by friction, and may also see lesser temperature rise due to shade. Deadweight deflections may account for even greater distortions, since the jacket is usually fabricated in a different attitude from that in which it is installed. Diagonal measurements often provide the best check.

When structures are afloat, there is always difficulty in establishing reference lines, especially the vertical lines. Lasers can be rigidly mounted at the base, accurately set normal to the base. They can then project a normal line, called "vertical," even if the structure is slightly listed due to ballasting or deadweight.

Even a relatively rigid structure such as a concrete gravity-base structure undergoes significant deflections during construction due to deadweight and ballasting eccentricities. Thus the shafts may deflect outward during ballasting to mount the deck.

Survey methods for final site location are discussed in Section 6.7. The key point of this section is to emphasize the role that survey control plays in the planning of construction.

21.12 Quality Control and Assurance

The establishment of a QC manual and a quality assurance (QA) program is an essential aspect of constructibility. The first task is to set up what the requirements are. They, of course, include those specified by the designer. If the designer has issued only general requirements, such as "compliance with the Specifications," it is necessary to determine which elements of that specification are applicable and, further, which will be determined or measured in the construction process. To these, constructors must add those requirements necessary to enable them to carry out their work in accordance with the materials selected and procedures adopted—for example, temperature and humidity control for painting; moisture control for above-water embankments, and early strength for concrete.

In establishing these lists, every effort should be made to reduce the number to the bare minimum. The nuclear reactor syndrome that "everything that can be measured must be measured" must be avoided. Paperwork must not become more important than the structure. The QA program should then provide for the identification and recording of

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the critical items which may be important for future reference. QA should not be used as a whip to ensure that the inspectors are doing their job.

Where defects can be immediately corrected, they should be.

Agreement should be reached before construction starts regarding what records are to be recorded and which data (e.g., radiographs) are to be kept. Those which are so kept must be properly identified and stored. Only that which is essential for the proper performance of the structure should be tested and inspected. Only those tests should be made which are necessary to ensure maintenance of quality on a statistically defensible basis. The reason for the above exhortations on limiting inspection and tests is that experience has shown that usually far more data has been collected than can be reduced, evaluated, and used, and that in the process, insufficient emphasis is given to the key properties which are truly important for performance.

Examples of mistaken programs are the taking and testing of an excessive number of cylinders for concrete compressive strength (where fewer cylinders supplemented by hammer testing might be more appropriate) and excessive reliance on x-ray for welding under conditions where cold lap may be the more likely defect.

21.13 Safety

The engineering of a safety plan for the large offshore project requires careful job-specific study by the construction and engineering personnel responsible for executing the project. They should develop a manual to apply to their project in which various safety risks are identified and appropriate preventive or mitigating measures adopted. Of the many safety precautions, procedures, and equipment required by various regulatory agencies, which are important to implement on this job? Which are irrelevant or non-applicable? Which may even be detrimental and hence require a special exemption?

A general law of one of the Australian states required that all man-hoists be powered down. This was written for building work on shore. However, its mistaken application offshore can be very dangerous, since in the transfer of workers by cage or Billy Pugh net, the ability to throw the clutch out of gear and to freely overhaul is essential for safe transfer between heaving vessels and platforms.

Is additional scaffolding required? Will lifelines and snap-on belts be required on high work? Should safety nets be provided, and if so, is their purpose to save a worker who falls or the protection of those working underneath or both? The design of the net and its supports should then be appropriate to the purpose.

What provision is made for workers to stand aside while loads of steel or concrete are being lowered? Will walkways or recesses be usable at such times?

Red or yellow plastic caps on projecting reinforcing bars will prevent deep scratches, protect eyes, and prevent puncture wounds. They should be attached before the bars are delivered.

What about a worker overboard? Arctic waters are at $-2°C$; a human can live only a few minutes in such water. At low temperatures, boats will not always start instantly. Should engines be left running and the boat be manned at all times? A continuously manned and operated lifeboat is now required around the platforms being constructed afloat in Norwegian waters.

Even if a person can swim, and has a life jacket on, what does the person grab hold of in a choppy sea and strong surface current? Fiber lines floating in the water can be trailed out from the structure. They should be well marked by buoys to prevent fouling of *boats'* propellers. Are there searchlights to illuminate the person in the water?

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Fire is the scourge of the sea, and especially so in the Arctic and sub-Arctic when piping and valves become frozen and intakes clogged with frazil ice. What secondary means are available for fighting fire?

If a worker is injured, what means are available for evacuating the worker from a congested location inside the structure to a shoreside hospital? As the structure nears completion, there will probably be excellent facilities, but in the early stages, temporary means must be planned. What about diving and the provision of decompression tanks? Finally, regarding training: major emergencies such as fire, collision, explosion, or imminent overturning require the coordinated action of several hundred workers, many of whom are not offshore-oriented and offshore-trained. Evacuation may necessary under conditions of darkness, wet decks, loss of power, high winds, and a stormy sea. The diverse groups of workers aboard need to be organized into crews, and the crews need instruction and rehearsal. This matter is especially difficult where numerous specialist subcontractor personnel (such as x-ray technicians) are on board on a temporary basis and hence unfamiliar with the organization of the vessel or platform. The lessons from Piper Alpha need to be carried into planning for emergencies.

21.14 Control of Construction: Feedback and Modification

An offshore structure is a major undertaking on two fronts, because of (1) the effect of the size, complexity, and interdisciplinary aspects, and (2) the dynamic movement, transport, launching, upending, and submergence that must be carried out on a grand scale, often involving over half a million tn. and a structure the size of our largest high-rise buildings.

Construction management will have carefully planned each operation. Now as the work goes on, how is the success or lack of it monitored? What warning signals will be sent, and how will they be recognized in time for corrective action?

Referring first to the productivity of fabrication and erection, careful monitoring can be carried out on the basis of schedule, unit costs or percentage of completion, man-hour or crew-day requirements, all compared with budgeted costs and time. It is not enough to try to control by flagging exceptions; the 10% overrun or underrun may apply to an insignificant item or one which will soon be completed and hence beyond timely correction. Rather, the major components of the work need to be identified: schedules and budgets assigned, with consideration of the learning curve, and the special conditions. These key items are then closely monitored, usually on a crew-day basis.

Constructibility planning must, of course, include an interface with the critical path scheduling. The critical path method (CPM) is a valuable technique for evaluating and controlling the various operations. The growing use of microcomputers in the field enhances the ability to identify critical elements of progress early, allowing appropriate action to be taken.

Critical path schedules are, of course, constantly updated. While most attention goes quite naturally to the items that lag, consideration must also be given to the opportunities that present themselves when work goes faster than scheduled. The Statfjord C platform was ahead of schedule on several early items; others were then accelerated to enable the completed structure to be placed on station several months ahead of target.

The second type of construction control relates to technically critical operations. What early indications will there be if serious engineering problems are imminent? Prior study

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has to be given to this matter for each critical operation. The instrumentation can be installed and observation schedule and procedures established to ensure that timely warning is received.

Examples of early feedback are unexplained discrepancies between weight control, ballast control, and observed draft. Another example is a trim or list that is inexplicable or beyond predictions. Rupture of erection bolts may indicate excessive built-in stresses. Cracking of welds may be due to poor welding or to excessive stress. Residual stress has been identified as a primary factor in the cracking of welds in buildings which have been subject to earthquake. Cracks in reinforced concrete may be local and caused by shrinking or may indicate major internal delamination.

In the upending process, is the attitude matching that predicted on the basis of ballasting calculations? If not, watertight closures may have ruptured, valves may be stuck open, or conductors or piles that were pre-loaded may have broken loose. From detailed consideration of each major observation, the needed data, their timeliness, and their relevance can be determined.

Experience on major projects onshore and offshore where serious accidents have occurred has shown in hindsight that warning phenomena had often been observed but had been disregarded because of assumptions that the engineering and construction control was infallible.

21.15 Contingency Planning

"Murphy's law" postulates that "what can go wrong, will go wrong, and at the worst possible time." For each detailed planning phase, a list of credible potential accidents and errors needs to be listed, including especially those due to human error. Human errors become more likely and more serious under the adverse conditions under which personnel must work. Each of these potential accidents and errors is then examined in detail. What can be done to prevent them? The preventive step may be physical (structural or mechanical), or it may be the assignment of a specially trained worker, a training or rehearsal program, or the provision of backup equipment.

Examples are numerous. To offset a stripped valve stem or jammed gate, valve position indicators may be installed, with a remote readout at the control station, to verify that the valve really is open or closed. Valves may be arranged in series, with a space between, to provide a backup in case some foreign object gets in. External screens may be provided over intakes. To prevent snagging and ripping off from a boat line, guards may be installed over the screens.

The above series of steps is now standard in the Norwegian North Sea, ever since a wire line got sucked into the Frigg ballasting line and kept two valves in series from closing. Fortunately, this occurred near the end of the installation and did not result in serious damage to the structure, but it could have been catastrophic.

We learn primarily from past mistakes, so the advice of experienced personnel is invaluable in preparing and reviewing contingency lists. However, an initial list can be prepared even by a less-experienced engineer, by extrapolating from more conventional problems on land and in surface vessels and by addressing imagination to the situation. Some of the contingent accidents cannot readily be prevented. For these, backup equipment should be provided. Special care must be taken to prevent progressive collapse—for example, when a crane boom collapses and falls across the falsework, which allows structural elements to drop, holing the bottom and subsequent flooding then overloads a bulkhead, leading to loss of buoyancy and sinking. Some

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contingent events will be judged so serious that they require a major change in construction procedure, even at an increase in cost or time.

21.16 Manuals

From the previous sections of this chapter, manuals are now prepared covering each major stage and each important or critical component of the construction process. A list of subjects includes the following:

1. For a steel jacket-pin pile structure:
   a. Welding procedures
   b. Node fabrication
   c. Erection of jacket legs
   d. Survey control
   e. J-tube installation
   f. Load-out
   g. Towing to site
   h. Launching
   i. Upending
   j. Positioning and landing
   k. Pile installation
   l. Grouting of piles to sleeves
   m. Conductor installation
   n. Deck girder erection
   o. Module erection
   p. Scour protection
   q. Riser installation
   r. Instrumentation
   s. Salvage and removal
2. For a typical concrete offshore platform:
   a. Skirt installation
   b. Base raft construction
   c. Air cushion
   d. Dock flooding
   e. Float-out
   f. Mooring at deep-water site
   g. Construction afloat
   h. Ballast control
   i. Weight control
   j. Geometry control
   k. Towing to mating site

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l. Mooring at deck-mating site
m. Deck supports (for deck fabrication)
n. Deck girder erection
o. Module erection
p. Deck load-out
q. Deck transport
r. Emergency mooring of deck
s. Deck mating
t. Deck outfitting
u. Inclining test
v. Towing to site
w. Installation at site
x. Penetration phase
y. Underbase grouting
z. Scour protection
aa. Conductor installation
- Riser pull-in
- Instrumentation

bb. Salvage and removal

Obviously, not every structure needs all the above manuals. Many of the items listed may be small enough in scope that they can be combined. As with the earlier division into stages, the important thing is not to overlook or gloss over a small item, for in accordance with a corollary to Murphy's law, this will turn out to be the critical one.

The preparation of each of these manuals requires the participation of all involved parties, including contractors and subcontractors, and all disciplines. Thus it turns out to be an effective means of communication and of making each group aware of the others' needs and concerns at that stage. A draft of the manual is then circulated to management, design engineering, field construction supervisors, consultants, key subcontractors, and insurance surveyors. They are asked to review in detail and comment.

Not only do constructive suggestions for improvement arise, but this review makes each party even more fully aware of the operation and enables each to focus on critical aspects:

1. The first section in each manual defines the scope of work to be covered and lists the other manuals which interface.
2. The next section includes the relevant drawings and specifications.
3. A few specially prepared summary drawings are included, relevant to the work covered in that manual.
4. Sources of material, as it will arrive, are identified.
5. Equipment available is identified.
6. Relevant weather and sea data are set forth.
7. The many substages of procedure are listed, with sketches of each such substage followed by calculated weights, ballast quantities, draft, and

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freeboard, as may be applicable. Important tolerances are listed. QC requirements are set forth. The survey and measurement program is described, along with acceptable tolerances and corrective methods.

8. QC requirements are set forth.
9. The survey and measurement program is described, along with acceptable tolerances and corrective methods.
10. Special safety requirements for each stage are set forth.
11. A contingency plan is attached.

Each of the previous sections of this chapter form the basis for a summary section in the manual. It is important that these manuals be issued in time for adequate review and revision if needed. Similarly, it is important for the reviewers to do their work promptly, allowing time for needed revision and recirculation.

21.17 On-Site Instruction Sheets

While during construction applicable drawings from the design and those showing temporary construction will be in the construction site office, as will the manuals, these are hardly suitable for use out on the platform structure. Complementary sets of construction drawings are therefore prepared, using the above documents as a source. One set is prepared for each substage or major operation. Unlike design drawings, these drawings show only those elements which are essential for that construction phase.

Isometric drawings may be used for certain steps. The tolerances applicable to each step are clearly shown. Critical requirements from the specifications and instructions are noted with arrows pointing to the affected location. Each step is shown on a single drawing. Serious errors have arisen when two or more steps are combined "to save paper."

Auxiliary gear and equipment are listed: slings, guides, tag lines, jacks, and so on. A bill of all auxiliary gear is tabulated so that availability can be checked. This bill includes safety equipment. These drawings are then issued on waterproof paper for actual use in the field by the construction personnel. These drawings are especially useful in the rigging, lifting, and launching operations. For these, successive drawings can show the different positions of the load, the booms, and the lines as the load or structure is transferred from one location to the next.

In the walls of a concrete offshore platform, multiple embedments are required—internally, to support utility shaft decks, conductor guides, piping, hangars, and penetrations and, externally, to provide for riser attachments and anodes. To ensure proper location of these during slip-forming, one contractor has prepared charts giving the embedment requirements and their locations for each "slice" of each shaft or cell. The slices are 1 m high; thus over 100 such drawings are required. In addition to the embedments, the prestressing duct requirements, reinforcing steel and mechanical installations are also detailed for each slice.

A new articulated pipeline "stinger" costing several million dollars was being connected to the stern of a pipe-laying barge in the Bass Strait, Australia. The connection was detailed with 60 high-strength bolts. These bolts were, of course, shown on the drawing, and the specifications for the bolts themselves were in an accompanying manual. Far down in the manual was a note on how to torque the bolts, but this section never reached the field superintendent or crew. Having no instructions, they did not

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torque the bolts at all. The stinger was attached, work started, and the connection failed in fatigue within 2 days, dropping the stinger in a crumpled heap and buckling the pipeline.

To avoid similar errors in the future, assembly drawings were sent to the field with the torquing instructions and other critical requirements clearly noted right on the drawings, not buried in an accompanying specification.

21.18 Risk and Reliability Evaluation

Risks associated with the various construction stages and procedures can be identified and a qualitative evaluation, at least, made of their reliability and safety involved. The word qualitative seems appropriate, even where some effort is made at quantification, because each operation has many unique aspects and because the database is generally inadequate.

Risks which have been identified on previous structures include:

1. Delay in materials, fabrication, hookup, testing, and approvals
2. Excess hydrostatic heads acting on compartments or through piping, ducts, etc.
3. Loss of compressed air pressurization
4. Flooding due to external damage, piping failure, valve failure, plug or bulkhead rupture
5. Overtopping due to waves
6. Free-surface water from spray, rain, leaking manholes
7. Structural cracking due to differential settlement or ballasting errors
8. Mooring line failure during storm
9. Anchor dragging
10. Fire and explosion
11. Storms—wind, waves, and high currents
12. Dynamic amplification of motion
13. Acceleration forces on deck equipment
14. Failure of tie-downs
15. Shifting of load
16. Tug breakdown
17. Broken towline
18. Ice jamming of towline
19. Ice jamming under and around structure
20. Excessive yaw and sway during tow
21. Excessive roll during tow
22. Grounding
23. Tug stopped—structure overruns tug
24. Loss of stability during final placement
25. Lateral “skidding” due to trapped water underneath base
26. Loss of reference markers

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27. Malfunction of instrumentation
28. Seafloor irregularities, hard spots, boulders, etc., previously unidentified
29. Excessively stiff soil or hard layers, e.g., ash
30. Excessively soft soils or low friction soils, e.g., calcareous or micaceous soils
31. Storm or fog during installation
32. Piles failing to develop resistance
33. Piles showing excessive resistance above design tip elevation
34. Excessive scour during installation
35. Inability to break suction effect during removal
36. Launched structure failing to float at proper draft or proper attitude in list or trim
37. Structural damage on launching
38. Lines fouled on projecting fittings
39. Failure of mooring line attachment due to fatigue from current-induced vibrations
40. Rupture and sinking of spring buoy
41. Drag of anchor
42. Loss of stability as vessel is unloaded
43. Errors or omissions in design and construction of local details, such as a lack of adequate through thickness reinforcement

The above list is obviously incomplete and includes both major and minor items. One of the frustrating aspects of offshore construction is that minor accidents or incidents, when they occur, often do so in groups and combine to create major problems.

The above list does not directly address human error, which is involved in many of the above risks, especially the last item on the list: errors and omissions during design and construction. Increasing attention is being given to engineering approaches that minimize the potential for human error—it extends to such obvious aspects as instrumentation read-out, to clearly indicate exceedance of safe values and/or safe rates of change. It, of course, also includes training and simulation. Special attention is given to redundancy to prevent mistakes from cumulating in progressive collapse. Most catastrophic accidents are the results of a chain of small events, occurring in a sequence that in retrospect seems to have been planned by some evil genius, since one break in the chain would have prevented the event.

The history of marine structures, both inland and offshore, is replete with catastrophes, commencing with Darius' loss of his floating bridge across the Bosporus in 400 BPE. The most recent such accident at the time of writing this book is the uncontrolled sinking of a 176-m-long precast concrete tunnel segment weighing 55,000 tn., during its installation in the Øresund Tunnel between Denmark and Sweden. Investigation showed that the reinforced concrete bolster girder failed, allowing premature flooding. Apparently, the drawings and work descriptions were not followed. Crucial reinforcing steel was missing. Fortunately, the sunken tunnel segment was not damaged, so that it was able to be salvaged and reinstalled. This was one of the last segments to be installed. Experience from a wide variety of structures involving multiple repetitive operations shows that a failure due to human error often occurs on one of the last elements to be installed due to carelessness. The sinking of the Baldpate jacket in 650 m of water due to malfunctioning valves is described in Section 22.5.3.

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Risk needs to be considered in terms of consequences. Even a low probability risk needs thorough consideration if the consequences are extreme, such as loss of the platform.

When the initial Seatank underwater storage tank was installed as a demonstration project in the Bay of Biscay in 1970, it failed catastrophically due to a long series of events:

1. As the structure was submerged just below the surface, the hydrodynamic "beach" effect of the waves caused it to "hang up," requiring additional ballasting.
2. This delayed the operation, while the weather worsened.
3. The tugs had difficulty positioning themselves and the buoys.
4. The added ballast to compensate for item 1 led to "plunging" later, i.e., rapid sinking when the structure was more deeply submerged.
5. This caused the boat lines to foul on manholes, ripping at least one open.
6. The radial lines leading to the buoys controlling submergence became entangled due to the waves.
7. The combination of items caused the structure to plunge to greater depths before sufficient compressed air could be injected for internal pressure compensation.
8. The structure imploded.

A similar sequence of multiple causes led to the original Sleipner failure (this list is the author's evaluation):

1. The owner-operator intentionally generated intensive competition between concrete and steel platform crews.
2. The concrete constructor cut his costs to the bone, both in design and estimating.
3. One item he eliminated was the third-party check, normally required by the Norwegian Petroleum Directorate but given special dispensation to allow in-house check due to the past history of 20 successful structures of similar (but not identical) design.
4. The design did not adequately address the progressive collapse limit state. There was no redundancy.
5. A two-dimensional finite element analysis was carried out. The mesh was incorrectly set to span over the critical apex of interior star cells.
6. The in-house check merely ran the same computer program over again.
7. The design showed T-headed bars, crossing each apex of the interior star cells and anchored behind the outer reinforcing bars.
8. To save time in placing the bars in this congested area, the field shortened the bars to anchor at the middle of the two walls. This apparently was not submitted to the design engineer to approve before implementation.
9. During test submergence before deck mounting, a shear failure occurred at one apex, followed by another as the compressive ring ruptured.
10. The structure sank rapidly, imploding as it descended past the hydrostatic capacity of the cells.
11. All 12 personnel escaped safely.
12. The contractor successfully rebuilt the concrete offshore platform, using additional reinforcing.

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The subsequent successful installation of several large concrete structures has shown that preventive measures can be taken and that they can prove fully adequate. They include limiting reliance on compressed air and augmentation of the safety factor, with the structure being designed in the ultimate state to withstand external hydrostatic heads even after the loss of air. Critical zones are being identified and properly reinforced. Positioning tugs are fitted with bow thrusters to enable them to position themselves in wind and waves. Manholes are recessed and fittings are protected by guards against snagging by towlines. Columns, shafts, or temporary tanks are installed to prevent sudden loss of stability and draft control during submergence below water.

Similar scenarios could be written about the catastrophic loss of steel structures during launching and installation. For the Magnus platform in the North Sea, the steel tubular pin piles were pre-installed in the jacket legs. As the jacket was up-ended, the piles broke loose and plunged through the end plates, hitting the seafloor so hard as to buckle and bend the piles. Replacement required several precious summer months.

Pipelines have been buckled due to slipped anchors, or crushed by an anchor from the laying vessel. Pulled pipelines have been twisted and buckled due to loss of weight coating during the pull.

Marine and Offshore Structures are subject to potential failures and even collapse due to the environmental forces and phenomena in which they must serve. It is the responsibility of both the Designers and the Constructor is to ensure that local failures do not occur and that if they do, they do not propagate to progressive collapse.

The Constructor's concerns in this regard include:

1. Lack of compliance with specifications and standards.
2. Failure to comply approved with QC Manual.
3. Exceedance of tolerances
4. Substandard welds, improper installation of HS bolts or other connection details.
5. Unauthorized or unapproved changes.
6. Failure to recognize early signs of distress and to act promptly.

Other areas of oversight by constructors in the past have included:

- Failure to meet all specifications, including those in referenced documents
- Failure to comply with approval QC/assurance manual
- Lack of implementation of safety program
- Failure to have temporary structures properly designed to meet conditions under which they will perform which may well differ from final in-service condition
- Unauthorized change of details

Careful evaluation of risks and reliability is essential to the selection of the appropriate method. Frequently, the results can be very positive; a procedure which appears excessively dangerous, such as launching a 30,000-tn. jacket the size of a highrise building, mounting a complex deck weighing 20,000 tn. on a pre-installed jacket in the North Sea, or mating an articulated loading column with its base while both are floating at an inclination, can, with thorough engineering, be made into a sound and reliable undertaking. Conversely, a relatively "simple" operation such as setting a module on the deck of a platform may be excessively hazardous if it is treated superficially and carelessly—if, for example, inadequate attention is given to padeyes and sling leg orientation. Fault-free

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analyses are valuable in preventing progressive collapse, the most catastrophic and most dreaded sequence of failure scenarios.

Risk and reliability evaluation is obviously closely related to contingency planning. The latter, however, is intended to establish specific procedures to prevent or mitigate risks after the overall plan has been established. Conversely, risk and reliability evaluation is intended to serve as a broad guide and overview to ensure that sub-optimization techniques have not led to adoption of excessively risky procedures and that areas of high risk will be re-investigated to reduce their probability of occurrence and mitigate the consequences.

The architects of these clipper ships were like poets who transmute nature's message into song, obeying what wind and wave had taught them, to create the noblest of all sailing vessels and the most beautiful creations of man, a perfect balance of spars and sails to the curving lines of the black hull, and this harmony of mass, form, and color was practiced to the music of dancing waves and of brave winds whistling in the rigging. For a few brief years they flashed their splendor around the world, then disappeared.

Samuel Eliot Morison, The Oxford History of the American People

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22

Construction in the Deep Sea

22.1 General

The deep sea is one of the newest and most exciting frontiers of the offshore construction industry. The discovery of giant fields for oil and gas in deep water has presented a major challenge to the industry, resulting in remarkable developments in the way of equipment, procedures, instrumentation, and remote operations.

What constitutes the deep sea? When international agreement was reached on national jurisdiction, 200 m was considered the limit beyond which development of resources would be prohibitively costly and beyond the capabilities of technology. When the first edition of this book was written in 1986, the demarcation had risen and the term "Deep Sea" was being applied to platforms and pipelines in 500 m. By the year 2000, the second edition was able to report that bottom-supported platforms were being constructed in over 500 m water depth, and subsea operations, supported by floating production, storage and offloading (FPSO) spars, Tension-Leg Platforms (TLPs), and drilling vessels, were in place in 1600 m. ROVs capable of 2500 m and even 3000 m were then available. Drill ships were being constructed to work in 3000 m, which is almost the maximum depth of the Gulf of Mexico. Currently, the previous limits are being exceeded on all fronts. Deep water operations are now being carried out in the Gulf of Mexico, West Africa, Brazil, and indeed worldwide.

As of 2005, the deepest structures of the several types were:

a. Tension leg semi-submersible, Magnolia, in 1425 m
b. Fixed Jacket-Pin Pile, Bullwinkle, 412 m
c. Compliant Piled Towers, Petronius, in 534 m
d. Spars, 1710 m. Devil's Tower. Red Hawk is in 1620 m, Constitution in 1515 m.
e. Semi-submersibles, Independence Head (moored with slightly inclined legs), 2438 m. Thunder Horse is in 1844 m.
f. Blind Faith will be in 2100 m depth.

Pipelines, flow lines, and risers have been installed to subsea completions such as the Mensa project (1700 m) and for the Exxon Spar project in 1600 m. Discoveries have been made in depths of 2000–2400 m.

Military activities, such as the deployment of acoustic sensors for recovery of armament and equipment, including retrieval of parts of a Soviet submarine, have been carried out at depths up to 6000 m. Test facilities for ocean thermal energy conversion (OTEC) have

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included pipelines suspended to a depth of 600 m. Tests of manganese nodule mining equipment have been conducted at a depth of 2000 m.

The deep-sea frontier is rapidly emerging as an area of great potential. Exploratory drilling for oil and gas has already been carried out at depths of over 3000 m. The Deep Sea Drilling Project included the successful drilling and reentry of a hole at a depth of 6000 m. Potential exploitation of the polysulfide mineral deposits from midocean rifts will require specialized dredging operations, with equipment and materials capable of operating in hot brine. Manganese nodules are concentrated on plateaus and basins lying at 2000–4000 m depth, requiring efficient dredging systems capable of operating at such depths. OTEC systems are generally based on the utilization of the cold water from 1000 m depth. The floating structures for this concept may require mooring in 4000 m. The deployment of sensor devices with cable moorings and of large surface and subsurface buoys has been carried out throughout almost the entire range of ocean depths.

Cables and pipelines have been installed for crossings of straits where the water depths range from 300 to 600 m. A bridge across the Strait of Gibraltar, for which the feasibility studies have been recently completed, will require piers in from 300 to 500 m of depth. In 2001, a gas line was laid across this same route. Studies and tests have been carried out by the U.S. Navy for mass concrete placements in the deep ocean. Scientific exploration, such as that proposed to identify the existence of neutrinos, will require extensive deployment of cables, sensors, and moorings at extreme water depths. The Deep Undersea Muon and Neutrino Detection (DUMAND) Project will involve the placement of an array of sensors $250 \times 250 \times 500$ m in plan at a depth of 4500 m.

The Offshore Oil Industry has formed a study group to identify the problems of operations in Ultra Deep Water, defined as 1500–3000 m. Not only does the hydrostatic pressure double as the depth increases from 1500 to 3000 m, but there are increased problems from cold water and riser fatigue. Abandonment and recovery operations have been carried out in water depths of 500 m.

For the purposes of this chapter, the deep sea will be defined as those depths at which manned intervention appears to be no longer economically practicable (i.e., 500 m and over) and where hydrostatic pressures dominate design and construction, so that specialized equipment, systems, and procedures become necessary.

22.2 Considerations and Phenomena for Deep-Sea Operations

Depth effects which are of concern to the constructor include the following:

1. Extreme hydrostatic pressures
2. Density changes in liquids, including seawater, due to high pressure and low temperature
3. Reduction in volume of solids due to bulk modulus effects (usually important only for low-modulus materials such as polyurethane foam)
4. Absorption of water into concrete and other solids
5. Absorption of gases into solids
6. Miscibility of water and other fluids
7. Change in strength of materials due to high triaxial stress states

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8. Density and other currents—at depths of 1000 m the currents may be of the following order: density currents: 0.2–0.5 kt.; internal wave-generated currents: up to 0.6 kt.; tsunami currents: up to 0.6 kt. Currents may produce vortex shedding resulting in dynamic responses in long risers and strumming vibrations in long cables
9. Internal waves
10. Density layers (stratification) of the water column
11. Leaks in seals of hydraulic systems, electrical connectors, etc., due to high pressure
12. Difficulty of control as a result of time lag in response of hydraulic systems due to the long length of lines. For this reason, deep sea well control devices use electro-hydraulic operation
13. Static and dynamic strains (stretch) in cables, casing, rods, etc., due to long length;
14. Buoyant weight of mooring lines and risers
15. Remote sensing and control requirements for positioning, orientation, guidance, etc.
16. Interaction of pressure and temperature on highly compressed gases, with rise in temperature when pressurized and sudden drop (even below freezing) when pressure is released
17. The seafloor, usually level and smooth in the coastal plane, has steep slopes and rugged topography. Landslides, mud flows, hydrate accumulations, active faults, salt domes, seafloor erosion, and chemo-synthetic communities abound in the deep water of the Gulf of Mexico. Pipeline routing becomes of major importance.

22.3 Techniques for Deep-Sea Construction

The constructor has available a number of techniques for meeting the special needs of the deep sea:

1. Deep-sea ROVs capable of carrying out investigations and survey, and performing tasks with manipulators adapted to specific tasks. Autonomous ROVs are increasingly used.
2. Electronic and acoustic sensing devices that enable accurate measurement and control of orientation and positions, both true and relative. These include gyros, Differential Global Positioning System (DGPS), inertial guidance, photographic and acoustic imaging, video, and sonic devices, this latter including side-scan sonar. For locating and exploring the remains of the Titanic, ultra-high-resolution photography and strobe lights were employed, based on technology originally developed for space missions. Photography can reveal features on the seafloor which have been missed by acoustic imaging.
3. Many devices that can be effectively deployed from ROVs or work submersibles using fiber optics for transmission of information. They can also be deployed on the structure itself. A number of these were successfully employed on the Cognac platform during its installation. Suitable systems have been field-proven

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in deep exploratory drilling operations, including reentry, by the Glomar Challenger at 20,000 ft. (6000 m).

4. Sparker and geophysical methods can be used to reveal anomalies and strata below the mud line.
5. Dynamic positioning, both at the surface of the sea, where propeller-driven thrusters are usually employed, and at depth, where jet thrusters are more applicable. These can be computer-controlled to maintain positions as determined by input data from satellites on the surface, inertial guidance and acoustic transponders in the sea and on the seafloor.
6. Use of de-aerated seawater as a hydraulic fluid.
7. Use of low-density fluids which still possess low compressibility and hence permit balancing of fluid pressures. These include gasolines, pentane, propane, oil, and solvents. Several solvents are available which are safe to handle, have minimal miscibility with water, and have specific gravities in the range of 0.55–0.60. After use, these solvents can be displaced by seawater and recovered to a tanker (Proceedings, Offshore Technology Conference, OTC, 8670, 1998).
8. Syntactic foams (closed-cell), possessing low density but capable of resisting hydrostatic pressures up to 6000 m and more.
9. High-density materials for weight control. These include barite-weighted drilling mud and iron ore slurry.
10. Development of near-neutral buoyancy materials of high strength and stiffness for mooring lines, such as polyester Kevlar and carbon fiber.
11. Use of drill casing and drill string for lowering of heavy objects. These can then also be used for transmission of fluids. A casing maintained empty may partially offset its deadweight by buoyancy.
12. Use of steel tubular pipe for vertical moorings of Tension-Leg Platforms (TLP).
13. Development of supporting techniques such as arc-flame cutting. Recent studies have been made of the effectiveness of underwater arcs and flames at great depths. Underwater flames involve an arc to ignite the preheating flames, the use of premixed flames fueled by hydrogen or methane, and an oxygen jet to burn away the preheated material. Underwater flames seem to have no inherent depth limitations and in fact may perform even better in deep water than in shallow water. Underwater arcs are more adversely affected by the pressure, water chemistry and heat sink. It appears possible to strike and maintain an arc, but further research and development will be needed to ensure efficient operations at great depths.
14. Development of self-contained power sources such as nickel–hydrogen, silver–hydrogen, and lithium thionyl chloride batteries, which are pressure-compensated.
15. An abrasive water jet cutting system to cut pipe or piles in deep water.
16. Suction anchors.
17. Lowering objects such as suction anchors and templates to the sea floor must consider the dynamics that occur due to the heave of the lowering vessel at the surface. This can set up resonance in the long lowering line, resulting in increasingly high peak forces. Intramoor has developed a system in which a large steel tank is placed in the lowering system. This tank is pressurized by nitrogen gas to a pressure determined by a computer program which takes into account the mass of

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the object, the mass of the lowering vessel, the sea state, the characteristics of the lowering, line and the depth of water.

22.4 Properties of Materials for the Deep Sea

The high hydrostatic pressures experienced with increasing depth cause the densities and other properties of many materials, such as fluids, to change from those associated with more normal, near-surface operations. In this new environment, these properties become important for construction operations. Table 22.1 gives the properties of seawater at various depths.

Propane, crude oil, gasoline, diesel oil, and solvents are fluids possessing buoyancy in seawater, and thus are capable of reducing the effective weight of large structures during installation. Their properties at various depths are given in Table 22.2 through Table 22.6. Attention is directed to the fact that these fluids have different degrees of miscibility with seawater. Table 22.7 gives the properties of syntactic foam. Both "syntectic" and "syntactic" are used in the technical literature, but syntactic is adopted in this book.

Heavy fluids and granulated or slurried solids are often required in order to ballast structures down against the buoyancy of the seawater. Table 22.8 gives the properties of weighted drilling muds, while Table 22.9 gives the properties of bulk solids.

To lower and position deep sea structures, wire rope, chain, drill pipe and drill casing have been used. The properties of these are listed in Table 22.10 through Table 22.12. Carbon fiber strands, as well as Kevlar rope, are commercially available.

Fiber ropes offer the advantage of relatively high strength with near-neutral buoyancy. Properties of several of the more commonly used rope materials are listed in Table 22.13. Kevlar (aramid fiber rope) offers the strength of steel wire line at only one fifth of the weight in air and only one-tenth of the weight in water. Carbon fiber is even stronger but is considerably more costly. Mooring and hoisting lines are available up to 1,200,000 lb (550 tn.) breaking strength. Hence, they are especially well suited for

TABLE 22.1

Properties of Seawater at Various Depths

		Water			
Depth	**Specific Gravity**	**Unit Weight**	**Temperature**	**Pressure**	
English System					
(ft.)		(lb/ft.^{3})	(°F)	(ksf)	(psi)
3,000	1.030	64.25	37	193	1340
6,000	1.034	64.5	34	386	2680
9,000	1.038	64.75	34	581	4040
12,000	1.042	65.0	33	776	5400
SI System					
(m)		(kN/m^3)	(°C)	(MN/m^2)	(MPa)
1,000	1.030	10.09	3	10.09	10.09
2,000	1.035	10.13	1	20.32	20.32
3,000	1.039	10.18	1	30.50	30.50
4,000	1.043	10.22	1	40.72	40.72

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TABLE 22.2

Properties of Propane at Various Depths

		Water		
Depth	**Temperature**	**Pressure**	**Unit Volume**	**Density**
English System				
(ft.)	(°F)	(psi)	($ft.^3/lb$)	($lb/ft.^3$)
0	60	100	0.0320	31.0
1,000	43	444	0.0306	32.7
2,000	39	888	0.0300	33.3
3,000	37	1332	0.0298	33.5
4,000	35	1776	0.0296	33.8
5,000	34	2220	0.0294	34.0
6,000	33	2664	0.0291	34.4
SI System				
(m)	(°C)	(MPa)	(m^3/kN)	(kN/m^3)
0	16	0.71	0.206	4.860
500	5	5.20	0.195	5.130
1,000	3	10.40	0.192	5.220
1,500	2	15.60	0.190	5.260
2,000	1	20.80	0.189	5.300
3,000	1	31.20	0.186	5.400

Note: Propane is miscible with seawater.

TABLE 22.3

Properties of Crude Oil

	Density	
API Gravity	**$lb/ft.^3$**	**kN/m^3**
20°	58.2	9.14
35°	53	8.30
42°	50.8	7.96

TABLE 22.4

Properties of Gasoline

API gravity = 67.5° at 1 atm at 60°F
Density = 44.4 $lb/ft.^3$ = 7.0 kN/m^3
At 3000 ft. and 33°F (1°C)
Density = 46.1 $lb/ft.^3$ = 7.25 kN/m^3
Other grades show density at 3000 ft. of 47.7 $lb/ft.^3$ = 7.5 kN/m^3

TABLE 22.5

Properties of Diesel Oil

API gravity = 40°
Density = 51.4 $lb/ft.^3$ = 8.06 kN/m^3

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TABLE 22.6

Properties of Solvent Mixture: Heptane Plus Hexane

Heptane: 62° API vapor pressure at 100°F (38°C) = 1.6 psi (11 kPa)
Hexane: 75.2° API vapor pressure at 100°F (38°C) = 5 psi (36 kPa)
Mixing 60% hexane plus 40% heptane gives API 70° at 68°F (20°C); nominal internal pressure about 5 psi (36 kPa)
Assumed increase in density with depth and lower temperature is similar to that of gasoline—i.e., 4%, giving a unit weight of 45.5 lb/ft.^{3} at 3000 ft. or 7.45 kN/m^3 at 1000/m depth

Note: Heptane and Hexane have low miscibility with seawater.

TABLE 22.7

Properties of Syntactic Foams

Lightweight Syntactic Foam
Density = 35 lb/ft.^{3} = 5.5 kN/m^3
Strength to withstand 2000 ft. (600 m) of external hydrostatic head
High-Strength Syntactic Foam
Density—42 lb/ft.^{3} = 6.6 kN/m^3
Strength to withstand 20,000 ft. (6000 m) of external hydrostatic head
Syntactic Foam can be used with glass or aluminum spheres embedded.

TABLE 22.8

Properties of Weighted Drilling Muds

Density			Mix Proportions per Barrel of 42 gal			
(lb/gal)	lb/ft.^{3}	kN/m^3	Bentonite	Barite	Lignosulfonate	Caustic
16	120	18.9	8.5	13	4	1
18	135	21.2	8.5	523	4	1
20	150	23.6	8.5	634	4	1

For Heavier Densities, Add Other Finely Ground Materials Material	Specific Gravity
Barite	4.2–4.3
Galena	6.5–6.7
Iron oxide	4.9–5.3
Iron particles	7.8
Lead powder	11.4

Note: Most of the above require environmental consideration.

TABLE 22.9

Properties of Bulk Solids for Weighting and Ballasting Purposes

	Density						
	Solid Specific Gravity	Solid State		Bulk Weight in Air		Bulk Weight in Seawater	
Material		lb/ft.^{3}	kN/m^3	lb/ft.^{3}	kN/m^3	lb/ft.^{3}	kN/m^3
Siliceous or limestone sand	2.64	165	26	105–115	17–18	41–51	7–8
Iron sands (oxides or sulfides)[a]	4.8–5	300–312	48–49	195–220	31–35	131–156	21–25

[a] Iron sulfides may have corrosive effects on steel and concrete reinforcement.

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TABLE 22.10

Weight of Chain and Wire Rope

	Weight in Air		Weight in Seawater		Proof Test	
Size, (in.)	**lb/ft.**	**kN/m**	**lb/ft.**	**kN/m**	**lb**	**MN**
3¼ chain	105	1.53	91	1.34	804,000	3.65
4 chain	152	2.22	132	1.93	1,200,000	5.45
5 chain	232	3.40	202	2.96	—	—
6 chain	323	4.70	281	4.11	—	—
4 wire rope	29.6	0.43	25.8	0.38	—	—

TABLE 22.11

Weight of Drill Pipe

	Weight in Air		Weight in Seawater	
Size, (in.)	**lb/ft.**	**kN/m**	**lb/ft.**	**KN/m**
3H	15.50	0.23	13.5	0.20
4	15.70	0.23	13.7	0.20
4H	20	0.29	17.4	0.25
5	19.5	0.285	17.0	0.25
6L	31.9	0.465	27.8	0.405
8L	40	0.58	34.9	0.504

Drill Pipe Is Available in the Following Grades

	Yield Strength		Ultimate Strength		Ultimate Elongation
Grade	**kips/in.^{2}**	**MPa**	**kips/in.^{2}**	**MPa**	**(%)**
D	55	400	95	680	18
E	75	530	100	710	18
G	105	750	120	860	15
S	135	960	150	1070	—

TABLE 22.12

Properties of Drill Casing

Size Diameter	Weight in Air		Weight in Seawater (open ended)		Buoyant Weight of Closed Casing	
(in.)	**lb/ft.**	**kN/m**	**lb/ft.**	**kN/m**	**lb/ft.**	**kN/m**
6⅝	32	0.47	28	0.41	−16.6	−0.24
8⅝	49	0.72	43	0.63	−23.0	−0.34
10¾	55.5	0.81	48	0.70	−15.0	−0.22
13⅜	85	1.24	74	1.08	−22.5	−0.33
18⅝	96.5	1.41	84	1.23	+25.5	+0.37
24½	113	1.65	98	1.44	+97	+1.42

Note: Minus sign (−) indicates weight exceeds buoyant force; plus sign (+) indicates buoyant force exceeds weight. These net values assume air-filled casing. If the 24H in. casing is filled with a buoyant fluid such as heptane plus hexane, the net weight in water becomes slightly negative. Weight of pipe in air, −113 lb/ft.; weight of fluid in air, −135 lb/ft.; displacement, +210 lb/ft.; net weight in water, −38 lb/ft. = −0.5 kN/m.

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TABLE 22.13

Properties of Typical Fiber Ropes

Size		Nystron Braid[a] (Nylon/Polyester)			Nylon/Multifilament Polypropylene Rope[b]			Polyester Rope[c]			Kevlar[d]		
Diameter (in.)	Circumference (in.)	Weight per 100 ft. (lb)	Minimum Breaking Strength (lb)	Elastic Elongation at 25% of Breaking Strength (%)	Weight per 100 ft. (lb)	Minimum Breaking Strength (lb)	Elastic Elongation at 25% of Breaking Strength (%)	Weight per 100 ft. (lb)	Minimum Breaking Strength (lb)	Elastic Elongation at 25% of Breaking Strength (%)	Weight per 100 ft. (lb)	Minimum Breaking Strength (lb)	Elastic Elongation at 25% of Breaking Strength (%)
2	6	114	121,000	7	93	88,400	7	124	105,400	3	132	172,000	1
2¼	7										180	224,000	1
3	9	268	272,000	7	210	193,000	7	294	236,300	3	—	—	—
4	12	470	460,000	7	371	329,000	7	515	399,500	3	—	—	—
5	15	719	683,000	7	590	505,000	7	788	593,300	3	—	—	—
6	18	988	921,000	7	836	698,000	7	985	731,850	3	—	—	—
7	21	1,348	1,233,000	7	1,080	884,000	7	1,478	1,071,850	3	—	—	—

[a] E at 25% B.S. = 140,000 psi.
[b] This rope has neutral buoyance E at 25% B.S. = 100,000 psi.
[c] E at 25% B.S. = 280,000 psi.
[d] E at 25% B.S. = 1,400,000 psi.
Source: From R. Samson. 1977. Manual No. 2.77, 2nd Ed., Boston: Samson Ocean Systems, Inc.

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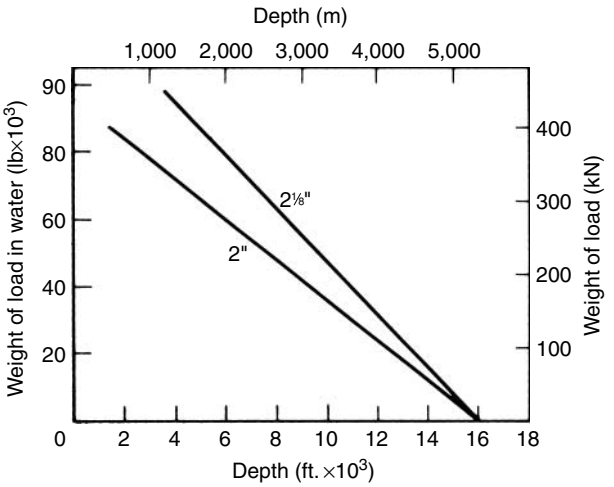

FIGURE 22.1
Load-carrying capacity for 6×41 fiber-care rope.

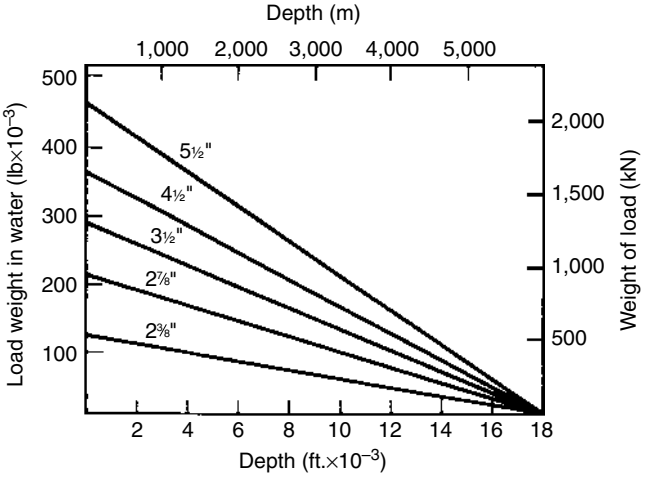

FIGURE 22.2
Load-carrying capacity of API drill pipe.

deep-water application. Figure 22.1 and Figure 22.2 give the effective load-carrying capacity of wire rope and drill pipe at various water depths.

22.5 Platforms in the Deep Sea: Compliant Structures

22.5.1 Description

At depths beyond 300 m, compliant structures are used. Their period is significantly longer than that of the design wave. These are structures which are intentionally flexible transversely, yet fixed to the seafloor for shear and axial support. Several concepts have been developed to achieve this response: the guyed tower, the free-standing flexible tower, and the articulated column (similar to that described in Section 13.4 but much larger and deeper). The towers themselves are typically trussed columns with a relatively constant cross section, although a large-diameter tube is also conceptually feasible. The base is

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either spudded or piled, in order to support the tower and provide restraint against lateral shear. Similar to the structures described in Section 13.4, the base may be set separately or attached to the tower during its installation.

22.5.2 Guyed Towers

Fabrication of the tower is carried out on a ways, as with any jacket. Since this structure is primarily for deep-water use, it will often be fabricated in two halves, just as was done for the Hondo platform (see Section 9.9). Preferably the two halves will be built as one, then later separated, to ensure perfect match.

However, if sufficient yard space is not available, a short section at the juncture, incorporating both mating sections, can be constructed first. Then this section is skidded to the far inshore end of the ways, separated, and the lower half fabricated in a normal manner. The mating section of the upper half can be skidded sideways onto a parallel launching ways, then down to the outboard end, and the upper half fabricated concurrently. There are obviously several variations of the above, depending on yard layout.

Each half is now transported to a protected deepwater site, as was done with Hondo, and then launched. Note that the launching can be either end-O, or sideways, since the cross section is uniform. Mating afloat can follow the methods previously described for the Hondo platform, using stabbing cones and watertight access tubes, enabling welding in the dry. Temporary buoyancy tanks ensure that the tower floats horizontally on its upper legs. A spud-can section is constructed at the lower end of the tower.

After mating and the welding of the mated legs to form an integral structure, the auxiliary buoyancy towers at the lower end of the tower are ballasted to slightly negative buoyancy. Now the structure, having a slightly inclined attitude, is towed to its installation site.

Ballast added to the spud can section and lower jacket legs causes the structure to upend. Bending moments in the tower need to be carefully computed for this operation and may require progressive ballasting of midlength compartments in the tower. The upper temporary buoyancy tanks are designed so that the structure will float vertically at the site. Further ballasting of the jacket legs and spud can cause the structure to touch bottom and then force the spud-can into the soil. By lowering the pressure inside the spud can, it maybe driven by the hydrostatic imbalance even deeper into the soil, similar to a suction anchor.

The anchors may then be installed using a drill ship or semisubmersible. In deep water, suction anchors are increasingly used. The first segment of the guy line will be attached to each anchor pile as it is set. Then the drill ship will lay out the segment, lower the clump anchor, attach the second segment and then lay it down. A pennant will be attached, with a marker buoy, so that the guy line can be retrieved when the tower is moved into position.

Once the tower has been set and the spud-can has achieved initial penetration, each guy line is fed in through a swiveling fairlead and run up to the deck and stopped off. A linear hoist is attached and initial tension taken up gradually around the series of guys. Then the upper temporary buoyancy tanks are ballasted to a slight negative buoyancy and removed (see Figure 22.3 through Figure 22.5).

Following final penetration, the tension in each leg is readjusted with the cable grip hoists. The deck structure and modules are then set by derrick barge. In other installations of guyed towers, a piled base structure was used, with the piles driven through sleeves in the guyed tower. This was the solution adopted for platform Lena, constructed in 1000 ft. (300 m) of water, described in the following section.

Not only was platform Lena a unique structure once in place, but its installation embodied a number of new ideas that proved successful. The jacket was 330 m (1080 ft.) long by

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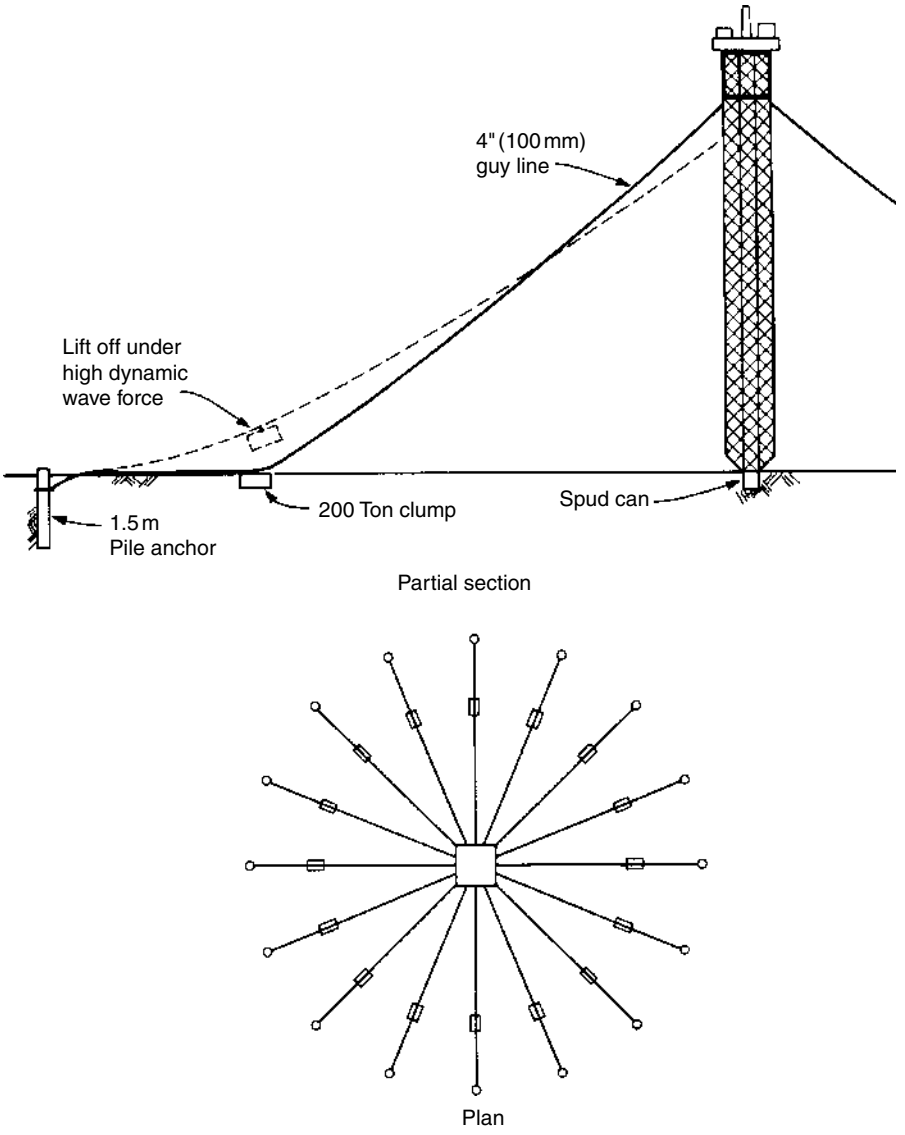

FIGURE 22.3
Guyed tower concept. (Courtesy of Exxon Exploration & Production.)

36 m (120 ft.) square. Launch weight, including main piles and torsion piles, was 27,000 tn. The jacket was loaded out from the fabrication yard in conventional end-O fashion and then lowered onto transverse skids. Launching near the site was sideways, using four launch runners with guides plus rocker arms. Holdbacks were used to restrain the jacket while the barge was ballasted to heel 7° to starboard. Then the 80 mm frangible nuts of the holdbacks were severed by explosive detonation and hydraulic jacks activated to overcome starting friction. The jacket launch took only about 10 s, one fourth the time normally required for a stern launch. The jacket had a maximum roll of 53°, the barge a maximum roll of 15°.

In all, 12 long buoyancy tanks, each 6 m in diameter by 36 m long, were built into the upper portion of the jacket. High-density iron ore slurry was placed into the base to assist

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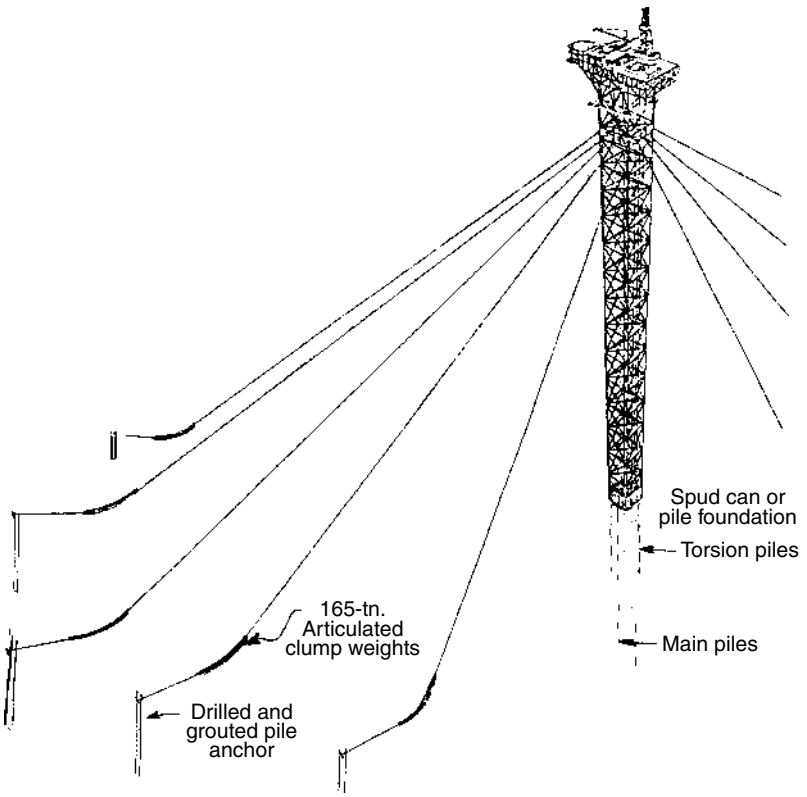

FIGURE 22.4
Guyed tower production system. (Courtesy of Exxon Exploration & Production.)

upending. A derrick barge was used to control the upending, seating, piling, and guy line attachment. This derrick barge maintained position by means of four computer-controlled thrusters.

The lower sections of the main piles of the structure, 1350 mm in diameter, were carried out with the jacket. After upending of the jacket and its seating, they were extended and driven to 170 m penetration.

For the torsion piles, whose upper end terminated in the base, a system of latches and lugs was used to connect pile and hammer together for lowering as a single unit. One 100 m-diameter multistrand wire rope was used to lower the combined unit, using a 600,000-lb capacity linear winch. Air, electric, and hydraulic lines were lowered with separate constant-tension winches and lines. Initially, hammer efficiency was reduced by the cushioning of compressed air below the hammer ram, but a change in the air exhaust system overcame this problem. After driving to full penetration, the latches and lugs were hydraulically released, allowing the hammer to be retrieved.

Earlier, the 20 guys and their anchors had been installed. Drilled-in pile anchors were placed and grouted, each with the guy line preattached. These guy lines had articulated clump anchors attached. A barge laid out the guy with its clumps and then secured it to a buoy temporarily held in position by a small, taut gravity anchor. Once the tower was installed, connection was made with four lines from the tower, leading out through underwater fairleads. These were then pretensioned with linear jacks. Then the additional 16 guys were completed and tensions equalized. The guy lines were 135-mm-diameter wire lines, each 550 m in total length and sheathed in polyethylene. These guys had a breaking

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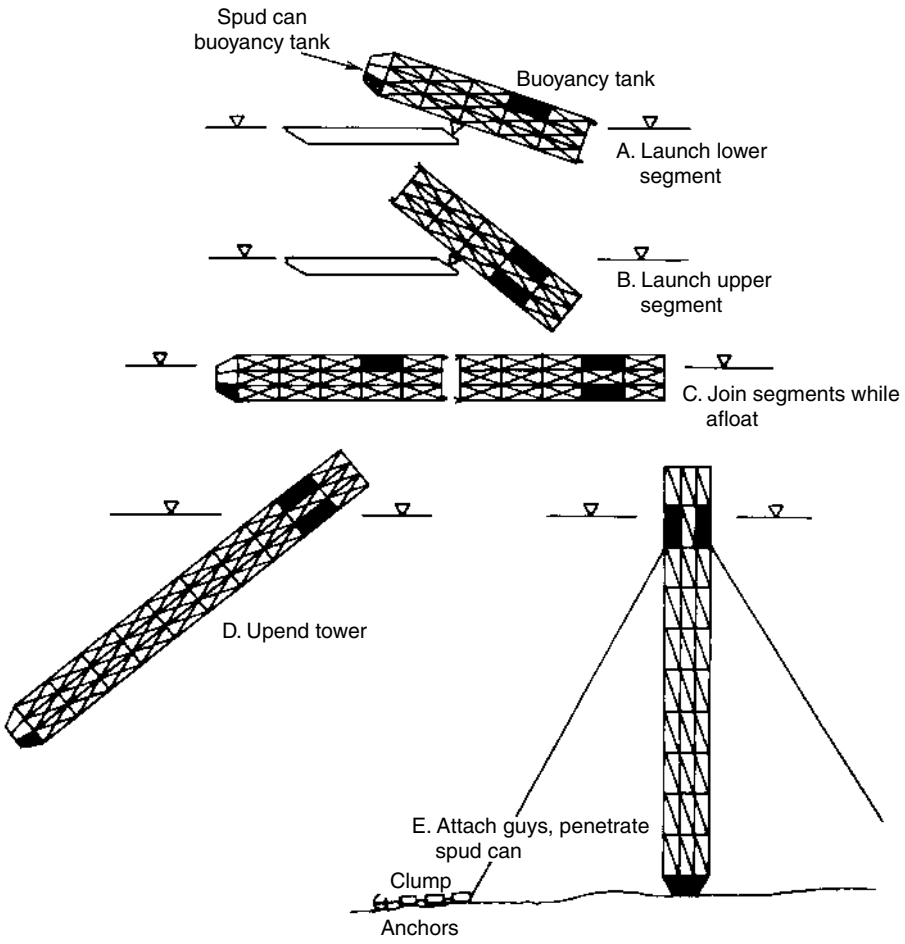

FIGURE 22.5
Installation of guyed tower. (Courtesy of Exxon Exploration & Production.)

strength of 1525 tn. each and were designed for 500–600 tn. maximum load. The clump anchors were each 200 tn., consisting of articulated weights attached to the guy line segments.

22.5.3 Compliant (Flexible) Tower

The Baldpate deep-water production platform consists of a 400 m rectangular tower mated to a 100 m tall base section. Eight tubes at the lower end allow the tower to articulate at 150 m above the seafloor (see Figure 22.6 through Figure 22.8). The tower base was installed by lowering from a heavy lift derrick barge (8000 tn. capacity). Two docking piles (dowels) were set on four leveling piles; 12–84 in. (2.15 m) piles were driven 140 m into the seafloor and grouted to the base.

The tower itself was transported as a single structural unit, by supporting it on two barges. The barge supporting the lower end was tilted by flooding, allowing that end to launch as the barge was towed clear. The upper end was then launched off a conventional rocker arm on its barge. The platform was launched in 650 m of water, 12 km from the final site. As planned, it came to a vertical orientation by itself. Small-diameter tubes at each

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FIGURE 22.6
Baldpate compliant tower for 500 m water depth. (Courtesy of J. Ray McDermott S.A.)

corner were designed to permit selective ballasting after tow to site for adjustment of verticality and to develop the small negative buoyancy needed to enable submergence for mating. Valves were provided for flooding at the lower end and other valves at the upper end for venting of trapped air. Unfortunately, on launching, two of the lower valves were open, instead of closed, as intended. Water slowly flooded in, compressing the air at the upper end. The tower sank slowly but inexorably under water, until it landed on the seafloor, still in its vertical attitude, with its top 250 m below water.

ROV surveys established that the structure was not sinking deeper into the seafloor ooze. Lines were attached by ROV and divers. A heavy-lift derrick barge was brought to the site and lifted the 8700 tn. tower to the surface. Due to adhesion of the soil, the initial

FIGURE 22.7
Baldpate compliant tower showing external rigid tubulars which are hinged above seafloor. (Courtesy of J. Ray McDermott S.A.)

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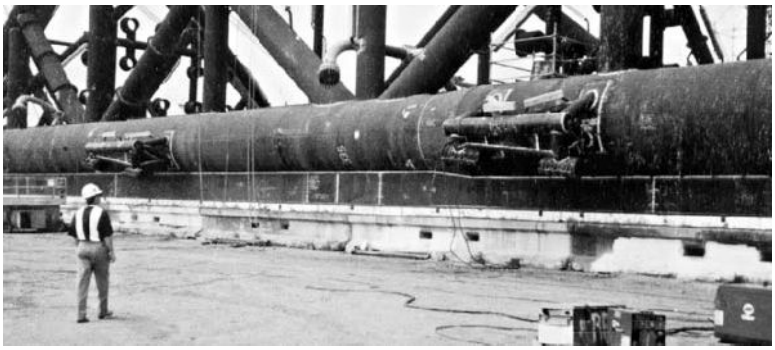

FIGURE 22.8
Connectors at hinge in rigid tubulars in order to allow upper section to be compliant under storm waves. (Courtesy of J. Ray McDermott S.A).

hoisting required 850 tn. of lift, which decreased to the buoyant tower weight of 700 tn. as soon as it was clear of the bottom. The tower was now towed to the site where it was positioned and submerged to mate with the dowels from the base, where the sleeves were grouted. Then the 4000 tn. topsides were lifted onto the tower.

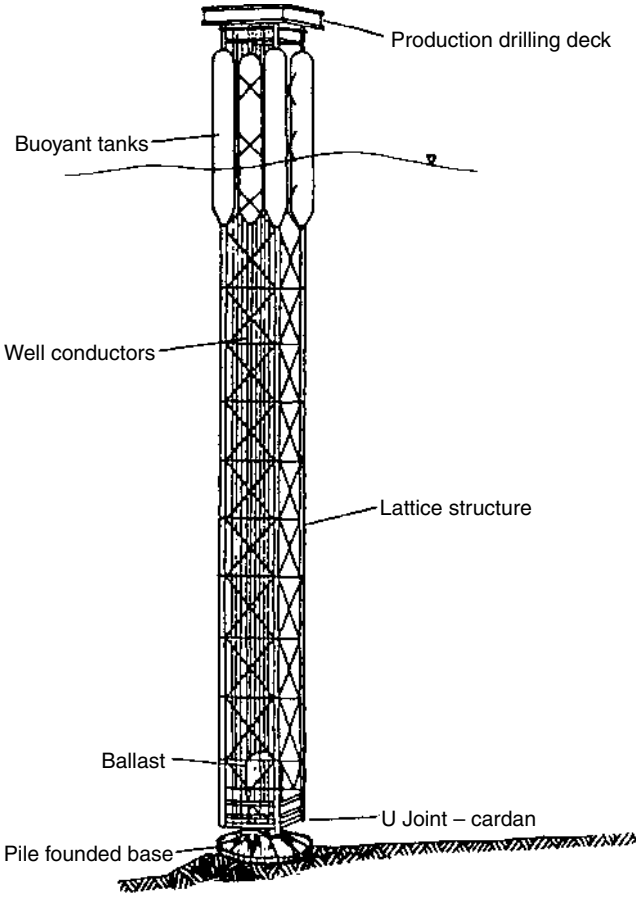

FIGURE 22.9
Doris compliant drilling and production platform. (Courtesy of C.G. Doris.)

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This accident with the valves, although dramatic in this extreme case, has occurred before in shallow water structures. In one North Sea case, a valve for ballasting was unable to close due to a short piece of wire line jamming. Thus, consideration should be given to having two valves in series.

22.5.4 Articulated Towers

A gravity base is used, on which is mounted a cardan (articulation joint), which will accommodate rotation of the tower in both orthogonal directions. A special torque restraint prevents twisting. Buoyancy is provided at the upper end to keep the tower upright. The tower itself may be an open truss, as shown in Figure 22.9.

22.6 Tension-Leg Platforms (TLP's)

The tension-leg platform system consists of a semisubmersible hull, moored by very high strength tethers under tension to a seafloor template or base. Although the early uses were in moderate-depth waters in the North Sea, the concept was primarily developed for the deep sea. Deep-water installations of TLPs have been made in the Gulf of Mexico in waters 1000 m deep and more.

The Hutton platform was installed in 1984 in the North Sea. The hull was a deep draft semisubmersible, consisting of a base raft and six large-diameter columnar shafts. All construction was of steel. The installation of the Hutton TLP was an example of excellent engineering, planning, and execution. While the platform substructure, the tethers, and deck were being fabricated, the well template was placed and the wells were predrilled. Then the foundation (base) templates were installed. To achieve the required tolerance in position of the foundation templates of 250 mm in plan, 2° in orientation, and 0.5° in level, a 900-tn. steel guiding frame was lowered and positioned on the previously placed well template. The frame was leveled by jacks operating against the mud mats. Foundation templates were supported temporarily by the frame and a single 30-m-long pin pile driven through each to fix its position. Then the frame was removed.

Then through each template, the eight main piles, each 1.8 m in diameter, were driven to 60 m penetration. A Menck MHU 1700 underwater hydraulic hammer was used. Piles were entered into their 7.5-m-long template sleeves and the hammer was set on them, using an acoustic positioning system and ROV video. After driving, the piles were grouted in the sleeves.

Installation of the platform itself was carried out in calm seas. The semisubmersible TLP was towed to the site and fixed in position by lines from two large semisubmersible derrick barges, which in turn were moored with 12-point anchor systems. The first leg was run down in each corner and the anchor connector at the lower end latched into the foundation template by hydraulically activated locks. The legs were forged steel hollow tubes, 260 m O.D., of 92.5-mm wall thickness proof-tested to 800 MPa. Tapered threaded joints were used to connect the 9.5-m-long segments.

The installation sequence follows. The acronym TMC stands for the four combination leg tensioner and motion compensators which were operated by pneumatic-hydraulic machines and were located in the mooring chamber of each shaft of the platform's hull.

1. The tension-leg platform was moored to the two semisubmersible crane barges and positioned 40 m to one side of final position.

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2. One leg was lowered at each corner, using special tapered threaded joints.
3. The legs were transferred to the tensioner and motion compensators and raised to the top of the stroke.
4. The platform was moved over the templates to final location and the first round of legs was stabbed into the seabed cones. Connectors were latched and 10-tn. tensions applied with the TMCs compensating for motion (Figure 22.10).
5. Tensioner and motion compensator valves were closed to suppress heave.
6. The platform was pulled down to the 32-m operating draft, with 500-tn. tension in legs.
7. Leg load was transferred from the TMCs to the permanent load block by means of a locking collar.
8. The platform was deballasted to increase tension to 1300 tn. in each corner leg.
9. The remaining 12 legs were stabbed and latched.
10. Tension was equalized with jacks in all legs.
11. Tension was adjusted to 815 tn. per leg by deballasting.

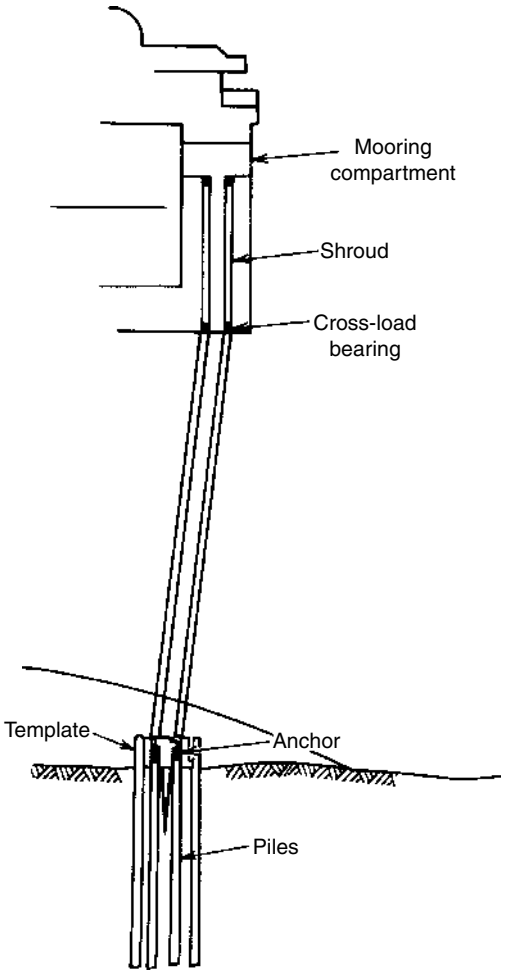

FIGURE 22.10
Conoco Hutton tension-leg mooring. (Courtesy of Conoco.)

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The Heidrun TLP was a concrete semisubmersible, fabricated in a construction basin in a manner similar to a gravity-base platform. Fabrication was complicated by the additional reinforcing steel required to meet recently discovered wave-structure interaction excitation known as "ringing phenomena."

The tethers were fabricated of high-strength tubulars, assembled on shore and launched in a manner similar to that used for pulled pipelines (see Section 15.3). They were towed to the site under tension, one tug pulling, the other dragging to maintain tension. Each tether was upended by flooding, while still keeping substantial tension in the line by tugs, one pulling forward, the other astern. The draft was maintained by compressed air in the top. Then each tether was stabbed into its mating sleeve on the preset concrete base, and pulled into a yoke on the TLP hull, which had been ballasted to below-operating draft. When all eight had been so connected, jacks were affixed in similar manner to those used on Hutton. The TLP hull was deballasted, and the jacks used for final equalization of load. As with Hutton, tensioning was a combination of deballasting, equalized by hydraulic jack.

Several steel TLPs have subsequently been fabricated of steel and installed in up to 1600 m water depth in the Gulf of Mexico. The TLP is considered a viable concept out to at least 2000 m depth and perhaps to 3000 m. Recent studies have shown potential advantages for carbon fiber tendons. They have not only high strength and light weight but also high stiffness and structural damping, making them less sensitive to vortex shedding.

22.7 SPARS

One of the newest and most exciting concepts for deep water is the SPAR, which is essentially a large-diameter vertical column of over 100 m in length, which is tethered by either catenary or taut moorings. The moorings are described in Section 22.8. The SPAR is constructed of either steel or concrete. One early concept was a double-walled concrete hull with an enlarged base. Steel hulls are also double-hull, reinforced internally to take the high hydrostatic pressures.

Spars are normally circular in cross-section and of constant diameter through their length. However, variations have been introduced, such as enlarged bases at the lower end, so as to enable construction afloat as well as to provide a counter force to the overturning force of the waves. In other cases, Spars have been terminated in trussed structural steel frames see Section 13.11).

In a recent Spar, fabrication costs were reduced by making the cross-section polygonal, each side being essentially a stiffened flat plate. In other recent Spars, clusters of smaller cells have been utilized.

Spars are subject to vortex excitation so helical fins are wound around the exterior.

Most spars are moored by a combination of vertical and steeply inclined catenaries, using suction anchors. Some recent Spars have used all polyester mooring lines.

22.8 Ship-Shaped FPSOs

These are surface-floating vessels, either ship hulls or, in calm waters, barge-shaped. They are often turret-moored, so that they weathervane to optimum orientation. This latter may be assisted by thrusters.

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Since the installation is generally set as to duration by the life of the field, they are usually moored by catenary lines leading to suction anchors. When on a fixed mooring, the lines must be of proper length and taut enough so that under a strong transverse wind, they still will not over-run the down-wind moorings.

22.9 Deep-Water Moorings

Catenary moorings in deep water typically lead from the anchor to a submerged spring buoy, thence in a conventional catenary to the vessel. The submerged buoy must be filled with syntactic foam. As with all spring buoys, the axial force must be transmitted through the buoy independently of the buoy itself, so that surges in force are not transmitted by the buoy but by the lines. The buoy holds the weight of the mooring line below (Figure 22.11 and Figure 22.12).

For the Mensa project, in 1700 m water depth, 3000 m of 96 mm wire rope and 1000 m of chain were placed to 18 tn. drag anchors designed for high vertical lift capacity. The anchor-handling vessel had 187 tn. bollard pull and developed 13,500 BHP.

Mooring systems for deep water have considerations which differ significantly from those employed in shallower water. These include the weight of the mooring lines, the increased influence of low-frequency motion of the vessel, and line dynamics. Since there are significant increases in total current force in the deep sea, both the drag and vortex shedding need to be considered. Catenary moorings are usually employed, with a spring buoy to hold the weight of the mooring line (see Figure 22.13).

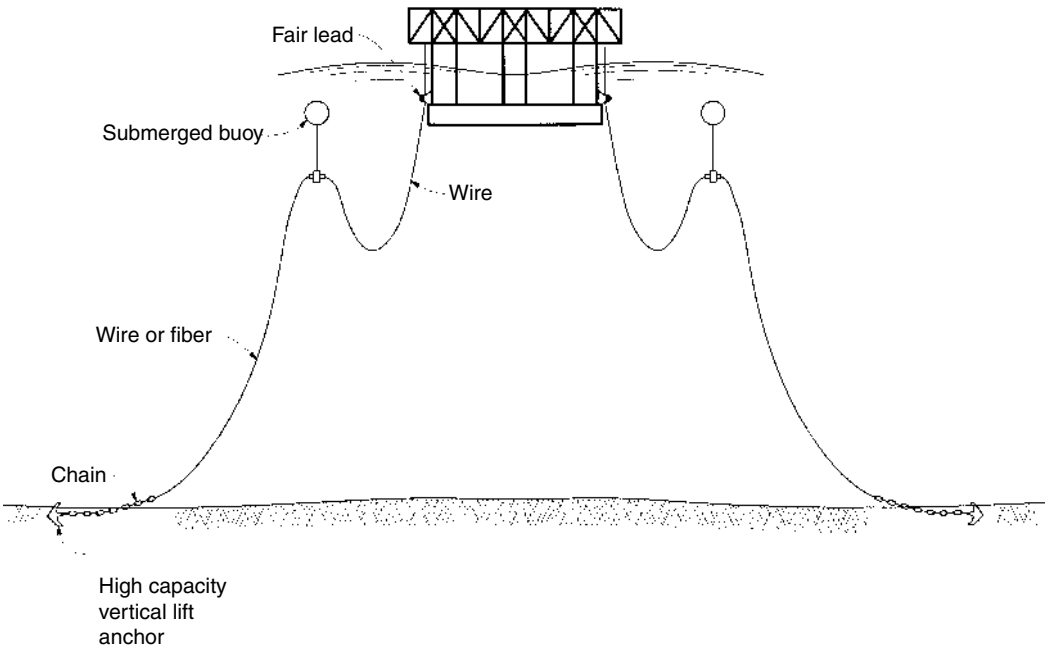

FIGURE 22.11
Catenary moor for deep water.

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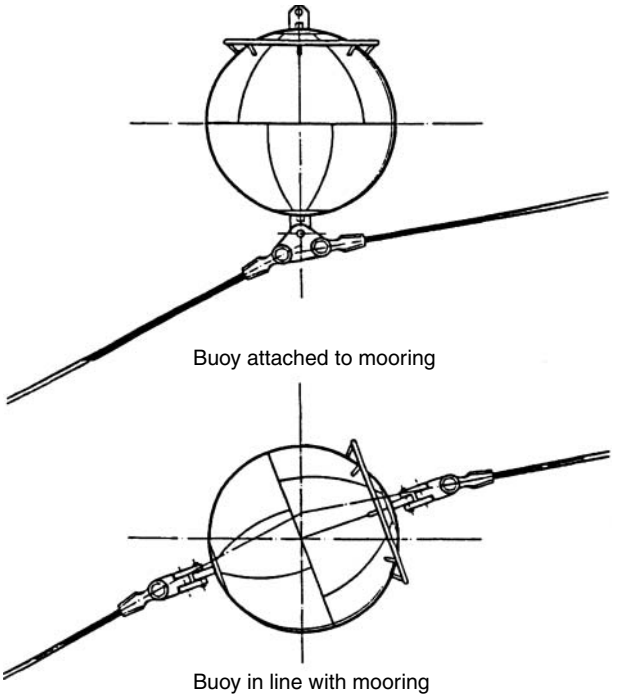

FIGURE 22.12
Submersible buoy configuration.

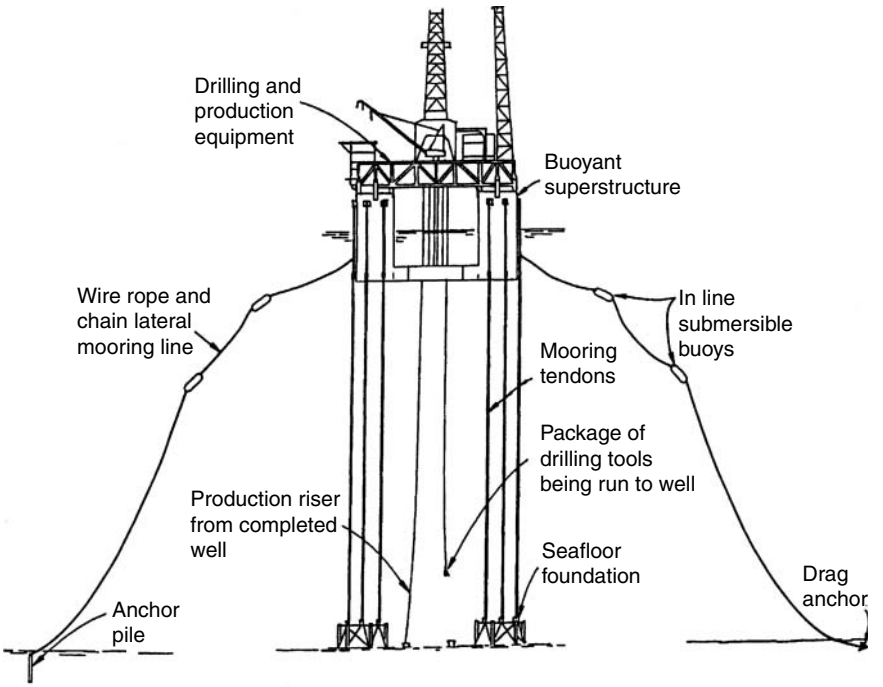

FIGURE 22.13
Tension-Leg Platforms (TLP) tension-leg mooring system.

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Since vertically loaded anchors are subject to a high number of cycles of large forces at the anchor itself, the connections must be designed for fatigue. Thus, special welding procedures are necessary.

Synthetic fiber ropes are being increasingly employed because of their light weight. The most commonly used fibers are polyester and Kevlar. They are deployed in a taut mooring system consisting of chain or wire in the first segment, to prevent chafing over the fairlead, then the synthetic fiber and finally a ground line of chain or wire. Wire has the advantage that it can cut more deeply through the soil. In installation, care must be taken to prevent damage from excessive bending, twisting, abrasion, heat from high-speed deployment, grounding on the seabed and excessive cyclic loading.

Not only do synthetic fiber lines reduce the vertical load, but they exhibit bi-modal performance, stretching as the moored vessel offsets under steady environmental loads, then oscillating with a much stiffer spring route about the first order offset location. This results in less fatigue due to second-order motions. Polyester lines have to be handled with care to avoid abrasion: there must be no crossing of lines and no dragging on the seafloor.

Although there do not appear to have been any recent serious problems with fish bite, it should be borne in mind that the Navy had trouble with fish bite on fiber moorings for buoys. This problem was solved by extruding a smooth polyurethane coating over the fiber rope.

For the Neptune and Genesis SPARS, a taut mooring system was employed, using 14 lines with studless chain shots at each end and steel wire rope in between. For Genesis, the chain links were 5 in. (133 mm) in diameter and the system was tensioned by 14–480-tn. (4.8-MN) capacity linear chain jacks, having a speed rating of 1.2 m/min. See Figure 22.14.

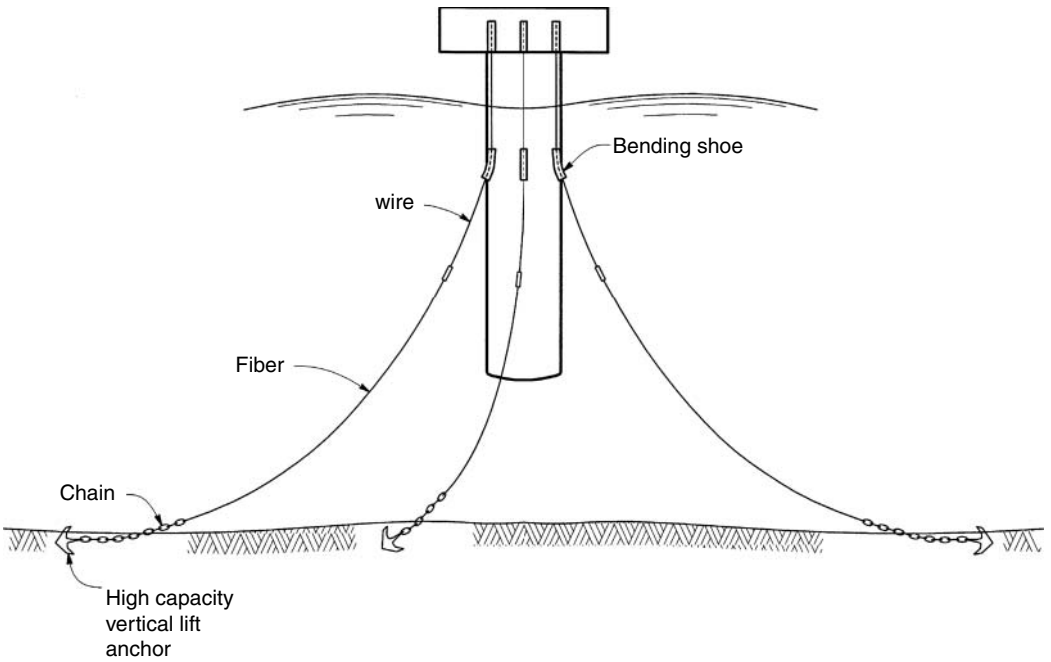

FIGURE 22.14
Taut mooring for SPAR in deep water.

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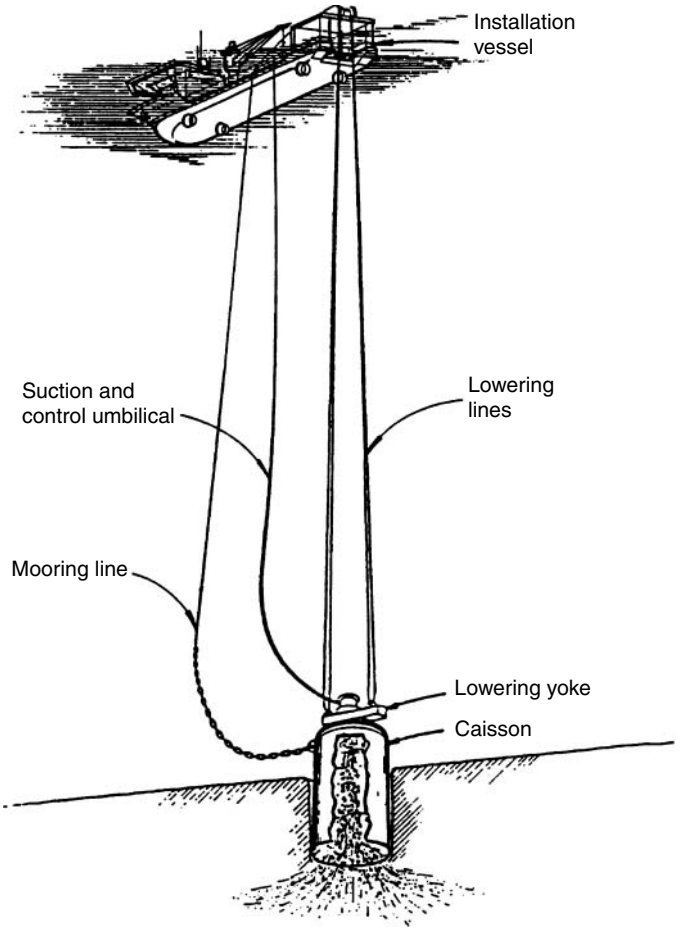

FIGURE 22.15
Installation of suction anchor.

Suction pile anchors are being used both in clays and sand (see Figure 22.15). Unlike driven piles, these suction piles are large diameter (4–20 m) and short (15–20 m), with a permanent cap on the upper end. They are lowered to seat in their final position on the seafloor and then allowed to penetrate through the surficial soils under their own weight. To gain an effective seal, it is necessary to penetrate 1–2 m. The trapped water must be vented.

Then pumping is implemented. A pump line is attached, which may be operated by an ROV or by an umbilical from the surface. It reduces the internal water pressure, thus mobilizing the external hydrostatic pressure to drive the pile to full penetration. Suction causes a temporary quick condition which facilitates penetration. In dense sands, a water injection system is employed to lubricate the sides. One such installation had 220 holes of 3 mm diameter around the periphery of the pile. If suction pile anchors are misplaced or unable to achieve penetration, they can be retrieved by reversing the process and pumping water into the top of the pile. Overpressure should be as low as practicable in order to avoid piping under the tip (see Section 6.2).

The U.S. Naval Civil Engineering Research Laboratory at Port Hueneme, California, has developed special anchors for hard, dense seafloors, which free fall to near the seafloor, and are then explosively driven into the soil.

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For drilled-in piles, either a drill ship or semisubmersible is used. The anchor chain is attached midway down the pile in order to develop as much lateral resistance as possible, even though the pull is almost vertical.

22.10 Construction Operations on the Deep Seafloor

As structures are submerged, they can be positioned by dynamic thrusters and locked in by onboard computer through acoustic transponders to surface vessels and thence to satellites. Alternatively, they can use pre-set seafloor transponders to maintain relative position.

Submerged buoyant structures can be kept afloat at prescribed elevations off the seafloor by the use of weighted tethers. If they rise, they pick up more tether weight (for example, chain) and hence return to their original elevation. Objects lowered on rope or casing must consider the dynamic response of the lowering vessel as it responds to the waves, as well as the inertial effects of the object, with its added mass of water that must also be accelerated. To overcome this, giant heave compensators were devised for the Glomar Explorer and were used to overcome roll, pitch, and heave effects. Free-fall deployment of seafloor structures, as well as anchors, may also be "controlled" by installing multiple buoys. The structure can then be progressively ballasted to descend in steps. French engineers have developed "breather" buoys, which decrease in volume and hence buoyancy as they descend. These have been successfully used in laying a test section of pipeline in 2500 m water depth, followed by its successful use in laying the gas line across the Strait of Gibraltar at a depth of 350 m. The Harding Gravity Base Tank (GBT), a seafloor storage tank, was placed by the use of multiple buoys, as described in Section 13.4 and Figure 13.5. "Glide" can be used with submersibles and ROVs to control the rate of descent. "Pulling down" of a buoyant structure against a seafloor anchor is an effective system for decoupling the system from the surface wave effects once the structure is below the surface.

The latest generation of large offshore crane barges is equipped to lower seafloor templates to substantial depths approaching 1000 m. For these and greater depths, the seafloor template is submerged by crane barge to below the hull of a deep-sea drilling vessel, and then slung in under the vessel's moon pool. It is then lowered to the seafloor by the drill string. Since the capacity of the drill string is usually limited to about 500 tn., including the dynamic effects of heave and acceleration, auxiliary buoyancy is incorporated into the seafloor template. Spars may be used to reduce the heave response, each fitted with a linear jack and using drill casing or wire rope.

When landing on a deep seafloor, there are potential problems due to excessive penetration into the seafloor ooze. This layer of very soft material ("soup"), which may actually be in colloidal suspension, may not have been revealed in the geotechnical investigations due to lack of acoustic reflection and inability to retain in sampling tubes. Legs extending downward from the structure in the form of large dowels may help to stabilize the initial penetration. The legs may have steps of increased diameter, so that total penetration is limited.

The turbidity cloud caused by landing in the soft ooze must be considered, as it will impede the use of video for positioning control. Prior steps may be taken to blanket the ooze, as outlined in Chapter 7 on seafloor modifications. The U.S. Naval Civil Engineering Laboratory at Port Hueneme, California, is continuing development of such means to suppress turbidity due to colloidal suspension of seafloor ooze.

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For placement of concrete at depths ranging from hundreds to thousands of meters, two methods have been developed. In one, the concrete is transported in a long tube, closed at the ends, for discharge at the seafloor. In the other, the concrete is pumped down a pipeline. An oversanded mix containing antiwashout admixture is used and the diameter of the pipeline is reduced so that the friction limits the velocity to about 3 m/s and thus prevents segregation. Aggregates should be pre-saturated to reduce the change in character of the mix and consequent stiffening due to absorption of water by the aggregates under pressure. The effects of low temperature (principally retardation) and high hydrostatic pressure must be considered.

The newly developed admixtures such as antiwashout and silica fume which prevent segregation may also make it possible to place concrete in deep water by use of closed buckets or other discrete devices. Cement slurries (grout) have long been placed by pumping at great depths by the oil-drilling industry, where they have been used to cement casing strings, and to plug wells. Concrete has been placed through tremie pipes to depths of 250 m in belled piles and to 1000 m and more in mine shafts.

When concrete or grout is used in large volumes, the heat of hydration must be considered, and special cementing mixtures such as blast furnace slag cement or cement plus pozzolan must be employed to reduce the heat and prevent the consequent disruption of the concrete or grout. Grout is especially vulnerable, due to the high cement content.

For breaking objects loose from the seafloor, waterflooding underneath is considered the most effective method. The pressure must be kept low enough to prevent piping to the sea. In soils of low permeability, many hours of such flooding may be required to raise the internal pore pressure in the soils sufficiently to overcome the suction effect and reduce the shear.

Deep-ocean dredging operations have been studied in detail for the mining of manganese nodules from the deep seafloor, and test operations have been carried out at depths up to 4000 m. The use of airlifts has been found to be an effective and efficient method. Because of the large volumetric expansion of air as the hydrostatic pressure reduces, the airlift is employed to raise the material only as far as a submerged pump capsule. Conventional pumps are then used to raise the nodules the additional distance to the surface vessel (see Figure 22.16).

Seafloor soils can be consolidated by suction drainage, which may be carried out after the structure is emplaced or even before installation, by drainage from under an impervious membrane.

For installing gravity-based structures in deep water where the seafloor is known to be irregular—as, for example, with rock outcrops—one solution is to dump rock to create a submerged embankment on which to seat the structure. The rock may be dumped from a bottom-dump barge in one mass to minimize segregation during the descent. Alternatively, it may be placed through a flexible tremie tube, which provides better control. The rock should be pre-saturated to dispel all air. After dumping, further consolidation can be obtained by dynamic compaction (i.e., the repeated dropping of a heavy ram or explosives).

Placement of rock by use of a chute ("flow pipe") to depths of 300 m and deeper to cover or support pipelines, has been described in Section 7.7. The use of this flexible chute, suspended from a ship, provides control and prevents segregation. It may prove feasible to extend the depth range of this type of placement and thus provide a high degree of control (Figure 22.17). If rock is used, the space under the base of the structure may be injected with grout, retained by filter fabric or sand bags around the perimeter. The grout for underbase fill should have a thixotropic admixture to reduce its flowing tendency once the pressure drops.

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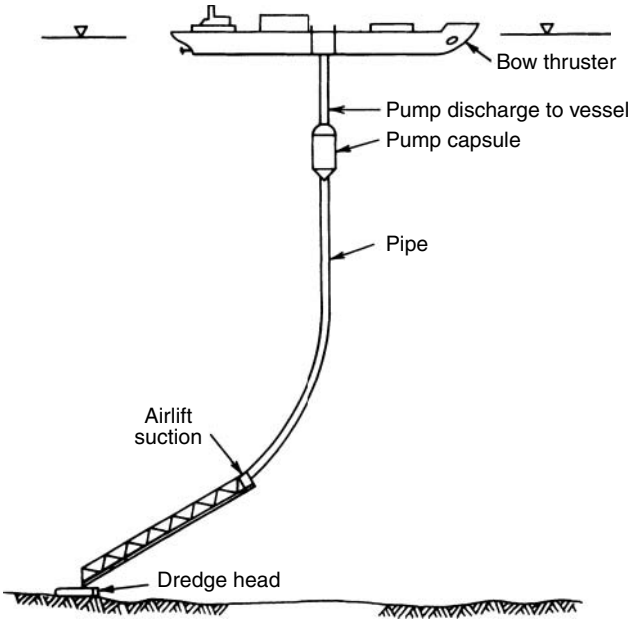

FIGURE 22.16
Deep-ocean dredging system for mining of manganese nodular from deep seafloor.

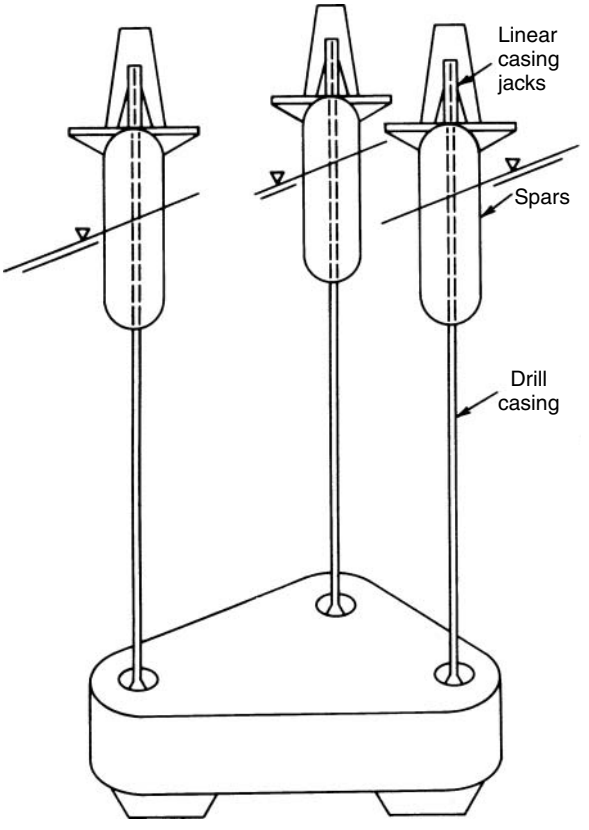

FIGURE 22.17
Lowering a heavy mass in deep water.

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This approach was successfully used in shallower water under caissons for an offshore terminal in Queensland and for scour erosion under the Oresund Bridge and the Akashi Strait Bridge. These methods have been incorporated in a study for the deep piers (300–500 m) for the bridge across the Strait of Gibraltar.

For structures such as subsea templates seated on the deep seafloor, it may be necessary to transfer manipulators and service modules between them and a surface vessel. Pop-up buoys may be attached to such a structure, to be released on acoustic signal and thus provide a guide line for subsequently lowering or guiding a manipulator or structural element to an exact mating with the previously installed element.

Tensioned guide lines of this type were extensively employed in the 1960s and 1970s at water depths up to 500 m for reentry of drilling strings into casing. They have now been largely replaced by acoustic and inertial guidance, as developed and used on the Glomar Challenger to re-enter the casing at 6000 m.

22.11 Deep-Water Pipe Laying

Deep-water pipelines and flow lines have been successfully laid in water depths as great as 1700 m (Mensa project). Even greater depths are in the planning stage. Below 1000 m, a typical pipeline has to be internally pressurized to balance the external hydrostatic head. Large pipelines may need pressurization at lesser depths due to out-of-roundness tolerances in manufacture and ovaling due to self-weight, both of which can initiate buckling under external pressure. In deep water, buckle propagation is a very serious concern. Buckle arrestors, in the form of heavier wall thickness pipe, are placed in the pipeline at regular intervals.

Sleeve-type buckle arrestors can be slipped onto the pipe segment, welded at both ends, and injected with epoxy. A more reliable method is to fusion-weld a short segment of heavy walled pipe at the end of the segment. Typical spacing in deep water is 60 m. In all cases, the internal diameter must be maintained so as to not obstruct the passage of a pig.

A number of methods have been used for deep-water installation. Several of these, the S-lay barge, the bottom-pull, and the reel barge methods, are modifications of the conventional methods which have been widely employed at lesser depths. With the S-lay method, the pipe will descend nearly vertically and hence the stinger will have to allow the pipe to develop an almost 90° bend. Typically, relatively short cantilever stingers are used. With this method, a 12 in. (300 m) pipe has been laid in 1600 m of water. The reel barge has been similarly used for deep-water laying of flow lines and small-diameter pipelines. By installing a series of two to four full reels on the barge, each feeding to the other, a substantial length of line may be laid in a single operation. A reel barge has laid 250 mm pipe in 1600 m of water.

The J-lay barge method differs from the S-lay barge in that the pipe segments are made up on a ramp that is inclined from 60 to 80° from the horizontal, thus eliminating the overbend. No stinger is required. As before, required tension is low. A hand-over-hand tensioner has been mounted in the inclined ramp. The joint is made and completed at one station, located just above the deck. Advanced rapid means of welding are employed. The most efficient and rapid laying is attained by racking up pre-assembled triple and quadruple lengths of pipe, even up to 60 m as a single length. See Section 15.9 for further description of this system and methods of rapid jointing.

The J-lay method is a highly specialized, highly developed method for laying long major pipelines in deep water and rough seas. Since there is no longer a significant horizontal line tension, dynamic thrusters can be used to position and move the barge, eliminating the continual anchor handling required for third-generation S-lay barges. The J-lay system

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was used on the Ursa project in 1300 m of water. Modular systems are being constructed to permit the installation of J-lay ramps on existing offshore pipe-laying barges without extensive modification to the barge.

The bottom-pull method described in Section 15.3 has been further developed to enable laying of deep-water pipelines in very deep water. It was successfully used to install the Troika pipeline in 800 m of water depth. A single 10 in. (250 m) pipe was carried in a 24 in. (600 m) casing with the annulus being filled with syntactic composite insulation. The 20-km-long line was made up along the beach at Matagorda, Texas, in 3–10 km strings. The casing and line were then pressurized with nitrogen. Using side-boom cats, the line was moved sideways into shallow water. A leading sled and trailing sled were attached, the latter to permit reverse pulling if it became necessary. The line was then towed 350 miles at a speed of 5.5 kt. Upon arrival, the supply vessel, equipped with dynamic thrusters, moved in between the legs of the semisubmersible to lay the end of the line at the seafloor manifold.

For the Bullwinkle platform, in 450 m water depth, a heavier catenary riser section 1300 m long was joined to the pipeline at its leading end. To provide buoyancy during tow, a 24 in. (600 mm) casing was attached over the full length of the riser. The subsequent filling and venting has to be carefully controlled to maintain the internal pressure at all times.

On one long bottom pull in the Gulf of Mexico, an inadequate seafloor survey resulted in the pipeline colliding with a submerged reef, leading to buckling. A thorough seafloor survey is essential.

Tows of long lines on the seafloor require that the net weight on bottom be reduced to the minimum. However, in many offshore areas, bottom currents are low and, in deeper water, there is no wave action. Provided a means or place for launching can be found which does not require passing through heavy surf, very light net weight (e.g., 7–10 kg/m) can be adopted. Tolerances then become even more critical.

For laying flow lines and pipelines in deep water, Exxon–Mobile has developed the reverse J-tube method, by which the line is made up stove-pipe fashion on a fixed platform deck and pulled down the J-tube and over the seafloor by a tug (see Figure 22.18, also Section 15.8).

In laying the Mensa flow lines, one had to be repaired at 1600 m depth. The line was severed by ROV-placed shaped charges. An ROV then placed lift frames and installed pipe recovery tools (Figure 22.19).

Currently (2005), a newly developed underwater trench cutter is excavating a 2×10 m trench in very hard glacial clay for the gas lines from the Ormen Lange Gas Field in

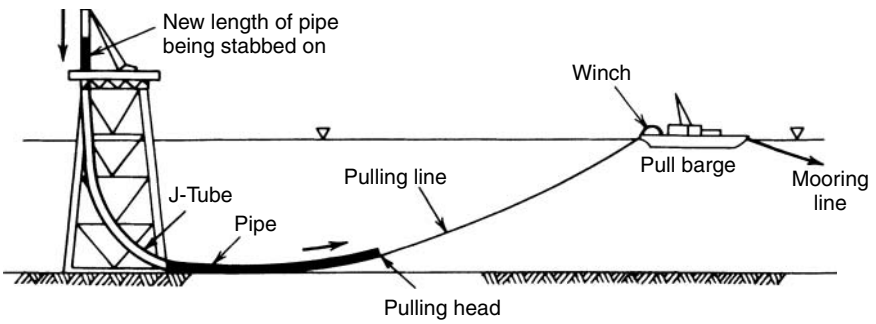

FIGURE 22.18
Reverse J-Tube pipe laying system. (Courtesy of Exxon.)

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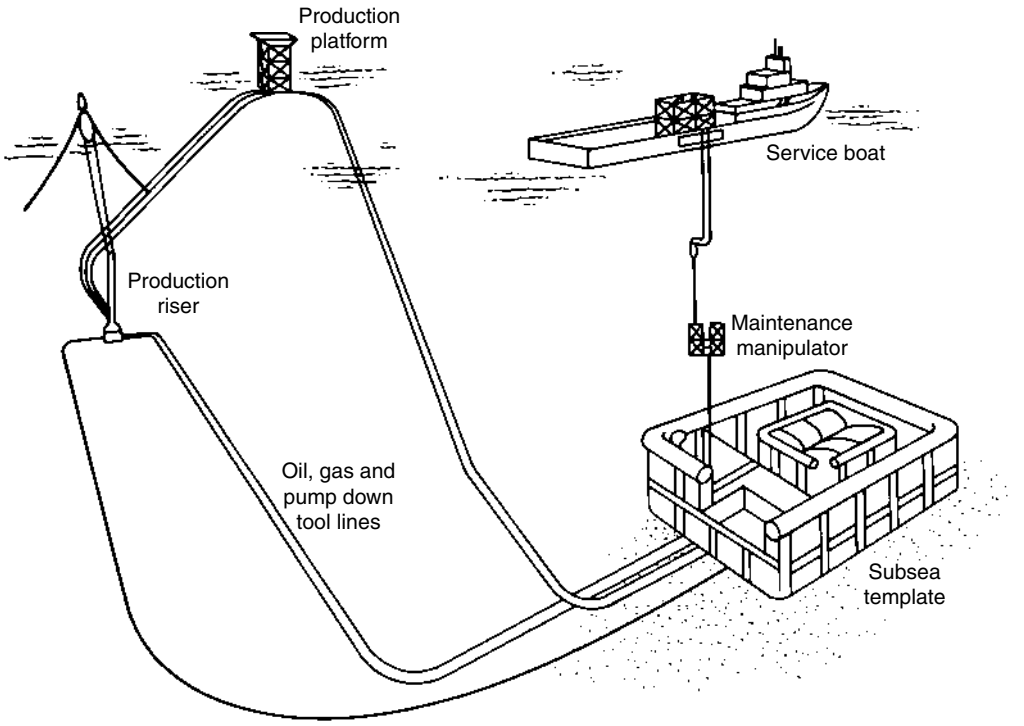

FIGURE 22.19
Subsea production system. (Courtesy of Exxon.)

offshore Norway. Water depths are 850–1100 m. This is a bottom-crawling walking leg machine, equipped with high pressure jets and eductors with a hydraulic backhoe bucket which can be fitted with various cutting tools.

While initially it can be controlled by video, the jets stir up silt clouds which render that inoperative. Controls then are monitored by computer-generated acoustic imaging displayed on the surface control vessel above.

The line will be laid by J-lay at an 80° angle which will optimize the bending stresses in the pipe. Under the rough seas that are anticipated, a location of the J-lay near midships of the semi-submersible will minimize the pitch heave and surge excursions.

A proposed pipeline across the Black Sea will transit depths as great as 2000 m. A 782 m diameter steel pipeline, with 41 m walls of high-strength steel, is under consideration. Both S-lay and J-lay methods have been found practicable. A diverless repair system has been developed, with mechanical connections to be carried out by an ROV.

Although all but a few small flow lines have been laid empty in order to take advantage of the buoyancy, this may no longer be the optimal solution for oil lines in deep water. For these, the wall thickness is largely determined by the external hydrostatic pressure.

Laying the line full of fluid and therefore under low external hydrostatic pressure permits a reduction in wall thickness that may partially offset the weight of the water fill. It will have to, of course, consider the net difference between the density of the fluid or oil and that of the seawater. A major saving in steel quantity may be realized.

The proposed Oman to India pipeline will be even deeper, up to 2500 m. Filling the line with pentane has been considered, in order to reduce the required wall thickness of steel pipe.

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22.12 Seafloor Well Completions

These are an integral part of most deep sea offshore oil and gas production systems. They basically consist of steel framed templates with appropriate well openings, values, and controls, set on the seafloor. The wells are drilled through the templates, using ROV guides and acoustic pingers; casings are now set and cemented (grouted) in to prevent gas blow out around the perimeter and to prevent fall in of the sea-floor sacrificial soils. The template is held in place by drilled-in spuds. This gives lateral stability and enables that pipe line connections to be pulled into conical receptacles where they are sealed gas tight. Guidance is by ROV.

22.13 Deep-Water Bridge Piers

A number of proposals have been developed through the conceptual engineering phase for bridge piers in deep water. Foremost among these are those for a bridge across the

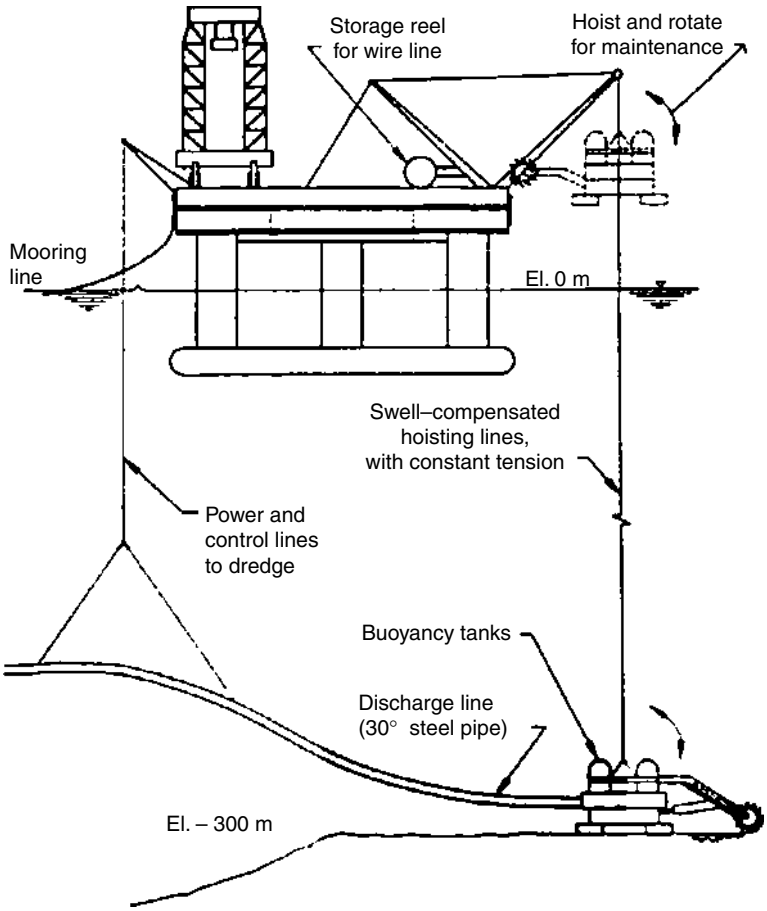

FIGURE 22.20
Concept for leveling foundations at depth of 300–500 m. Strait of Gibraltar Bridge.

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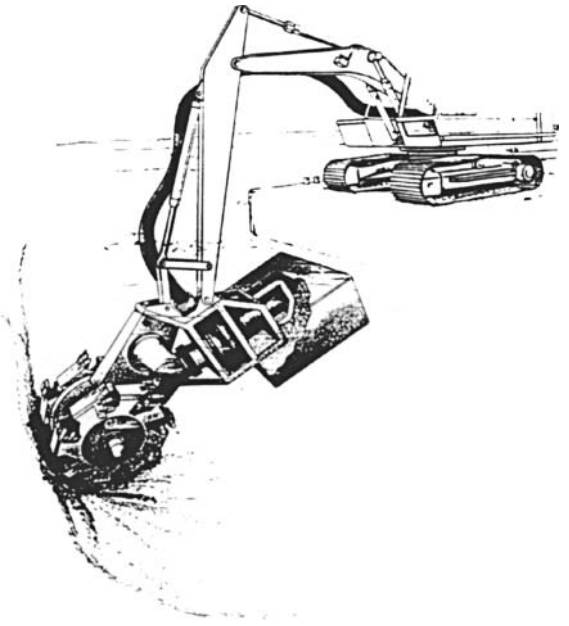
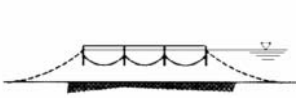


FIGURE 22.21
Dredge head of seafloor dredge.

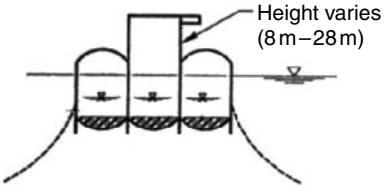
① Construct base caission in construction basin

Pier	Lift	Draft
1	15 m	11.4 m
2	20 m	15.9 m
3	20 m	15.6 m
4	20 m	14.4 m



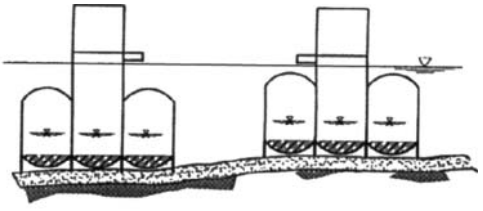
② Float out to moorings

FIGURE 22.22
Construction of prefabricated pier caisson for Strait of Gibraltar Bridge.



③ Construction afloat

Pier	Draft	Critical gm
1	36 m	21 m
2	45 m	21 m
3	45 m	21 m
4	40 m	18 m



④ Found all four base caissons on temporary stone bed foundations

Pier	Draft	Critical gm_2
1	56 m	12 m
2	64 m	9 m
3	57 m	8 m
4	56 m	9 m

FIGURE 22.23
Stages 3 and 4 of prefabrication. Temporary seating on shallow seafloor, which is profiled to that of prepared foundation.

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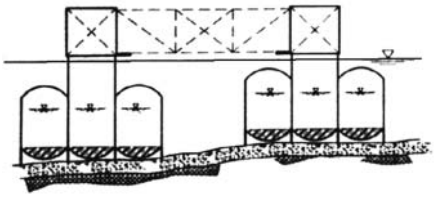
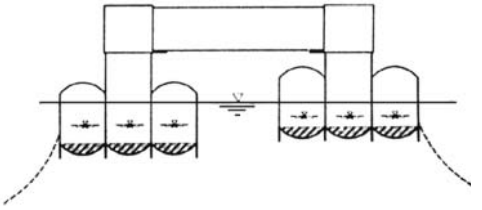


⑤ Set prefabricated steel trusses on projecting seats



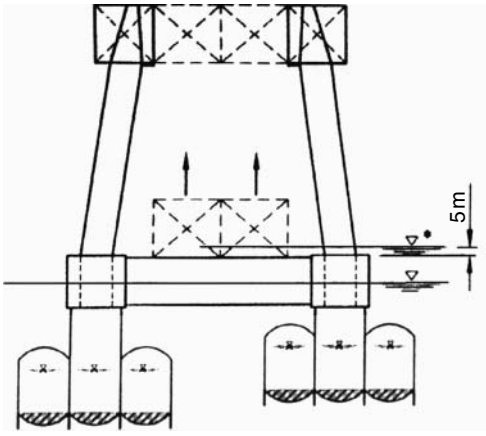
⑥ Construct cross arms. Deballast and tow to deep water mooring

Pier	Draft	Critical gm_3
1	36 m	63 m
2	46 m	146 m
3	46 m	151 m
4	41 m	106 m

FIGURE 22.24
Stages 5 and 6.

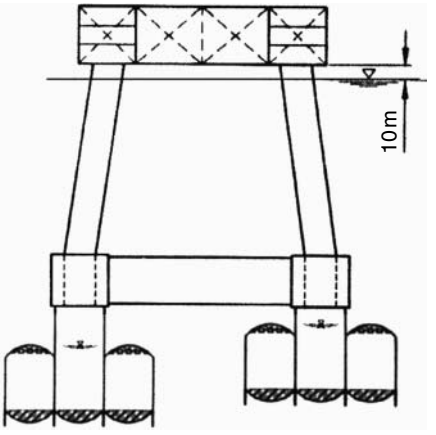
Strait of Gibraltar. Depending on the route selected, the piers will need to be founded at water depths between 300 and 500 m. Strong currents, heavy waves, and irregular seafloor make this an extremely challenging undertaking. The original studies in the 1990s assumed economic feasibility would be realized about 2050, which makes it timely now to begin serious investigation and feasibility studies.

The seafloor would need to be leveled over the footprint of each gravity-base caisson. Dredging in the calcareous sandstone and siltstone is believed to be feasible using a very large semisubmersible hydraulic cutter head dredge with a wheel cutter head.



⑦ Ballast down to mid-height of cross-arms. construct all 4 shafts and conical tops. Raise plateform fabrication trusses and secure to shaft top cones and upper falsework (truss weights ~ 2000T)

Pier	Draft *	Critical gm_4
1	96 m	22 m
2	106 m	34 m
3	95 m	36 m
4	97 m	27 m



⑧ Ballast down to deep draft. Complete upper platform, vertical cross walls formed with rebar tied but no concrete placed

Pier	Draft *	Critical gm_4
1	103 m	5 m
2	281 m	21 m
3	246 m	6 m
4	196 m	6 m

FIGURE 22.25
Stages 7 and 8. Submergence in protected deep water of Algeciras Bay, Spain.

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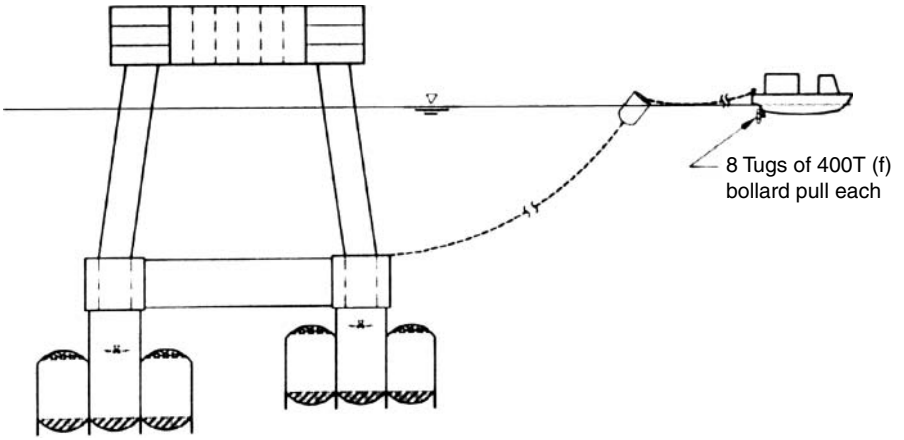
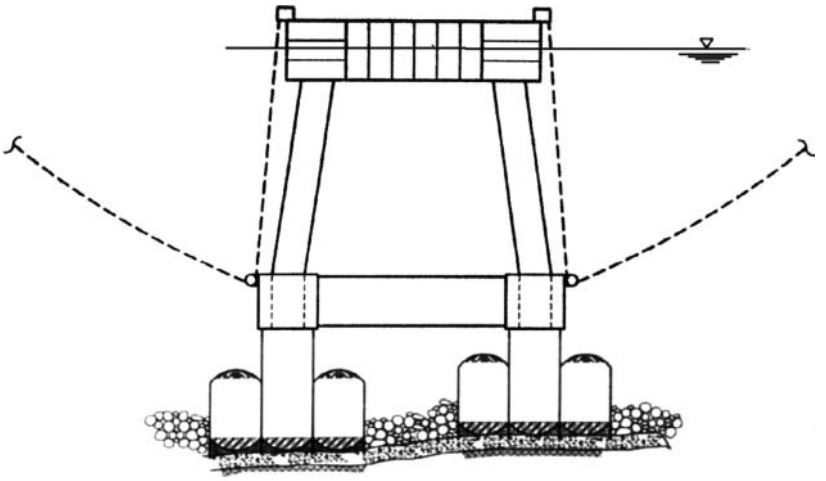


⑨ Deballast to optimum draft and tow to site

Pier	Min. draft	gm_6
1	90.8 m	3.1 m
2	216.0 m	2.2 m
3	191.0 m	2.3 m
4	186.0 m	3.0 m

FIGURE 22.26
Stage 9. Tow to site using 6 tugs, each with 400 tn. bollard pull.



⑩ Moor to pre-set mooring buoys, ballast down to found on prepared stone bed foundation
⑪ Grout under base
⑫ Install scour protection as necessary
⑬ Place concrete in platform cross walls

FIGURE 22.27
Ballasting down to seat on prepared base.

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The piers must withstand ship collision. The legs of the piers must withstand collision from a nuclear submarine at any depth. Therefore, a concept was developed for a four-legged concrete GBS, along with the construction method outlined in Figure 22.20 through Figure 22.27. Future studies will consider the use of low density fluids to reduce the wall thickness required during installation (see Section 22.3 and Section 22.4).

Other proposals, involving both steel and concrete, utilize concepts developed for deep water oil platforms such as the flexible tower, with inclined mooring lines to restrict displacements.

I must go down to the seas again, for the call of the running tide
Is a wild call and a clear call that cannot be denied;
And all I ask is a windy day, with the white clouds flying,
And the flung spray, and the blown spume, and the sea-gulls crying.

John Masefeld, "Sea Fever"

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23

Arctic Marine Structures

23.1 General

The Arctic Ocean and the adjacent sub-Arctic seas are major challenges for offshore construction. These frontier areas are dominated by perhaps the most severe environmental conditions yet addressed for offshore development. The Canadian and Alaskan Beaufort Sea were the primary areas of interest in the 1980s. In the 1990s the focus has shifted to the east coast of Canada, the Barents and Kara Seas, and offshore Sakhalin.

This chapter is largely a synthesis collecting the construction procedures, data, and guidance that relate to the Arctic Offshore. Some of this material has been previously presented in individual topic-oriented chapters. Because of the important role of the Arctic and because work in Arctic regions requires a multidisciplinary approach, it seems appropriate to gather together in one chapter the multiple aspects to which consideration must be given if structures are to be economically and safely built there.

Development of the Arctic offshore areas was greatly accelerated by the leasing programs of Canada and the United States in the 1980s. Platforms subject to moving sea ice were constructed in Cook Inlet, Alaska in the 1970s and have given very satisfactory service, although occasionally subject to severe vibration from ice rafts jamming between the legs. Significant but not yet serious loss of underwater plate thickness is reported from the highly abrasive siliceous silts in the constantly moving current.

Lighthouses were constructed even earlier in the Baltic Sea, generally of concrete caissons for the substructure with steel or composite steel–concrete towers. A number of failures have occurred due to vibration-induced fatigue of steel shafts, under the cyclic crushing of the ice. Experience from studies of these lighthouses and of icebreaking vessels indicates that the crushing failure tends to become locked in resonance with the natural period of the structure.

Abrasion–erosion of the concrete has occurred in these Baltic lighthouses in a narrow band from half a meter above to 2 m below sea level (the tidal range in the Baltic is only about 1 m). Not only has the concrete been eroded but the reinforcing steel has been torn loose. This abrasion problem has occurred only in the narrow neck of the upper Baltic where hard, low-salinity ice has moved past the structure during the spring outflow. Farther north, the hard ice has been largely fast ice; farther south, the ice has been softer, higher salinity ice, so in both these cases, there has been no significant abrasion.

The above-water decks of the concrete caisson substructures have also been deteriorated by freeze–thaw attack, especially where salt-water splash has ponded. Corrosion of the reinforcement has occurred to a moderate degree where the freeze–thaw attack and abrasion have removed or reduced the concrete cover.

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The Russians constructed a prototype tidal power plant at Kislogybusk, 150 km east of Murmansk, in 1970. Although the structure is subjected to 4 or 5 months per year of primarily fast ice, there has been no reported damage or deterioration.

Reference should be made to the detailed description of sea ice and icebergs in Section 1.10; to the discussions of geotechnical properties of the Arctic seafloor in Section 2.5 through Section 2.8 (overconsolidated silts, permafrost, weak Arctic silts and clays, and ice scour); and to special ecological considerations in the Arctic in Chapter 3, especially noise, open leads in ice, oil spill in ice covered waters, and disturbance of critical migration routes.

23.2 Sea Ice and Icebergs

The Arctic Ocean proper is dominated by the polar pack, a disk of permanent ice 1500 km in diameter circulating clockwise in a gyre centered at about 80°N and 150°W. The polar pack is largely composed of multiyear ice floes interspersed with ridges and fractured by leads. The average thickness is about 4 m. Recently it has been thinning due to global warming.

The Joint Ice Center of the U.S. Navy and the National Oceanographic and Atmospheric Administration publishes maps of ice conditions in the Arctic and sub-Arctic and a prediction of future ice conditions. Examples of such maps for summer (August) and winter (December) are shown in Figure 23.1 through Figure 23.3. Note in the 7 August map (Figure 23.1) the relatively narrow corridor of ice-free water around the north of Point

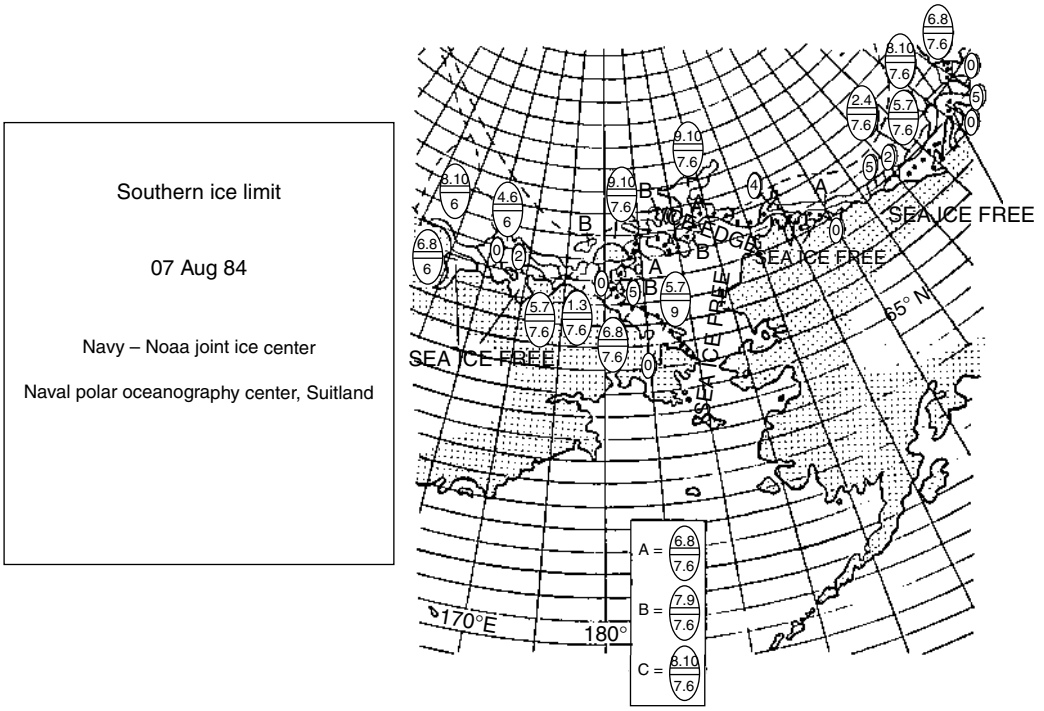

FIGURE 23.1
Sea-ice conditions for Western Arctic. August.

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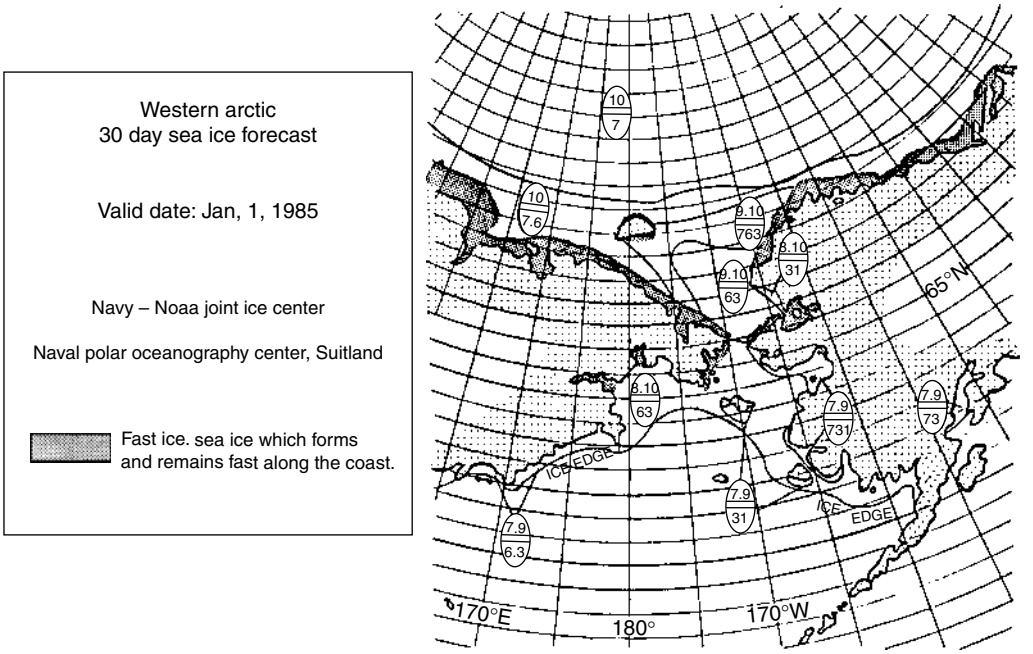

FIGURE 23.2
Sea-ice conditions, Western Arctic. January.

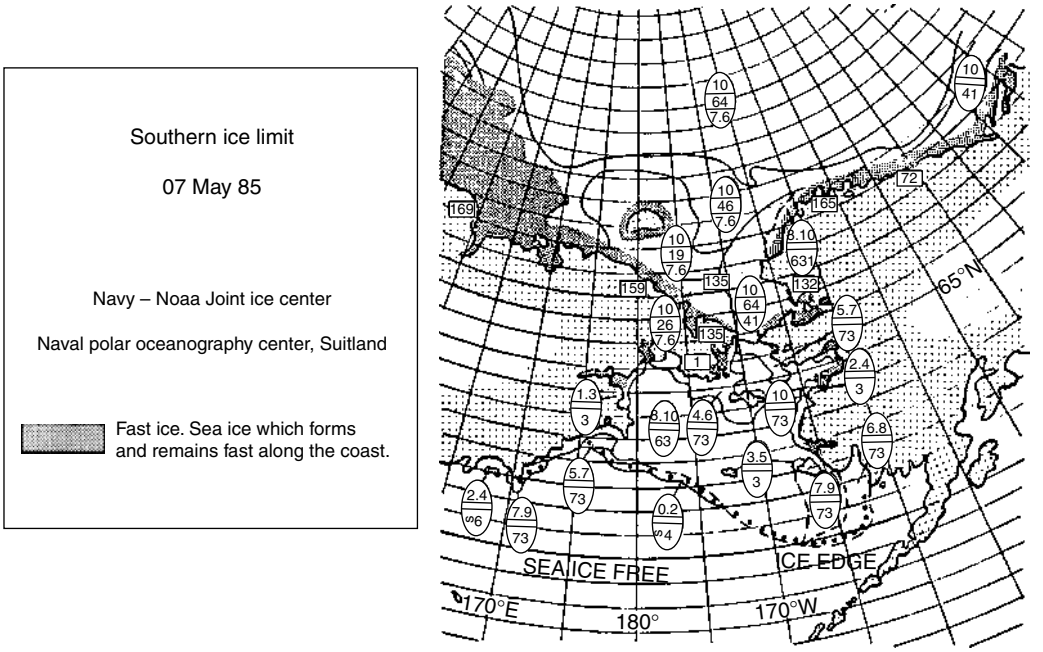

FIGURE 23.3
Sea-ice conditions, Western Arctic. May.

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Barrow. It is under these conditions that the successful tows of the Molikpaq and CIDS (GBS-I) were made. These tows are described in more detail in Section 23.11 of this chapter on "Deployment of Structures in the Arctic." Figure 23.1 through Figure 23.3 show how the ice conditions in the Arctic and sub-Arctic vary with the seasons. The Joint Ice Center has also published an Ice Atlas giving typical ice conditions worldwide.

Around the periphery of the Arctic Ocean, in the shallow waters of the continental shelves, is the zone which varies from open water in the brief summer to annual ice in the winter. This annual ice is relatively immobile—hence its name, "fast ice." Between the pack and the fast ice is the highly dynamic "shear zone," dominated by both annual and multiyear ridges and by massive floes which break off from the pack. This is also known as the "stahmuki zone."

Sea ice pressures show a significant scale effect, decreasing for large and global contact zones to values of 1 MPa or less, but increasing significantly to as high as 4 MPa and even 6 MPa for small local areas. A pressure of 3 MPa has been used for design of large-diameter steel tubular piles. Icebergs, being fresh water ice, exert higher local unit pressures. When the moving ice encounters an obstruction, whether a shoal or a structure, rubble piles build up. These rubble piles may protect (buffer) the structure from further impact by floes, but, conversely, they remain as massive ice features well into the following summer, thus impeding access for resupply.

When the ice sheet is driven onto a sloping shore, it tends to ride up onto and over the beach. When the sea level has been raised by a storm surge, the ride-up may proceed 30–100 m inland from the normal shoreline. Ride-up may similarly be a threat to artificial islands and platforms.

Seasonally, the shear zone, which is where much of the potential offshore petroleum development lies, typically undergoes the following sequence:

July–September: open water, with several intense storms. One or more invasions by multiyear ice floes.

September–November: freeze-up. Thin ice forms, restricting movement of ice floes, but also restricting operations. Ice breakers and ice-strengthened vessels are needed.

November–May: winter ice conditions. Movement of ice slow and erratic, driven by wind and currents. Thick ice. Rafted ice.

May–July: spring breakup. Dynamic movement of ice, with both annual and multiyear features. Open leads. Formation of pressure ridges.

Within the Arctic polar pack are several anomalies. One is the polyna, which is a large open-water area which forms even in the winter. Another is that of ice islands and ice island fragments. These start as huge tabular bergs which break off from a glacial ice sheet on Ellesmere Island and are caught up in the polar pack. Being of glacial origin, they are freshwater ice. As they ground in shallow water, or as thermal and impact stresses cause fractures, they break up into smaller fragments.

In the sub-Arctic areas, other ice-dominated environments exist. Between Greenland and eastern Canada, in Baffin Bay, Davis Strait, in the Barents Sea, and as far south as Newfoundland, icebergs are a primary consideration icebergs also occur in the Norwegian and Barents sea. These are largely blocky bergs, ranging in size up to 10 million tons or even more. Their depth is typically limited only by the water depth. As they melt and break up, growlers of several thousand tons or so form, and finally the bergy bits, small enough to be accelerated by storm waves yet difficult to detect either visually or by radar. In the winter these bergs are locked in the annual sea ice which forms, yet movement of the agglomerated mass still continues. Other bergs ground and roll over or break up.

The worst incidence of icebergs is in April and May, which is also the worst time for fog. Annual incidence of bergs varies by almost two orders of magnitude with 100–2400

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per year crossing latitude 48°N. Reasons for this extreme variability are not clearly understood.

In the Chukchi Sea, the open-water season is 3–4 months long. The area is subject to frequent multiyear ice floe invasions, punctuated by intense storms.

The Bering Sea sees only annual ice. However, south of Nome, the ice is very dynamic, with sheet ice forming and then being driven south by the wind, leaving open water to form a new sheet. The sheets are rafted upon one another, and many annual ridges are formed. Ice driven up onto the shallow flats of the Yukon Delta forms monstrous rubble piles which last until late in the following summer. Then, in the early summer, the area is subject to occasional major storms, with high waves steepened by the shallow water and significant storm surges up to 3 m. Occasionally, the ice rubble mounds are lifted up to float off and drift northward from Norton Sound as "floe bergs." Farther south in the Bering Sea, in the Navarin Basin, the winter ice is less thick and of lower strength due to the higher water temperature. Due to the effect of storm winds, large annual ridges can still form, with consolidated zones in their mid-depth.

On the northeast sides of both Sakhalin (off the coast of Siberia) and Greenland, the sea ice forms closely spaced ridges, which constitute a compressive zone, presenting perhaps the worst ice conditions for structures and vessels. On this northeast coast of Sakhalin, southwesterly currents drive the ridges ashore and, at the same time, cause severe beach erosion. Large annual and occasional multiyear ridges are reported from the Kara Sea, as well as icebergs in the eastern Barents Sea.

23.3 Atmospheric Conditions

Several atmospheric conditions have a significant effect on construction activities in the Arctic. Low temperatures are, of course, a controlling phenomenon of the Arctic, reaching to −50°C in winter. Summer temperatures are typically 10°C. Water temperature is generally −2°C, rising in late summer to +8° near shore. Low temperatures affect not only operations but lead to a brittle failure mode for conventional carbon steels.

Fog tends to form at the edge of the pack ice during summer. This is due to the cold air off the ice flowing over the warm open water. Since the edge of the pack is never far offshore, this means that during the construction season, visibility can be severely restricted. Offshore eastern Canada, this condition may extend well out over the open sea. This type of fog tends to hang close to the sea surface and often is limited to 10–15 m in height, above which visibility may be fine. An even more serious impedance to helicopter operations is that of the "whiteout," when sky, land, and sea become one vast white haze, with no reference features.

Atmospheric icing of superstructures, boat masts, rigging, and crane booms can build up rapidly and create serious problems of topside weight. Typically it can build to 75–100 m of solid ice or 300 m and more of porous ice. It is most serious in the southern reaches of the sub-Arctic where the air has more moisture. API Bulletin 2N warns that accretion of ice on the superstructure of vessels or structures can cause local overstressing or reduction of overall stability by increasing topside weight and the exposed areas to wind. Atmospheric icing buildup can be reduced by using tubulars instead of shapes and by means of low-friction coatings such as polyethylene.

Radio communication can be adversely affected by the Aurora Borealis ("Northern Lights") and associated electromagnetic disturbances. These are usually most severe near the time of the equinoxes (March 15 and September 15).

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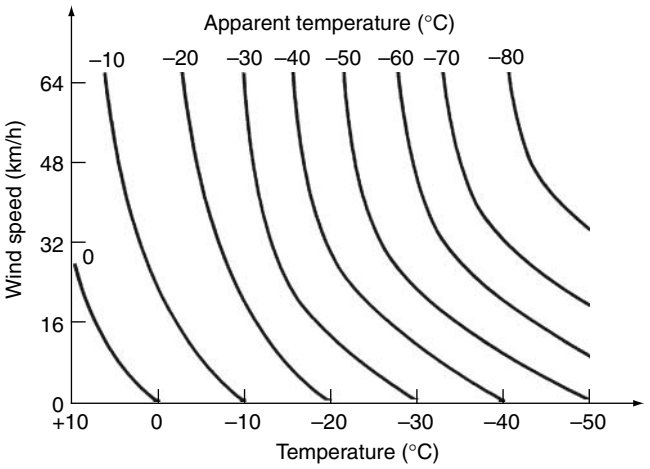

FIGURE 23.4
Apparent temperature due to wind chill.

Magnetic compasses are essentially useless since the north magnetic pole is on Baffin Island. Loran C coverage is poor in the Arctic. However, satellite navigation is available. The Global Positioning System (GPS) gives excellent navigational control.

Windstorms are often relatively local in extent and very intense, reaching 30 m/s velocity or occasionally even more, but with a usual duration of only 6–12 h. They are difficult to forecast and hence arrive with only a few hours' warning or no warning at all. When these winds occur at times of low temperature, wind chill effects can be extreme, reaching the equivalent of −70°C (see Figure 23.4).

23.4 Arctic Seafloor and Geotechnics

The seafloor soils of the Arctic are among the most complex and difficult in the world. Along the shores of the Arctic proper, the shelf is very shallow for a long distance out, up to 60 km and more, at which point the depth may be only 100 m. On this shelf are recent Holocene deposits of silt (from such glacier-fed streams as the Mackenzie), degrading to clayey silts and silty clays the farther the location from the river mouth.

These weak recent sediments may be from 2 to 20 m in thickness. Near the edge of the shelf, extensive slumping has been identified, even though the slope is very flat. Below these recent sediments, in many cases over-consolidated silts are encountered up to 10 m or more in thickness. These represent an extremely difficult problem for the constructor. They are so dense that it is almost impossible to drive a pile into them and extremely difficult to excavate or cut. Yet if they are broken up, the silt goes into suspension as an almost colloidal material. The most effective means of penetrating them is with a high-pressure water jet. Mechanical drills and cutters will work, but they experience excessive abrasion unless augmented by a jet.

These over-consolidated silts are in turn often underlain by dense to very dense sands. Subsea permafrost may be encountered, even well out into the shear zone. In the permafrost zone, the upper levels are usually partially ice-bonded, with lenses of ice, whereas at greater depths the sands are fully bonded.

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Between the top of the sands and the over-consolidated silt deposits described above often lies a thin stratum, only a few meters thick, where the silt is extremely weak, almost fluid. Many explanations have been proposed for this phenomenon; one is that the water from the melting permafrost and associated gases have permeated up through the partially bonded sands to be trapped under the impermeable silts that overlie, thus breaking down the silt structure and increasing the pore pressures. Shear friction values are reduced to near negligible.

Shallow methane gas has been encountered at a number of locations. Care must be taken in bore-hole drilling and pile driving to avoid accidents due to small explosions. Gas detection equipment is commercially available.

Occasional ice lenses have also been reported in the silt deposits, well above the body of subsea permafrost. Another anomaly is the reported presence of boulders on the surface of the seafloor, out near the edge of the shelf. These are reportedly of relatively small size (cobbles to small boulders) and hence may not be of significant concern.

The surface of the Arctic shelf, out to a depth of 50 m or so, has been repeatedly plowed by the keels of ice ridges. Furrows are generally 1–2 m deep and 10–20 m wide, with ridges forced up on the sides. Old furrows have refilled with loose sediments. The scour patterns are varied and crossed as the direction of ice movement has changed. It is postulated that the entire shelf, out to 50 or 60 m depth, has been continuously scoured over recent geologic times. Another scour phenomenon is that of strudl-scour. In the early spring, when the rivers thaw, large quantities of water flow out over the shore fast ice. Eventually they find a weak point and break through to the sea below. The velocities are high enough to scour a small crater, 20–40 m in diameter, 10 m or more in depth. Pingos, the hillocks that arise on the coastal plains due to frost heave, are also occasionally encountered underwater, as relics from the period when the sea surface was lower. Pingos can have a base diameter of 100 m and a height of 50 m. Forward-looking sonar should be employed when navigating in Arctic shelf waters. Relic pingos have left pock marks up to 100 m in diameter and 10 m deep on the floor of the North Sea and other sub-Arctic areas. These pock marks are believed to have been formed when the ice core melted, allowing the pingo to collapse.

Offshore Labrador and eastern Canada, glaciers have scoured the seafloor clean in many shallow areas. Scour marks from iceberg keels are evident at such locations as the Strait of Belle Isle. Deep scours at depths of 300 m or more have been found. These are perhaps similar to those found in the Antarctic out to a depth of 500 m, which are believed due to glacial tongues which have extended out into the sea in relatively recent geological times.

In Norton Sound, extensive sand deposits occur, often densified by wave action to depths of several meters. Overlying these sands is 1–2 m of very loose recent deposits from the Yukon and Kuskokwim rivers. These loose silty sands are very susceptible to movement by currents and waves. The underlying sands are often gas-charged with methane gas.

The Navarin Basin is composed of deep deposits of extremely weak silty clay deltaic deposits, susceptible to slumping even on relatively shallow slopes.

In the southern areas of the sub-Arctic, that is, offshore Newfoundland and in the North Aleutian Basin of the Bering Sea, very dense sands are encountered, the densification apparently being due to storm wave action. The sands at the location of the Hibernia platform had an internal angle of friction of 42°.

When natural loose sands support an offshore platform, the potential for liquefaction due to pore pressure build-up exists. This can be occasioned by the cyclic pounding by storm waves or by the continuous crushing of sea ice against either the wall of a gravity base caisson or the legs of a steel jacket.

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Another geotechnical phenomenon, of more concern to drillers than constructors, is that of clathrates. These methane hydrates exist as stable solids under natural conditions of temperature and pressure, but expand to 500 times greater volume as methane gas, once the pressure is reduced. Clathrates are believed to exist only at depths of several hundred meters or so. However, gas from decomposed clathrates may be one cause of the very weak silt stratum at the interface with the sands described earlier.

The sudden release of methane gas from clathrates during drilling may adversely affect foundation soils and may cause severe downdrag on piles and conductors. Filling of the resultant crater may accentuate the downdrag problem. Drilling techniques have been developed to prevent this phenomenon from occurring—for example, by allowing early release of pressure before casing is installed. The sudden release of gas, known as "blow-out" may reduce the water density and hence its buoyancy, allowing the vessel to sink.

The Arctic is generally an area of low seismicity. However, there are local areas of seismic activity such as near Camden Bay on the North Slope of Alaska and an active fault in the northeastern Beaufort Sea, just west of the Canadian Arctic Islands. Earthquakes in this last area may affect structures and facilities in the eastern Beaufort Sea as far south as Tuktoyaktuk, thus requiring consideration of such phenomena as liquefaction of silts and sands. Sakhalin lies between two major subduction zones and has experienced many major seismic events. The Mid-Atlantic rift runs east of the underwater plateau of northeast Greenland, but at least one epicenter has been recorded in the area of potential interest.

In the southern Bering Sea, earthquakes must again be considered due to the proximity of the active plate margin just south of the Aleutian chain. For the constructor, operations in these seismic environments may involve more strict requirements for gradation of imported sands for ballast and for their densification and consolidation to prevent liquefaction.

The shoreline of the Arctic coasts and barrier islands is especially sensitive to physical and thermal changes. The permafrost line is near to the surface, actually only 300 m or so below the onshore tundra. It is this frozen soil which protects the coast, often forming a low bluff under summer wave action, with a thin layer of gravel on the beach proper. Excavations and other constructional activity can upset this equilibrium, leading to large-scale and progressive erosion of the fine sediments.

The barrier islands of the Beaufort Sea are under a more or less continuous process of erosion and deposition. Along the North Slope of Alaska, the currents are driven by the Beaufort gyre. Man-built structures, islands, and groins interrupt this longshore process locally, leading to accretion on one side, erosion on the other.

An area of current development is the northeast coast of Sakhalin. It is dominated by sea ice up to 2 m thick for up to 6 months per year, as well as high seismicity.

23.5 Oceanographic

The frequent storms in the summer affect the open-water zone, which has very limited fetch in the north–south direction but much greater fetch in the east–west direction. Typically, the waves will be of only 2 m significant height, but when the ice pack has been driven northward, so as to leave 200 km of open water, higher waves are possible.

The wave period will be similarly affected, being usually 5 s but increasing to 6 or 7 s when the open-water fetch is extensive. Waves will tend to be quite steep, due to the shallow water in the zones of most current construction interest.

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Around structures and especially embankments, refraction patterns and directional spread from the rapidly moving storms may create confused and turbulent seas. The high winds pick up the top of the breaking waves as spray. The spray may become of significant amount, transporting several hundred tons of water onto a structure during a storm, creating problems of drainage. The drains may be clogged by ice. Since the spray can shoot up to 30 m or more around a structure, it can pose problems for helicopter rescue during a storm. During early fall, when the sea is still open but the air is below freezing, severe icing problems may ensue. The durations of these storms are usually relatively short, with a total duration of 24–36 h and a period of peak intensity of 6 h or less.

There are several interactive phenomena which may lead to significant storm surges, creating both raised water level ($+2$ m) and depressed water level (-1 m). These differences in water level are caused by the low barometric pressure, the strong winds, and the currents. The effect of surges is abnormally great in the Arctic because of the long, shallow shelf and the narrow "river" that exists for the water to escape between the land and the pack ice. This same combination leads to rather severe currents in the open-water zone, especially between Harrison Bay and Point Barrow, where the bathymetry and close presence of the ice pack lead to currents of almost constant velocity over the full water column from the surface to the seafloor.

The Bering Sea is subjected to severe cyclonic storms, especially in its southern stretches. As the storm-driven waves reach the shallow waters of the Yukon delta and Norton Sound, they are depth-limited, becoming very short and steep and piling up several meters of storm surge.

23.6 Ecological Considerations

The Arctic has its own unique ecology, with the preponderance of activity taking place on the coastal plains and in the adjacent open water. Large colonies of geese and other shorebirds nest along the coast. Within the leads and open water, photoplankton "bloom" in the summer flourishes, leading to an ecological chain that culminates in the bowhead and other whales, seals, and the polar bear. Native Inuits are also part of the ecology. Over the past 12,000 years, they have developed a unique culture that has allowed survival despite the harsh environment.

Construction activities and development obviously have an impact upon this ecology. Many impacts are positive. The indigenous population has better food, education, health, and communications than it has ever enjoyed. Many of the larger fish, birds, and mammals seem to be doing well, with little adverse effects on their numbers, although they have obviously had to adapt to the changes. The caribou, for example, have learned to cross the Alyeska pipeline, and the shorebirds and bears pay little attention to the helicopters.

However, other concerns raised by environmental protectionists are real and have resulted in rules which can and must be followed. Grayling and other fish migrate very close to the shoreline. Jetties and causeways may adversely affect them. Loud noises are believed to adversely affect the bowhead whales, at least if they are in the immediate vicinity, although there is a great difference of opinion as to distance and degree. Low-flying aircraft, high-powered boats, and the pumping of gravel through a pipeline are believed to affect nearby whales. Low-flying aircraft disturb nesting geese and ducks. Dragline dredging of gravel from rivers and river deltas must be carried out in furrows parallel to the flow, not cross-channel, so as not to interfere with fish swimming upstream to breed.

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Indigenous people of Sakhalin are concerned about the effects of submarine pipelines passing through the areas of spawning fish, their main source of food. They worry about not only the effects of construction but also about potential oil spillage due to damage to the line by the keels of ice ridges.

Great concern has been raised about oil spills, especially in the spring, when there is substantial coverage of broken ice. Industry in both Alaska and Canada has therefore formed special oil cleanup task forces to be able to respond quickly in case of a spill. The effect of oil in Arctic waters is a subject to which considerable study has been directed. Much debate has ensued as to the effects and the ability to contain and clean up. In the winter, the rough underside of the ice, especially the keels of ridges, will contain the oil. The cold water will cause the heavier fractions to congeal and drop to the seafloor. In the open-water season, the problem is similar to that in temperate zones. Fortunately, most bird activity is inland on the multiple shallow freshwater lakes and tundra, rather than on the sea proper. The spring breakup is the time of greatest concern, with current efforts by industry concentrating on development of a capability to contain and clean up any spill, even in the broken ice. Canadian authorities and industry have jointly carried out tests of burning spilled oil in broken ice.

A constructor will therefore be under special requirements to conduct the operations in such a way as to prevent disruption and adverse impact upon the ecology. This may limit certain activities that would otherwise seem appropriate. For example, many activities will be prohibited during the fall migration of the bowhead whale in September and October, which are the same periods when the constructor is trying to finish the work for the season. These limitations are regional and national in character, with some activities permitted in Canadian waters which are prohibited in Alaskan waters, and vice versa.

Two elements of the ecology are aggressive to humans, the polar bear, and the mosquito. The polar bear considers humans its natural prey. The animal is quick and agile on the ice and in and under the water. Isolated survey and geotechnical crews must be accompanied by an indigenous hunter ("bear watch"). Care must be taken in approaching an abandoned camp or facility. Bears are attracted to human activities and are very curious. They have been known to climb aboard vessels and rigs which were locked in the ice. They are attracted to food. Their keen sense of smell reportedly enables them to locate food over distances up to 50 km. The mosquito is less deadly, carrying no serious diseases, but nevertheless emerging in summer in such great numbers as to impede human activity. Fortunately, its realm rarely extends more than a half mile offshore.

23.7 Logistics and Operations

The Arctic is like the desert, with great distances between the small habitations, a generally flat and featureless terrain, low precipitation, great extremes of temperature, a small indigenous population, and very little infrastructure to support activities. Aggravating the problems is the short construction season.

Point Barrow, at the northern tip of Alaska, is a critical logistics pivot around which all floating structures, barges, and vessels must pass en route to the Beaufort Sea. Point Barrow itself is some 2200 miles from the Pacific Northwest of the United States, almost 3000 miles from Japan and Korea. Vessels and floating structures must cross the North Pacific just to reach the passes through the Aleutian Chain and then proceed through the shallow but sometimes rough waters of the Bering and Chukchi Seas to reach Barrow (see Figure 23.5). At Barrow, the ice usually recedes about the first of August. Thus passage is delayed, even though farther east the Beaufort Sea may have had partial open water in late

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FIGURE 23.5
Towing Global Marine Arctic drilling platform past Point Barron. (Courtesy of Global Marine.)

June or early July. The polar pack never recedes far from Point Barrow, thus restricting the channel to a few miles. Unfortunately, these few miles have limited water depth (about 10 m) extending out some 7 miles. The channel typically remains open until about September 15, when freeze-up starts.

The other water route to the Beaufort Sea is by barge down the Mackenzie River, which generally breaks up in mid-June, thus allowing access 6 weeks to 2 months earlier than Barrow—but, of course, restricted as to size of cargo. The Alyeska Pipeline Road is available for commodity shipment through most of the year, with limitations in late spring due to frost heave. Air service is generally good to the Beaufort Sea, with adequate fields and communication. Planes have landed successfully on ice airfields. Airdrops have been used for many commodities—even timber piles, for example.

While other areas of the Arctic and sub-Arctic may have somewhat less demanding logistical requirements, none is easy or inexpensive. Staffing points are required, support bases must be developed, and winter layover areas must be established. Shore bases must have a year-round water supply, year-round fire protection, provision for disposal of garbage so as not to attract bears, and provision for sewage disposal. This last will generally be chemical, since the permafrost is near the surface, preventing drainage into the soil. Sudden winds can be very serious, especially if they collapse or tear off a roof in winter, when the outside temperature is $-40°C$.

Because of the conditions described above, aircraft and special types of vehicles have been extensively used in construction. Several thousand rock gabions were transported by helicopter to provide additional protection against wave action for Tarsiut Island. Icebreaker supply boats have been used to haul construction materials and equipment. Fixed-wing aircraft, up to the C-5 Hercules, have made airdrops on the ice. An air-supported vehicle of the hovercraft type has proved its ability not only to ride over

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both ice and water but incidentally to use its air to break thin ice ahead of itself, allowing barges and boats to operate as late as November 1. Rollagons, large trucks with huge tires, softly inflated, have been used in snow and on the tundra, causing almost no damage to the fragile tundra.

Helicopters have been used to tow loaded hovercraft and ice sleds carrying cargo. An Archimedes-screw-propelled vehicle has proved able to cross both open water and ice in essentially all seasons. For under-ice surveys, ROVs appear to offer the best potential, although under-ice diving has been extensively employed in the Arctic Islands of Canada. An ice-strengthened cargo barge pushed by two pusher tugs has proved effective in opening the channel around Point Barrow in thin ice.

Meanwhile, advanced icebreakers for industrial and commercial support are being built and deployed by Canada, Finland, and Russia. The U.S. Coast Guard icebreakers are unfortunately designed for multipurpose activities, making them largely unavailable for Arctic marine construction. The offshore petroleum industry has developed designs for icebreakers and ice-strengthened supply boats ready for procurement as the need arises. Construction operations in the Arctic must be planned in extreme detail. Literally each tool and each material item needs to be considered as to where and when it is needed, how it gets there, and how it will be used in the extreme cold, wind chill, and darkness. Equipment needs to be specifically designed for the Arctic. Ordinary greases freeze, so silicon grease must be used instead.

23.8 Earthwork in the Arctic Offshore

Most of the offshore structures constructed to the date of this writing have been in the shallow waters of the Mackenzie delta and adjacent Canadian Beaufort Sea and in Prudhoe Bay. The Canadian structures have been largely built of sand, dredged, and placed by hydraulic dredges when a sand source could be found nearby, or with trailing suction hopper dredges when the source was distant. In some cases, the source has been 50–80 km away from the site. Many of the islands north of Alaska have been constructed of river gravel. The deepest water in which such an island has been constructed to date is 19 m at Issungnak, and 20 m at Mukluk, both of which involved large quantities of material (see Figure 23.6).

Deposition has been largely by hydraulic or barge discharge at the surface, resulting in very flat side slopes, as flat as 15:1. Use of special discharge tremie pipes, with devices at the tip to slow the velocity, has resulted in denser placement of sand and steeper slopes, about 5:1 to 6:1.

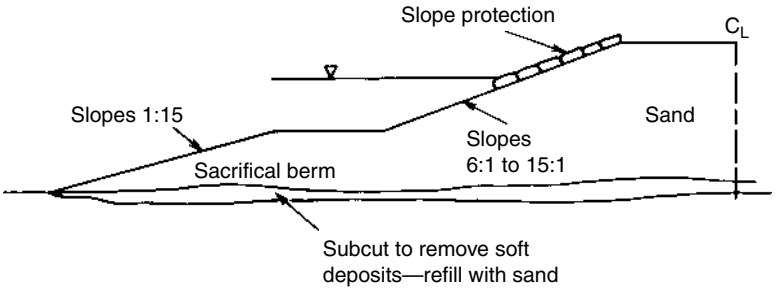

FIGURE 23.6
Typical sand island.

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It is, of course, not only a matter of placement technique but also of gradation which determines the slope and density. Some islands have experienced progressive liquefaction when they were built up too rapidly with material that did not achieve the required density. In one case, multiple flow slides were triggered, one after the other, and the embankment spread out so far that it had to be abandoned.

Slope protection is required as the island emerges from the water. The sudden storms of the open-water season can quickly erode thousands and even tens of thousands of cubic meters of sand. One solution that has been adopted—with only moderate success, however—has been to build the island up to just below sea level, hold until late in the season when most of the storm activity is over, and then rapidly complete the above-water portion.

The short duration of the open-water season requires a large dredging capacity and continuous operation. Trailer suction hopper dredges with ice-strengthened hulls have been able to work up to November 1 in the eastern Beaufort Sea.

One potential problem for above-water embankments is the inclusion of ice fragments in the fill. These may thaw in the following summer or when hot oil is produced, leading to sudden slumping. This has occurred in similar construction as far south as Cook Inlet.

For the Alaskan Beaufort, gravel has been available from the river deltas of the Sagavanirktok, Kuparak, and Colville Rivers. This gravel has often been transported by ice roads on the fast ice during January, February, and March for direct dumping through the ice. Side slopes of 3:1 are achieved. For islands further offshore, such as Mukluk, the gravel was stockpiled in the winter and then hauled by a fleet of barges for placement the following summer. When scraped or pushed off barges, gravel has achieved slopes of about 5:1 (see Figure 23.7).

Slopes have been protected by the use of plastic bags, filled with sand, placed in one or two courses over a heavy filter fabric. The bags are UV-stabilized, 2–4 cu.m. in size and double-strength. These have worked well for temporary exploratory islands but suffer tearing and dislocation from the ice during ice movement and breakup, requiring substantial annual maintenance (see Figure 23.8).

Mukluk in Harrison Bay, unfortunately, found no oil and was abandoned. Within a few years, all above-water traces of the island disappeared due to storms, except for one lonely concrete caisson which had been installed as an offloading dock.

Articulated concrete mats, attached to or laid over filter fabric, have performed well in tests and appear to be more permanent and require less maintenance than sandbags. As a result, they have been adopted for use on North Star Island and Endicott Island, two of the first production islands to be built in the offshore Alaskan Arctic and reportedly at Northeast Sakhalin (see Figure 23.9).

Freezing of embankments, both above and below water, has been studied by many engineers. Natural freeze-back occurs in above-water and near-surface soils. Artificial freezing has been proposed to stabilize and strengthen underwater embankments.

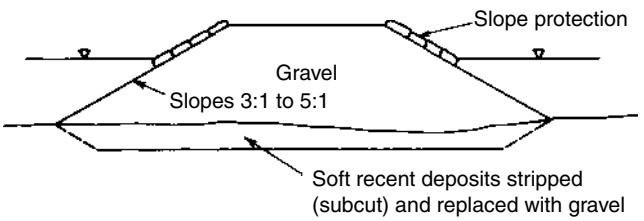

FIGURE 23.7
Gravel island.

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FIGURE 23.8
Tarsiut Island in Beaufort Sea, caisson-retained. A sand island with slope protection by sand-filled plastic bags. (Courtesy of Dome Petroleum, Inc.)

However, there are several difficulties. First, although the natural air temperature is low, the heat of fusion is the same whether in the Arctic or in a temperate zone. Second and more seriously, there is little experience with saline soils. Such as there is indicates that brine channels are formed, with the freeze front progressively driving increasing brine concentrations ahead to form unfrozen brine lenses and hence potential failure planes. A construction dike formed around the site for the Prudhoe Bay Salt Water Treatment Plant (for the Waterflood Facility) was designed on the basis of natural freeze-back. Portions of this failed, however, apparently due to unfrozen lenses of high-salinity water.

To freeze an embankment artificially, freeze pipes can be installed from the embankment or structure in drilled or jetted-in pipes or laid as mats between layers of embankment. To be effective in increasing shear and bearing capacity for Arctic offshore structures, the temperature within the embankment should probably be lowered to at least $-10°C$. Frost heave during freezing must be considered. The adverse effects can probably be minimized by following a progressive pattern to drive the expansion out from under the structure. Freezing takes time, usually several months on a large installation such as those contemplated for offshore structures. Once frozen, the condition can be rather easily maintained by either active brine circulation at low volumes or by passive means such as the thermopiles used on the Alyeska pipeline in Alaska. Frozen soils sustain short-term loads well

FIGURE 23.9
Articulated concrete armor in place on gravel island in Beaufort Sea. (Courtesy of Coastal Frontiers, Inc.)

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but have high creep under sustained loads such as may occur under winter ice movements.

The soft, silty clay soils of the Arctic seafloor place severe requirements on concrete and steel gravity-base structures designed for use in the shear zone. Therefore, a great deal of attention has been devoted in recent years to the development of effective and economical methods of improving the soils. Since the silty clay and clayey silt sediments are anisotropic in character, with greater horizontal than vertical permeability, consolidation through drainage and surcharge appears attractive. To accomplish this requires the placement of an impervious membrane, and the installation of wick or sand drains. Practical considerations indicate that perhaps the best way to carry out the consolidation may be after the structure is installed, using the base of the structure plus its skirts as the membrane and draining it to the interior.

It is also considered practicable to install the vertical drains prior to arrival of the structure. An initial sand or sand and gravel embankment may be placed over the impermeable surficial soils of the seafloor. Another scheme is based on the installation of sand piles prior to arrival of the structure, designed to enhance the bearing and shear resistance of the soil initially, to be supplemented by drainage and consolidation later. Such underwater sand piles might be installed using a 1-m-diameter steel pipe mandrel with a hinged or expandable plug in its tip. This would be driven to the required depth by an impact or vibratory hammer. During driving, the mandrel would be filled with sand by means of a hopper. The top of the pipe would then be closed and low air pressure applied to keep the sand in place in the hole while the mandrel is withdrawn. Only 10–20 psi of air pressure is required. This method is similar to the consolidation operations employed at the Kansai Airport in Osaka, Japan, where multiple mandrels were mounted on a barge.

A method being applied for the Venice Storm Surge Barrier and which was successfully employed on the Rion-Antirion Bridge may be applicable in certain Arctic cases as well. This is the driving of closely spaced piles to consolidate the soil and to tie the strata together as a continuous block. Gravel or crushed stone is then dumped over the piles, embedding their tops. The piles are not connected to the structure except through shear. Piles may be timber, precast concrete, or steel.

Dredging of the softer materials and their subsequent backfilling with imported sand is a highly practicable and effective solution if the depth of the soft material is limited to a few meters and if the soft material will stand on reasonable side slopes during the dredging. Both these requirements are met in many areas of the eastern Beaufort Sea. The advantages of such a system are that it is positive, and that it is not affected significantly by the characteristics of the weak soils that will be removed. Densification may still be required to prevent liquefaction.

Surcharging around the perimeter of a structure is an effective means of enhancing the resistance to sliding by extending the shear path. This surcharge can best be applied after the structure has been installed and thus can use the structure as a reference marker for control of the dumping operations.

Another means of enhancing shear resistance is that of penetration, in which the structure is equipped with long skirts to penetrate through the soft soils. In the soft soils of the Arctic and sub-Arctic such as the Navarin Basin, the penetration may be aided by removal of the soft soils from within the skirts, for example, by jet and airlift or by jet and eductor (see Figure 23.10). The eductor is essentially independent of column height, whereas the airlift is most effective with a high column.

In deep water, penetration may be increased by draining the pressure from the underbase zone, thus utilizing the hydrostatic forces to push the skirts to greater depth.

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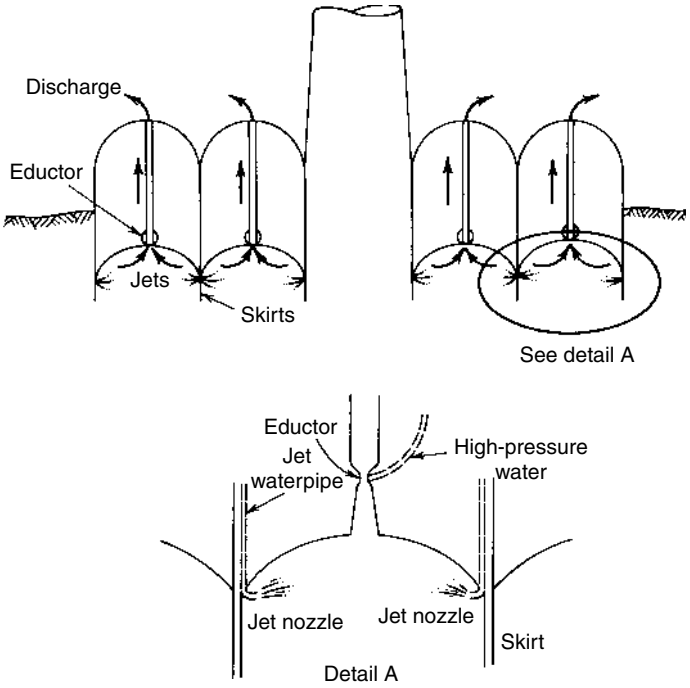

FIGURE 23.10
Use of jets and eductors (or airlifts) to remove soil from under skirt compartments.

23.9 Ice Structures

It is only natural that early attention was given to the use of ice as a structural material for temporary structures in the Arctic. In World War I, a considerable amount of research was directed toward a mixture of ice and sawdust, known as Pi-Crete, which it was hoped would be suitable for use in constructing temporary floating airfields to enable the planes of that day to transit the Atlantic. In more recent times ice has been utilized for ice roads out over the fast ice and as temporary drilling platforms in the fast ice wedged in the channels between the Arctic Islands. The ice roads are constructed by progressive flooding, allowing each layer to freeze. Dikes are built of snow and broken ice rubble. The water is obtained from holes drilled through the ice. Such roads have carried heavy trucks and have been used for the delivery of gravel to offshore sites. The gravel is then dumped through slots cut in the ice at the site. The movement of trucks, especially when in convoy at fairly high speeds, can produce resonant undulations in the flexible floating strip and lead to failure. Spacing and speed need to be controlled to prevent this.

API Bulletin 2N, in the Section "Design of Ice Roads," states: "Attention is directed to the problems of edge loading, and of cracks opening due to tidal, thermal, wind and other forces. Thermal cracks generally appear in ice roads following construction. Such cracks are particularly noticeable following snow clearing and large changes in temperature. Wet cracks may heal—dry cracks often have to be repaired with slush or water. A particularly dangerous situation can occur if two wet cracks join to form a wedge. Attention is directed to the spacing of vehicles traveling in the same and opposing directions, to dynamic

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amplification due to waves created by the vehicles, and to fatigue, caused by heavily loaded trucks traveling at close intervals. The life of drilling pads built up of layers of ice by progressive flooding and freezing has been extended into the early summer by the use of polyurethane insulating blankets.

Considerable experimentation has been carried out by Union Oil, Exxon, Sohio (BP Alaska), and Amoco in developing artificial ice islands for use as year-round drilling islands in the shallower waters of the Beaufort Sea. Water has been sprayed out over the ice using the large circular spray systems more usually employed in farm irrigation. This allows almost instant freezing and thus a continuous operation. A major problem with these islands occurs in spring, as the opening water thaws and erodes under the perimeter ice edges, causing the resultant overhangs to break off. Other problems encountered are propagating cracks due to thermal stresses.

The behavior of frozen soils and embankments must be monitored and repair operations undertaken promptly. Cracks in the frozen structure may be repaired by filling them with water. An adequate internal temperature monitoring system is required. Maintenance of desired internal temperatures may be achieved by:

a. Balancing natural cooling and heating
b. Selective insulation, such as installing a cover in the spring, and removing it in the fall
c. Convective refrigeration, such as "freeze piles"
d. Active refrigeration

Artificial ice rubble is being generated around more conventional drilling structures of steel and concrete in order to provide a cushion against the impact of large ice floes. This has been done in several ways:

1. Constructing an underwater berm or embankment of sand or gravel on which the ice features will ground, thus progressively forming an encasing rubble pile.
2. Accelerating this natural action by mechanically adding ice to weight the ice sheet adjacent to the structure until it grounds. This can be done by icebreakers pushing the ice ahead during the early winter or by bulldozers working on the ice that is at least temporarily fast to the structure. Cranes have been used to lift and place blocks of ice cut from the adjacent sheet.
3. Spraying water out from monitors mounted on the structure. The water freezes as it falls through the air and quickly generates massive rubble. This system was developed by Sohio (BP Alaska) and Exxon and was used to provide a rubble encasement around the Super CIDS exploratory drilling structure at a location offshore Cape Halkett, Alaska, in a water depth of 16 m. Amoco has successfully constructed an exploratory drilling platform of ice in 7 m of water.
4. Considerable experimentation has been carried out by Sohio (BP Alaska) on the use of steel dolphins (pyramidal frames) seated on the seafloor, designed to intercept the moving ice and cause it to raft, thereby forming a rubble pile.

The use of ice for temporary structures will undoubtedly be extended as further work is carried out in the Arctic. Its limitations are due to the fact that ice is a solid at a temperature near to its melting point, that it is subject to brittle fracture, and that thermal strains and water erosion can lead to early and sometimes sudden failure. To overcome these, research is investigating potential means of reinforcement.

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23.10 Steel and Concrete Structures for the Arctic

A wide variety of structures of steel and concrete have been designed for the Arctic and sub-Arctic, designed to resist the high lateral forces from the ice and to transmit these forces down to the foundation soils.

23.10.1 Steel Tower Platforms

Tower-type structures have been used for many years in Cook Inlet, Alaska. These steel structures are constructed like jackets, but with legs of much larger diameter. They are built in a fabricating yard, launched and delivered as a self-floating structure and upended to seat on the seafloor, with steel piles driven or drilled and grouted, either through the large legs or as skirt piles (see Figure 23.11). These structures are suited for moderate depth waters and moderate ice conditions such as those of the southern Bering Sea. The legs of the jacket must be spread widely apart to prevent the ice sheet from arching between them, leading to rafting of the ice sheet and potential vibration. This can be so severe as to lead to fatigue. Through the ice zone, the legs require special reinforcement, e.g., sandwich steel-concrete construction, in order to resist ice impact. Conductors are enclosed within the legs for protection against the ice.

23.10.2 Caisson-Retained Islands

Caisson-retained islands have been developed as an extension of the surface-piercing island concept described earlier. The perimeter caissons provide protection through the

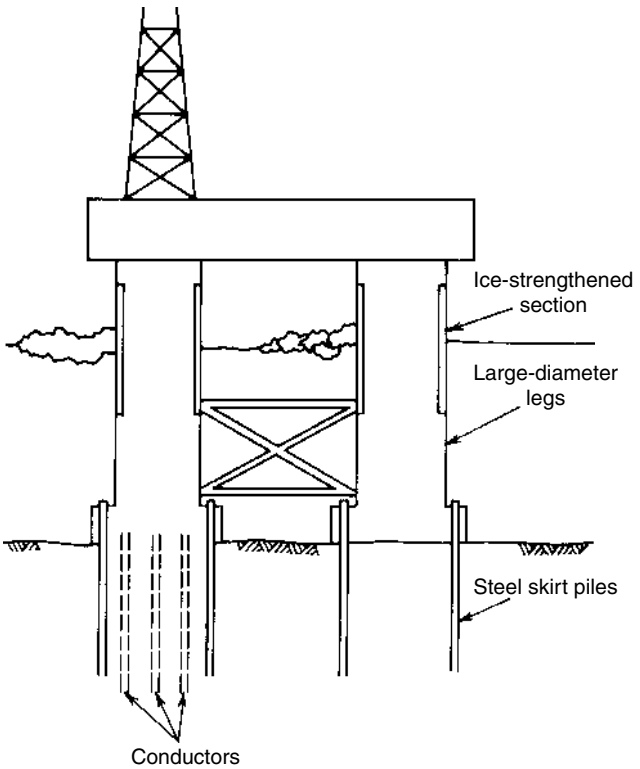

FIGURE 23.11
Tower structure for Cook Inlet, Alaska, and other moderate sea-ice conditions.

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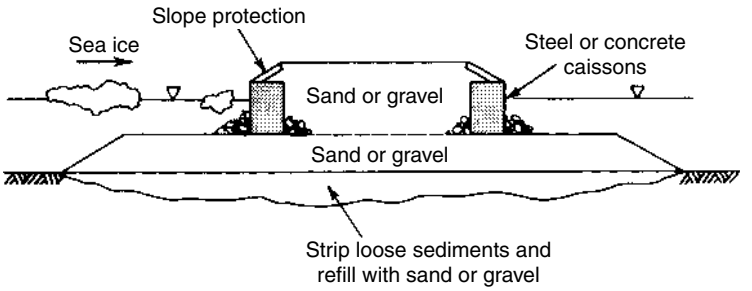

FIGURE 23.12
Caisson-retained island.

air–water interface, preventing wave erosion during construction, reducing fill quantities significantly, and facilitating access to the structure. They reduce the overall construction time by permitting concurrent construction operations (see Figure 23.12).

The caissons themselves may be of steel or concrete, filled with sand. Junctures are constructed between the adjoining caissons to prevent them from being wedged apart by the ice.

A concrete caisson-retained structure of modified density concrete was built for Tarsiut Island while a steel caisson-retained structure was employed by Esso, both in the Canadian Beaufort Sea. The junctures have turned out to be the critical design and construction problem. During the joining, tolerances must be accommodated in all six degrees of freedom. In service, the joints must be sufficiently closed to prevent excessive spray and to prevent ice wedging. A series of concrete caissons, arranged in a zigzag pattern, have been proposed as an offshore breakwater, to create a harbor for icebreaking support vessels. A large caisson-retained island, using concrete caissons, was designed for Amauligak.

23.10.3 Shallow-Water Gravity-Base Caissons

Gravity-base caissons are large structures of concrete and/or steel founded either on the existing seafloor or on a prepared underwater embankment. The Dome SSDC-1 was such a structure, constructed from a section of a Very Large Crude oil Carrier (VLCC) hull, reinforced internally by concrete and steel. The Global Marine Super CIDS (GBS-1) is a gravity-based caisson constructed of three sections, the base and deck being steel, the midsection being of prestressed lightweight concrete, with the three sections joined together to act monolithically. The Gulf Canada (now Chevron) Molikpaq is a gravity-based caisson of steel, filled with sand when on site.

The CIDS was built in a shipyard in Tsu, Southern Honshu. That portion of the structure which would be in the ice belt was of reinforced lightweight concrete with the ice wall prestressed on two axes, vertically and axially. This structure was mounted on a steel barge. Then a steel superstructure, fully integrated, was set on top. The structure was towed through the North Pacific, then the Bering Sea, and finally seated in the Beaufort Sea, where drilling was carried out for two seasons before the structure was refloated and stored. Recently, 2003, it was towed to a shipyard in Eastern Russia where a deeper steel underwater hull was fabricated. After remounting, the CIDS was installed in the coastal waters offshore northeastern Sakhalin, where it will be exposed to the violent storms in fall and the compressive ice pressure ridges in early spring. Two larger concrete ciassons, one for 48 m of water depth and the second for 30 m, have been built in Vladivistok and towed

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1700 km to sites off the northeast coast of Sakhalin. Large base rafts with heavy shafts are submerged under the ice zone, while comparatively heavy shafts penetrate to support the deck. Sakhalin 2 in 48 m of water has a base raft 105 m × 88 m and 13.5 m thick, supporting four legs of 24 m diameter and 40 m high. It displaces 100,000 tn.

One model employs a hybrid design, with steel on the inside of the peripheral wall, joined in composite action with a concrete exterior wall. Many concepts have been developed, but they all tend toward axisymmetrical structures, circular or polygonal in plan, with sloping or vertical walls. The cone is designed to fail the ice in bending (flexure), whereas the near-vertical-sided caissons are designed to fail the ice in crushing and shear. The "stepped pyramid" concept enables the extension of the gravity-base structure to deeper water.

Additional resistance against sliding in weak soils can be developed by the use of piles or spuds, such as those designed for the Sohio (BP Alaska) Sohio Arctic Mobile Structure (SAMS). The SAMS was designed to be installed as a GBS. To enhance its shear resistance, steel tubular piles (2 m diameter.) were to be driven into the firm clay about 8 m below the seabed. The weight of the caisson bearing on the seabed would increase the shear resistance of the spuds. For subsequent removal in the succeeding open water season, steam was to be injected into the piles, to release any frozen soil. Then a cap would be fitted to the top and the pile filled almost to the top with water. Air pressure over the water would jack the pile out. To prevent water from escaping under the tip, 1–2 m of clay would be left in the pile tip. Once all the piles were removed, the structure could be floated for re-installation at another exploratory location.

23.10.4 Jack-Up Structures

Considerable conceptual design effort has been given to the development of jack-up rigs of various types designed to float into location on a large hull, which will then be seated on the soil, raising its deck above the sea and ice. The narrow legs then present a minimum face to the ice and are intended to reduce ice forces acting on the structures.

While the concept may eventually be developed into reality, none has yet been built because of concerns over whether the extreme ice loads, such as those from a multiyear floe or a short heavy ridge, arching across the legs of the structure, will really be reduced by the narrow shafts and because of potential problems of dynamic amplification under continuous ice crushing, which has led to the failure of several lighthouses in the Baltic Sea.

23.10.5 Bottom-Founded Deep-Water Structures

Bottom-founded structures for deeper water are being conceived as cones, stepped pyramids, or monopods in order to reduce quantities of material, limit the weight on the seafloor, and reduce the ice forces. The shafts must be able to withstand the impact of the relevant ice features. Their construction and installation would be similar to those for the North Sea gravity-based platforms (see Figure 23.13 and Figure 23.14). Installation of these structures requires very careful calculations of stability as the structure is ballasted down, due to the decreasing water plane. Means of countering this, such as the use of temporary buoyancy tanks, are described later in this chapter, Section 23.12, "Installation at Site."

The Hibernia platform off Newfoundland is a vertically sided cylinder, 90 m in diameter. It is constructed with a fluted exterior, with the teeth designed to penetrate the oncoming iceberg by concentrating the resisting forces at discrete points, thus

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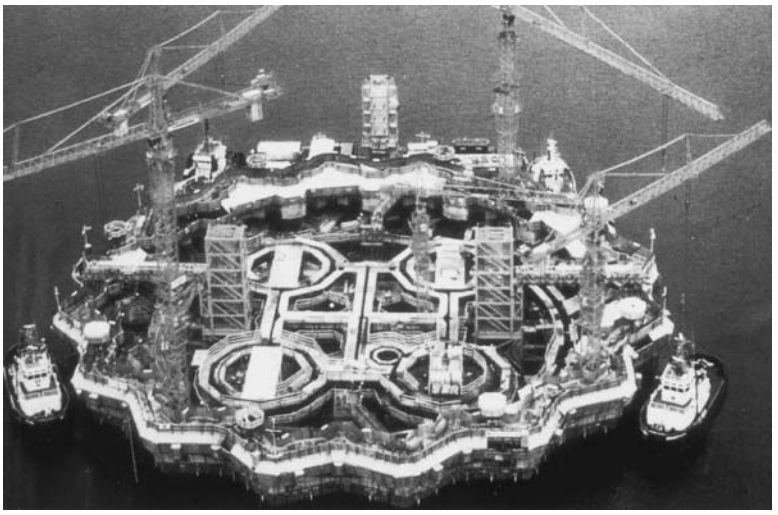

FIGURE 23.13
Hibernia gravity-base structure under construction while afloat. This structure is designed for iceberg impact. (Courtesy of Mobile Offshore.)

extending the length and duration of impact. Maximum force is thereby reduced from 2200 to 1400 MN.

The Hibernia offshore platform was designed to resist the impact of large icebergs. It became apparent early in the conceptual stage that local punching shear, acting over a

FIGURE 23.14
Hibernia GBS near completion. Ready for mounting of deck. (Courtesy of Mobile Offshore.)

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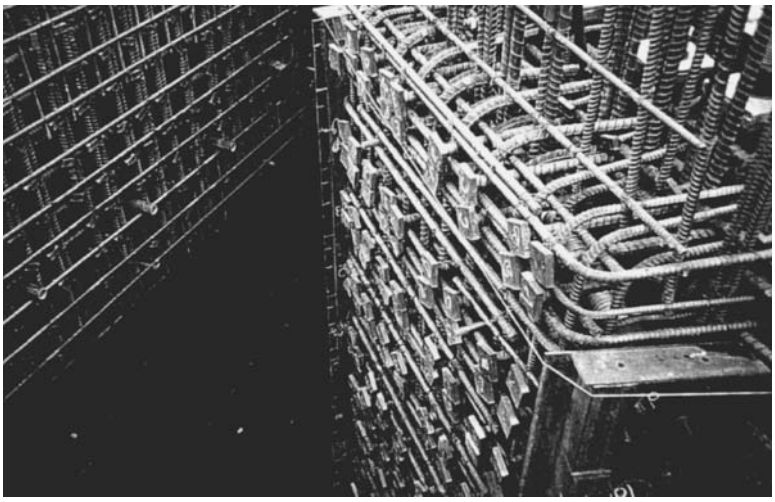

FIGURE 23.15
T-headed bars provide shear reinforcement to resist iceberg impact.

limited area anywhere on the external periphery of over 22,000 m^2 would need special attention. As the result of extensive research and testing, headed bars (T-headed bars) were arranged to extend through the wall at close spacing. These proved effective in providing both elastic and especially post-elastic ductility (see Figure 23.13 through Figure 23.15).

Modified-density concrete was employed in a major portion of the structure to reduce draft during deployment.

23.10.6 Floating Structures

For deeper waters, floating structures have been developed to be permanently or temporarily moored to high-capacity anchors. These structures generally resemble the caisson structures and are built of steel, concrete, or a combination of the two. The Kulluk exploratory drilling platform, designed to work in moderate sea ice, is a floating inverted cone, so that the ice is broken and deflected downward. It was moored by 8 anchor lines. This structure was built of steel; an earlier design of this concept based on concrete has also been developed. The Kulluk has successfully extended its drilling season to the first part of December.

A spar-type structure in concrete was designed for the Navarin Basin in the Bering Sea, to break the ice upward or downward depending on the draft (see Figure 23.16). This structure was designed for sub-Arctic areas, with unconsolidated ridges up to 10–15 m.

For less severe ice conditions, ship-shape hulls with turret moorings have been proposed. These are designed to weathervane with the ice movement. Thrusters may be installed to optimize this rotation and to take care of adverse combinations of wind, current, and ice. The bow will usually be wider than the body to facilitate clearance of the broken ice. Sides of the hull may be inclined inward to break the ice downward.

The major problem for all floating structures in the Arctic and Sub-Arctic is the design of high-capacity moorings and anchors, since the global forces in heavy ice are several

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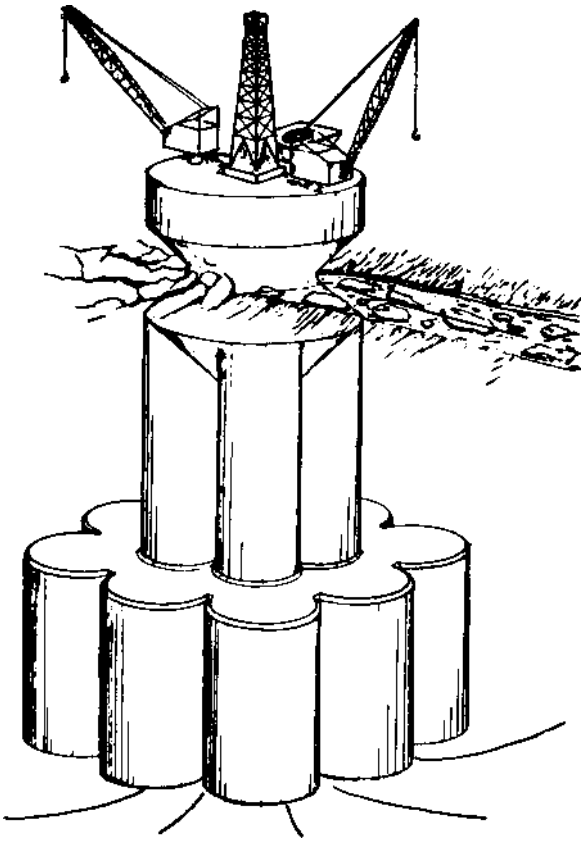

FIGURE 23.16
Floating SPAR for use in sub-Arctic waters. Designed to break ice both downward and upward.

times higher than those experienced from wave forces in such locations as the North Sea.

23.10.7 Well Protectors and Seafloor Templates

Once the structure is properly moored on location, a glory hole is usually excavated in the seafloor enabling well control gear to be placed below the level of potential ice scour. This has been accomplished by very powerful jets and by the use of a vertical dredge head. Glory holes are obviously well suited to floating drill vessels. Suction hydraulic dredges with long dredge ladders and powerful cutter heads have been used in the dense sands at offshore Newfoundland, in depths up to 80 m.

An alternative to the glory hole, better suited to seafloor-founded installations, is to sink a well protector caisson (of perhaps 10 m diameter) down into the seafloor using a combination of jets and the weight of the shell along with airlift or eductor excavation. Such well protector caissons can be either of steel or concrete. A double-steel shell can be easily transported to the site, hung under the drilling derrick, and pumped full of iron-ore slurry or concrete which will set at low temperatures. Then the weight of the fill is available to aid the penetration. Another alternative is the use of precast concrete segments, post-tensioned together vertically with prestressing bars. Off Newfoundland, large glory holes for subsea templates are excavated in the dense, partially cemented sands by vertical drills, having an enlarged drill bit (5 m) augmented by airlift. Most recently, at Hibernia, a large diameter (5 m) drill, with air assist, has been used, although the progress has reportedly been slow.

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23.11 Deployment of Structures in the Arctic

The various caisson-type structures are typically planned for construction in the warm-water ports of the temperate regions for delivery afloat to the Arctic. While the caissons for the Tarsiut Island Caisson Retained Island were delivered on a large submersible barge, most have been and will continue to be delivered as self-floating structures, complete and monolithic, fully outfitted, to minimize the need for subsequent operations in the Arctic.

The tow across the North Pacific, whether from Japan, Korea, or from the Pacific Coast of North America, is so long as to have a high probability of encountering a major summer storm en route, with the relative long-period waves typical of the Pacific. It is therefore necessary to investigate thoroughly the dynamic response of the structure in such sea states. While recent advances in hydrodynamic analysis, such as strip theory, make it possible to analyze the response of such a structure, it is difficult to include damping aspects without a physical model test. Fortunately, in many cases these will show that the theoretical amplifications are significantly damped, so that the response of the structure remains within acceptable limits. Special attention has to be directed to cones and similar structures where the wave run-up over the sloping surface may lead to erratic lurching.

In a major storm, the tugs may have to cast the structure free. The route should therefore be selected in such a way to keep adequate sea room. After the storm the tugs can pick up the tow once again. Free-floating hawsers (Kevlar or nylon) attached to spare pendants will facilitate the pickup. Most tows will need to be manned, in order to operate pumps if necessary. Full provision must be made for the safety of personnel on board, in compliance with the regulations of the relevant agencies, for example, the U.S. and the Canadian Coast Guards.

Because of the length of the tow, provision may have to be made for refueling of the tugs en route. The size and number of tugs will vary with the size and displacement of the structure, but most frequently more than one tug of high horsepower (greater than 16,000 HP) will be required. Arrangements for multiple tugs will be similar to those in the North Sea. Because of the longer length of Pacific waves, towlines will probably be longer. Axisymmetrical structures such as those used in the Arctic have little directional stability, tending to yaw excessively. Skegs will usually be found effective.

Upon arrival at Point Barrow, tows will stand by awaiting the opening of a suitable channel through the ice. The depth at Point Barrow inside the shoal 7 miles offshore is a maximum of 10–11 m. Unfortunately, the pack ice usually hugs this shoal until late August, leading to a low probability of successful passage around the seaward side of the shoal. When several class 4 to class 6 icebreakers become available, then deeper-draft tows will be able to pass north of the shoal. Their draft then will be limited only by the depth at the installation site.

The towing of large offshore platform structures in the shallow waters of the Arctic, under conditions of broken ice, presents many new problems (Figure 23.17 and Figure 23.18). However, it has been satisfactorily accomplished on several major wide-beam structures, especially the Prudhoe Bay Seawater Treatment Facility, the Kulluk, the SSDC-1, the Molikpaq, and the CIDS (GBS-1).

The Molikpaq and GBS-1 were towed in convoy, led by a class 4 icebreaker vessel. Ice conditions became extremely heavy as the convoy approached Harrison Bay. Towing with one boat on a diagonal proved most effective. Towline snagging on ice resulted in severe impact forces on the towline, attachments and boat. The tow speed averaged 1.3 knots over the almost 500 miles of distance from Point Barrow to Herschel Island.

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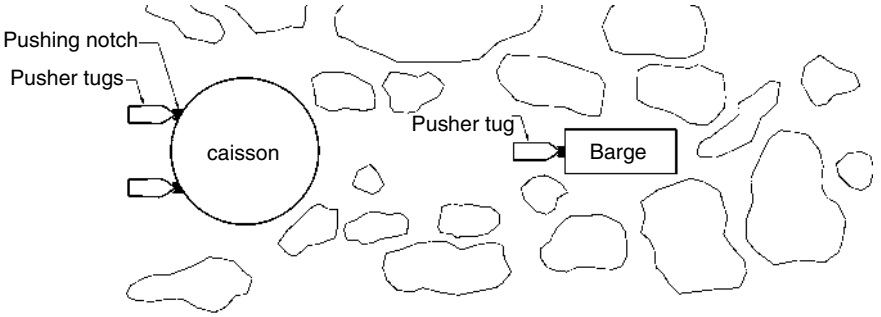

FIGURE 23.17
Towing Caisson through broken ice.

For the GBS-1, the tow from Point Barrow to Point Halkett took 4 days. Drafts of the two structures were 9–10 m.

From Point Barrow eastward through the Beaufort Sea the water depth is very limited. The underkeel clearance is minimal, so most ice must clear around the sides. If the structure is polygonal, towing with one point forward is best so that the broken ice will be forced to the side, preventing buildup of a rubble pile ahead.

Due to shallow water in the Beaufort Sea, sway, yaw, and squat are potential problems. In actual experience, low speed, the use of several boats, and the confinement of the ice have tended to minimize these adverse effects. Towing skegs may be effective in limiting the sway and yaw. The towlines for crossing the Pacific will normally have been secured below the waterline, slightly below the center of rotation, in order to keep a favorable trim, which is usually slightly down by the stern. For the Point Barrow passage, trim will need to be equalized in order to minimize draft. However, if the towlines are below water, they may tend to snag under ice blocks due to the irregular speed of the tugs in the ice. If attached above water, they may create an unfavorable trim down by the head.

FIGURE 23.18
Deployment of Kulluk floating exploratory drilling vessel in thin ice in Canadian Beaufort Sea. (Courtesy of Chevron Resources Canada Ltd.)

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On the Molikpaq tow, the initial passage was made by pulling with two icebreaker tugs. As one tug encountered heavy ice, it would slow. Then as the ice broke and the tug surged forward, the towline would snap up, causing severe impact on the fittings of both tug and tow and threatening to break the towline. In the case of the Molikpaq, they switched to using one icebreaker to break ice and another to tow, which improved the situation.

Crowley Maritime has used pusher tugs effectively to push an ice-strengthened barge to break a channel in thin ice. Consideration should be given to installing pusher notches and fittings on the stern of the structure itself and to using it as an icebreaker, since most structures are adequately designed to resist the impact of ice.

If, in the future, deep-water structures will have to be towed through or deployed in pack ice, then the structure may well be configured at the towing draft waterline to efficiently break ice and to deflect the broken ice to the side so that it will not jam the passage. Past experience (e.g., the Manhattan voyage) has shown that the ice pack can be opened to pass a 30-m-wide ship. In the future, structures may be over 100 m in beam. Where skirts are installed under the structure, an air cushion could be used to reduce draft during the Point Barrow passage. Since with partial ice coverage there is usually a low sea state, the "water plug" could probably be reduced to about 1 m.

API Bulletin 2N, Section 7.3, notes: "The effects of ice accretion on mooring lines, both above and below water, should be considered. Mooring line buoyancy and drag forces, and consequently tension forces, may be affected by icing."

23.12 Installation at Site

Caisson structures for the Arctic will usually be installed in a manner similar to that employed in the North Sea. Upon arrival at the site, the tugs will fan out in a star formation, controlling lateral position and orientation as the structure is ballasted down to the seafloor. Positioning is controlled by a combination of GPS satellite navigation, medium-range electronic-positioning systems such as SYDELIS, and seafloor transponders.

Some of the conical structures and monopods proposed for the Arctic encounter stability problems at this stage. There are two solutions. In waters of limited depth, a conical structure may be tipped down, keeping water plane stability until one side has touched down and then using the seafloor for stability as the structure is ballasted down to its permanent attitude. Such a system has been used in shallow waters in the Gulf of Mexico and is presumed suitable for exploratory drilling platforms in the Arctic. However, it is highly questionable for permanent production platforms because of the angle to which the deck and consequently the processing equipment must be tilted and because of potential disturbance to the foundation. The other method is to install temporary buoyant columns which give righting moment stability to the conical structure during the critical submergence phase. After installation, these temporary columns are removed.

For all gravity-base structures, as they near the seafloor, a cushion of water will be trapped underneath which must be vented internally or displaced laterally. Dowels or spuds can be dropped to engage the bottom and hold the structure in position. These spuds will have to penetrate the soil; in some cases they may have to be locked to the structure in order to use its weight to force penetration. The rate of descent can be reduced to allow the water to escape without creating piping channels. Slow descent and adequate venting of the trapped water will also help prevent lateral "skidding" of the structure.

The structure will be seated down upon a seafloor which overall is usually level, but which will have been plowed by ice scour and thus have numerous local deviations in profile and strength, with ridges and furrows, creating very high local pressures acting on

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the base. In order to provide a uniform foundation for a gravity-based structure, attempts have been made to pre-place sandfill and then screed it off as a level base to receive the structure. The procedure was successfully carried out underwater for the Prudhoe Bay Waterflood Project Seawater Treatment Facility for a barge-mounted facility 186×46 m in plan; however, this was in the shallow water of Prudhoe Bay, within a diked-off area where the sea was essentially still.

Out in the open sea, attempts so far to screed from floating equipment have achieved only a 300 m tolerance, despite the use of heave compensators. In the 1984 installation of Molikpaq, a tolerance of 150 m was achieved by use of a computer-controlled drag arm from the dredge. For several of the exploratory drilling structures, such as the SSDC-1 structures, the constructors have elected underbase filling after the structure has been placed. Filling is with a sand slurry, flowed in through piping installed within the structure. The sand is first mixed into a slurry, then pumped with a very low head, approximately that of gravity, to flow out under the base of the structure. This is similar to underbase grouting as practiced under the North Sea structures, but has the advantage of being a permeable, cohesionless foundation, able to adjust to settlements, to be refilled if necessary; and to facilitate subsequent refloating.

When using sand underbase fill, the structure must first be seated, either on three or four pads that have been pre-leveled or on mud mat-type feet extending down from the structure. For the SSDC1, the pads were concrete slabs, with a polyurethane mattress affixed, which provided support under the low bearing pressures during founding but collapsed when the structure was fully ballasted, to transfer the load directly to the sand. Skirts are generally required to seal around the perimeter of the caisson to prevent scour, enhance shear transfer to the soil, and contain sand underbase fill. Upon landing, the skirts must be forced to penetrate by the weight of the structure. In the shallow waters typical of the Arctic, the structure weight may be limited. Hence, maximum ballasting will often be required.

Some exploratory and probably all production structures will be ballasted with sand slurry to provide the needed weight for stability under maximum ice loads. This slurry will be pumped in through pre-installed pipes and discharged into the various ballast tanks (Figure 23.19).

After the structure is seated; scour protection may be required. This can best be done if filter fabric has been pre-attached to the sides of the structure just above the base. After seating, the fabric can be laid out laterally, and stone riprap or articulated stone-filled mattresses or articulated concrete mats can be laid. The concepts can be merged into one, with the articulated mats attached to the filter fabric, then affixed and transported by the structure until founding, after which they are laid out laterally.

If bearing piles or spud piles are to be installed after seating, their driving procedures will depend on the soil conditions which are anticipated. The concept developed for Sohio-BP known as SAMS incorporates steel spuds 84 in. (2.12 m) in diameter, with walls up to 3 in. (75 m) thick. After landing, the spuds are to be driven or jacked through weak overlying soils into stronger soils below (see Figure 23.20 through Figure 23.24).

In over-consolidated silts, high-pressure jetting within the pile appears to be essential to break up the plug. Concurrent driving with an impact or vibratory hammer should then achieve penetration. Similarly, in fully ice-bonded sands and silts, (permafrost), high-pressure jetting is believed to be the most effective means of breaking down the material in order to achieve penetration of piles.

For subsequent removal of piles driven into permafrost, any freeze-back can be destroyed by injecting steam into the water of the core of the pile. Raising the height of

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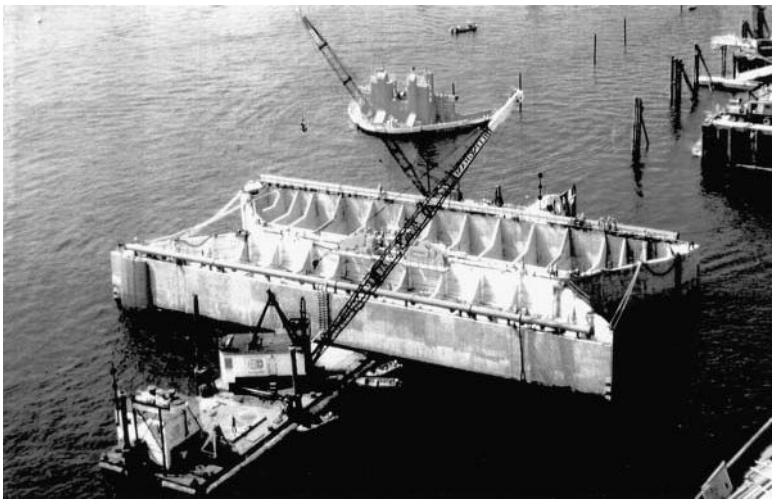

FIGURE 23.19
Deployment of caisson for Tarsiut caisson-retained island, Beaufort Sea, Canada. (Courtesy of Dome Petroleum.)

water in the pile to above sea level will help to break the bond of the soils by increasing the effective pore pressure in the surrounding soils.

The pile may then be removed by vibratory pulling or by jacking. Alternatively, the pile may be capped, filled with water, and internal pressure applied. As noted earlier, this system was successfully used to remove the 2-m-diameter, 100-m-long steel cylinder piles installed as temporary dolphins for the Oosterschelde Storm Surge Barrier in The Netherlands. To prevent the water from escaping under the tip of the pile, a plug of very fine sand was placed, which allowed the permeation of water but resisted piping, thus allowing the development of a hydrostatic head against the cap that was sufficient to remove the pile.

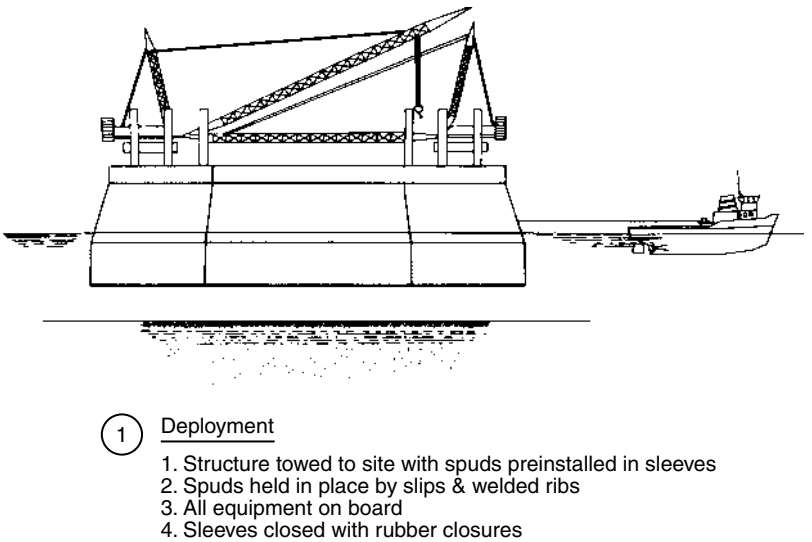

FIGURE 23.20
SOHIO's mobile exploratory drilling platform deploying to installation site.

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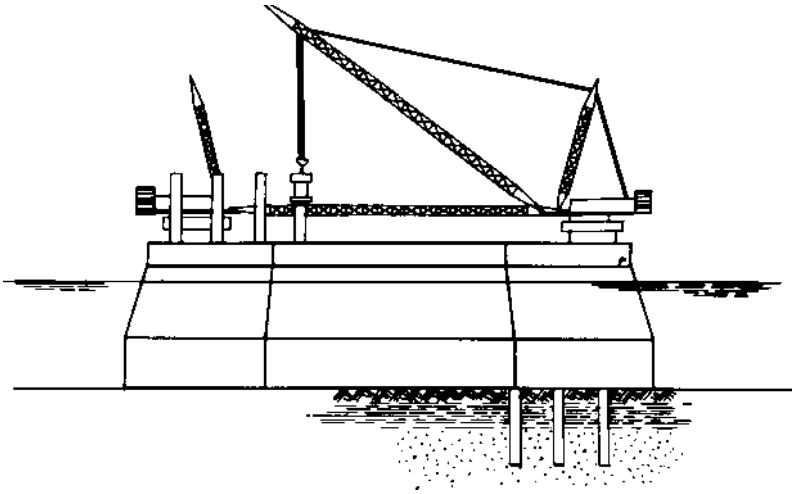
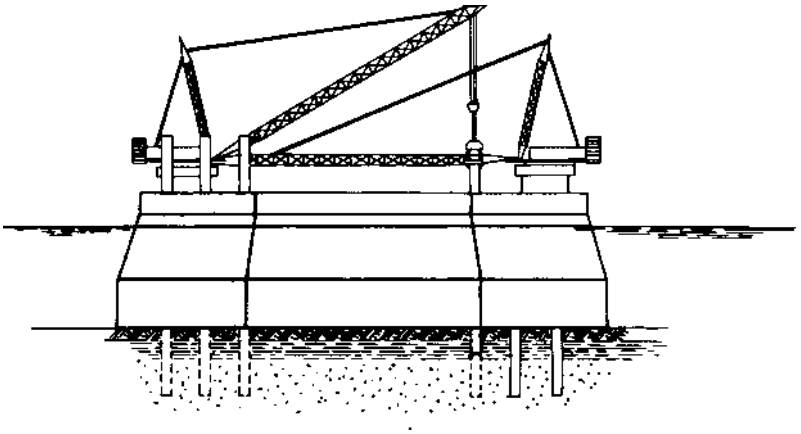


(2) Installation

1. Spuds vibrated down (shown) or jetted to grade.
2. Top of spuds driven below top of deck.
3. Spuds shimmed and bolted or welded in place.

FIGURE 23.21
Installing spud piles for Sohio Arctic Mobile Structure (SAMS) exploratory drilling platform. (Courtesy of Berger-ABAM Engineers.)



(3) Removal

1. Spuds removed by jacking (shown), vibratory hammer, jetting or deballasting caisson.
2. Spuds either held by slips (shown) or lifted clear and placed on deck.
3. Entire spud removed or two-piece spud used and bottom piece abandoned (optional).

FIGURE 23.22
Removal of spud piles of Sohio Arctic Mobile Structure (SAMS) and preparing for relocation. (Courtesy of Berger-ABAM Engineers.)

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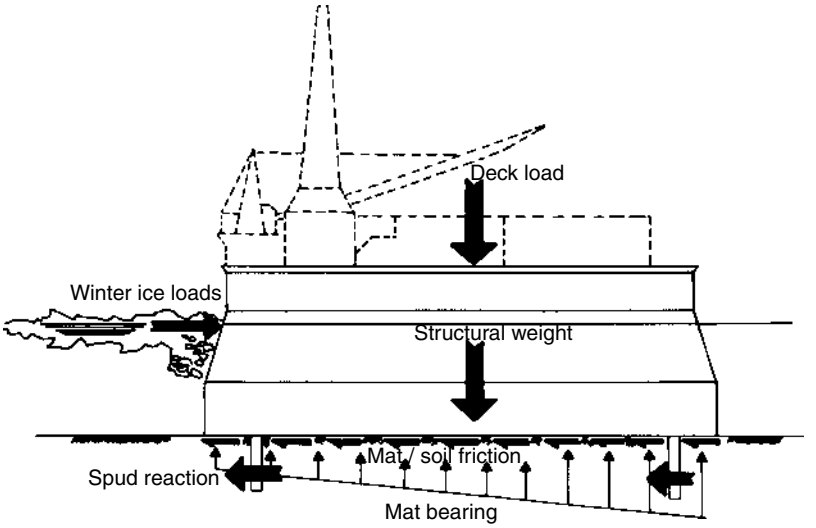

FIGURE 23.23
Shear transfer of lateral ice pressure to soils by spud piles. (Courtesy of Berger-ABAM.)

Finally, to break the structure free for relocation or salvage, sand ballast is removed by slurry pump and the structure deballasted to near-neutral buoyancy. Sustained low-pressure water flooding under the structure will be found very effective. Too high a pressure may cause piping and prevent subsequent maintenance of a low overpressure. When the waterflooding has equalized underbase pore pressures, one end of the structure is deballasted further to lift off, and then the other end is raised. The GBS-1 (CIDS) was rather easily broken loose from its clay foundation at its first site in the Beaufort Sea by underbase water flooding. Thus it appears that the suction effect can be effectively overcome by this method.

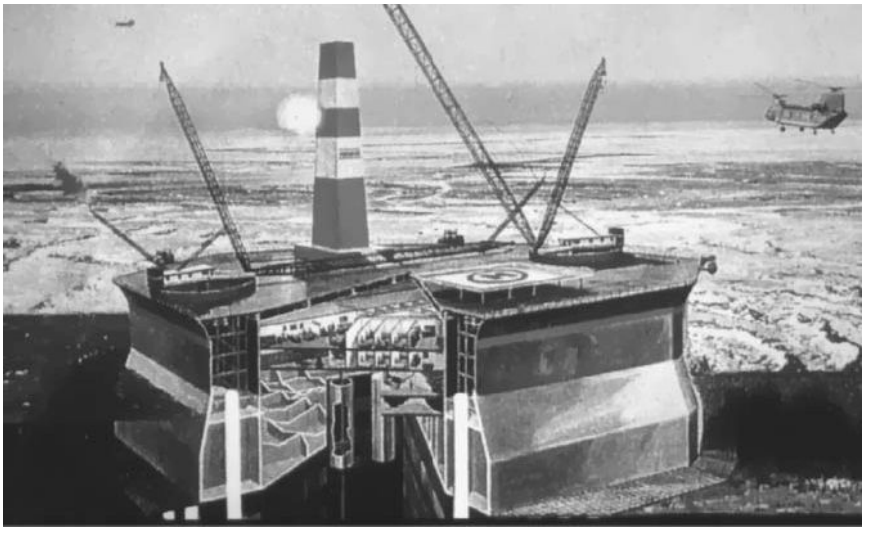

FIGURE 23.24
"SAMS"-SOHIO Arctic Mobile Drilling Platform. (Courtesy of Berger-ABAM.)

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As exploratory drilling is extended into deeper waters, a sub-base can be first seated on which an exploratory drilling caisson is set. Both steel and concrete sub-bases have been proposed. Their use is based on the successful mating of the Super CIDS concrete ice-belt caisson to the steel "mud base." While this was carried out in fully protected waters in Japan, it appears practicable even in exposed locations of the Beaufort Sea under favorable weather conditions. The concept has now been successfully applied in order to enable the SSDC to drill in deeper water.

Since the sub-base will be completely submerged, its submergence to sit on the seafloor presents a problem of stability. Use of several temporary buoyancy tanks, attached at the corners, will enable complete control as well as serving as guides for the mating. Tipping down in shallow water is an alternative. The next problem is that of the bearing of the caisson, 100–150 m in diameter, on the sub-base. The CIDS used a special rubber asphalt mix to equalize the bearing. A sub-base has been designed which would employ a compressible foam interface. The interface must be able to transfer shear after installation.

During construction of offshore structures in the Arctic, consideration must be given to the possibility of damage or destruction of the partially completed structure due to waves or sea ice. This is especially critical if the structure must be left uncompleted during the winter. Caisson-retained islands, using relatively small caisson elements, are especially subject to such piecemeal destruction. Caisson-retained islands have, however, been adopted as a viable concept because they solve the problem of construction of an embankment through the sea–air interface, where so much damage has occurred to embankments. These caissons are seated on an underwater embankment close to each other. Their articulation enables them to accommodate minor differences in the embankment surface.

The installation and removal of the Tarsiut Island caissons are very instructive for future Arctic construction. The four caissons, constructed of prestressed and reinforced lightweight concrete, were constructed in Vancouver and transported to the Beaufort Sea by a very large submersible carrier barge, which submerged to load the caissons and then deballasted to lighter draft for the tow. Near Herschel Island, the barge was holed by ice. The barge was therefore ballasted down and the caissons were floated off and towed the remaining distance as self-floaters (see Figure 23.19).

The sand island embankment had been constructed to an elevation of 6 m. Seats for the caissons had been screeded level to a tolerance of about ± 150 m. Due to an error in global surveying, the caissons were not set down on their pre-screeded seats but some 20 m off location. This resulted in some severe bending in at least one of the caissons and consequent flexural cracking. More serious was the fact that it did not fully bed in the sand. This survey error was due to the relocation of the survey tower during the sand island placement. It does point out the desirability of providing backup surveys and gross visual markers that will enable the working tugs and crews to guide the caissons to position. Spar buoys, articulated buoys, or driven pile markers should be used on future installations of this nature as well as GPS.

The caissons were skillfully positioned relative to each other, using winches mounted on the caissons and mooring lines to anchors and, later, to previously set caissons. Care was taken to keep all mooring lines as long as possible to absorb surge energy in the stretch of the lines. Tolerances achieved were extremely good, validating this system for relative positioning. Steel gates were then used to close the openings; however, due to the lateness of the season, they were backfilled with sand instead of the gravel originally planned (see Figure 23.25 through Figure 23.27).

During a summer storm, the waves were focused into the reentrant angle at the joints and caused piping of the sand from under the caissons, causing the gates to be dislodged.

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FIGURE 23.25
Tarsiut caisson-retained island under construction. (Courtesy of Dome Petroleum.)

Under continued battering and with the loss of the supporting sand behind, the gates failed inward, and 16,000 m^3 of sand were washed out (see Figure 23.27). The several lessons learned point to the following:

1. The need to provide adjustable seals for the gates
2. The need to prevent piping and erosion underneath the caissons, especially near their ends
3. The need to structurally secure the closure against wave and ice forces, including cyclic loading
4. The need to eliminate reentrant angles in the perimeter to avoid concentration of wave energy

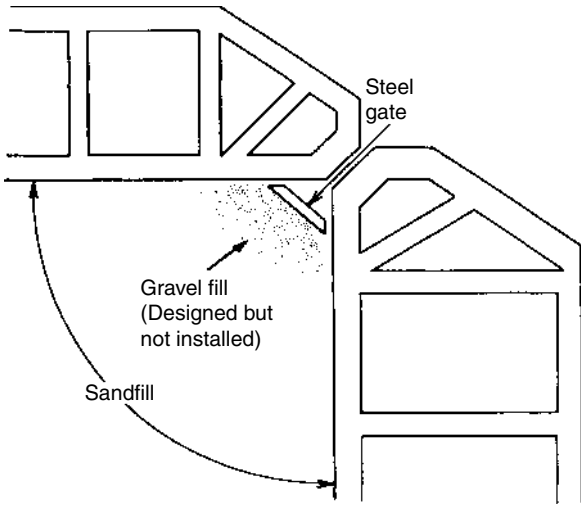

FIGURE 23.26
Joint detail for caissons of Tarsiut caission-retained island.

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FIGURE 23.27
Destruction of Tarsiut caisson-retained island by violent storm. (Courtesy of Dome Petroleum.)

This author believes that a preferable method of closure is by inner and outer arcs of steel sheet piles, connecting with interlocks in the caissons; while these allow greater tolerances in the joint's closure, shear and locking must still be provided so as to prevent differential movement. Another potential solution is the use of rock-filled gabions, which was the remedial solution finally adopted at Tarsiut.

Other problems encountered at Tarsiut Island were wave energy focusing due to the shallow draft of the berm, wave run-up and mach-stem effects of high wave buildup as the waves ran along the sides of the caissons, then flooded over the top, pyramidal waves at the lee corner which had been originally designated as a supply and personnel evacuation location, and spray. The spray and run-up deposited large quantities of water on the island, which caused serious local erosion as it drained. On the other hand, the berm caused the early ice to ground and form a protective rubble pile (Figure 23.28).

The island was completed the first season by heroic construction efforts extending into November and carried on under very adverse circumstances (see Figure 23.29). That winter, the height of the peripheral walls was raised by installing gabions (rock-filled nets of steel), which provided good protection and drainage (see Figure 23.30). Unfortunately, they were not adequately tied together, so that in a subsequent summer storm the waves dislodged some of the top gabions. Subsequently, they were all linked together (see Figure 23.31).

The island not only survived, but fulfilled its purpose well (see Figure 23.32). When removal was required, the partially frozen sand fill was excavated by backhoe, the caissons were washed clean, and upon dewatering, they floated free, with almost negligible leakage despite the external structural cracking which had occurred during initial setdown in the erroneous location. The experiences at Tarsiut Island and elsewhere have shown that embankments and caissons must be designed and constructed to be stable during the construction period under summer storms, and under a summer pack ice incursion, as well as under ice attack in winter and spring.

The Esso Canada caisson-retained island uses multiple steel caissons which, although articulated, are stressed together to transfer shear and axial forces and thus act as a flexible but monolithic whole. At any such joint, there are potentially six degrees of

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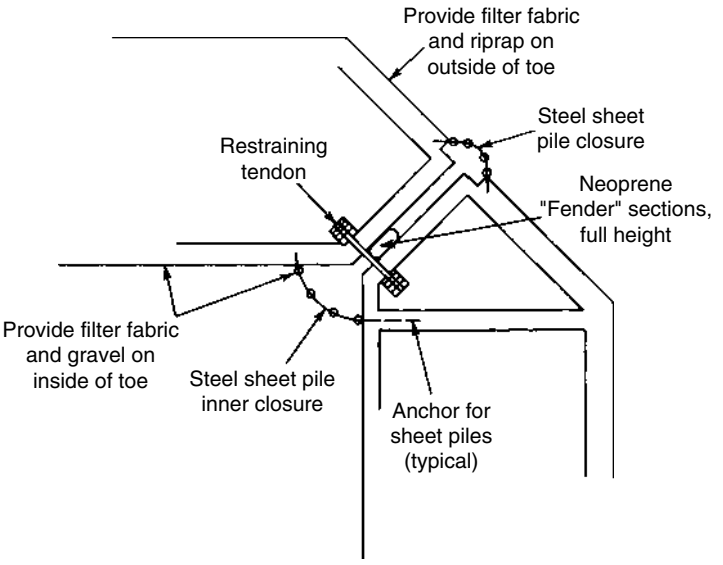

FIGURE 23.28
Joint detail for caissons Tarsiut of caisson-retained island.

freedom, thus 6 degrees of mis-match which must be accommodated. The design of the joint and connection details must accommodate these tolerances and enable the joint connection to be constructed rapidly, yet still provide the structural and sealing functions. The Esso caisson was assembled afloat, towed to location, and seated on a prepared berm. It was then filled with sand which froze during the winter, forming a highly stable seawall.

The Kulluk operated as a floating drilling structure into November, supported by an icebreaker which broke up the floes. However, ice conditions became too severe and

FIGURE 23.29
Dredge works into winter season to complete reconstruction of Tarsiut. (Courtesy of Dome Petroleum.)

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FIGURE 23.30
Gabions placed at periphery of Tarsiut to prevent wave overtopping.

reportedly caused two of the winches to fail, which then threatened movement over the "downstream" anchors and tilting of the structure. Quick and effective action by both the icebreaker and the drilling vessel prevented a potentially serious event.

During late spring, a multiyear pressure ridge moved against the Molikpaq, driven by the polar pack behind. The ice piled up to deck level and above, with continuous crushing at about one second intervals. This created intense vibration in the hull and partially liquefied the sand fill inside. Fortunately the movement of the ridge ceased. After the event, the sand fill inside was densified by repeated small explosive charges.

FIGURE 23.31
Summer storm overtops gabions and damages tanks. (Courtesy of Dome Petroleum.)

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FIGURE 23.32
Tarsiut island surrounded by winter sea-ice. Note rubber pile in foreground. (Courtesy of Dome Petroleum.)

23.13 Ice Condition Surveys and Ice Management

Area-wide ice conditions can best be obtained by satellite. Synthetic aperture radar (SAR), flown on the European and Japanese satellites, can be read out in real time at the U.S. facilities at Fairbanks, Alaska and Washington, SAR has the ability to penetrate the fog and the clouds which obscure the ice from visual observation and passive microwave sensing. Multiyear ice and ice islands can be distinguished from first-year ice.

Within local regions, the individual companies developing the offshore oil leases fly fixed-wing aircraft with side-looking radar (SLAR) or SAR. Progress is being made in using laser and radar methods to obtain estimates of ice thickness. Structure-mounted lasers now make it possible to obtain indications of sail heights of approaching ridges, which give a rough measure of the overall ridge size. Underwater sonar measurements of approaching ridge keels are being developed.

Under-ice surveys on a large-scale statistical basis can be obtained by submarine with upward-looking sonar. ROVs using fiber optics are capable of monitoring close-in underwater ice profiles in the winter, when the ice pack is slow moving.

Active ice management has been attempted both in the deep Arctic and in iceberg areas. API Bulletin 2N, Section 9, "Active Ice Management," recommends the following active defense mechanisms:

1. A moat around the structure
2. A narrow slot
3. Removal of ice rubble
4. Heating the surface of the structure
5. Installing an air bubbler system to cause water circulation
6. Use of air cushion systems to aid in breaking ice—air under an ice sheet is effective in fracturing it

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7. Mechanically cutting or grinding the ice
8. Use of insulating mats or snow to minimize ice thickness

With regard to item 3, it is more common in recent practice to purposefully form rubble piles as an aid to reduce ice impact and cushion local forces. If the rubble pile can be grounded, then it will also transmit some of the global ice force into the seafloor directly. Use of spray to generate artificial rubble has been applied on the GBS-1 (CIDS) in Harrison Bay.

A potential for future development is the application of small amounts of methyl alcohol to first year ice sheets to reduce the surface tension of the ice and initiate early fracturing. Canmar Drilling and Gulf Canada (now Chevron) have been using icebreaker vessels as an effective means of breaking up the oncoming ice floes. At present, they have only been able to break up the first-year ice, but they have been able to extend the use of floating drilling vessels to about November 30. In the deep Arctic, icebreakers can circulate upstream of the structure, breaking the floes. This has become proven technology in support of the floating drill vessel Kulluk, as well as with the more conventional drill ships.

In fast-ice regions, where the movement is only a few tens of meters per winter, slots can be cut by powered saw and the resultant blocks lifted out by crane. Continuous-saw-cutting machinery has been proposed, as has the use of high-pressure water jets.

Off the eastern seaboard of Canada, the towing of icebergs with masses up to 1,000,000 tn. to prevent encounter with floating drilling vessels is a well-established practice. The tugs either "lasso" the berg by carrying a floating nylon or Kevlar line around it or use explosively driven embedment anchors. The tugs can practically exert only 100–200 tn. of pull in total, so their main action is to divert or deflect the berg from a collision course. Considerable effort has gone into computer programs to help predict the movement of close-in bergs both prior to hook-on and after the tugs exert their thrust.

Major problems are the presence of fog around the berg, the poor reflectivity of radar from the ice, the unseen underwater extensions of the berg and finally, the tendency for pyramidal bergs to roll over suddenly, endangering the tugs. Explosives as a means of fracturing bergs and multiyear ice have been tried with little or no success due to the great absorptivity of the ice.

23.14 Durability

The Arctic exposure is unique for both steel and concrete. For steel, low temperatures raise the problem of brittle fracture and low energy absorption, not only of the steel plates but also of the welds. Major advances have been made in overcoming these problems, so that today reliable low-temperature-rated steels and welding procedures are commercially available.

External steel plates are subjected to abrasion from the ice and corrosion from the salt water-air environments. The water has a high percentage of dissolved oxygen due to its low temperatures. Abrasion removes the products of corrosion, exposing fresh surfaces so that new corrosion can commence. Thus, the rate of corrosion–abrasion in fast currents and silt-laden water has been as much as 0.3 m/year.

Both dense epoxy and dense polyurethane coatings are available on the market, which give excellent protection to the steel surface as well as reducing friction and adfreeze bond. These coatings require touch-up every year or two. A small portable cofferdam could be

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provided to enable dewatering of local areas for touch up. For internal steel compartments, sacrificial anodes appear appropriate. Externally, they can be effective only if located below the deepest ice; otherwise they will be ripped off.

For concrete structures, the concrete will, of course, have been designed to be of very high quality, with low permeability and optimal entrained air. Both normal hard rock and high-grade lightweight aggregates are used. Silica fume should be used to improve bond strength and abrasion-resistance of the matrix. Abrasion-resistant aggregates should be selected.

The major sources of problems for concrete structures are thermal cracking, freeze–thaw disintegration of the concrete, and abrasion by ice. Thermal cracking during fabrication is due to restrained cooling after the temperature rise due to heat of hydration. Proper mix design, selection of cement, use of pozzolans, and insulation of the forms can eliminate or minimize such cracking. Proper detailing of reinforcement on the exposed face can serve to control cracks due to thermal strains both in fabrication and in service.

Freeze–thaw disintegration of external concrete surfaces can be prevented by use of air entrainment of the proper amount and pore spacing (this latter is the more important), by using a dense impermeable mix, and by using aggregate of low water absorption. Blast furnace slag cements appear more susceptible to freeze–thaw attack than Portland cement with fly ash. Structural lightweight aggregate with silica fume has given satisfactory performance in tests in Baltic sea.

A special problem may arise when outer compartments are filled with seawater above the external sea level. Water penetration into the concrete, combined with very low air temperatures, can create a freeze-front inside the concrete wall, leading eventually to delamination. The internal walls in this zone should therefore be coated with an impermeable membrane.

Abrasion by ice appears to be a complex interaction of frictional wear and adfreeze plucking (see Figure 23.33). Use of a very dense concrete, such as that obtained by adding condensed silica fume to the mix, appears to give moderately satisfactory results, based on both laboratory and field exposure tests. Steel armor plate has been used on the Baltic Sea lighthouses. Polyurethane coating and ceramics are potential solutions.

FIGURE 23.33
Abrasion at waterline by moving sea-ice. Baltic Sea Lighthouse, Sweden.

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Corrosion of reinforcement should not occur with a dense concrete such as that obtained by the combined addition of silica fume and superplasticizer (high-range water reducer). If the deck of the structure is concrete and if salts such as calcium chloride will be used by the operating personnel to prevent atmospheric and spray icing, then the decks should be well sloped for drainage and the top layers of reinforcing steel should be epoxy coated.

23.15 Constructibility

Constructibility planning as set forth in Chapter 21 is even more important in the Arctic than elsewhere, due to the very limited open-water season, the extreme logistical difficulties, and the large capital investment.

Contingency plans must be prepared for the cases of late opening of the ice for passage around Point Barrow and for sudden summer storms and even summer pack ice invasions during the construction period. Summer ice incursions are of special concern in the Beaufort and Chukchi Seas. Boats may have their propellers damaged by ice. Critical equipment may not start in extremely cold weather. Windblown spray may add many hundreds of tons of water onto an island under construction.

API Bulletin 2N, Section 7.5, states: "In areas subject to heavy sea ice, bad weather and ice conditions may mean delays in completing the tow to the final location. Possible temporary mooring sites should be selected along the towing route for refuge in case of such delays. Exceptionally poor ice conditions or weather conditions may cause sufficient delay to prevent installing the structure during the scheduled summer construction season. For this reason, it may be necessary to over-winter at a temporary location."

Safety of personnel must be given major consideration. A human can survive only a few minutes in water at -2°C.

Provisions must be made for firefighting in below-freezing weather. Intakes for water must not clog with frazil ice or broken ice. Provisions must be made for snow clearance, for although the amount of snow is small, the high winds can cause substantial drifts around structures. In the sub-Arctic regions, atmospheric icing may make crane booms unusable and endanger the stability of boats. Decks and walkways can become iced. Measures of preventing or removing ice need to be planned.

API Bulletin 2N, Section 8.1.2, "Construction Conditions during the Arctic Summer," states:

"Construction planning in offshore areas subject to ice incursion should allow for this contingency (ice invasion) by providing proper equipment and personnel training. Contingency plans should include provision for ice surveillance and forecasting, by satellite and radar, active and/or passive defense systems, separation of vessels, and ice strengthening of vessels."

Section 8.1.4b, "Fog," continues: "Construction plans should account for the effect of fog on logistics and other visibility-dependent operations."

Section 8.1.4c, "Break-up and Freeze-up," states: "Construction plans should account for the effects of ice movement and logistics interruptions associated with break-up and freeze-up."

With regard to winter construction of ice roads, API-2N in Section 8.2 and Section 8.3 calls attention to the need for pre-construction reconnaissance, with special reference to leads and cracks in the ice, and to the need for lighted signs and roadway delineation markers. They should also indicate the distance and direction to a safe refuge or aid, in

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event of an accident or unforeseen event. Survival shacks, marked by a light, and survival drums should be placed at adequate intervals along the road.

23.16 Pipeline Installation

In areas of the Arctic where 40–60 days of open water can be assumed, a conventional lay barge could be employed. The shallow water and mild wave climate eliminate many of the problems normally associated with deep-water pipe-laying. However, special welding procedures may be needed because of the low ambient temperatures in early fall.

Pipelines in the Arctic will generally have to be trenched 3–6 m deep in order to protect them against ice scour, at least out to a water depth of 50 m. The fastest means for trenching appears to be by the use of a heavy-duty plow. It may be desirable to equip the plow with high-pressure water jets to break down the overconsolidated silt and to break down any permafrost or ice lenses encountered in the near-shore areas.

Pipelines in other areas can be pulled, working even under the ice. The pulling line must be laid out on the seafloor, possibly using a submersible or ROV to lay a messenger line. In fast ice in winter, holes may be cut at intervals and the ice used as a platform for feeding in and pulling of pipe (see Section 15.13). For the Panarctic Drake tie-in line, a 140-tn. manifold was modified to be pulled as a sled. The 300 m shoreline approach was encased in a 24 in. (600 mm) casing with a 3 in. (75 m) methanol line, for circulating methanol at −10°C to grow a 1-m diameter ice bulb to protect the line. Winches were installed on the ice at 1-km intervals and side-boom cats used to control overbend radii (see Figure 23.34). The pipeline was pulled into a 2-m-deep plowed ditch, which was backfilled with gravel to form the frozen bulb.

The pipeline tie-in to a structure requires careful detailing. Some structures can best be fitted with J-tubes. In other cases, a curved casing can be directionally drilled into the seafloor and used as a pull-in tube for the pipeline. In sands, a slant casing may be

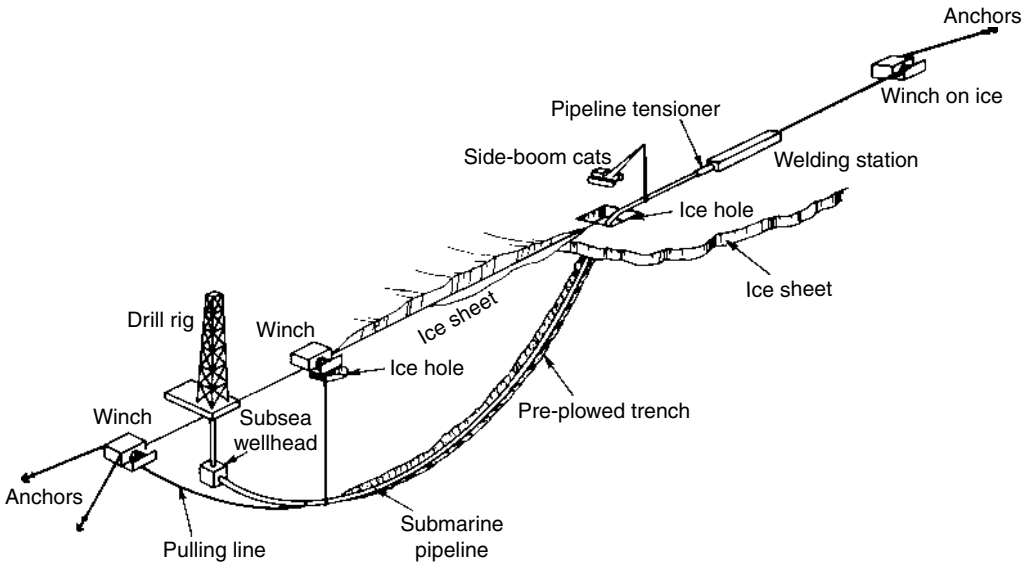

FIGURE 23.34
Pan-Arctic test installation of sub-ice flow line. (Courtesy of R.J. Brown.)

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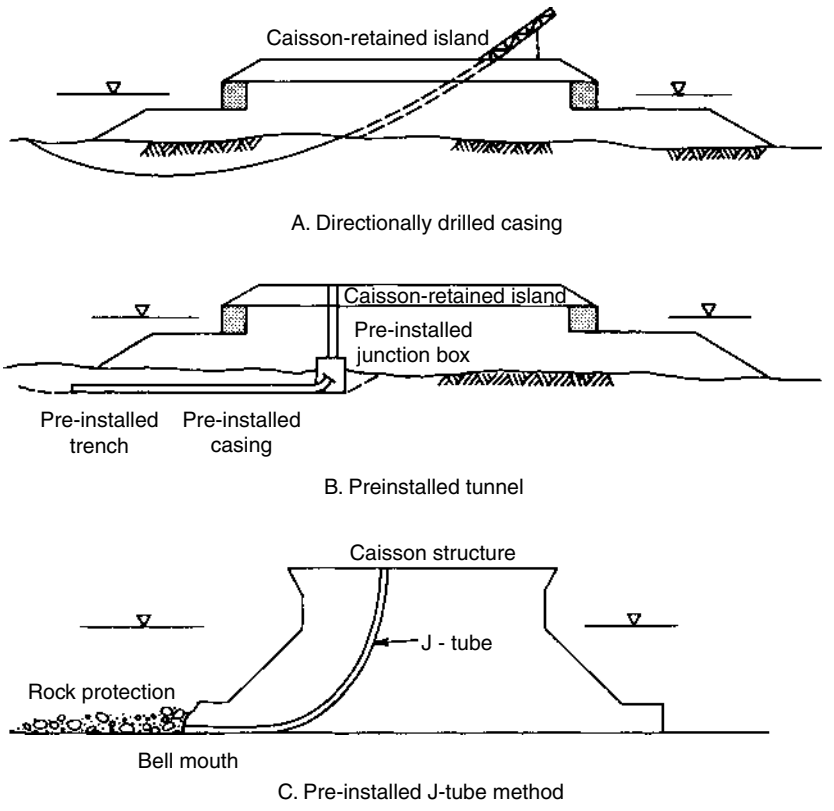

FIGURE 23.35
Pipeline tie-ins to Arctic offshore platforms. (Courtesy of R.J. Brown.)

installed and a pre-bent elbow fitted. This necessitates an underwater joint at the toe of the caisson. Such an underwater joint can be made by use of a habitat. Alternatively, the initial connection may be made by a gasketed flange, with a worker descending into the unwatered line from the platform to make the final weld (see Figure 23.35).

In sands, local stabilization may be required around the connection zone, using chemical grouts suitable for the low temperature. Another solution is to build in a pipe tunnel at the base of the structure. After the pipeline has been pulled in, the tunnel is dewatered to permit welding of the line. At the shore end, the pipeline will enter a zone of permafrost. The protection of the shore from thermokarst erosion due to thawing of the frozen soil and subsequent wave erosion is an extremely important design matter. The constructor can expect that measures such as double casing with refrigerant or special insulation or a rock-filled causeway will be required.

23.17 Current Arctic Developments

1. Eastern Canada. The Terra Nova field, north of Sable Island, on the Grand Banks off the coast of Nova Scotia, is an area subject to high waves and high currents, as well as sea ice and rare icebergs. This field will be developed by floating

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production, storage, and offloading structures (FPSOs), moored to anchors placed in 12-m-deep glory holes. Because of the rough seas, the glory holes are being excavated by drilling overlapping holes with a 5.6-m-diameter cutting head. The glory holes range from 16×16 m to 56×16 m. A 20 in. (0.5 m) drill string operates the cutting head and raises the cuttings for discharge through a 300-m floating pipeline. Water depth is up to 100 m. The seabed is dense sand overlying glacial debris with clay, cobbles, cemented sands, and gravel. Occasional boulders are encountered.

2. Sakhalin. One of the newest frontier oil provinces is Sakhalin. Although the former Soviet Union and more recently Russia have been producing a modest amount of oil from a coastal field on northeast Sakhalin, the interest today is on a number of apparently large structures in water depths from 30 to over 200 m along the eastern and northern coasts. For 6 months of the year the seas are ice-covered. Potential difficulties include the prevailing winds and currents which make this a compression field, with ridges jammed against one another. In the open-water season, storms create high waves. Currents are strong. Sakhalin is in a seismic zone.
   Despite these adverse environmental factors, the potential yield of these fields and their proximity to the Japanese and Korean markets makes this an area of strong interest. Initial development of two offshore fields is being carried out using the Molikpaq, the steel gravity-base mobile platform that previously worked in the Canadian Beaufort Sea and the CIDS from the Alaska Beaufort. They have been re-mounted on new steel hulls fabricated in Eastern Russia, so as to enable their use in deeper waters. The sub-bases will be submerged, and the caissons floated over the top. Then the sub-bases will be de-ballasted, raising the joints above water. The joints will be welded, with exterior closure plates.
   When fully outfitted, the caissons will be towed to the site and submerged onto a gravel pad controlled by external buoyancy tanks. Then the interior will be filled with dredged sand and gravel. Riprap and articulated concrete mats will be placed around the periphery to prevent scour.
3. Barents and Kara Seas, north of Russia. Sea and ice conditions appear similar to those of the Beaufort Sea off Alaska. In addition, icebergs are encountered, having broken off from the glaciers on Northern Novaya Zemblya. The area is also subject to seismic activity. Concerns over fish and marine mammals must also be addressed. Logistics will be difficult.

And now there came both mist and snow,
And it grew wondrous cold:
And ice mast-high, came floating by,
As green as emerald.
The ice was here, the ice was there,
The ice was all around,
It cracked and growled and roared and howled
Like noises in a swound.

Coleridge "The Rime of the Ancient Mariner"

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