Design & Construction Considerations for Offshore Wind Turbine Foundations in North America

Increased need for alternative energy sources that are cost effective and that have little impact on the environment has resulted in greater interest in offshore wind farms.

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Ballooning energy prices in the United States have increased focus on alternative energy sources such as wind energy. While in the past few years onshore wind farms were being developed in the Midwest and elsewhere, offshore wind energy has now become an attractive proposition because there are vast wind resources located offshore that pose little negative visual or noise impacts to local communities. But developing these wind resources 15 to 30 kilometers (9.3 to 18.6 miles) from shore would require large support structures and foundations that could be cost prohibitive. To make such a proposition cost effective requires even larger wind turbines in order to eke out more power per dollar of capital investment. In return, these large wind turbines will place large demands on their support structures and foundations. Of the thirty-odd offshore wind farms being developed, or proposed, in the United States, several are planned in water depths of 30 to 50 meters (98 to 164 feet). The combination of water depth and the increased windmill tower heights and large rotor blade diameters will create loads that make foundation design very complex. Offshore wind turbine structures are exposed to additional loads such as ocean currents, storm wave loading, ice loads and potential ship impact loads. All of these factors pose significant challenges in

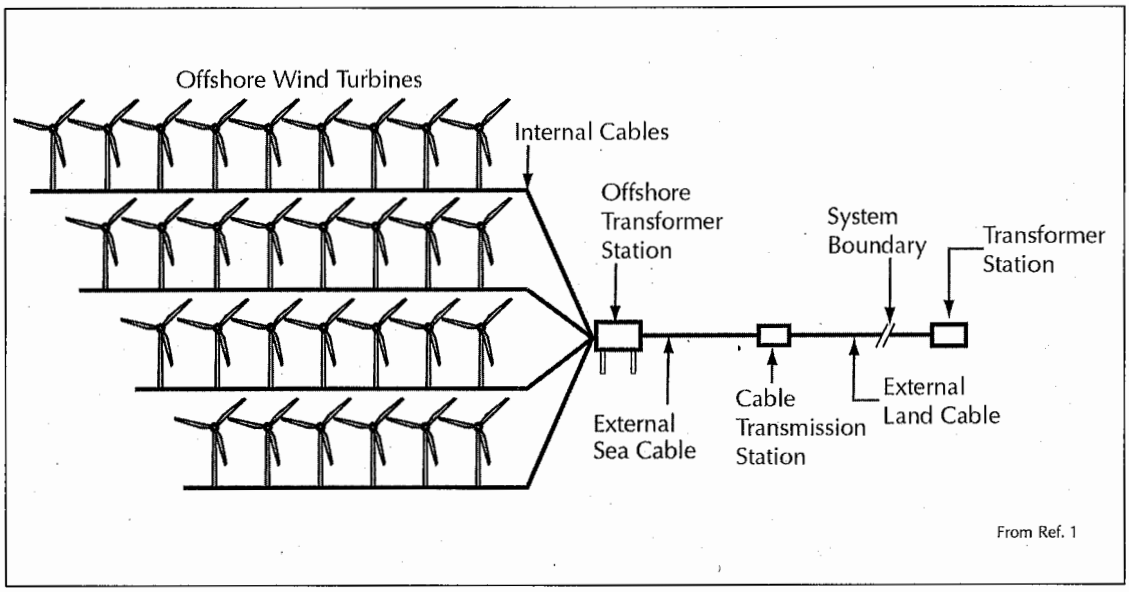

FIGURE 1. Wind farm components and their layout.

the design and construction of wind turbine foundations.

Background

Until now in the United States, offshore wind power development has not been a focus area because there is great potential for wind power on land. However, high-quality onshore wind resources are mostly located in the Midwest and Central United States while the demand centers are located along the coasts, thereby making the cost of transmission high. On the northeast coast of the United States, offshore development is an attractive alternative because electricity costs are high and transmission line construction from the Midwest faces many obstacles. Higher quality wind resources, proximity to coastal population centers, potential for reducing land use, aesthetic concerns and ease of transportation and installation are a few of the compelling reasons why power companies are turning their attention to offshore development. Offshore turbines are being made larger to economize on the foundation and power collection costs. As the technology for wind turbines improves, the industry has developed wind turbines with rotor diameters as large as 150 meters (492 feet) and power ratings of over 7.5 megawatts. Since an increasing number of wind farms are being planned offshore in water depths of over 50 meters (164 feet), the combination of water depth, the increasing wind tower heights and rotor blade diameters create loads that complicate foundation design and consequently place a greater burden on the engineer to develop more innovative and cost-effective foundations. The U.S. Department of Energy predicts that 50 gigawatts of installed offshore wind energy will be developed in the next twenty years.[2,3] This means at least $100 billion of capital investment, with about $50 billion going to offshore design and construction contracts.

Wind Turbine Farm Layout

Primary components of a typical offshore wind farm include several wind turbines located in the water that are connected by a series of cables to an offshore transformer station that, in turn, is connected by an undersea cable to an onshore transformer station linked to the existing power grid (see Figure 1). The wind turbines are usually spaced laterally at several (four to eight) times the rotor diameter and staggered in order to minimize wake effects. Placing turbines closer to each other reduces the quantity of electric cable required but it increases turbulence and wake effects, thereby reducing power generation. There-

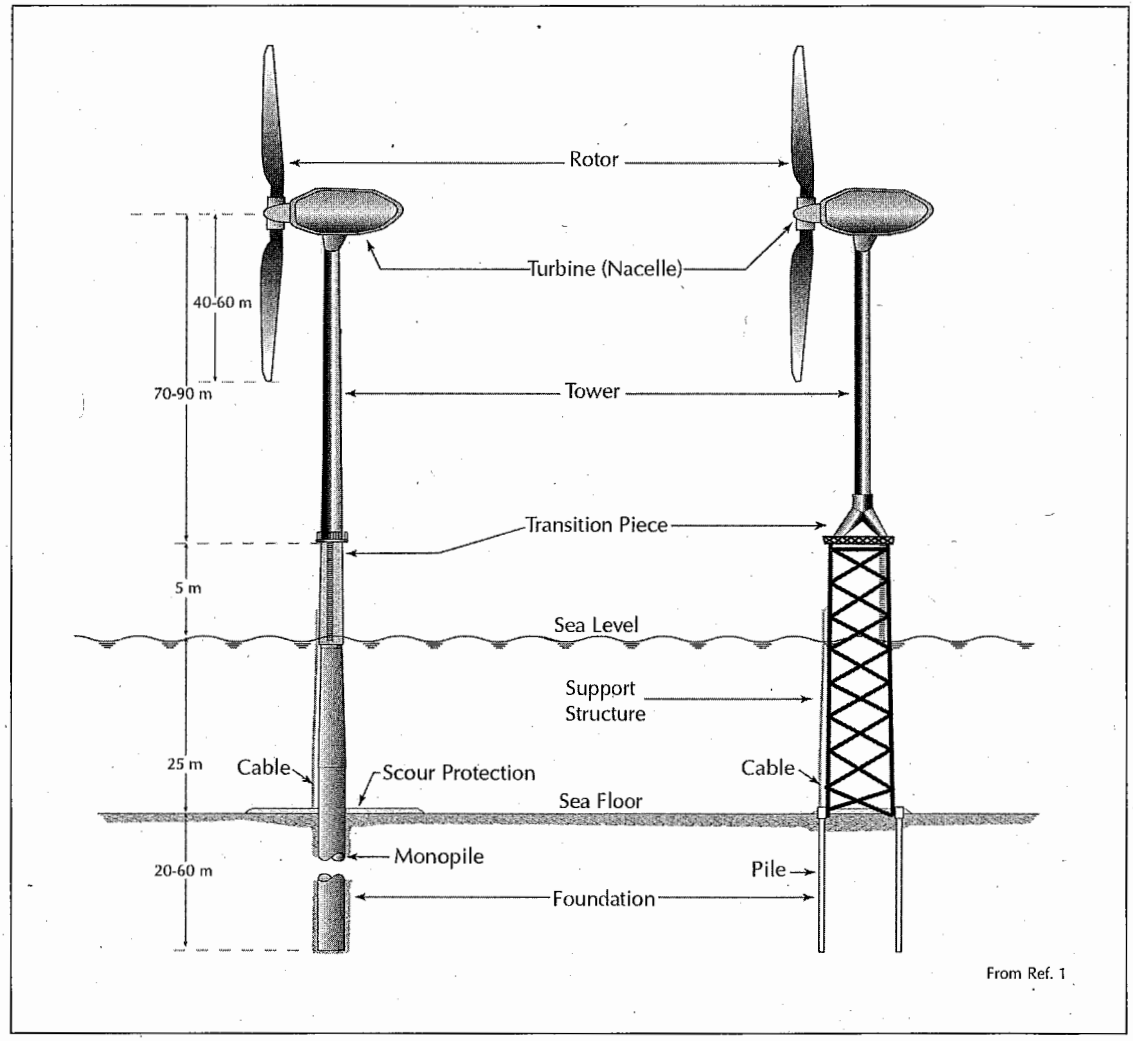

FIGURE 2. Wind turbine system components.

fore, laying out wind turbine farms includes minimizing the length of cabling required yet maximizing power generation in order to optimize the costs per unit of the power that can be produced.

Wind Turbine Components

The primary components of a wind turbine system (see Figure 2) include the foundation, the support structure, the transition piece, the tower, the rotor blades and the nacelle. The foundation carries the support structure, transition piece, tower and turbine, anchoring them to the seabed. In turn, the support structure connects the transition piece and the tower to the foundation at the seabed level. In some cases, as in monopiles and gravity structures, the foundations serve as support structures as well by extending from the seabed level to above the water level and they are directly connected to the transition piece or tower. The foundation system and support structure, which are used to keep the turbine in its proper position while being exposed to the forces of nature such as wind and sea waves, can be fabricated by using a variety of materials such as reinforced concrete or steel. The transition piece connects the tower to the support structure or directly to the foundation (as in cases where monopiles are used). The transition piece serves the purpose of correcting any misalignment of the monopile that

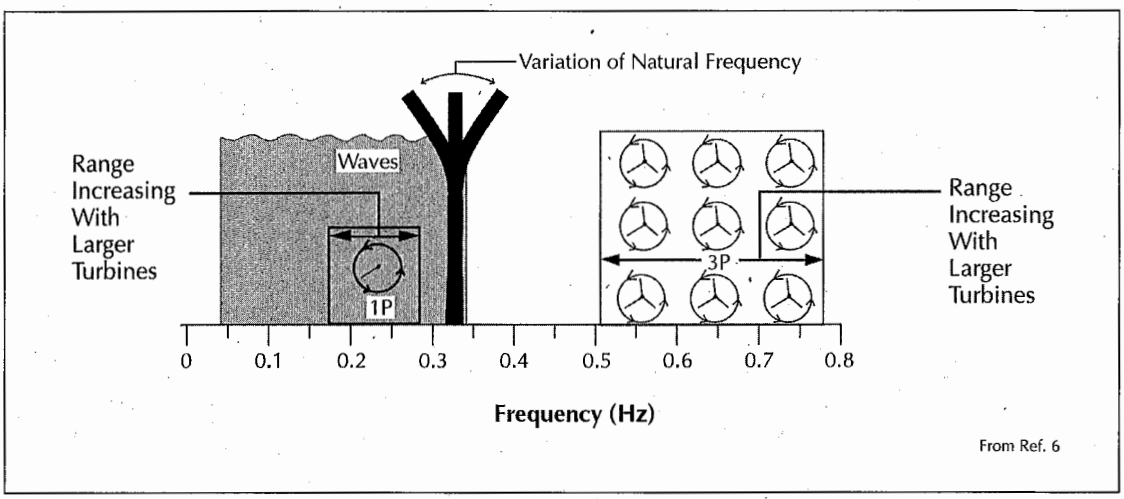

FIGURE 3. Typical ranges for frequencies for waves, rotors, blade passing and structure.

may have occurred during installation. The towers are made of steel plate rolled into conical subsections that are cut and rolled into the right shape, and then welded together. The nacelle contains the key electro-mechanical components of the wind turbine, including the gearbox and generator. The rotor blades are made of fiberglass mats impregnated with polyester or carbon fiber composites. The power cable from each turbine is inserted in a J-shaped plastic tube that carries the cable to the cable trench in the seabed below.

Wind Turbine Operation

As wind flows through a turbine it forces the rotor blades to rotate, transforming the kinetic energy of the wind to mechanical energy of the rotating turbine. The rotation of the turbine drives a shaft through which a gear box drives a power generator that generates current through the principle of electromagnetic induction. The shaft, gearbox and generator are located in the nacelle. The nacelle is able to revolve about a vertical axis in order to optimally direct the turbine to face the prevailing wind. The electric current thus generated is converted to a higher voltage via a transformer at the base of the tower. The power that can be harnessed from the wind is proportional to the cube of wind speed. However, today's wind turbines convert only a fraction of the available wind power to electricity and are shut down beyond a certain wind speed because of structural limitations and concern for wear and tear. So far, it is considered cost optimal to start power regulation at a ten-minute wind speed of 9 to 10 meters (29.5 to 33 feet) per second, have full regulation at mean wind speeds above 14 to 15 meters (46 to 49 feet) per second and shut-down or idle mode at 25 meters (82 feet) per second. To minimize fluctuation and to control the power flow, the pitch of the blades of offshore wind turbines is regulated. At lower wind speeds, variable rotor speed regulation is used to smooth out power output. To maximize operating efficiency, the yaw of the turbine is also varied every thirty to sixty seconds. The pitching and yawing creates non-linear aerodynamic response and hysteretic loads that have to be modeled in turbine response calculations.

Wind Turbine Foundation Performance Requirements

Deformation tolerances are usually specified by the wind turbine manufacturer and are based on the requirements for the operation of the wind turbine. Typically, these tolerances include a maximum allowable rotation at pile head after installation, and also a maximum accumulated permanent rotation resulting from cyclic loading over the turbine's design life. For an onshore wind turbine, the maximum allowable tilt at pile head after installation is between 0.003 to 0.008 radian (0.2 to 0.45 degrees). A somewhat larger tilt 0.009

radian (0.5 degrees) may be allowed for offshore wind turbines. Any permanent tilt related to construction tolerances must be subtracted from these specified tolerances. Typical values of construction tolerances range from 0.0030 to 0.0044 radians (0.20 to 0.25 degrees). Allowable rotation of the support structure/foundation during operation is generally defined in terms of rotational stiffness, which typically ranges between 25 to 30 GNm/radian.[4]

Foundation Dynamics

As the offshore wind turbine rotates, the blades travel past the tower, thus creating significant vibrations. When a three-bladed rotor encounters a turbulent eddy, it resists peak forces at frequencies of 1 and $3P$, where P is the rotational frequency of the rotor. For a typical three-bladed variable speed turbine, the rotational frequency is between an approximate range of 0.17 and 0.26 Hertz, and the blade passing frequency is between about 0.51 and 0.78 Hertz. Meanwhile, cyclic loading from sea waves typically occurs at a frequency between 0.04 and 0.34 Hertz.[5] Therefore, to avoid resonance the offshore wind turbine (turbine, tower, support structure and foundation) has to be designed with a natural frequency that is different from the rotor frequencies as well as wave frequencies as shown in Figure 3.

Larger turbine diameters will require higher hub heights and heavier nacelles, which will impose even greater demands on the design of the foundation and support structure. The range of rotational frequencies will increase linearly with the blade diameter. Since the natural frequency of the tower system is inversely proportional to the height of the tower squared, the frequency of the higher towers decreases rapidly and lies in the region of the wave frequencies. Accordingly, the support structure and foundation system would need to be made relatively stiff. A stiffer foundation would require more materials and thereby greater expense than a flexible foundation.

Design Process

The design process involves an initial site selection followed by an assessment of external conditions, selection of wind turbine size, a subsurface investigation, an assessment of geo-hazards, the selection of the foundation and support structure, the development of design load cases, and geotechnical and structural analyses. A flow diagram for the design process of a typical offshore wind turbine is shown in Figure 4. So far, for achieving economies of scale, wind turbines are generally mass produced and available in four predefined classes based on wind speed. Consequently, the designer simply selects one of the predefined turbine classes that may apply to the wind farm site. Because the water depth, seabed conditions, sea state statistics (wave heights and current velocities), ice climate, etc., may vary widely between sites, the use of a generic support structure concept is not feasible. Therefore, the tower, support structure and foundation must be designed for site-specific conditions. The foundation system is selected based on several factors such as the level of design loads, depth of water at the site, the site geology and potential impact to the marine environment. As larger, customized wind turbines are developed, they will require an integrated analytical model of the turbine, support structure and foundation system, as well as rigorous analyses with site-specific wind and wave regimes.

Site Selection

In addition to wind resource availability, a variety of factors that govern the selection of a wind farm site include:

- site availability;
- distance from shore;
- proximity to power demand sites;
- proximity to local electricity distribution companies;
- potential impact to existing shipping routes and dredged channels;
- interference with telecom installations;
- interference with line of sight from onshore;
- buried under-sea cables and gas lines;
- distance from local airports to avoid potential interference with aircraft flight paths; and,
- other factors such as interference with

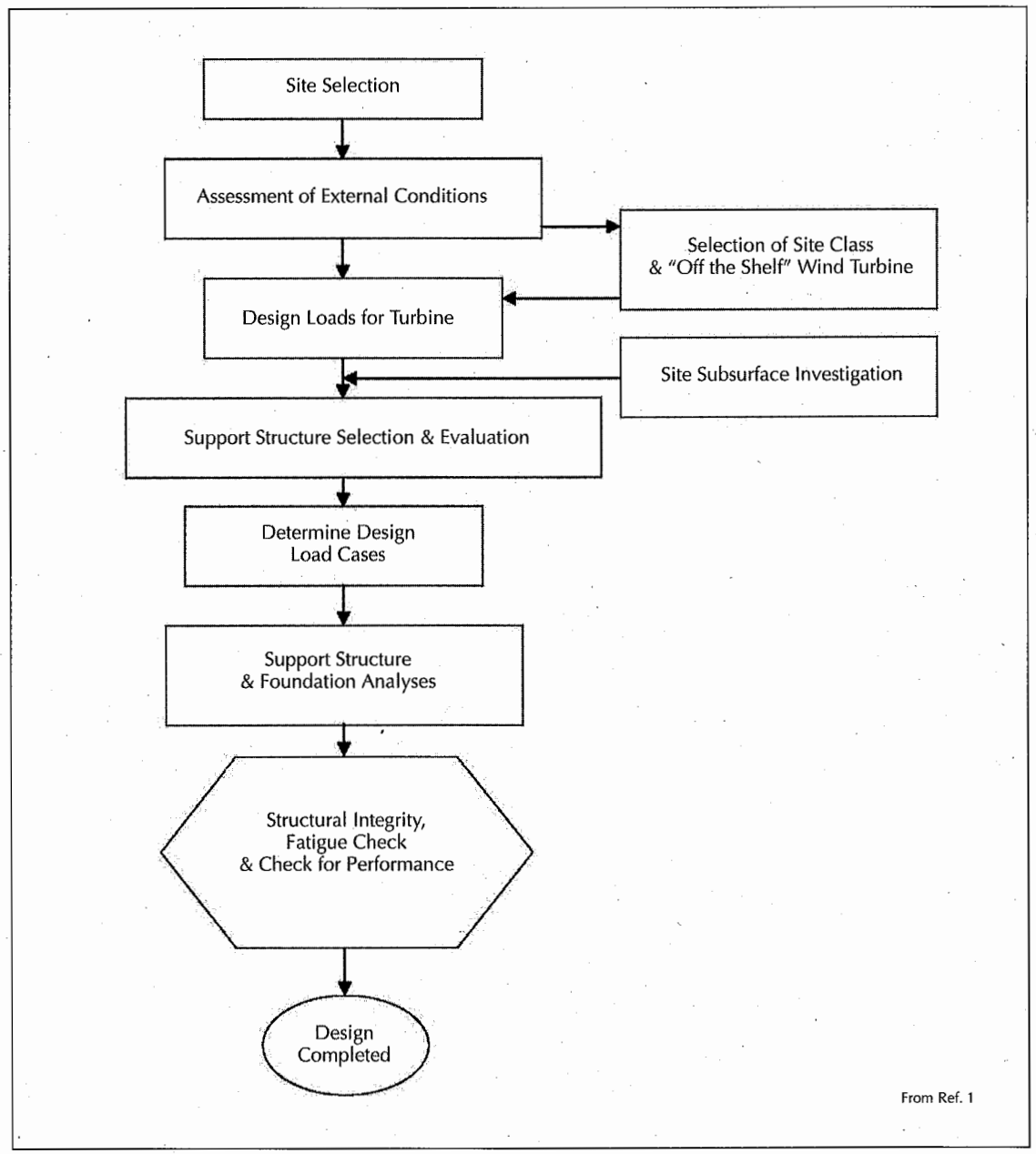

FIGURE 4. The design process for a typical offshore wind turbine.

bird flight paths also have to be evaluated through avian studies.

Assessment of External Conditions

Following initial site selection, the developer makes an assessment of external conditions such as the level of existing wind conditions, water depth, currents, tides, wave conditions, and ice loading, the site geology and associated geo-hazards such as sea-floor mudslides, scour and seismic hazards.

Design Loads. Since wind loading is the dominant loading on an offshore wind turbine structure, it results in dynamic characteristics that are different from the wave and current loading that dominate the design of foundations for typical oil and gas installations. The loading on wind turbine foundations is char-

TABLE 1.
Permanent Loads from a Typical Offshore Wind Turbine

	Typical 3 MW Turbine, 80 m (263 ft) Hub Height (tonnes/tons)	Typical 3.6 MW Turbine, 80 m (263 ft) Hub Height (tonnes/tons)	Typical 5 MW Turbine, 90 m (295 ft) Hub Height (tonnes/tons)	Future 7.5 MW Turbine, 100 m (328 ft) Hub Height (tonnes/tons)
Tower	156/172	178/196	347/383	~550/606
Nacelle	68/75	70/77	240/265	~300/331
Rotor	40/44	40/44	110/121	~180/198

Note: From Ref. 1.

acterized by relatively small vertical loading and larger horizontal and moment loads (which are also dynamic). The design loads are classified into permanent, variable and environmental loads.

Permanent loads include the mass of the structure in air (including the mass of grout and ballast, equipment or attachments that are permanently mounted onto the access platform) and hydrostatic forces (including buoyancy forces) on the various members below the waterline. Permanent loads from typical offshore wind turbines are presented in Table 1.

Variable loads are loads that may vary in magnitude, position and direction during the period under consideration. These loads include personnel, crane operational loads, ship impacts from service vessels, loads from fendering, access ladders, platforms and variable ballast and also actuation loads. Actuation loads result from the operation of the wind turbine. These loads include torque control from the generator, yaw and pitch actuator loads and mechanical braking loads. In addition to the above, gravity loads on the rotor blades, centrifugal and Coriolis forces, and gyroscopic forces due to yawing must be included in design. Loads that arise during the fabrication and installation of the wind turbine or its components are also classified as variable loads. During fabrication, erection lifts of various structural components generate lifting forces, while in the installation phase forces are generated during load out, transportation to the site, launching and upending, as well as during lifts related to installation. The necessary data for computing all operating loads are provided by the operator and the equipment manufacturers. These data need to be critically evaluated by the designer. Forces generated during operations are often dynamic or impulsive in nature and must be treated as such. For vessel mooring, design forces are computed for the largest ship likely to approach at operational speeds. Generally, permanent and variable loads can be quantified with some certainty.

Environmental loads depend on the site climate and include loads from wind, wave, ice, currents and earthquakes. These loads have a great degree of uncertainty associated with them (see Figure 5). These loads are time dependent, covering a wide range of time periods ranging from a fraction of a second to several hours. These loads act on the wind tower through different load combinations and directions under different design conditions and are then resolved into an axial force, horizontal base shear and an overturning moment to be resisted by the foundation.

Wind Loading. Site-specific wind data collected over sufficiently long periods are usually required to develop the wind speed statistics to be used as the basis of design. The design wind is represented by a mean wind speed, a standard deviation and a probability distribution for each of these parameters.

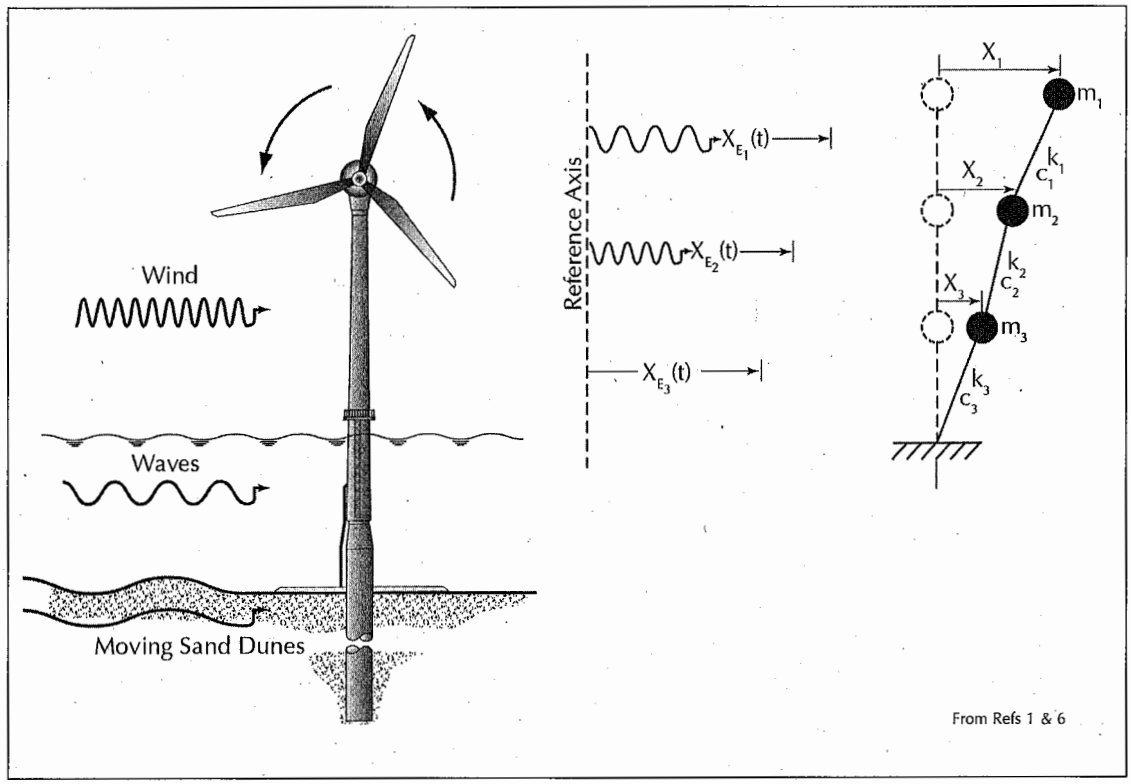

FIGURE 5. Loads from wind, waves, currents and moving sand dunes.

Wind speed data are height dependent. To develop a design wind speed profile, a logarithmic or an exponential wind speed profile is often used. In areas where hurricanes are known to occur, the annual maximum wind speed should be based on hurricane data.

Hydrodynamic Loads. Site-specific measured wave data collected over long continuous periods are preferable. When site-specific wave data are unavailable, data from adjacent sites must be transformed to account for possible differences due to water depths and different seabed topographies. Because waves are caused by winds, the wave data and wind data should correlate. However, extreme waves may not occur in the same direction as an extreme wind. Therefore, the directionality of the waves and wind should be recorded.

Loads from Currents. Tidal and wind generated currents such as those caused by storm surge have to be included in the design. In shallower waters, usually a significant component of the hydrodynamic load is from currents.

Ice Loads. In areas where ice is expected to develop or where ice may drift, ice loads have to be considered in the design. The relevant data for sea ice conditions include the concentration and distribution of ice, the type of ice, mechanical properties of ice, velocity and direction of drifting ice, and thickness of ice.

Seismic Loads. For wind turbines that have to be located in seismic areas, a site response spectrum is usually developed for horizontal and vertical directions. For these analyses, the wind turbine is represented by a lumped mass at the top of the tower and it includes the mass of the nacelle, the rotors and part of the tower. Buckling analyses of the tower are conducted with the loads from the vertical ground acceleration.

Environmental Loading Conditions in the United States

Environmental loading conditions peculiar to the United States include hurricanes in the southeastern United States and the Gulf of Mexico, and northeaster storms along the east

coast from Maine to Virginia, and floating freshwater ice in the Great Lakes region. Hurricanes are large, revolving tropical cyclones that form well defined spirals with a distinct low pressure center and can be as large as 1,000 kilometers (621 miles) in diameter, traveling at a velocity of up to 11 meters (36 feet) per second. Wind data for a number of hurricanes that made landfall in the United States over a fifty year period are available. However, measured wave data from hurricanes are quite limited and simplified methods are often employed to estimate design load parameters.

Northeaster winter storms are generated in the winter at higher latitudes with colder air at their core and do not have a well defined spiral and are often much larger in diameter than hurricanes. Even though these storms produce winds with lower velocities than hurricanes, their larger diameter can develop bigger high energy waves. Approximately thirty northeaster storms occur in the northern portion of the Atlantic coast every year. Therefore, these storms must be considered in the determination of the wind turbine design loads.

In most European waters, sea ice is not a common phenomenon. It mostly occurs in the Barents Sea, northern and western parts of the Norwegian Sea and inland waters such as the Baltic and Skagerak. Moreover, most offshore wind turbines have been installed in saline water either in the North Sea or the Baltic Sea. The Great Lakes region of the United States consists of large bodies of fresh water and is more susceptible to the formation of floating ice than are salt seas. Floating fresh water ice is considered to be harder than salt water ice and will need to be considered in the design of support structures for turbines in these locations.

Application of Available Design Standards

If the established practice for the design of conventional fixed offshore platforms, as outlined in the API guidelines, is extended to the design of offshore wind turbine support structures, the designer must first appreciate the differences in the two types of structures and how they respond to applied dynamic loads.[7]

The assessment of the dynamic response of offshore wind turbines will differ from that of offshore oil and gas platforms and also onshore wind turbines. Offshore platforms are designed using static or quasi-static response calculations for external design loads, whereas offshore wind turbines are driven by a combination of wind, wave and current loading in non-linear dynamic analyses. The natural frequency of the offshore wind turbine is somewhere between the wave and rotor frequencies. On the other hand, fixed platforms for the offshore oil industry are usually designed to have natural frequencies well above the wave frequencies. Unlike common practice in the offshore platforms, frequency domain analysis of dynamic response is seldom used for offshore wind turbines. To assess the non-linear behavior of aerodynamically loaded rotors, time domain simulations are generally required for an accurate assessment of both fatigue and ultimate limits states. Since the operating state of the wind turbine varies along with variable wind conditions, a number of load cases need to be analyzed. Compared with onshore wind turbines, wave and current climate cause a large extension of the number of load cases. Moreover, the influence of the foundation and support structure on the overall dynamic behavior is much larger compared to that of an onshore wind turbine. For an offshore wind turbine structure, both wind and wave loading are dominant; while for typical oil and gas installations, wave and current loading dominate the design of foundations.

Extreme wave loads generally govern the design of conventional fixed offshore platforms with wind loads contributing a mere 10 percent to the total load. Therefore, existing offshore standards emphasize wave loading but pay little attention to the combination with wind loads. In contrast, the design of offshore wind turbines is generally governed by extreme wind, wave and current loads, with almost equal weight being given to wind and wave loads depending on the site location. In addition, given the highly flexible response of the wind turbine structure, fatigue loads are critical.

So far, a key assumption in the design of wind turbines in Europe is that the turbines

must be able to withstand extreme events with a return period of fifty years,[8] whereas the oil and gas industry structures are designed to withstand one hundred year events. Therefore, the resulting reliability for offshore wind turbines and conventional fixed offshore platforms is understandably different. Extending the use of design loads obtained from API in the design of support structure and foundation will result in a higher degree of conservatism for the foundation design than for the wind turbine and, consequently, lead to higher construction costs. For the design of wind turbines a ten-minute average wind speed is considered long enough to cover all high frequency fluctuations of the wind speed and short enough to have statistically stable values. This approach is significantly different from offshore platform design where one-hour average values are used.

For the design of offshore structures in the United States, three exposure category levels corresponding to the consequence of failure are considered.[9] Consequences would include environmental impact, danger to human life or economic loss. The failure of manned facilities or those with oil and gas storage facilities are considered of high consequence. Failure of platforms that may be manned but are evacuated during storms or do not have oil and gas storage is considered to be of medium consequence. Structures that are never manned and have low consequence of failure fall in the low consequence category. For the Gulf of Mexico, associated with each of these categories are a minimum wave height and period, wind speed and current speed to be used for design.

Offshore wind turbines are generally unmanned in storm situations so that the risk of human injury is low compared to typical manned offshore structures. Moreover, the economic consequences of collapse and the related environmental pollution are low. For now, offshore wind turbines are likely to fall in the low consequence category. But as they become more integrated into the power grid and supply more power to the grid, the consequences of their failure are likely to increase.

API guidelines suggest that the recurrence interval for the oceanic design criteria should be several times the design life of the offshore platform.[9] Typical offshore platforms have a design life of about twenty years and are designed using one hundred year return period oceanic criteria. However, for offshore wind turbine foundation design, a fifty year recurrence period is being used in Europe and appears appropriate for the United States as well.

Typical Support Structures

Support structures connect the transition piece or the tower to the foundation at the seabed level. In some cases, as in monopiles and gravity structures, the foundations serve as support structures as well by extending from the seabed level to above the water level and are directly connected to the transition piece or tower. Support structures for offshore wind towers can be categorized by their configuration and method of installation into six basic types, described below.

Gravity Structures. As the name implies, these foundations resist the overturning loads solely by means of their own gravity. These are typically used at sites where the installation of piles in the underlying seabed is difficult, such as on a hard rock ledge or on competent soil sites in relatively shallow waters. Gravity caissons are typically concrete shell structures, weighing as much as 1,300 tonnes (1,433 tons). These structures are cost effective when the environmental loads are relatively low, and the dead load is significant, or when additional ballast can be provided at a reasonable cost.

Monopiles. This design is a simple one in which the wind tower, made up of steel pipe, is supported by the monopile, either directly or through a transition piece. The monopile consists of a large-diameter steel pipe pile of up to 6 meters (20 feet) in diameter with wall thicknesses as much as 150 millimeters (6 inches). The weight of a typical monopile can range between 180 to 220 tonnes (198 to 243 tons). Depending on the subsurface conditions, the pile is typically driven into the seabed by either large impact or vibratory hammers, or the piles are grouted into sockets drilled into rock. Compared to the gravity base foundation, the monopile has minimal and localized environmental impact. By far,

the monopile is the most commonly used foundation for offshore wind turbines.

Guyed Monopile Towers. The limitation of excessive deflection of a monopile in deeper waters is overcome by stabilizing the monopile with tensioned guy wires.

Tripods. Where guyed towers are not feasible, tripods can be used to limit the deflections of the wind towers. The pre-fabricated frame is triangular in plan view and consists of steel pipe members connecting each corner. A jacket leg is installed at each corner that is diagonally and horizontally braced to a transition piece in the center. The tripod braced frame and the piles are constructed onshore and transported by barge to the site. One advantage of these types of foundations is that they do not require any seabed preparation.

Braced Lattice Frames. A modification of the tripod frame, the lattice frame has more structural members. The jacket consists of a three-leg or four-leg structure made of steel pipes interconnected with bracing to provide the required stiffness. In deeper waters (for example, greater than 50 meters [164 feet]), multiple braced frames can be used to form a giant tripod. The batter of each leg could be varied to obtain the required stiffness. Braced lattice frames have been used for deeper water installations in Scotland and are currently planned for wind farms offshore New Jersey.

Floating Tension Leg Platforms. These platforms are structures that are floated to the site and submerged by means of tensioned vertical anchor legs. The base structure helps dampen the motion of the system. Installation is simple because the structure can be floated to the site and connected to anchor piles. The structure can be subsequently lowered by use of ballast tanks and/or tension systems. The entire structure can be disconnected from the anchor piles and floated back to shore for major maintenance or repair of the wind turbine.

Considerations for the selection of support structures for offshore wind turbines include:

- required dynamic response in the given water depth;
- constructability and logistics of installation, including contractor experience and availability of equipment; and,
- Costs of fabrication, availability of steel and other materials.

Of these, the required dynamic response of the overall system in the given water depth is the main consideration. Since the dynamic response of a typical wind turbine depends on the stiffness of the support structure, which, in turn, is inversely proportional to the cube of its free standing height (or water depth), water depth can be used as a main factor for selecting the support structure in initial design. A survey of over thirty-seven wind farms in Europe provides typical water depths in which these support structures have been used (see Figure 6 on the next page).[1,10]

Typical Foundations

Foundations anchor the support structures to the seabed, and typically fall into the six types described herein.

Gravity Caissons. This type of foundation has been used for several offshore wind farms in Europe. For economical fabrication of gravity caissons, a shipyard or a drydock near the site is required that allows the massive foundation structures to be floated out to the site and sunk (see Figure 7 on page 19). Site preparation and placement required for gravity caissons typically involves dredging several meters of generally loose, soft seabed sediment and replacement with compacted crushed stone to prepare a level bed for the gravity caisson to rest on. Special screeds and accurate surveying is required to accomplish this task. Installation of these structures is relatively time consuming. For example, at the Nysted wind farm in Denmark it took approximately twenty-six to twenty-nine days to complete four gravity foundations.[11,12]

Driven Pipe Pile. The driven steel pipe pile option is an efficient foundation solution in deep waters. The typical method of offshore and near-shore installation of piled structures is to float the structure (monopile, tripod or braced frame) into position and then to drive the piles into the seabed using a hydraulic hammer (see Figure 8 on page 20). The handling of the piles requires the use of a crane of sufficient capacity, preferably a floating crane vessel. The use of open-ended driven pipe

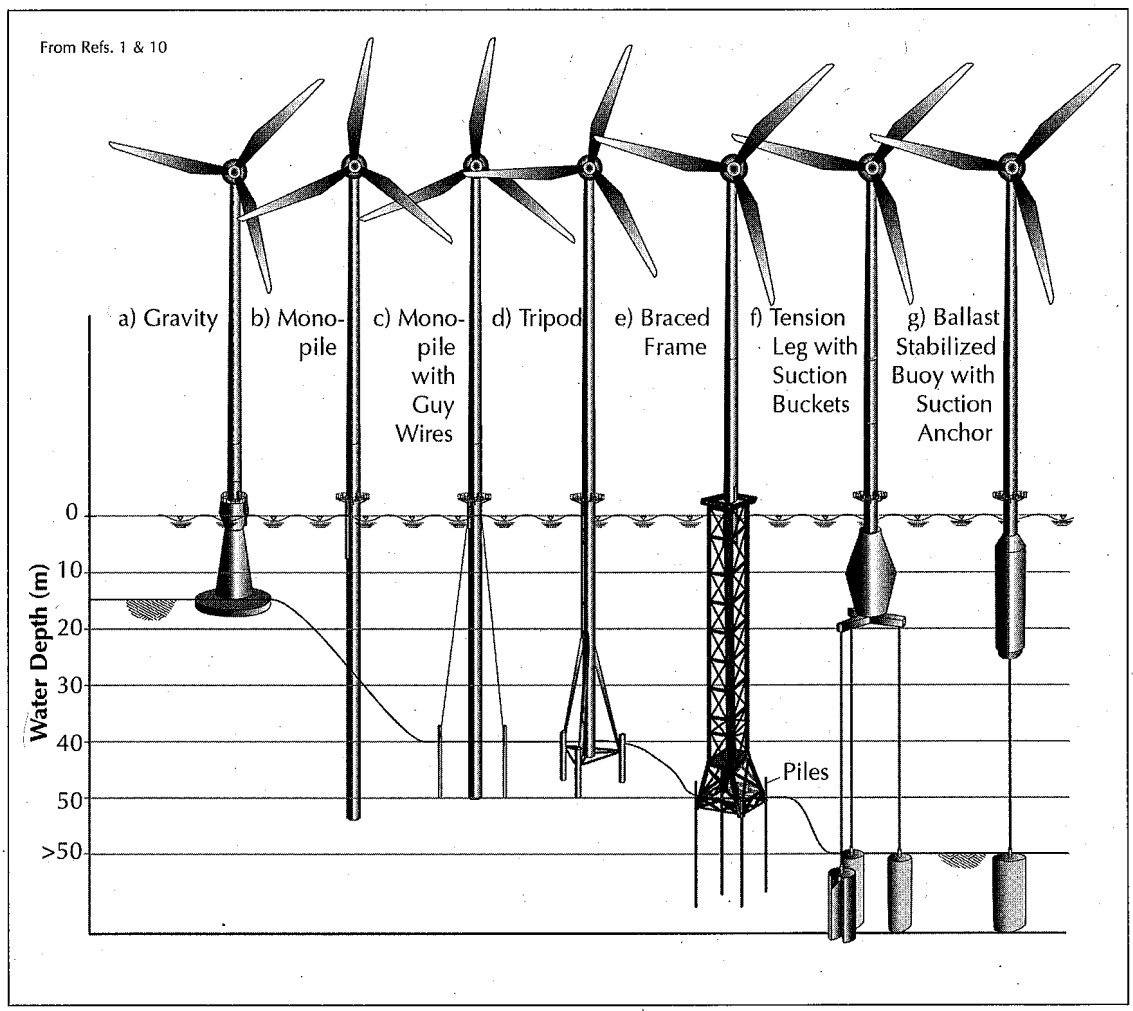

FIGURE 6. Various types of support structures and their applicable water depth.

piles allows the sea bottom sediment to be encased inside the pipe, thus minimizing disturbance. However, the noise generated during pile driving in the marine environment might cause a short-term adverse impact to aquatic life. Since the number of piles is typically few and spread apart, these adverse impacts are only short term and relatively minor. Installation times for driven monopiles are relatively short, taking less than twenty-four hours per monopile.[13,14]

Post-Grouted Closed-End Pile in Predrilled Hole. In this design, a closed-ended steel pipe pile is placed into a predrilled hole and then grouted in place (see Figure 9 on page 21). This option is often used for offshore pile foundations less than 5 meters (16.4 feet) in diameter and offers significant advantages over the cast-in-place drilled shaft option, including advance fabrication of the pile, better quality control and much shorter construction time on the water. This option requires a specially fabricated large-diameter reverse circulation drill. It also requires the handling and placement of a long, large-diameter pile with considerable weight. Closed-end piles can be floated to the site and lowered into the drill hole by slowly filling them with water. Installation times for drilled and post-grouted monopiles are relatively long, averaging about fifty hours per monopile.

Drilled Shafts. The installation of bored, cast-in-place concrete pile requires driving a relatively thin-walled (25 millimeters [1 inch])

casing through the soft sediment to the underlying denser material (if necessary to establish a seal), then drilling through and below the casing to the required base elevation. Bending resistance is provided by a heavy reinforcing cage utilizing high-strength, large-diameter bars, with double ring where necessary. The casing provides excavation support, guides the drilling tool, contains the fluid concrete and serves as sacrificial corrosion protection. This approach requires a large, specially fabricated reverse circulation drill.

Composite "Drive-Drill-Drive" Pile. Where pile driving is not feasible to achieve the design depth, an adaptation of existing drilling and piling techniques involves a combination drive-drill-drive sequence to achieve the design depth. Installation times for drilled and driven monopiles are relatively long, averaging about seventy to ninety hours per monopile as in the North Hoyle Wind Farm in the United Kingdom.[13]

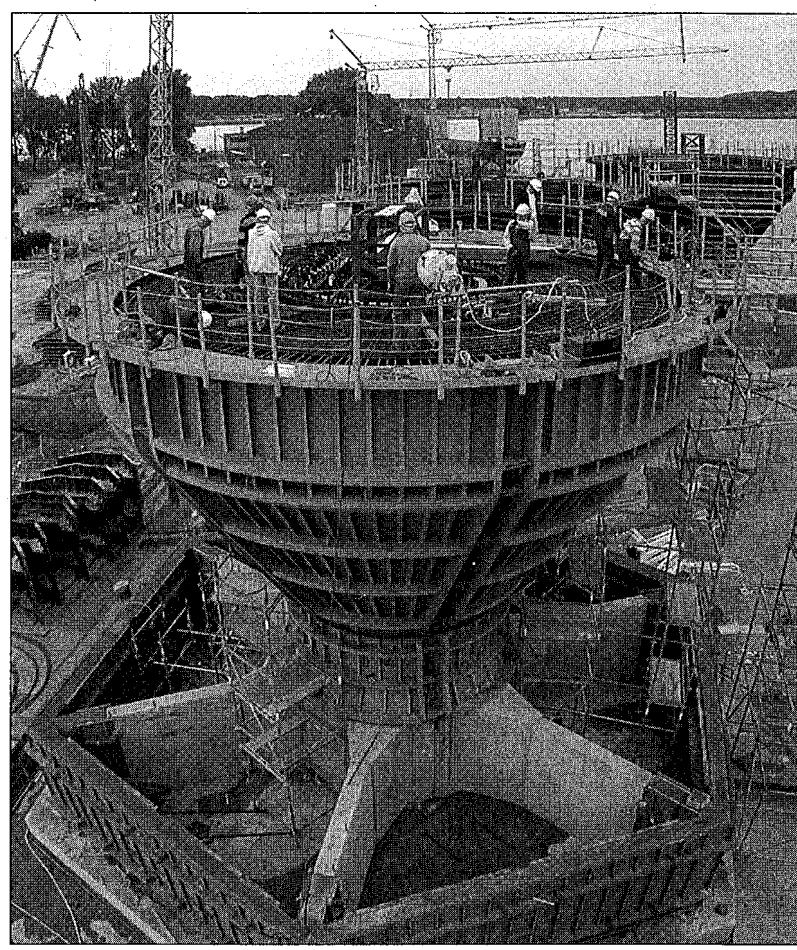

FIGURE 7. Gravity base foundation being constructed for the Nysted Offshore Wind Farm at Rødsand, Denmark. ***(Courtesy of Bob Bittner, Ben C. Gerwick, Inc.)***

Suction Anchor. Suction anchors consist of a steel canister with an open bottom and closed top. Like piles, suction anchors (see Figure 10 on page 21) are cylindrical in shape but have larger diameters (10 to 15 meters [33 to 49 feet]) and subsequently shallower penetration depths. They are installed by sinking into the seabed and then pumping the water out of the pile using a submersible pump (see Figure 11 on page 22). Pumping the water creates a pressure difference across the sealed top, resulting in a downward hydrostatic force on the pile top. The hydrostatic force thus developed pushes the anchor to the design depth. Once the design depth is achieved, the pumps are disconnected and retrieved. The installation process is relatively time consuming, as evidenced at the Horns Rev II Wind Farm where it took about thirty-two hours to complete installation, of which ten hours were spent on penetration.[15] Suction anchors resist tension loads by relying on the weight of the soil encased by the steel bucket along with side friction on the walls and hydrostatic pressure. The stability of the system is ensured because there is not enough time for the bucket to be pulled out of the soil during a wave passage. As the bucket is pulled up, a cavity is formed

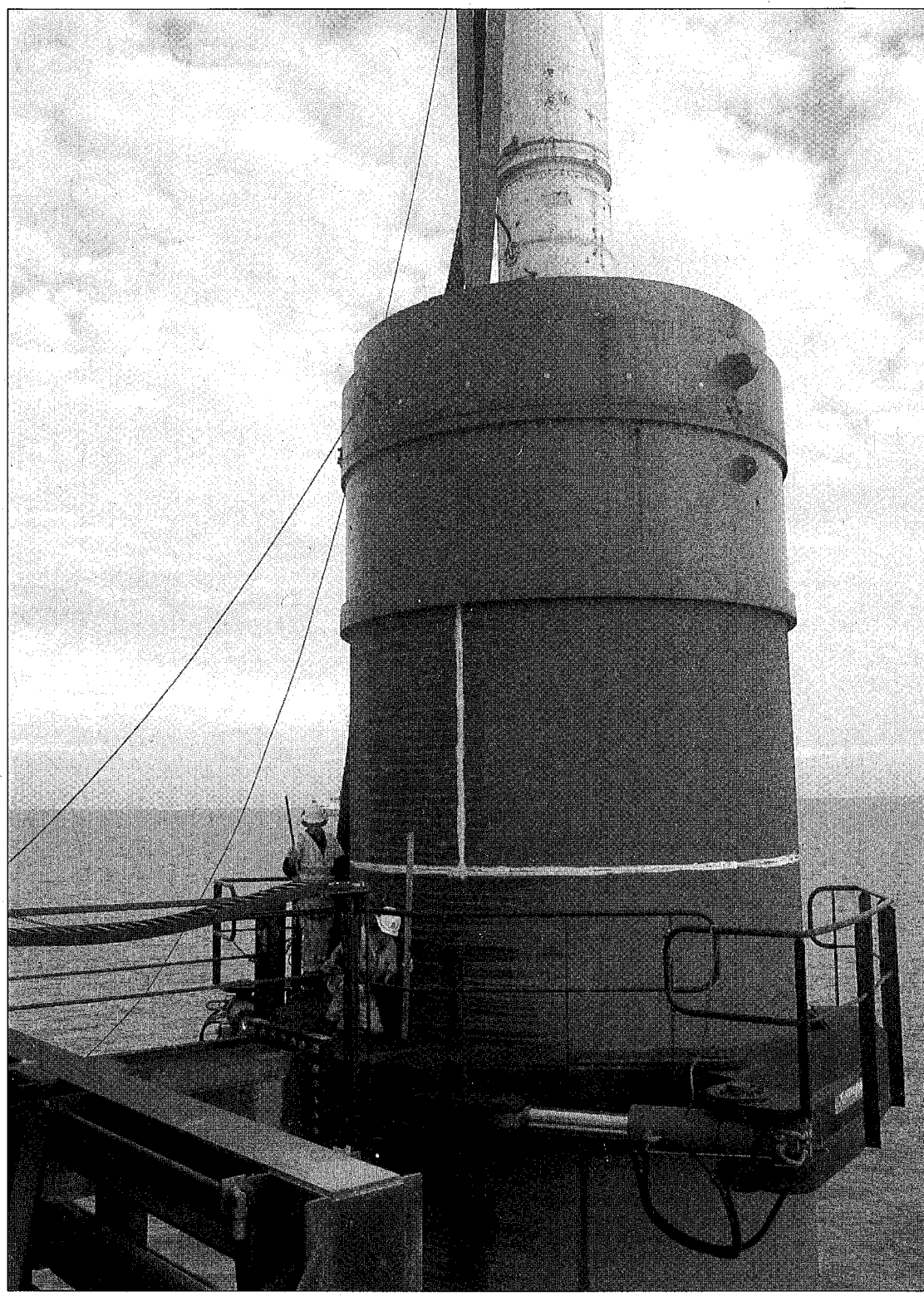

FIGURE 8. Pile driving hammer atop a steel pipe pile at the Kentish Flats Offshore Wind Farm, United Kingdom. ***(Courtesy of Elsam.)***

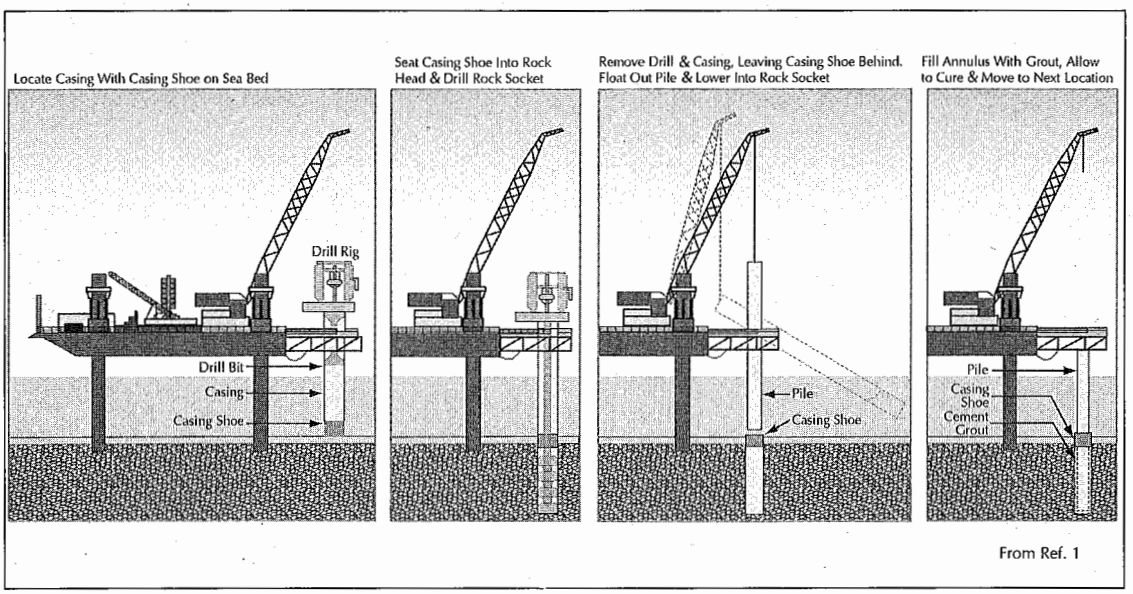

FIGURE 9. Typical installation sequence for a post-grouted closed-end pipe pile in a predrilled hole.

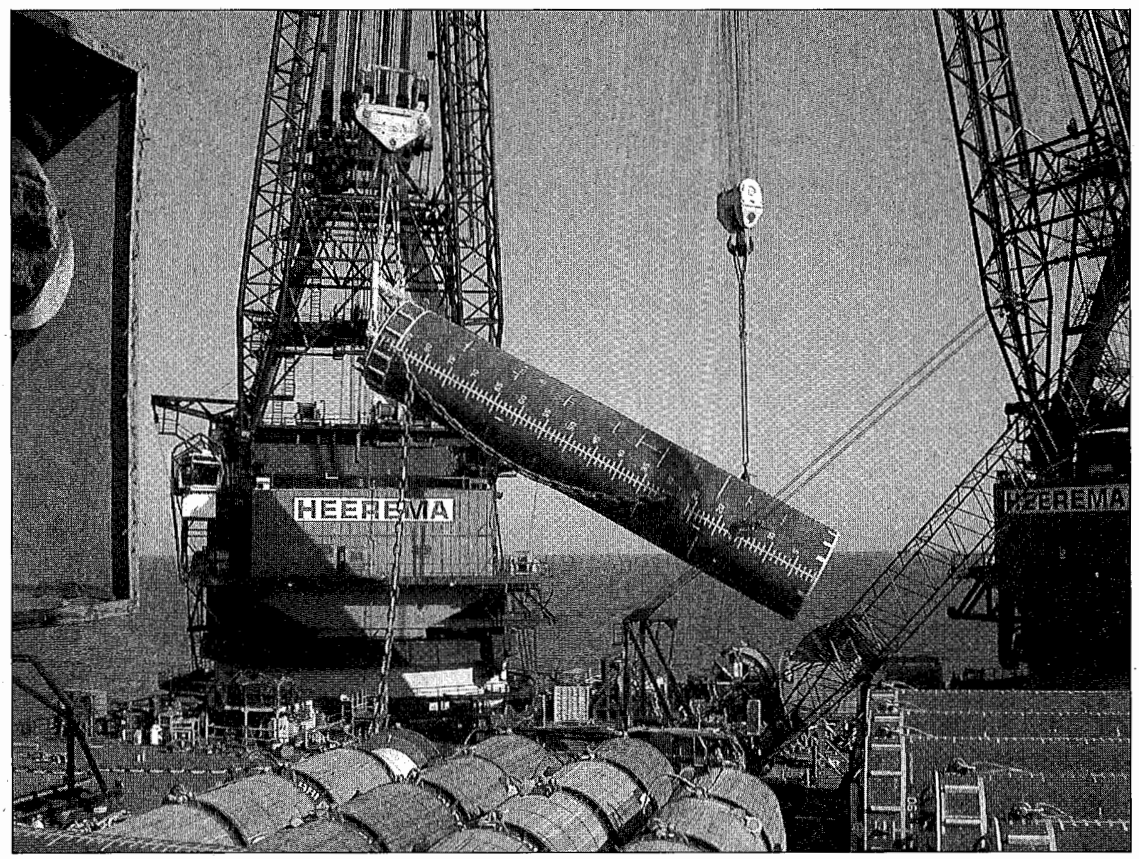

FIGURE 10. Suction anchors for an offshore platform being transported to site in the Gulf of Mexico. ***(Courtesy of Professor Aubeny.)***

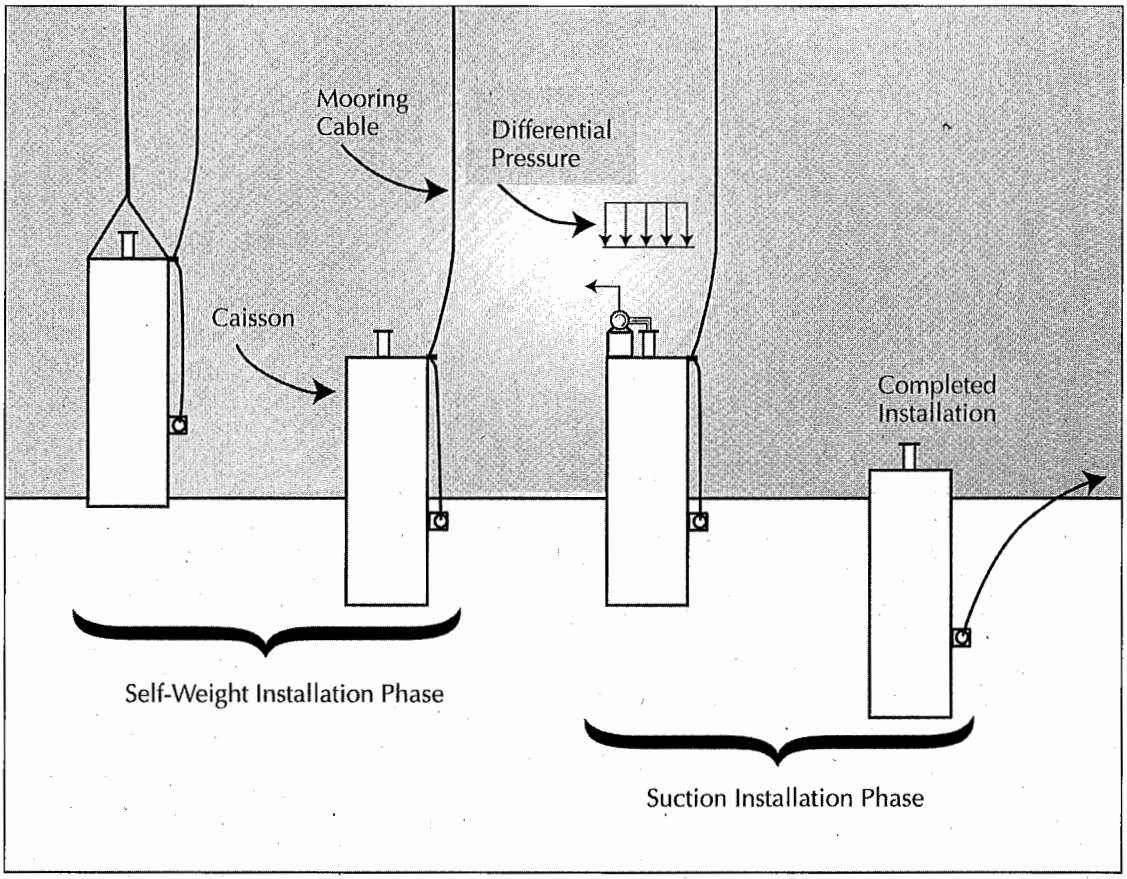

FIGURE 11. Installation stages of a suction anchor.

between the soil surface and the bottom of the bucket that creates a suction pressure that resists the uplift loads. These foundations carry compression loads by side friction and end bearing. Since they do not encounter severe dynamic stresses caused by driving, the wall thickness for suction anchors is generally less than driven piles. Suction anchors are expected to be particularly suitable for foundations in the type of soft cohesive sediments found around the U.S. coasts. These foundations cannot be used in rock, in gravel or in very dense sand. When compared to piles, suction anchors are cheaper to install since they do not require underwater pile drivers. Moreover, the installation process is relatively quiet, thereby reducing noise effects on marine life. At the end of a wind turbine's life, a suction anchor may be removed completely from the seabed, unlike piled foundations. This design approach provides room for recycling and reuse.

Foundation selection considerations for offshore wind turbines include:

- soil conditions that facilitate installation and performance;
- driveability for driven piles and penetrability for suction anchors;
- constructability and logistics of installation, including contractor experience and availability of equipment;
- costs of fabrication, availability of steel and other materials; and,
- environmental impact considerations.

Soil conditions at a project site will generally drive the method of installation and constructability aspects. Driven monopiles are most adaptable to a variety of soil conditions. They are currently the most commonly used

foundation for offshore wind turbine projects. Their construction procedure can be modified to suit the site conditions encountered. For example, in the presence of cobbles and boulders or very dense sands, the contractor may use a sequence of drilling and driving to achieve the required design depth for the monopile. In cohesive till and in soft rock, drilled shafts or post-grouted closed-end pipe in a drilled hole may be most suitable. Gravity base foundations will be feasible in shallow water, where competent bearing stratum, such as a rock ledge or glacial till is available at shallow depth. Suction caissons are feasible in soft clay strata and medium dense sands. The final selection of the foundation may be driven by other factors such as environmental impact, costs of construction, availability of equipment and contractor preference.

Environmental Impacts of Foundations

Foundation construction and installation will affect the marine environment and their impacts must be assessed. If drilled shafts are selected as the foundation, then the issue of disposing the excavated material will need to be addressed. The larger areas of seabed required for gravity base foundations also pose significant disturbance to the seabed environment. To limit the area of dredging required for the gravity base foundation, some form of ground improvement should be considered. The use of open-ended driven pipe piles allows the sea bottom sediment to be encased inside the pipe, thus minimizing excess spoil.

Airborne Noise. During the construction of offshore farms, airborne noise from construction work (vessels, ramming, pile driving, etc.) will likely affect birds and marine mammals, but since the construction operations are of limited duration, these effects are expected only to be temporary. However, sensitive time periods like breeding or nursery periods should be avoided if the construction site is placed near important biological areas — which may be in conflict with the intentions of the developers to establish offshore wind farms when stormy weather is least probable.

Underwater Construction Noise. During construction, underwater noise from construction vessels and drilling or piling equipment may have a detrimental effect on marine mammals, fish and benthos. These effects are especially evident when driving monopiles. Noise from pile driving can either cause behavioral changes, injury or mortality in fish when they are very close to the source and exposed to either sufficiently prolonged durations of noise or elevated pressure levels. Accurately analyzing and addressing these effects is somewhat complicated. The sound pressure levels and acoustic particle motion produced from pile installation can vary depending on pile type, pile size, soil conditions and type of hammer. Furthermore, the diversity in fish anatomy, hearing sensitivity and behavior, as well as the acoustic nature of the environment itself (such as water depth, bathymetry and tides) further complicate the issue of how fish are affected by pile driving noise and how severe those effects may be. Experiences from a variety of marine projects in the United States and offshore wind farm projects in Sweden indicate that baro-trauma from pile driving noise results either in mortality or trauma in fish, resulting in loss of consciousness and drifting on the water surface as if they were dead. However, the effect is considered temporary. Noise from construction work during sensitive periods such as larvae season may result in a very high fish mortality rate. Accordingly, construction work during larvae season should be avoided. Generally, peak sound pressure levels of more than about 160 decibels at a reference pressure of 1 micro-Pascal are considered harmful to aquatic life and marine mammals.[16]

Available approaches to mitigating pile driving noise include:

- prolonging hammer impact duration;
- using vibratory hammers;
- using an air bubble curtain or bubble tree; and,
- isolating the pile with a foam-coated casing.

When hammer impact is prolonged, it results in smaller velocity amplitudes of pile vibrations and lower frequencies, thereby lowering overall noise levels. Installing piles using

vibratory hammers operating with continuous vibration frequencies between 20 and 40 Hertz also reduces noise that is 15 to 20 decibels less than impact driving. However, such vibratory hammers are effective only under a limited range of soil conditions and for relatively small piles. A bubble curtain involves pumping air into a network of perforated pipes surrounding the pile. As the air escapes the perforations it forms an almost continuous curtain of bubbles around the pile. The bubbles prevent the sound waves from being transmitted into the surrounding water. Noise reductions on the order of 10 to 20 decibels have been realized with bubble curtains. With foam coated steel casing, noise reductions of 5 to 25 decibels (depending on frequency) have been achieved.[16]

Underwater Operational Noise. During operation, noise from offshore turbines can be transmitted into the water in two ways: the noise either enters the water via the air as airborne sound, or the noise is transmitted into the water from tower and foundation as structural noise. The frequency and level of underwater noise is therefore to a certain degree determined by the way the tower is constructed and by the choice of foundation type and material (steel monopile or concrete gravity caisson). Underwater noise from offshore wind turbines must of course exceed the level of underwater background noise (ambient noise, especially from ships) in order to have any effects on marine fauna.

Measurements from offshore farms Vindeby in Denmark (caisson foundation type) and Bockstigen in Sweden (monopile) indicate that underwater noise is primarily a result of the structural noise from the tower and foundation.[17] When the results were scaled up, based on measurements from a 2 megawatt onshore wind turbine, it was concluded that the underwater noise might be audible to marine mammals within a radius of 20 meters (66 feet) from the foundation. Generally, it is believed that for frequencies above 1 kiloHertz, the underwater noise from offshore turbines will not exceed the ambient noise, whereas it is expected that for frequencies below 1 kiloHertz, noise from turbines will have a higher level than the background noise. Only measurements and impact studies after the construction can reveal if underwater noise will really affect marine mammals. The effects on fish from low frequency sounds (infrasound, below 20 Hertz) is uncertain. The effects from noise and electromagnetic fields on fish communities living at the seabed are still a subject of further study.

Foundation Design Considerations

Geotechnical Investigation. Since foundation construction costs can balloon from unanticipated subsurface conditions (such as paleochannels), the presence of boulders or foreign objects (such as shipwrecks, abandoned anchors, pipelines or lost casings from prior explorations), a thorough bathymetric survey and a geotechnical investigation program are essential to managing such risks. With the purpose of defining the soil stratification and investigating the seabed for unidentified objects, a geotechnical investigation program should be performed in at least three phases: a geological desktop study, followed by a bathymetric and geophysical survey, and then by cone penetration test (CPT) soundings and geotechnical borings. A wind farm site typically extends over a substantial area and there may be significant variation in subsurface conditions across the site. The generally limited number of geotechnical borings may not reveal the true variation in subsurface conditions across the site. In this regard, a geologic study and geophysical methods become invaluable in alerting the designer to the range of variation in subsurface conditions across the site. The wind turbine array may need to be modified based on the subsurface conditions (see Figure 12). Geophysical methods usually based on seismic and sonar methods, including side-scan sonar and sub-bottom profilers, can be used to obtain information on seabed stratigraphy and topography over the proposed site along with information on any foreign objects. With the purpose of confirming the results of the geophysical profiling, cone soundings should be performed in an array across the site. These soundings should be supplemented by several borings located strategically across the study area. Proper surveying methods and locating sys-

tems must be employed to ensure that the bathymetry, geophysical methods, geotechnical soundings and borings, and the actual installation are all controlled to the same positioning relative to each other. To determine the geotechnical engineering characteristics of the soils, a program of laboratory soil testing must be performed.

The geotechnical investigations must extend throughout the depth and areal extent of soils that will affect or will be affected by the installation of the planned structure and foundation. The number of borings will depend on the foundation type selected and the anticipated geologic conditions. In general, one boring per turbine foundation is recommended. Since the main expense of performing an offshore drilling program is related to mobilizing the drill barge, performing fewer borings may not be cost effective. Moreover, obtaining geotechnical information at foundation locations goes a long way in reducing risk. If pile foundations are selected, the borings and soundings should extend at least 5 to 10 meters (16 to 33 feet) beyond the preliminary estimated depth of pile foundations. If a gravity structure is selected, the borings should extend to a depth where the stress increase in the soil and the resulting strain caused by the gravity structure is negligible. Moreover, multiple borings may be required for each gravity structure to account for variation in subsurface conditions. From all borings, soil samples must be collected with great care in order to minimize the degree of disturbance to the samples. All in all, a geotechnical engineer experienced in offshore investigations should be retained for the planning and executing of such investigations. The geotechnical engineer should be able to modify the initially proposed plan based on the site conditions encountered. The program should be equipped to switch between soil sampling, rotary coring and in-situ testing where needed. For cable routing studies, relatively shallow cone soundings or vibro-core borings may be performed to obtain information for the cable trench.

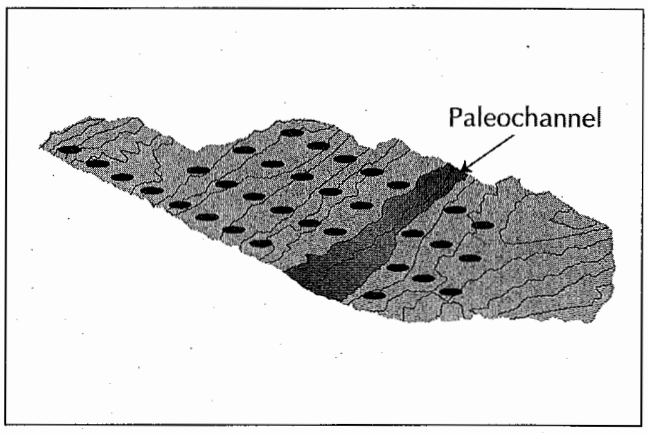

FIGURE 12. A paleochannel filled with soft mud at a wind farm site requiring a change in the layout.

Local Geology & Geologic Hazards. Geologic hazards that have the potential for causing the failure of offshore engineered structures include slope failures, fault ruptures and adverse soil conditions. Such hazards can be triggered by external events, including earthquakes and surface storms. For instance, in the Gulf of Mexico, hurricanes Ivan, Katrina and Rita destroyed 118 oil drilling platforms and caused substantial damage to underwater pipelines by triggering slope failures.[18] Pipelines and undersea cables face several geohazards including:

- submarine slope failures including rotational or translational failures and sheet flows;
- sea floor rupture associated with seismic activity; and,
- rough sea floor where the cable or pipeline may have to bridge over rock outcrops and channels.

Seafloor Mudslides. If the seafloor at the proposed site of the wind farm is even mildly sloping, it could be susceptible to localized or areal sea-wave or earthquake-induced mudslides or submarine slides that could adversely affect the foundations. Numerous submarine slides have been documented on the coasts of continental United States (see Figure 13 on the next page). Identifying slope stability hazards in the marine environment is a challenging and important task. Marine slope failures can occur on slopes with gradients as low as 2 to 6 degrees. But their most notable feature is their large aerial extent.

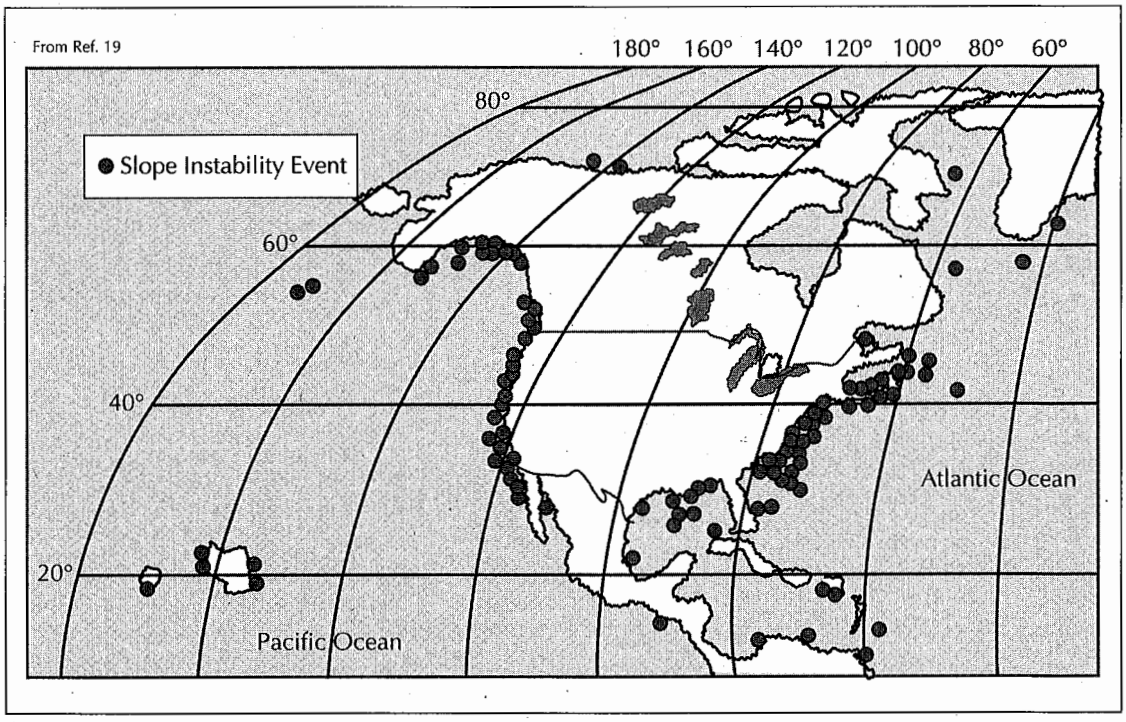

FIGURE 13. Documented submarine slope failures in the United States.

For example, the Sigsbee Escarpment in the Gulf of Mexico and the Storegga Slide in the Norwegian Sea are among the largest documented submarine slides in the world. The Storrega Slide has occurred over a 290 kilometer (180 mile) stretch of the coastal shelf, stretching out almost 800 kilometers (497 miles) and covering an area of about 34,000 square kilometers (13,130 square miles).[18] A submarine slide can either result in mud flow or the sliding mass will stop in a short distance. Debris flow analyses require an estimation of the depth of debris and run-out distances. The seismic stability of submarine slopes should also be evaluated. Moreover, slide masses originating offsite could potentially impact the wind farm foundations. Therefore, these issues should be identified early in the process since they can significantly influence site selection. Where identified, the potential loading from seafloor mudslides should be incorporated in foundation design. Seafloor mudslides can potentially overstress the foundations by lateral loads and damage buried cables.

Scour & Moving Sand Dunes. The removal and deposition of seabed soils caused by waves and currents is an ongoing geologic process that may be further exacerbated when the natural flow regime is interrupted by engineered structures. For an offshore wind farm, different types of scour need to be considered:

- local scour occurring immediately around the foundation;
- global scour occurring to a large extent as a shallow scoured basin due to effects of multiple foundations; and,
- overall seafloor movements, including sand waves, ridges and shoals.

Another phenomenon that occurs as a result of large-scale morphological events is the overall movement of the seabed resulting in sand waves. The movement of sand waves can result in several meters of accumulation in a few years. For example, at the Scroby Sands Wind Farm in Norfolk, United Kingdom, the sand bank level is expected to vary as much as 9 meters (30 feet) during the design life of twenty years. A similar scenario was expected at the offshore wind farm off Long Island, New York.

The consequence of scour is either the reduction of lateral and vertical support for

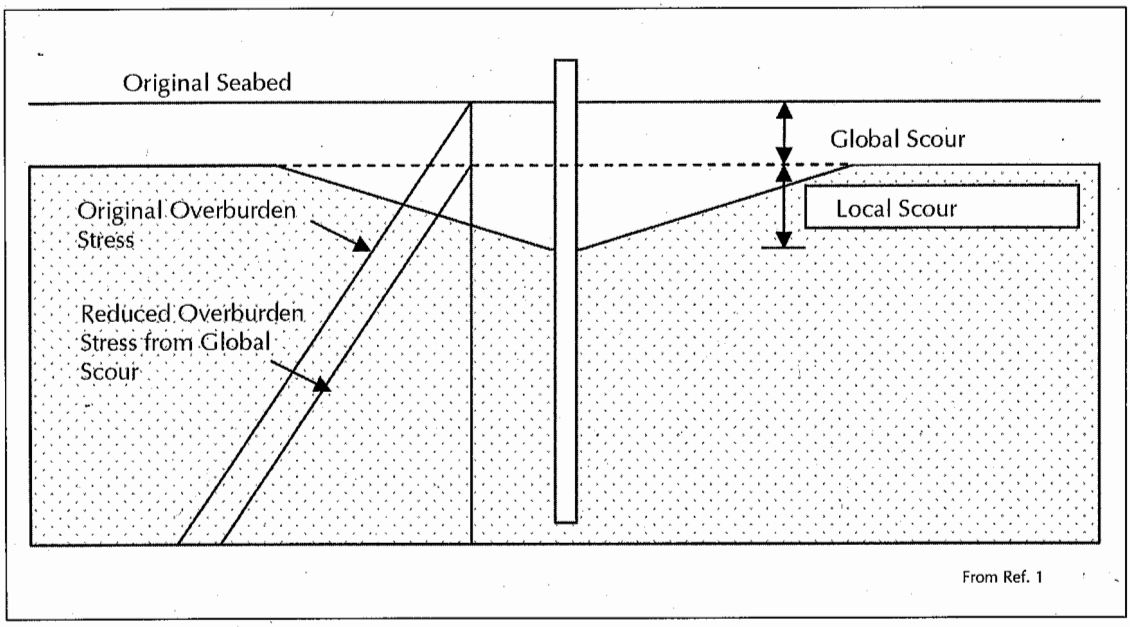

FIGURE 14. Overburden stress reduction due to scour.

foundations, or the imposition of additional forces resulting in settlement or overstressing of the foundations (see Figure 14). Scour will have an impact on the length of foundation embedment, the natural frequency of the wind turbine system and the J-tube. For monopiles, the lateral load resistance is offered by soils primarily in the upper five to ten pile diameters, and the effect of scour on monopile design is rather pronounced. Moreover, changes in seabed level conditions — whether by scour or by sand waves — will result in a decreased or increased lateral pile stiffness and natural frequency. In design, the most probable combination of maximum scour depth and maximum extreme loads must be used for foundation analyses. As noted earlier, the power cable from each turbine is inserted in a J-shaped plastic tube that carries the cable to the cable trench. In the event of scour, the cable would be freely spanning over the scour hole. Since the function of the cable is crucial to the whole operation, any damage to the cable, whether by scour or other factors, must be prevented. Fatigue response of the structure is particularly sensitive to any changes in the dynamic stiffness of the foundation.

Seismic Considerations. The level of seismicity of the wind farm site should be assessed either through available data or detailed investigations. If the area is determined to be seismically active and the wind turbine will be affected by such activity, then site-specific response spectra and design criteria should be developed. Depending on the location of the site, seismic hazards of liquefaction, seismic settlement, lateral spreading and earthquake loads could adversely impact the wind tower foundations and should be addressed in the design. The potential for seismically induced sea waves, also known as tsunamis, should also be assessed. Earthquake loads seldom drive the design for wind turbines. For conceptual level studies, seismic ground accelerations may be obtained from prior studies or relevant seismic hazard maps.

Liquefaction Potential. Soil liquefaction, defined as a significant reduction in soil strength and stiffness as a result of an increase in pore pressure during dynamic loading, is a major cause of damage to built structures during earthquakes. Typically, the hazard from liquefaction occurs in four ways:

- bearing failure;
- settlement;
- localized differential lateral movements; and,

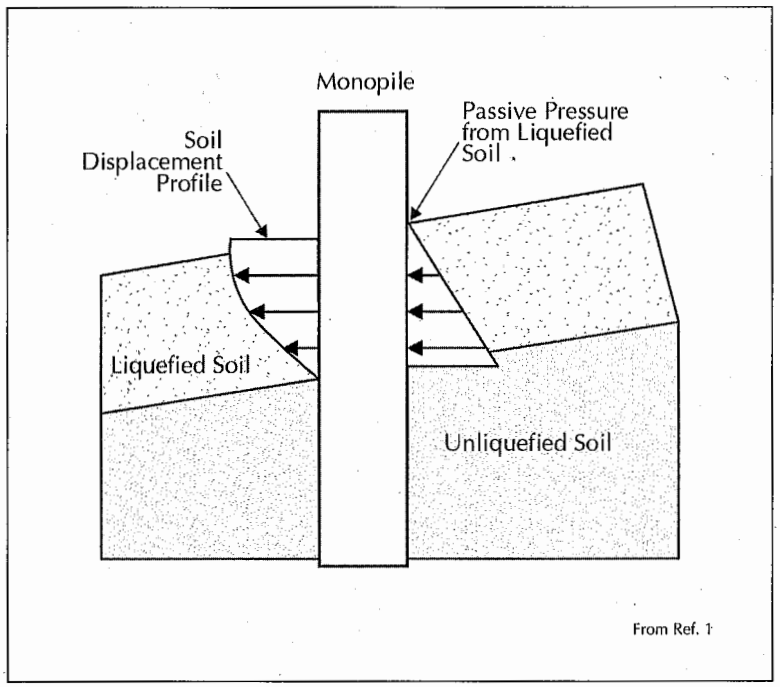

FIGURE 15. Forces on a monopile foundation from lateral spreading of liquefied seabed soils.

- ground loss or highly localized subsidence associated with the expulsion of material such as "sand boils."

Usually, for soil liquefaction to occur offshore, two conditions must exist:

- presence of loose, sandy soils or silty soils of low plasticity; and,
- a source of sudden or rapid loading, typically associated with earthquakes.

Generally, soil conditions found at offshore wind farm sites would be susceptible to liquefaction should an earthquake occur. Therefore, the liquefaction potential at all wind farm sites should be assessed in the early phase of design development.

Lateral Spread Potential. Lateral spreading occurs primarily by the horizontal displacement of soils at the seabed due to the liquefaction of underlying granular deposits. The degradation in the undrained shear resistance arising from liquefaction may lead to limited lateral spreads induced by earthquake inertial loading. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear biases on essentially horizontal ground. The determination of lateral spread potential and an assessment of its likely magnitude ought to be addressed as a part of the hazard assessment process for an offshore wind farm site. In the event that there is a potential for lateral spreading at a wind farm site, it should be incorporated into the design of the foundation. For instance, if pile foundations are employed, passive forces from the moving soil mass should be applied and the foundation checked for moment and shear capacity and related deflection (see Figure 15). Alternately, the expected soil displacement may be applied to the pile model in order to evaluate the related shear and moment developed in the pile.

Foundation Analyses

Gravity Base Foundations. In gravity base foundations, the passage of waves produces dynamic horizontal forces concentrated on the top of the structure. At the same time, pressure fluctuations of the passing crest and trough reduces the apparent weight of the gravity structure and creates a mode for vertical vibration. With or without earthquake forces, the eccentricity of the wave forces results in coupled rocking and sliding modes of vibrations. During a major storm, the simultaneous occurrence of vertical, sliding and rocking loads would create high stress concentrations at the edges of the foundation, resulting in excess pore pressures and possible localized yielding of foundation soils below the foundation edge opposing the storm. Following the passage of the storm, the excess pore pressures would dissipate, causing settlement and related differential settlement possibly resulting in

the tilting of the foundation. Therefore, for gravity foundations, uplift, overturning, sliding, bearing capacity, lateral displacement and settlement are potential failure modes that require evaluation. Immediate, primary and secondary consolidation settlement, permanent horizontal displacements and resulting differential settlements also have to be evaluated. Differential settlements should incorporate the lateral variation of the soil conditions, unsymmetrical weight distributions, predominant directional loading and seismically induced settlement. The required sliding resistance determines the minimum weight of the system based on Coulomb's relation for frictional material. It has been found that the heave force on the gravity base is a more dominant factor leading to instability by sliding than the overturning moment from the aerodynamic loads.[20] The heave force can be calculated using Bernoulli's theorem for the undisturbed wave kinematics (instead of the more complicated diffraction theory). Vertical bearing capacity is computed using bearing capacity theory developed by Prandtl, Brinch Hansen and Terzaghi. In bearing capacity calculations, the inclined and eccentric loads on the gravity base can create a severe reduction in the effective area and allowable bearing capacity. Therefore, reduction factors for inclined and eccentric loads have to be applied. Because of its sensitivity to the ratio of the vertical and lateral loading, an evaluation of bearing capacity and hydrodynamic heave and lateral forces is required for many phases of the oncoming design wave (see Figure 16 on the next page). The design of the gravity base foundation is iterative in the sense that the large contribution from hydrodynamic loading on the gravity base depends on the shape and size of the gravity structure, which is in turn is governed by its load bearing function (see Figure 17 on page 31).

Dynamic analyses of gravity base foundations require considering soil-structure interaction effects. For homogeneous soil conditions, the soils can be modeled as an elastic half space with an equivalent shear modulus. Foundation stiffness coefficients can be determined based on elastic theory. These stiffness coefficients can be used in structural analyses for wind and wave loading on the turbine and its support structure. To account for the strain dependency of shear modulus and internal soil damping, a range of shear modulii and damping should be considered in developing foundation springs. In the case where heave from wave passage and excessive overturning moments lead to the formation of a gap at the base of the gravity foundation, these procedures overestimate the foundation stiffness. Gravity base structures are generally massive and constitute a majority of the weight of the wind tower system. Therefore, the fundamental period of vibration of the gravity base is small compared to the rest of the wind tower system. Since the modes of vibration of the wind tower and the gravity base are substantially different, an uncoupled analysis would be expected to provide reasonable estimates of foundation behavior (see Figure 18 on page 32).

Deep Foundations. Geotechnical design methods for driven pile foundations focus on three aspects:

- axial load carrying capacity in compression and tension;
- response under cyclic lateral loads; and,
- installation.

The main objective of the design is to select a foundation size that can develop the required axial capacity, perform adequately under lateral loads without excessive deflection and rotation at mudline, and withstand the installation stresses needed to penetrate the pile without buckling the walls. A typical driven pile design will require the prediction of:

- penetration resistance with depth;
- depth to fixity under lateral loads;
- minimum required penetration depth for fixity and axial capacity considerations; and,
- driveability evaluation.

The bearing capacity and lateral load deformation performance of the pile is evaluated using finite element procedures or other available procedures using the *p-y* approach. Another paper provides a detailed treatment of axial pile capacity for pipe piles.[21]

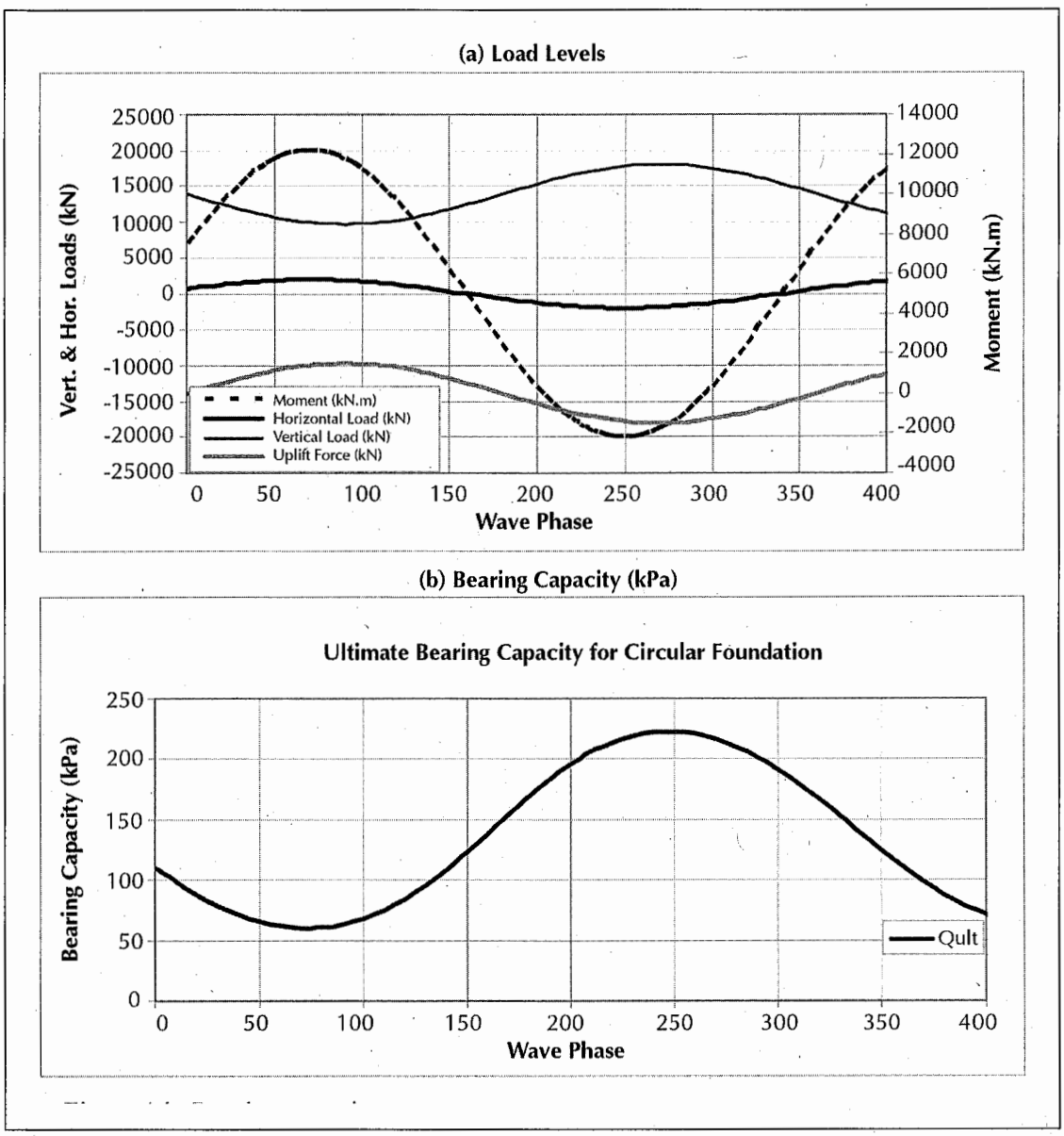

FIGURE 16. Bearing capacity of a gravity base with varying load levels.

Lateral load analyses of pile foundations are performed with a variety of methods (see Figure 19 on page 33). The most common methods to perform these analyses are:

- a finite difference model of a beam supported on non-linear *p-y*, *t-z* and *Q-w* springs to represent the pile-soil interaction;
- coupled foundation stiffness matrix;
- uncoupled stiffness matrix; and,
- equivalent cantilever with an effective depth to fixity.

Soil-pile-structure interaction under such cyclic loading is a complex process involving several simultaneously occurring phenomena such as cyclic degradation, pore water pressure generation and gap-slap-scour near the mudline. All these factors may occur simultaneously to modify the axial and lateral stiffness and the response of the pile foundations.

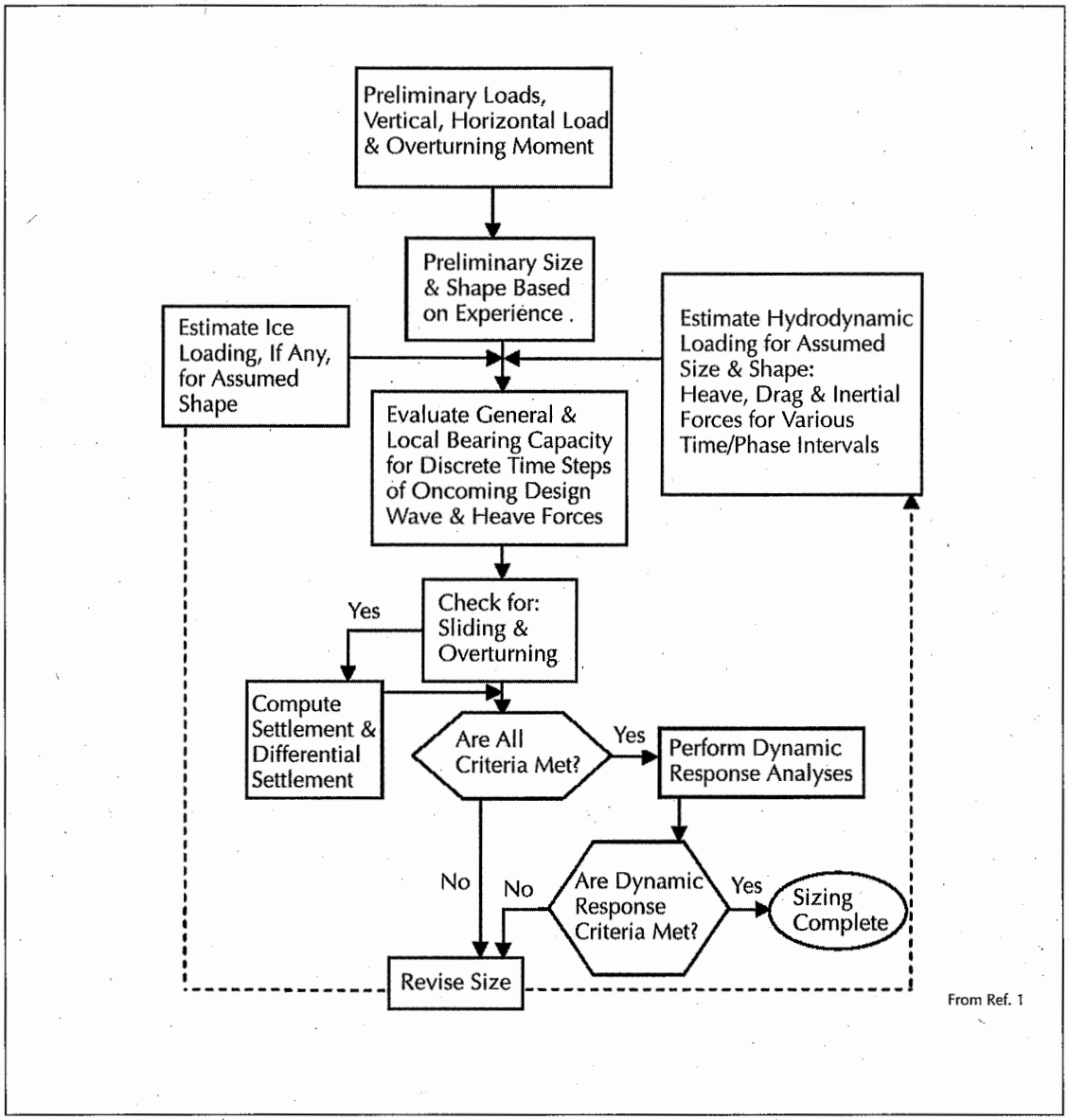

FIGURE 17. The design process for an offshore gravity base structure.

Space limitations prevent inclusion of a detailed discussion of these phenomena.[22]

Axial Response. The axial response of a pile driven into soil and subjected to dynamic loading depends on the characteristics of the loads, the pile geometry, the method of pile installation and the stress-strain characteristics of the soils surrounding the pile. Cyclic vertical loads generate cyclic shear stresses in the soils adjacent to the piles and alternating compressive-tensile stresses in the soils adjacent to the pile tip. If the cycling is intense enough, the pile-soil bonds along the shaft may be broken down so that only a dynamic friction exists together with a tensile and compression resistance at the tip. The ultimate skin friction would be influenced by factors such as cyclic degradation and rate effects. It has been shown that there is a definite trend towards an increase in pile head settlement with the number of cycles and the level of cyclic loading. Therefore, for foundations subjected to contin-

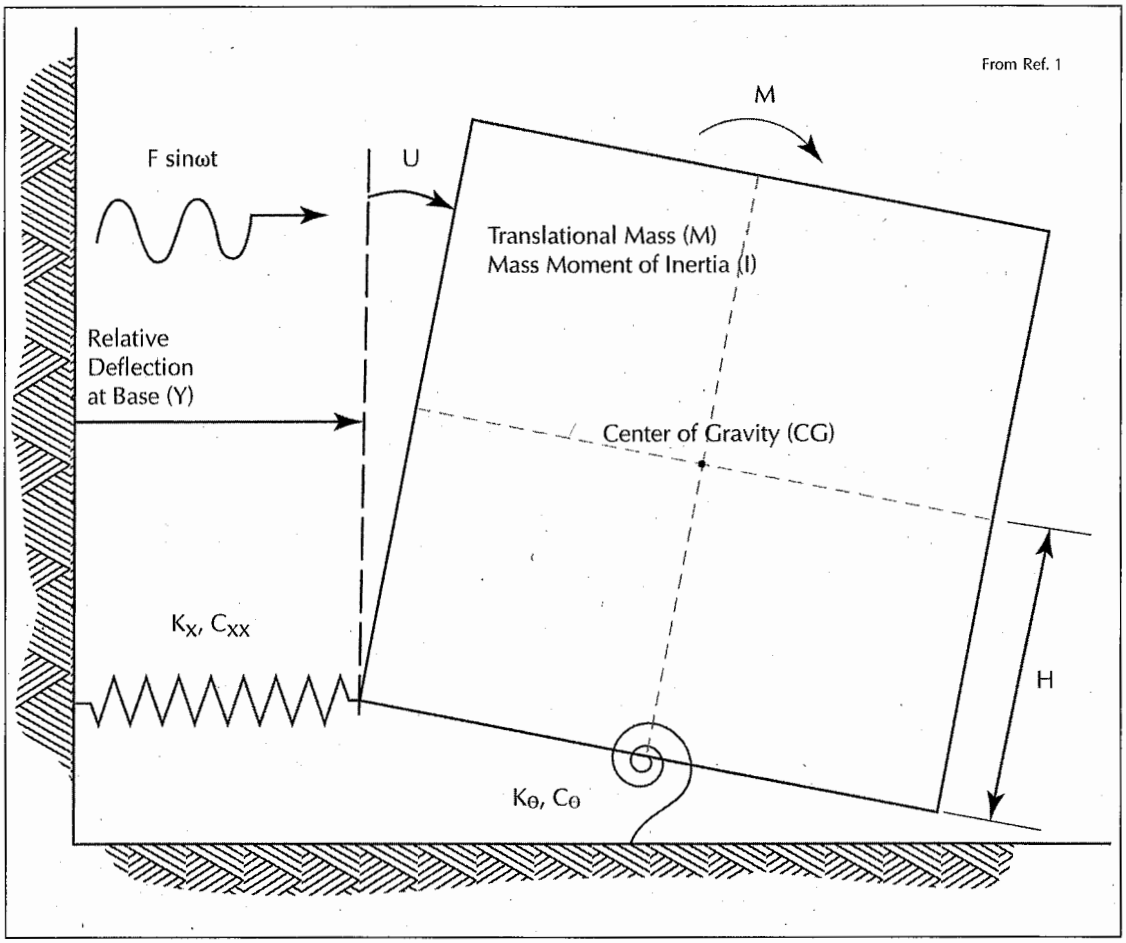

FIGURE 18. An analytical model of a gravity base showing uncoupled translational and rotational springs.

ual cyclic loads such as those from waves and turbines, pile head displacements could accumulate, causing foundation settlement. Available literature indicates that significant degradation can occur with cyclic loading of piles in sands.[23,24] Model tests performed by Chan and Hanna showed significant loss of resistance reflected by rapid increases in pile displacement under the cyclic loads that are a fraction of the static load capacity.[23] The degradation of pile resistance to cyclic loading is attributed to increases in induced pore pressure, destruction of interparticle bonds, realignment and rearrangement of the soil particles. All these factors combine to reduce the side resistance on the pile.

Lateral Response. The lateral pile response is affected by cyclic degradation, pore pressure generation and loss of resistance due to gap formation near the mudline. The pore pressures will increase because of cyclic loads on the foundation. These increases in pore water pressure result in a decrease in vertical effective stress and a consequent decrease in shear strength that, in turn, results in a softer lateral response and axial response. It is anticipated that the loading from the turbine will also produce pore pressures that result in a softer lateral response.

Cyclic loading from waves, and also the turbine, will produce pile motion in opposing directions. When the soil starts deforming beyond the elastic range, the contact between the adjacent soil is reduced effectively, forming a gap between the pile and the soil. When gap formation occurs at shallow depths, it will

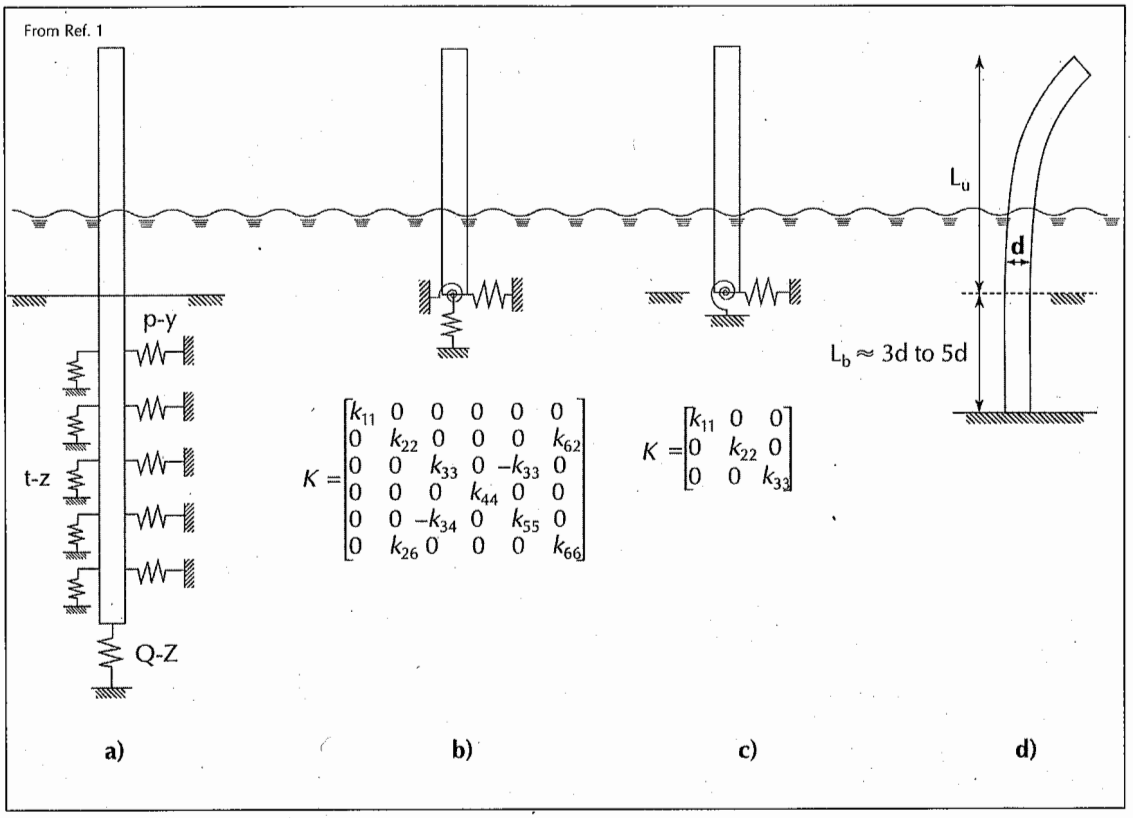

FIGURE 19. Available analytical models for deep foundations: a) beam column on non-linear spring; b) coupled stiffness matrix; c) uncoupled stiffness matrix; and, d) equivalent cantilever with effective depth of fixity.

further reduce the lateral resistance of the piles. The response of soil to lateral loads is usually modeled by load p versus deflection y curves that are derived from pile load tests conducted at various sites. These lateral load tests were performed on small-diameter piles either under static conditions or they involved cycles of loads where each cycle lasted several hours at a time. The available p-y curves for cyclic loads are derived from slow-cyclic tests with each cycle lasting several hours, which allowed ample time for pore pressure dissipation. However, some gapping and cyclic degradation were observed in these tests. Therefore, the cyclic p-y curves have inherent in them the effects of gap formation and some cyclic degradation, but the effects of pore water pressure generation are not included. Using the same set of p-y curves for larger (3 to 6 meter [10 to 20 foot]) diameter piles may not be entirely appropriate. For larger diameters, a significant side shear force component and a related moment and an additional base shear force and a moment associated with the normal pressures at the base of the pile would be generated, making the pile response stiffer. No field data on load transfer at the base is available currently and this topic requires further study.

Torsional Response. Since the side friction/adhesion will not only resist axial loads but also torsional moment loads, cyclic degradation, pore pressure generation and the aforementioned phenomenon of gap formation will also modify the torsional response of the monopile. Therefore, appropriate conservatism should be employed in static design.

Pile Driveability. Driveability evaluations are a crucial part of the design process. Wave equation analyses are typically performed to assess the range of pile hammer energies

required to drive the piles. The major purpose of this study is to:

- estimate the feasibility of driving the piles to the design depth with the selected hammer;
- to evaluate pile tensile and compressive stresses and pile driving resistances for the selected range in hammer energies; and,
- to develop preliminary driving criteria.

While available offshore pile driving hammers with a rated energy of 3,000 kiloJoules or more are capable of installing piles with diameters as large as 4.5 meters (15 feet), newer higher capacity hammers with adaptors for even larger pile diameters are being developed. Space limitations prevent inclusion of a case study on the driveability of very large diameter piles.[1]

Issues Pertaining to Lateral Response of Large-Diameter Piles. Another issue of current interest is the effect of increased pile diameter on lateral response and how it is modeled with available numerical models. Typical loading conditions for the 5 megawatt (and larger) offshore turbines in water depths of 20 meters (66 feet) or more will require pile diameters up to 6 meters (20 feet), which are pushing the envelope of current experience in design and installation. Current procedures for the analyses of laterally loaded piles are of semi-empirical nature[25] and only indirectly consider the effect of the diameter on the pile behavior. Moreover, they have not been validated with load tests on very large-diameter piles. Although piles with diameters of up to 4.5 meters (15 feet) have been recently designed and installed,[26] no data on their long-term pile response are available. Hence, any extrapolation of available methods of lateral load analyses to large-diameter monopiles for offshore wind turbines requires an evaluation of the effects of scale on the pile response under lateral loads.

A review of available literature shows that only a few studies have been published to date. Stevens and Audibert found that the conventional *p-y* curves significantly over-predicted the displacements for larger diameters and published a correction for normalizing the standard curves.[27] A number of studies have reported on lateral load tests on piles with diameters of 0.27, 1.22 and 1.83 meters (1, 4 and 6 feet) at a site where the clay was over-consolidated.[28-30] They found that Matlock's *p-y* curves over-predicted displacement and suggested a modification. Other researchers have also performed three-dimensional finite element modeling to evaluate laterally loaded piles of large diameters in sand.[31-33]

Juirnarongrit and Ashford concluded that the effect of pile diameter was relatively insignificant in the small deflection range but increased somewhat near ultimate loading where shaft rotation and non-linearity increased.[31] Due to the simplicity of the three-dimensional finite element model used in their study, some other important aspects could not be incorporated that could have made significant contributions to the effect of pile diameter on the pile response. These aspects include soil-pile separation, increased friction between the soil and pile, and the effect of soil confinement.

Abdel-Rahman and Achmus evaluated a relatively short 7.5 meter (25 foot) diameter pile, 20 meters (66 feet) long and found that *p-y* models under-predicted deflections — *i.e.*, showed a stiffer lateral response compared to a finite element model.[33] The details of the finite element model, however, were not available. Lesny *et al.* evaluated a pile with diameter of 6 meters (20 feet) and length of 38.9 meters (128 feet) and drew a similar conclusion.[32] Their results indicated that, compared to the load transfer approach, the pile response determined by finite element analysis is somewhat softer and results in a deeper depth of fixity. Both researchers, however, indicated that their results have not been verified with actual field test observations. It is somewhat unclear whether appropriate soil characteristics at the applicable stress range and yield models relating to cyclic loading, gap-separation and alteration from pile driving were selected in these analyses.

More recently, numerical modeling studies for large caissons for bridge foundations have been performed by Professor George Gazetas and his co-workers.[34-37] They have developed a non-linear Winkler model that, through a

pre-defined pinching function, can model gapping and slippage. However, these models are complicated and cannot readily be used by the practitioner.

The key to obtaining useful solutions using the finite element method is characterizing the material properties accurately. Even if appropriate material properties can be obtained from sophisticated testing in the laboratory, the behavior of deep foundations poses two significant problems:

- how substantially the soil properties were modified during pile driving; and,
- how soil properties change during continual cyclic loading.

Neither of these two issues have been addressed sufficiently enough for a practitioner to readily quantify them in a numerical model. Perhaps the main shortcoming of the finite element method in analyzing a pile under lateral load is that it requires that the engineer obtain clearly defined stress-deformation relationships for the soil prior to, and also after, pile installation at various distances from the pile. In addition, if the loading is repetitive or extreme, it would require further knowledge about the soil response to cyclic loads and large strains.

In any finite element method analysis, the use of an appropriate interface element (gapping and slippage) between the pile and the soil is of utmost importance. Constitutive models of soil and pile, an interface element between pile and soil, and the degradation of soil stiffness under cyclic loading can simulate the actual behavior of pile and soil only once it has been calibrated to actual field load tests. The application of the finite element method to deep foundations under various loading conditions must by necessity use the results of complicated testing that determines the characteristics of the soil as a function of distance from the structure and incorporates the effects of installation and the nature of loading. Nevertheless, the approach is quite useful in performing parametric studies and sensitivity analyses in pile design.

Physically, the effect of diameter on pile response could be explained as follows: For a slender pile of small diameter, as the pile flexes under lateral load, there is some rotation of the pile segments. Any vertical shear resistance offered by the soil to the rotation of the pile segment is considered insignificant. However, for larger diameter piles the greater surface area causes the vertical shear resistance and related moment offered by the soil to the rotation of the pile segment to become large enough to be significant. In addition, the installation method (such as post-grouted piles, drilled shafts or driven piles) would likely alter the lateral response of the pile. If the piles are relatively short, a rigid body rotation mode dominates the lateral response of the pile. In this case, the response of the pile soil system is affected by the deformation of the soil not only near the ground surface, but also adjacent to the pile tip and beneath the pile tip. Base shear along with the resisting moment associated with the normal pressures at the base of the pile become significant. If base slippage or base separation occurs, it should be included in the model. Accordingly, any laterally loaded pile model should incorporate components of axial load transfer.

Load transfer curves obtained from field testing offer a relatively simple and practical way of modeling laterally loaded pile behavior.[38] A load transfer model used in the design of large-diameter suction caissons was developed by Wang and Arrellaga.[39] It incorporates well accepted *t-z*, *Q-w* and *p-y* curves and can accommodate axial tip resistance and base separation along the rim of the caisson. Such a model could be used for large-diameter piles as well.

In summary, the issue of the effect of pile diameter on lateral response requires further study involving laboratory testing, scale models, field testing and numerical models. Until that time, a degree of conservatism is required in soil parameter selection and in designing pile lengths with the load transfer approach. Further discussion on the subject is provided in another study.[1]

Suction Caissons. Geotechnical design methods for suction caissons focuses on two aspects:[39]

- installation and retrieval; and,
- compressive and uplift capacity.

The main objective of the design is to select a caisson size that can develop the required compressive and uplift capacity and withstand the installation suction needed to penetrate the caisson without buckling the walls or failing the soil plug. A typical suction caisson design will require the prediction of:

- penetration resistance with depth;
- required suction with depth;
- optimum location of pad eye for lateral loads (if any);
- self-weight penetration depth; and,
- maximum penetration depth.

Once the caisson is sized, a caisson penetrability study is performed to ensure that the soil resistance offered is less than the driving force on the caisson. The driving force on the caisson consists of the buoyant weight of the caisson and the applied suction. The maximum attainable value of suction is limited by:

- the absolute pressure at which the water cavitates, resulting in pump failure;
- the minimum absolute pressure that can be achieved by the given pump; and,
- the minimum relative pressure that can be achieved by the pump.

In addition to the limit imposed by the maximum available suction, there is a limit to the depth of penetration that can be achieved by applying suction. If the difference between the vertical stress inside and outside the caisson at the level of the caisson tip exceeds a certain amount, then local plastic failure may occur and further penetration may not be possible. This mechanism may be thought of as a "reverse" bearing capacity problem in which the soil flows into the caisson. A detailed discussion of the design process with design examples (see Figure 20) are provided in Malhotra.[1] The capacity of a suction caisson to resist tensile loads is sensitive to the dynamic characteristics of the load. Moreover, sediment movement and local scour are important considerations that can affect caisson performance. Other factors that must be considered for suction caisson design are:

- the set-up time required to develop full shaft resistance; and,
- formation of a gap under inclined loading and potential loss of side resistance.

For preliminary design purposes, suction caissons are analyzed in their final installed condition as gravity base foundations with the theoretical foundation base at the caisson tip level. However, since the suction caissons encounter cyclic oblique tensile loads, other methods of analyses have been developed. The performance of suction caissons to cyclic oblique tensile loads has been a subject of intensive study in the last two decades. Procedures and design equations have been developed from the results of model tests performed at the University of Texas and at Oxford University.[40,41] During detailed design, the bearing capacity and load deformation performance of the caisson is evaluated using finite element procedures or other available procedures using the *p-y* approach. For initial sizing purposes, the designer may assume a length to diameter ratio between 5 and 8. Buckling considerations will govern wall thickness.

Fatigue

Fatigue is the process of gradual damage done to materials when subjected to continually changing stresses. Due to these repeated stress changes, the material slowly deteriorates, initiating cracks that will eventually propagate and lead to eventual failure. Offshore wind turbines are by default subjected to loads varying in time from wind as well as waves. This condition means that the stresses on the support structure will also vary continuously, making them prone to fatigue. Tubular steel structures (such as monopiles, braced lattice frames and tripods in deep water and exposed to wave loading) are particularly susceptible to fatigue. To be able to take fatigue into account in the design process, an empirical design method for the design of steel structures is commonly used.[7] Fatigue evaluation during design involves comparing the intended design life of the structure with its predicted fatigue life as limited by "hot-spot" stresses — *i.e.*, areas of

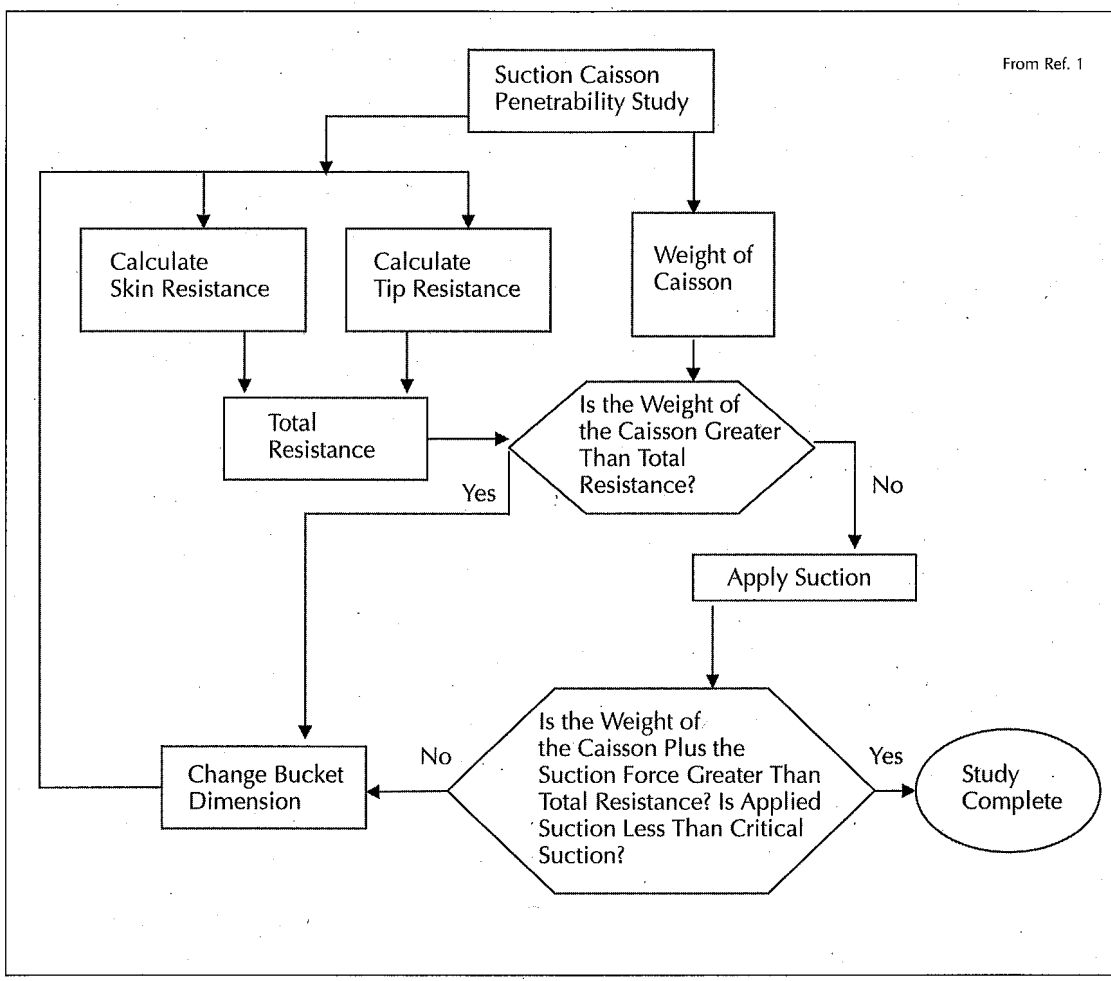

FIGURE 20. Typical flow diagram in the design of suction caissons.

high local stress reversals. Fatigue, which spans the lifetime of the structure, is particularly sensitive to any changes in the dynamic stiffness of the foundation. Small increases in stress levels will result in exponential increase in fatigue damage. Experimental data on tubular steel joints indicate an exponent of about 4.[7]

Minor amounts of scour can result in relatively minor changes in foundation stiffness and consequent cyclic stresses, yet result in major changes in fatigue response. It can be shown that, for a typical monopile, lowering the natural frequency of the foundation by a small amount (5 to 8 percent), will have a dramatic effect on fatigue (almost 100 percent).[1] At the lower natural frequency of the pile, more wave energy will create a resonant response of the wind turbine and increase fatigue. Therefore, great emphasis is placed on foundation selection and foundation dynamics.

Other Considerations

Corrosion Protection. Since the most intensive corrosion typically occurs in the splash zone, the transition piece is usually provided with a heavy duty protective coating. To protect the underwater part of the transition piece and piles against corrosion, sacrificial cathodic protection is used in addition to the shop-applied coating.

Scour Potential. Since scouring can lower the foundation stiffness and fatigue performance, potential scouring of the seafloor should be carefully evaluated during the design phase

with a site-specific study. Typical scour protection measures include scour mats made up of buoyant polypropylene fronds and polyester webbing that is anchored securely to the seabed or crushed rock mattresses.

Marine Growth. The plant and animal life on the site causes marine growth on structural components in the water and in the splash zone. The potential for marine growth should be addressed, since it adds weight to the structural components of the system and may increase hydrodynamic loads. Special paints and coatings are available that prevent marine growth.

Impact from Ship Collision. Accidents between ships and wind farms can result in damage to the wind farm, the ship and the local environment in a variety of ways. Although the probability of occurrence of ship collisions is relatively low, the consequences could be serious. Mitigating measures are therefore warranted. Generally, mitigation techniques involve a two-pronged approach:

- reduce the frequency of collision; and,
- reduce the consequences of collision.

Collision risks can be reduced by passive measures, such as proper marking of the wind farm and the individual wind turbines using classical techniques as marking lights, painting, buoys or by active measures such radar-based ship detection in combination with emergency towing capabilities. Collision damage can be reduced by classical fendering techniques designed based on the existing experience with pier, ship, tug and buoy fendering and specifically developed for offshore wind farm applications. Damage reduction solutions for offshore wind farms are different from classic applications used for marine bridge foundations. For individual monopile-supported wind turbines, the designer has to balance the need to absorb, without damage, the impact of small vessels with the requirement of total collapse of the monopile when the colliding ship is a larger oil or chemical tanker in order to avoid damage to the tanker. The collapse mechanism is facilitated with a structural fuse, such as a plastic hinge mechanism forming at or below the sea-bed.

Construction Considerations

Transportation. Monopiles, tripods, braced frames and suction caissons are usually transported to the site on barges. For the larger diameter monopiles, the ends are capped and sealed and they are floated to the site (see Figure 21). Gravity caissons either are floated to the site, and then filled with ballast or transported by barge (see Figure 22). The turbine is usually transported in one piece. However, the newer turbines require that the components be transported separately and be assembled on the barge or on the tower.

Erection. A jack-up rig serves as a fairly stable platform for carrying out the operation of installing the piles, the tower, nacelle and rotors. However, its inherent stability brings with it a lack of maneuverability during installation of the tower (see Figure 23 on page 40). Ship-shaped vessels and flat-bottom barges are most commonly available but provide the least stability for construction work. Ship-shaped vessels with rotating cranes offer the optimum mix of stability and maneuverability in carrying out the construction work (see Figure 24 on page 40).

Offshore Electric Cabling. Undersea power transmission cables are deployed using an environmentally sensitive process called "hydro-plowing" in which the cables are buried 2 to 3 meters (6.5 to 10 feet) under the ocean floor. This process uses high-powered jets to fluidize a pathway in the sea floor. The cables are then laid in the pathway and are buried as the sediment settles around them. This technique is minimally invasive and quickly returns the sea bed to its original form.

Construction Operations Planning

Significant savings in cost and schedule can be achieved by evaluating each step of the construction process and assuring that it is indeed necessary and being performed in the most efficient manner. Minor changes in design can often lead to significant changes in the construction process, such as reducing the number of construction steps or removing a con-

struction step altogether. Therefore, designers should be familiar with the construction processes involved and be able to consider a constructability review as well as a schedule and cost analysis.

The wind farm at Scroby Sands in Norfolk, United Kingdom, is an example of how minor design changes can lead significant savings in construction schedule and costs. The designers decided to make minor modifications to the monopile by welding a flange to which the wind tower could be bolted, thereby eliminating the transition piece and expensive grouting used to connect it to the monopile altogether.[13]

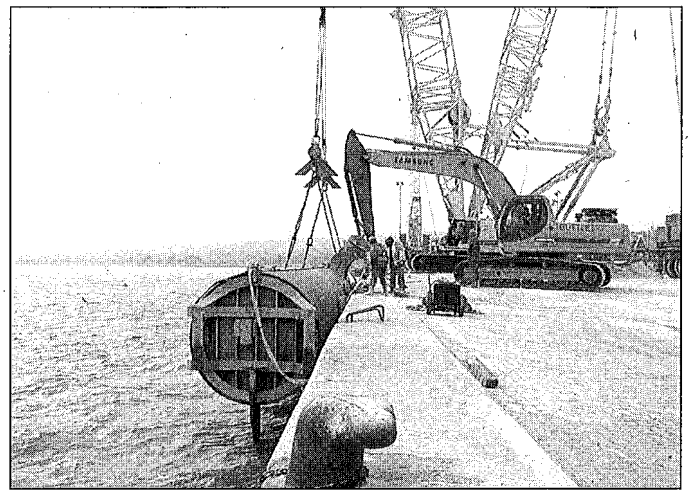

FIGURE 21. Monopile being floated to site at North Hoyle. ***(Courtesy of RWE npower.)***

The Nysted Wind Farm at Rodsand in Denmark involved fabricating the gravity bases on the barge itself at a casting yard off Poland's Baltic Coast.[11] The significantly lower labor cost, along with the more efficient construction process, led to considerable cost savings.

The gravity bases used at Middlegrunden Wind Farm, near Copenhagen, Denmark, were designed using composite construction — *i.e.*, a steel cylinder was encased in reinforced concrete and filled with ballast to create the gravity base.[42] Composite construction saved substantial construction time and led to cost savings.

Similarly, transportation and erection means and methods can also be streamlined by minimizing the number of operations required to perform the work. For example, the monopiles and the wind tower can all be placed on the same jack-up barge before towing to the site, thereby reducing the need for another ship.

Trends in Wind Farm Economics

The economic feasabilty of offshore wind farms could be improved by extending the design life of the turbines and amortizing their cost over a longer period and by investing in towers and foundations designed to enable future upgrades to larger, higher capacity turbines. As a late entrant in the offshore wind farm industry, the United States could benefit considerably by using fewer larger 6 to 10 megawatt turbines located further from shore (instead of more smaller turbines closer to shore), resulting in much less visual and environmental impact. Other trends include the use of multiple technologies to garner energy

FIGURE 22. Gravity bases being lowered from barge by a special crane at Nysted. ***(Courtesy of Bob Bittner, Ben C. Gerwick, Inc.)***

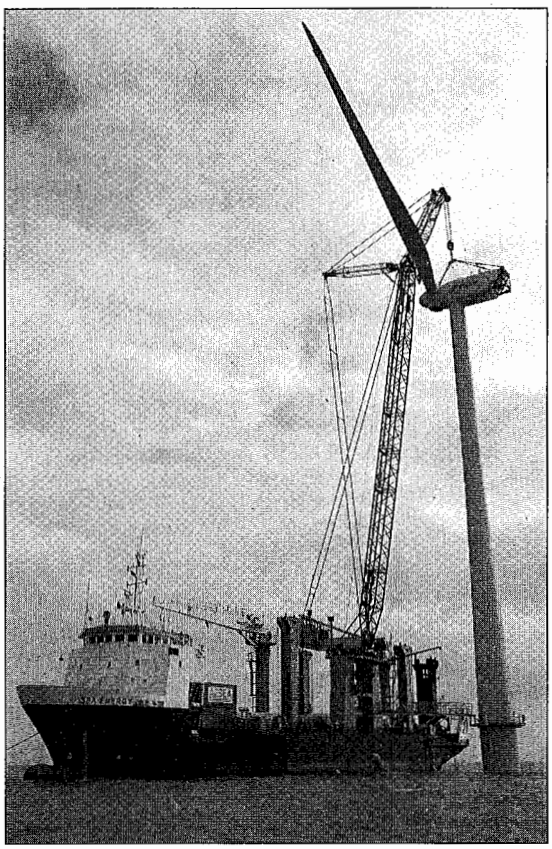

FIGURE 23. Wind turbine being erected with a crane from a special ship at Kentish Flats Offshore Wind Farm in the United Kingdom. ***(Courtesy of Elsam.)***

FIGURE 24. Wind turbine being erected with a crane on a jack-up barge at Thornton Bank Offshore Wind Farm in Belgium. ***(Courtesy of C-Power.)***

from wind and waves and to use floating foundations in deeper waters.

Conclusions

Increasing wind turbine and tower sizes and installations in deeper waters have clearly demonstrated a need for more innovative and cost-effective foundations. There is room for improvement in all areas of wind farm development — in design, through the innovative use of composite materials, support structures and foundations, and in construction processes, through improvements in installation techniques, fabrication and transportation. Six basic types of offshore wind turbine support structures and foundations have been described, along with their advantages and drawbacks. General considerations in the selection, design, analyses and construction of these structures have been presented to assist wind farm developers and engineers in identifying various issues that are likely to arise in the development phase of an offshore wind farm.

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