# Design and Analysis of Stationkeeping Systems for Floating Structures

API RECOMMENDED PRACTICE 2SK
THIRD EDITION, OCTOBER 2005

ADDENDUM, MAY 2008

REAFFIRMED, JUNE 2015

AMERICAN PETROLEUM INSTITUTE

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**Upstream Segment**

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# Design and Analysis of Stationkeeping Systems for Floating Structures

## 1 Scope

The purpose of this document is to present a rational method for analyzing, designing or evaluating station-keeping systems used for floating units. This method provides a uniform analysis tool which, when combined with an understanding of the environment at a particular location, the characteristics of the unit being moored, and other factors, can be used to determine the adequacy and safety of the mooring system. This document addresses station-keeping system (mooring, dynamic positioning, or thruster-assisted mooring) design, analysis, and operation. Different design requirements for mobile and permanent moorings are provided.

The design procedure specified in this document is based on a deterministic approach where the mooring system responses such as line tensions, vessel offsets, and anchor loads are evaluated for a design environment defined by a return period. The mooring system responses are then checked against the mooring strength, offset limit, and anchor capacity to ensure a factor of safety against mooring breakage or excessive vessel excursion. It should be noted that mooring designs based on this approach may not have the same level of reliability, as discussed in Appendix G.

The technology of mooring floating units is growing rapidly. In those areas where data considered adequate were available, specific and detailed recommendations are given. In other areas general statements are used to indicate that consideration should be given to those particular points. Designers are encouraged to utilize all research advances available to them. As offshore knowledge continues to grow, this document will be revised. It is hoped that the general statements contained herein will gradually be replaced by detailed recommendations.

This document does not address mooring inspection/maintenance requirements and synthetic fiber rope mooring. These issues are addressed in the following API documents:

- API RP 2I, *Recommended Practice for In-Service Inspection of Mooring Hardware for Floating Drilling Units* (Reference 1).
- API RP 2SM, *Recommended Practice for Design, Manufacturing, and Maintenance of Synthetic Fiber Ropes for Offshore Mooring* (Reference 2).

## 2 Basic Considerations

### 2.1 INTRODUCTION TO STATIONKEEPING SYSTEMS

The stationkeeping system for a floating structure can be either a single point mooring or a spread mooring. Single point moorings tend to be used more frequently for ship shaped vessels, while spread moorings are used mostly for semi-submersibles and spars. A third type of stationkeeping system is dynamic positioning (DP). Dynamic positioning can be used as the sole source of stationkeeping or used to assist a catenary mooring. Dynamic positioning can be used with either ship shaped or semi-submersible vessels.

#### 2.1.1 Spread Mooring

In a typical spread mooring system, groups of mooring lines are terminated at the corners of the vessel, holding a stable vessel heading. Figure 1 is an illustration of a catenary spread moored semi-submersible. Since the environmental force on a semi-submersible or a spar is relatively insensitive to direction, a spread mooring system can be designed to hold the vessel on location regardless of the direction of the environment. However, this system can also be applied to ship-shaped vessels, which are more sensitive to environmental directions. The mooring line can be chain, wire rope, fiber rope, or a combination of the three. Either conventional drag anchors or anchor piles can be used to terminate the mooring lines.

The combination of a spread mooring with vertical mooring tendons to restrain a Tension Leg Platform (TLP) on location, as shown in Figure 2, has been used to enhance both the operability and reliability of the basic TLP concept. The configuration and design of this spread mooring are similar to a spread mooring system for semi-submersible based floating production systems.

The Differentiated Compliance Anchoring System (DICAS) is a new mooring system that was initially developed for FPSO operations off Brazil. The system is a spread mooring system with different lateral mooring stiffness at the bow and stern of the FPSO, allowing the FPSO to partially weathervane. Because of this feature, production swivels and turrets are not needed, leading to a reduction in the mooring capital cost. However, the larger vessel offsets, particularly the yaw motion of the vessel, will require a more complex and costly riser system.

An early example of DICAS is a mooring system consisting of 15 mooring lines, which are arranged in three groups of five lines, with two groups located at the bow of the vessel and one group at the stern (Figure 3). Another example of DICAS is a mooring design consisting of 18 mooring lines, which are arranged in four groups, with two groups of 6 lines located at the bow of the vessel and two groups of 3 lines at the stern. The bow mooring groups provide the majority of the restoring force. However, system stiffness also depends upon the lateral stiffness of the stern mooring groups. As the stiffness of the stern mooring groups increase, the ability of the vessel to weathervane decreases.

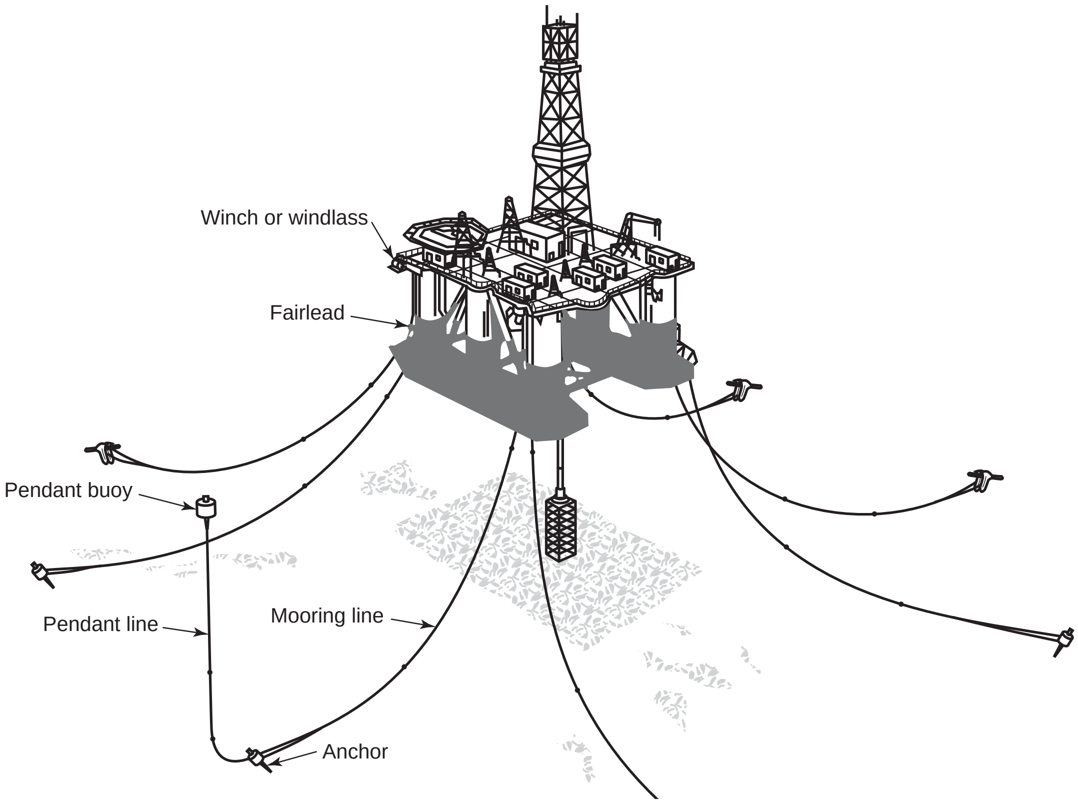



Figure 1—Spread Mooring

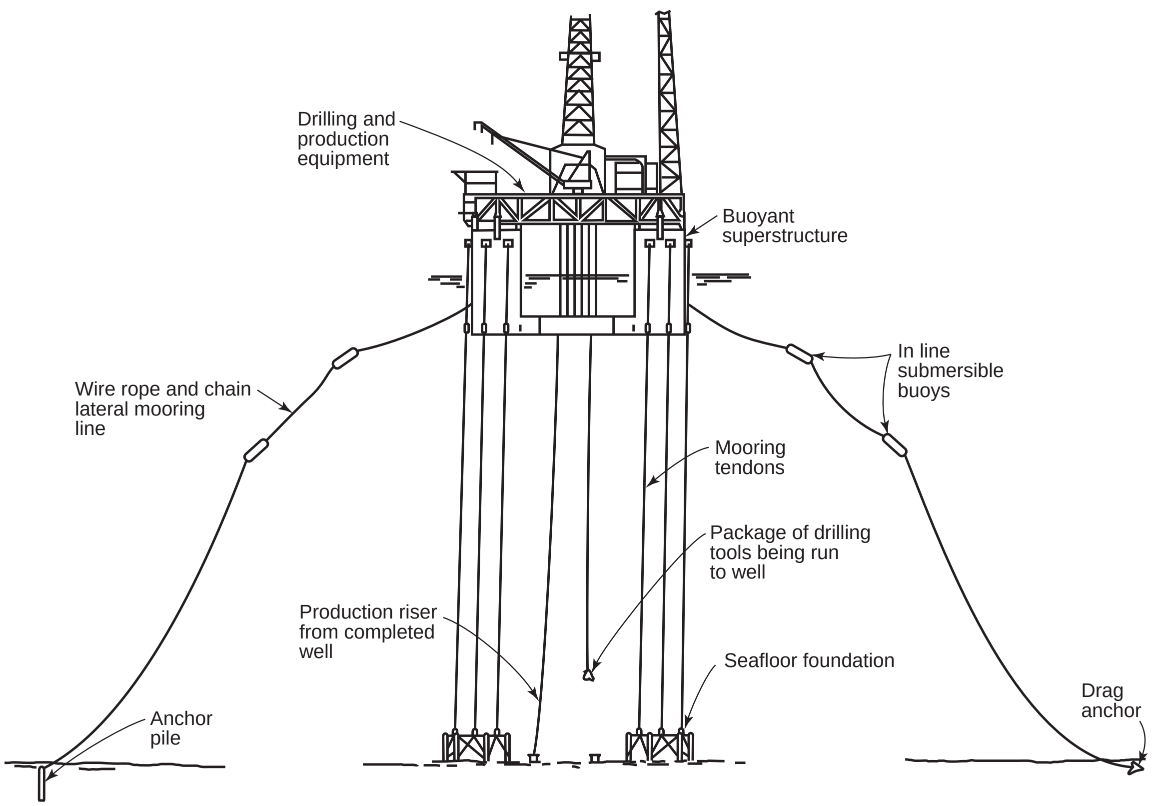



Figure 2—TLP Lateral Mooring System

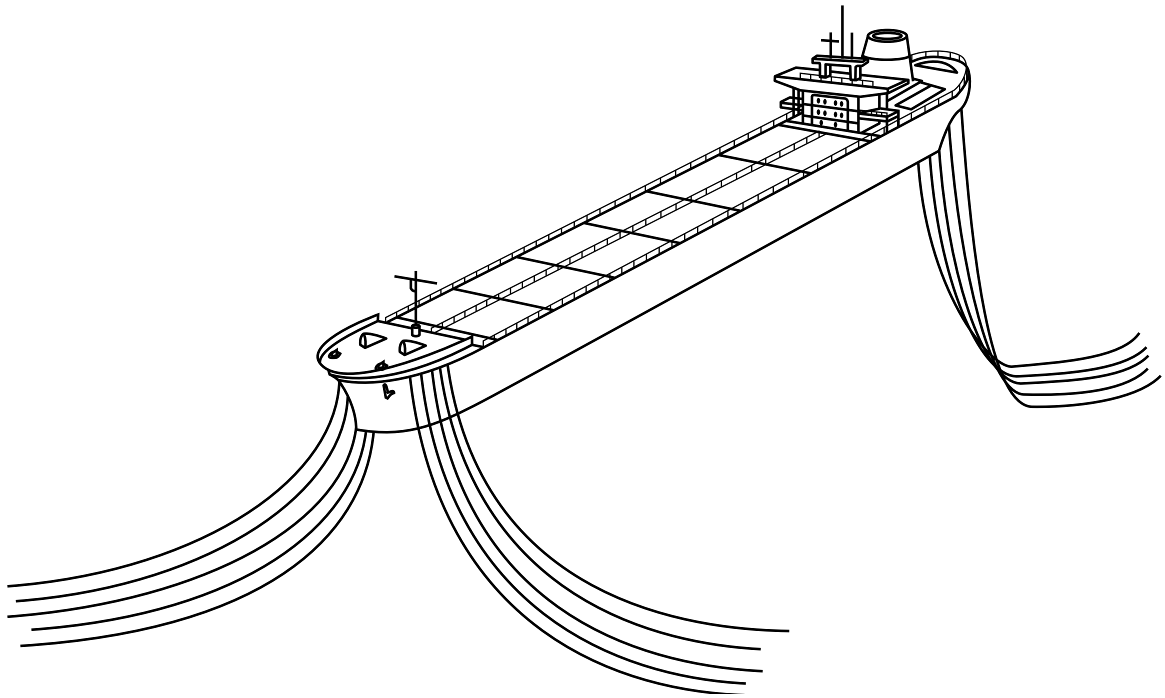


Figure 3—Differential Compliance Anchoring System (DICAS)

### 2.1.2 Single Point Mooring

Single point moorings are used primarily for ship shaped vessels. They allow the vessel to weather vane. This is necessary to minimize environmental loads on the ship shaped vessel by heading into the prevailing weather. There is wide variety in the design of single point moorings, as discussed below.

#### 2.1.2.1 Turret Mooring

A turret mooring system is defined as any mooring system where a number of catenary mooring legs are attached to a turret, which includes bearings to allow the vessel to rotate around the anchor legs.

The turret can be mounted externally from the vessel bow or stern with appropriate reinforcements (Figure 4) or internally within the vessel (Figure 5). The chain table can be above or below the waterline. The turret also could be integrated into a vertical riser system which is attached to the bow or stern of the vessel (or internally) through some kind of mechanism that allows articulation (gimbaled table, "U" joint or chain connections). The base of the riser is often weighted through additional weight within the riser or suspended beneath the riser (counterweight). These items affect the performance of the mooring system. The configuration of the riser could include steel tubular, chain or wire rope components and can vary considerably in diameter and length. The position of the chain table relative to the riser also can vary according to the design. Figure 6 shows some variations in the turret design offered by the industry.

#### 2.1.2.2 CALM (Catenary Anchor Leg Mooring)

The CALM system consists of a large buoy, which supports a number of catenary chain legs anchored to the sea floor (Figure 7). Riser systems or flow lines that emerge from the sea floor are attached to the underside of the CALM buoy. Some of the systems use a hawser, typically a synthetic rope, between the vessel and the buoy. Since the response of the CALM buoy is totally different than that of the vessel under the influence of waves, this system is limited in its ability to withstand environmental conditions. When seastates attain a certain magnitude it is necessary to cast the vessel off.

In order to overcome this limitation, rigid structural yokes with articulations are used in some designs to tie the vessel to the top of the buoy. An example is shown in Figure 8. This rigid articulation virtually eliminates horizontal motions between the buoy and the vessel. Another development, shown in Figure 9, is a buoyant yoke with a "soft" mooring connection using chains attached to the yoke.

#### 2.1.2.3 SALM (Single Anchor Leg Mooring)

This system employs a vertical riser system that has a large amount of buoyancy near the surface, and sometimes on the surface, which is held by a pre-tensioned riser. The system typically employs a tubular, articulated riser with a fixed yoke (Figure 10). It is possible also to use a chain riser with soft mooring connections (Figure 11). The vertical buoyancy force acting on the top of the riser functions as an inverted pendulum. When the system is displaced to the side, the pendulum action tends to restore the riser to the vertical position.

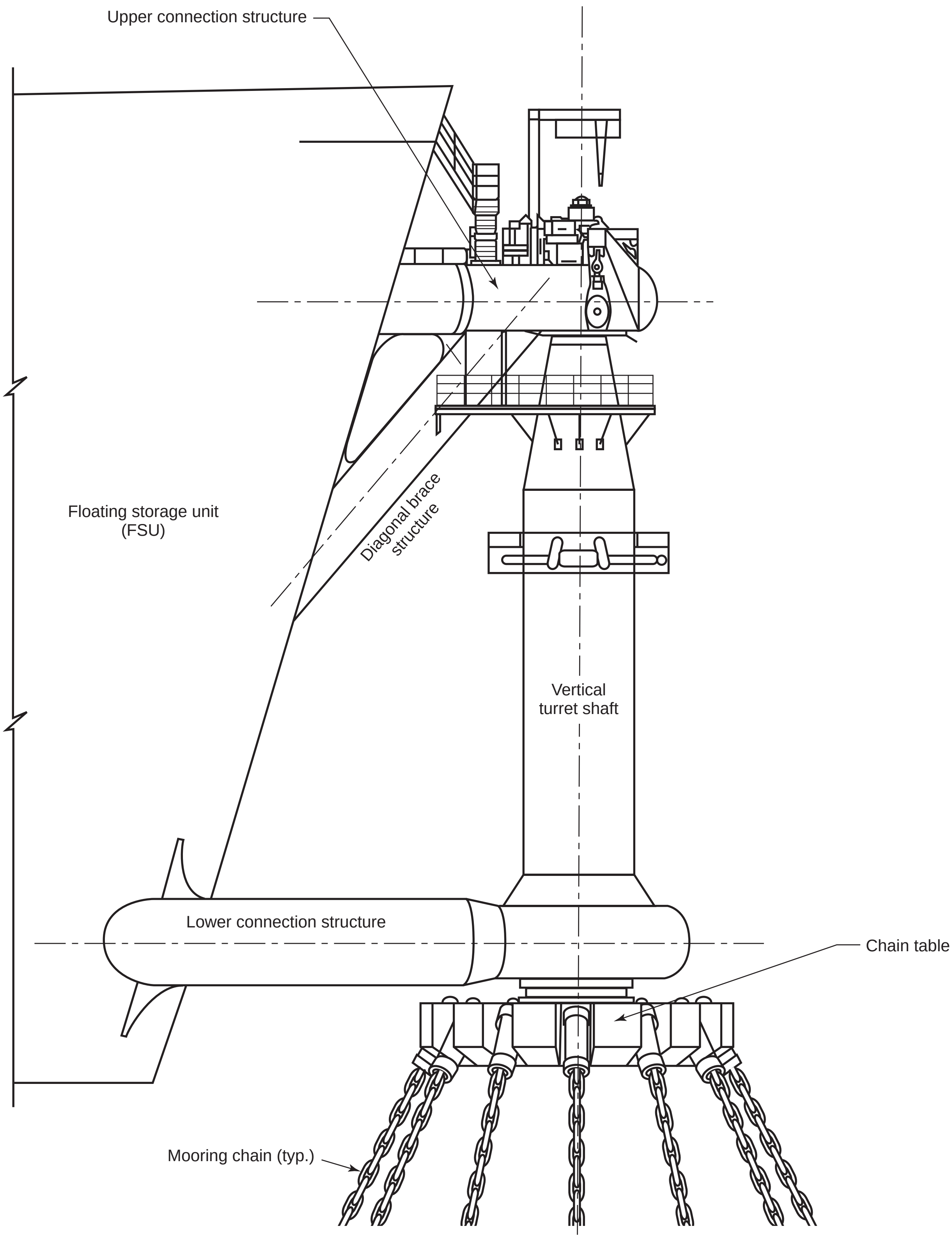



Figure 4—Typical External Turret Mooring Arrangement

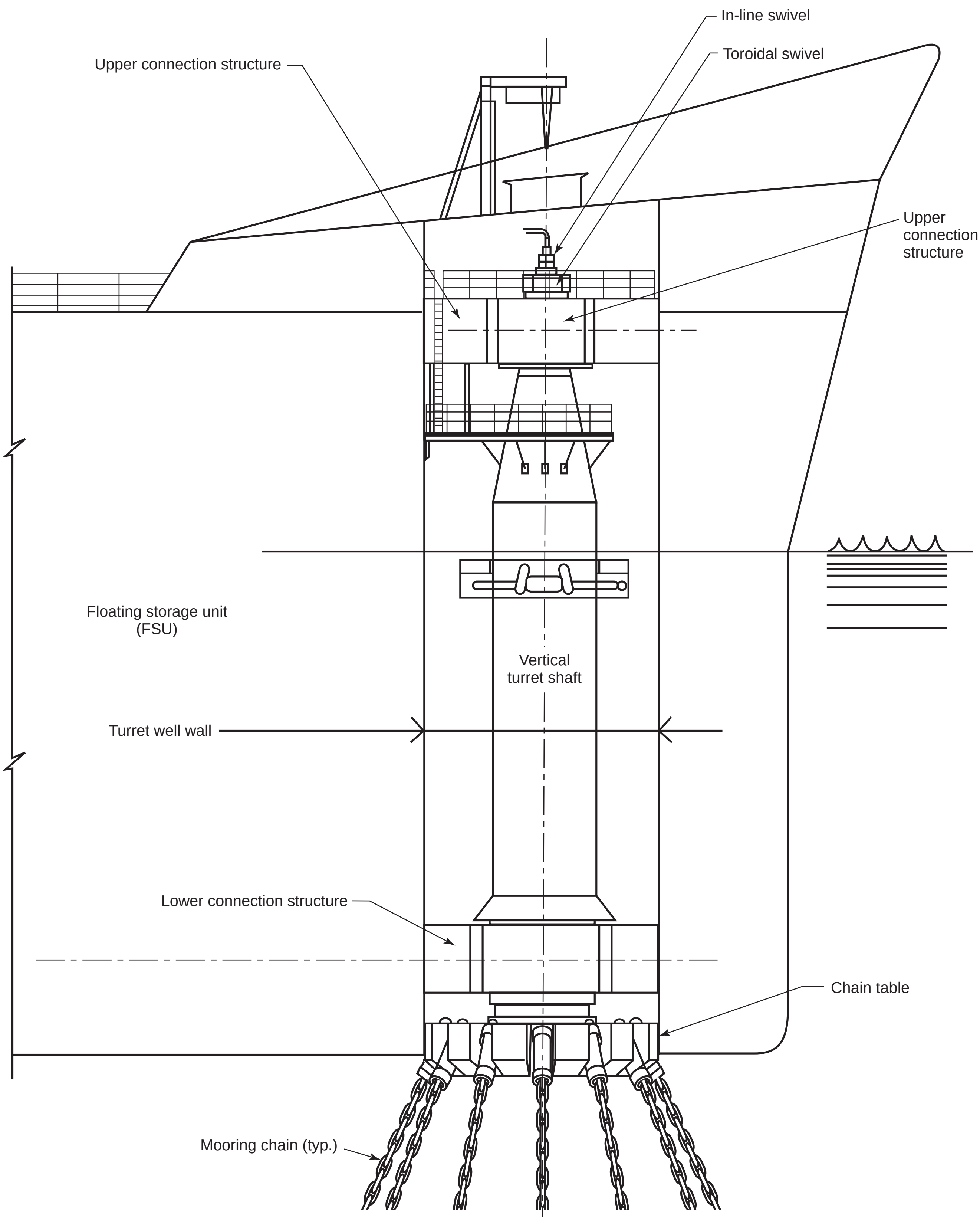



Figure 5—Typical Internal Turret Mooring Arrangement

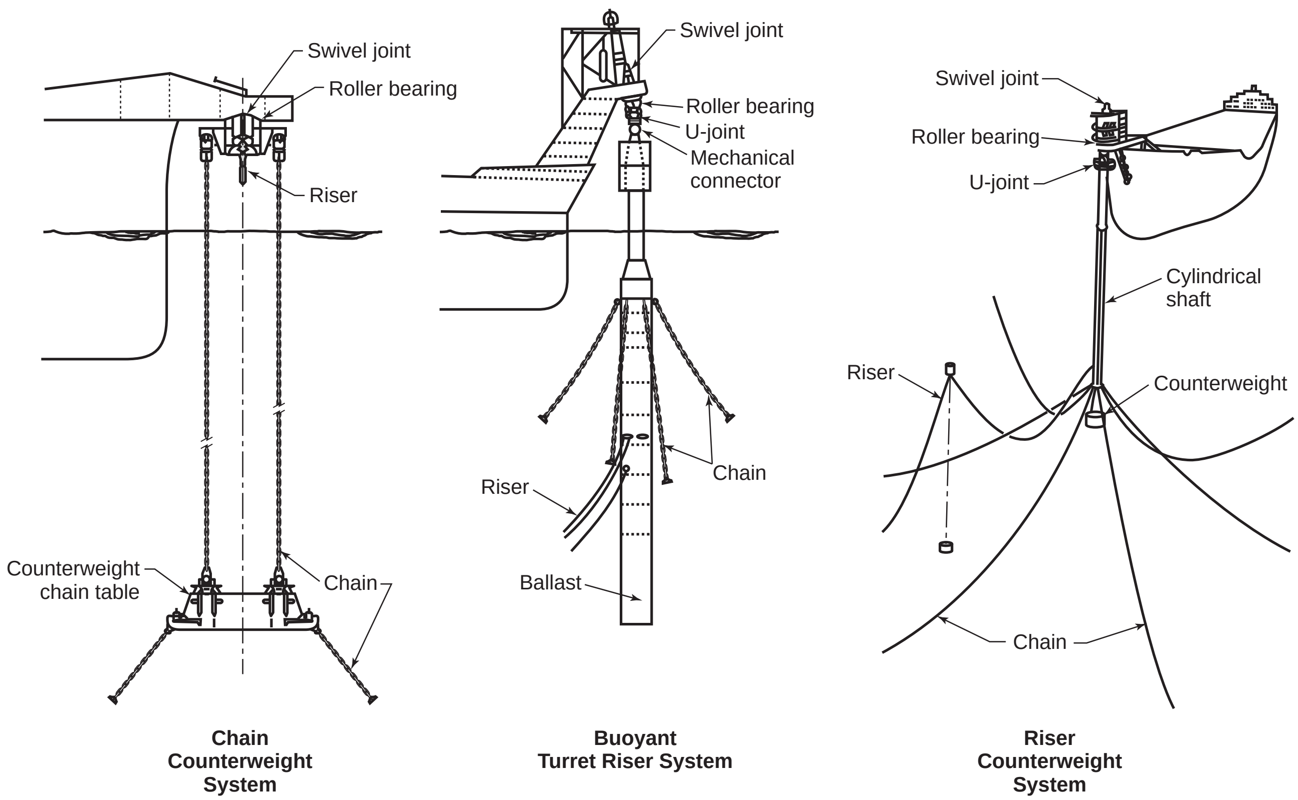



Figure 6—Variations on Turret/Riser System

The tanker can be secured to the top of this SALM buoy with either a flexible hawser or a rigid yoke as discussed in the CALM description. The base of the riser is usually attached through a U-joint to a piled or deadweight concrete or steel structure on the sea floor. In deep water, the riser system usually has mid-span articulation.

#### 2.1.3 Dynamic Positioning

Dynamic positioning (Figure 12) is a technique of automatically maintaining the position of a floating vessel within a specified tolerance by controlling onboard thrusters which generate thrust vectors to counter the wind, wave and current forces. Dynamic positioning is particularly well suited for a vessel designed to arrive and leave location frequently, such as an extended well test system.

#### 2.1.4 Thruster Assisted Mooring

Many floating vessels designed to operate with moorings are also equipped with thrusters and thruster control systems. The thrusters can be used to control the vessel heading, reduce mooring load under severe environment, or increase the workability of the floating vessel.

### 2.2 MOORING COMPONENTS

A mooring system consists of a number of components such as chain, wire rope, synthetic rope, connecting hardware, clump weight, buoy, winch, fairlead, and anchor. A description of mooring components commonly used by the offshore industry can be found in Appendix A.

### 2.3 PERMANENT AND MOBILE MOORING SYSTEMS

Permanent moorings are normally used for production operations with longer design lives. The mooring for a floating production system (FPS), for example, is normally a permanent mooring since FPSs typically have design lives of over 10 years. Mobile moorings often stay on one location for a short period. Examples of mobile moorings include those for mobile offshore drilling units (MODUs) and for tenders moored next to another platform such as floatels, drilling tender, and service vessels. The division between mobile and permanent moorings may not be clear for operations with design lives of a few years. In this case, the user should make a judgment based on the risk of exposure to severe environments and the consequence of a mooring failure. Differences

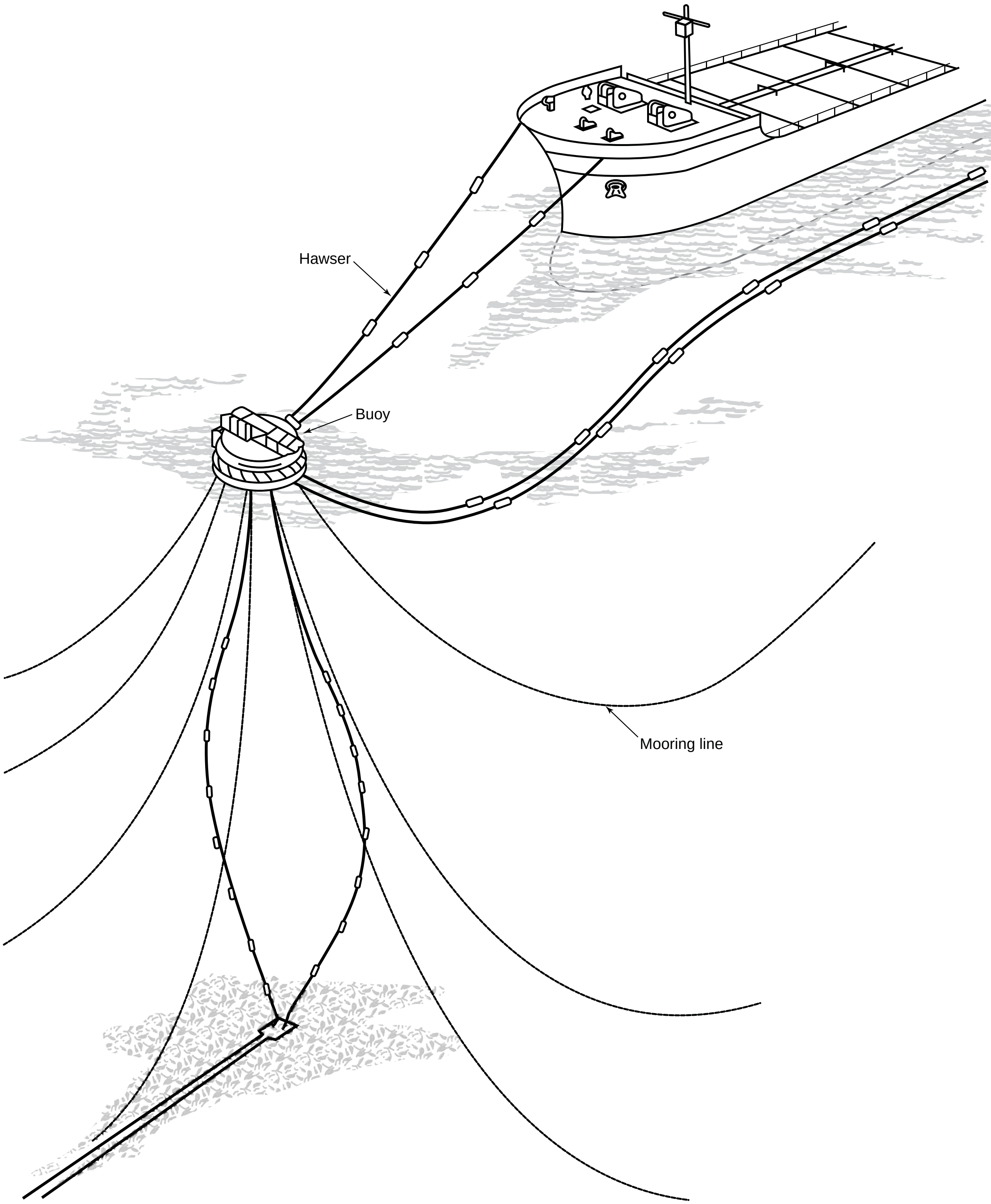



Figure 7—Catenary Anchor Leg Mooring (CALM) with Hawsers

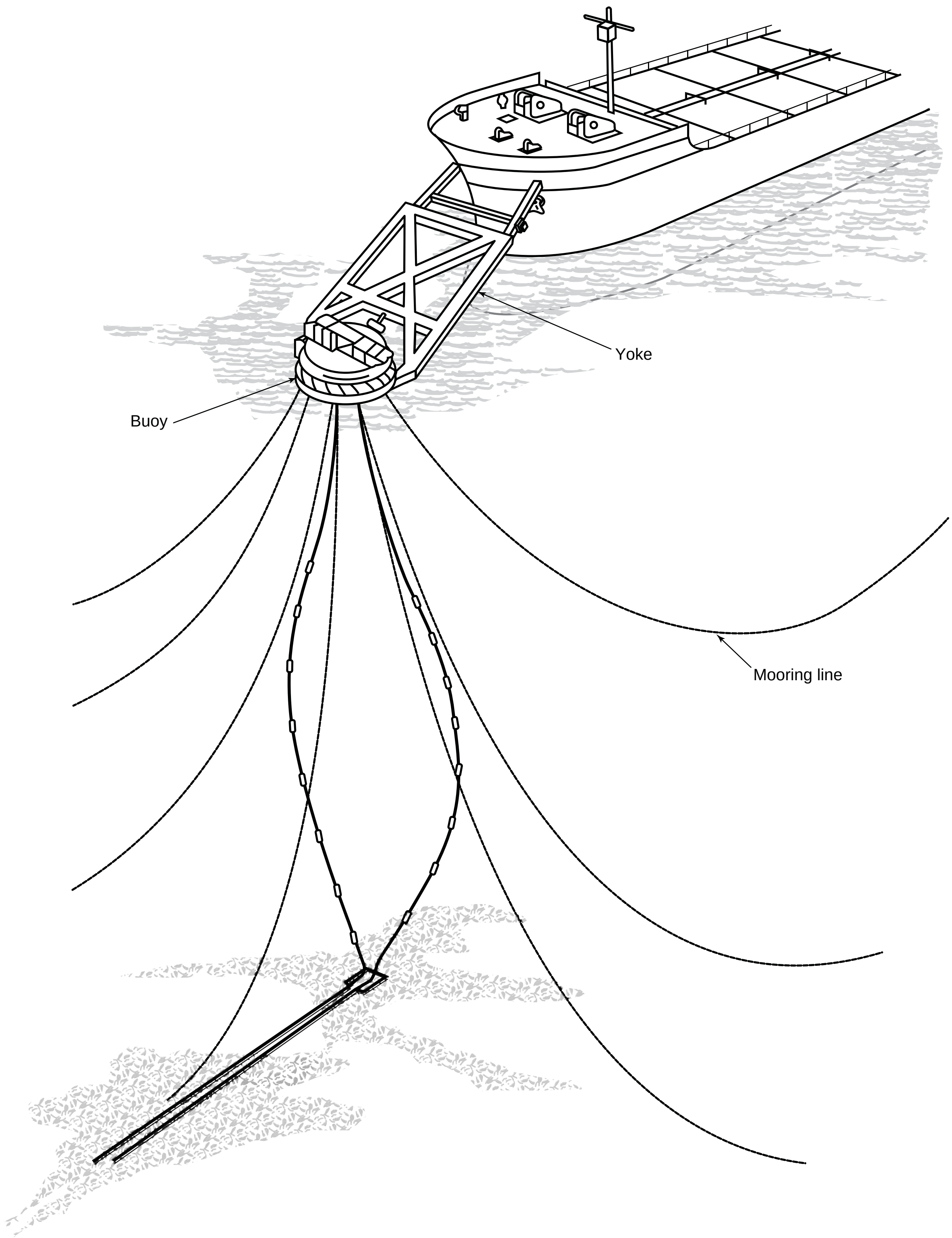



Figure 8—Catenary Anchor Leg Mooring (CALM) with Fixed Yoke

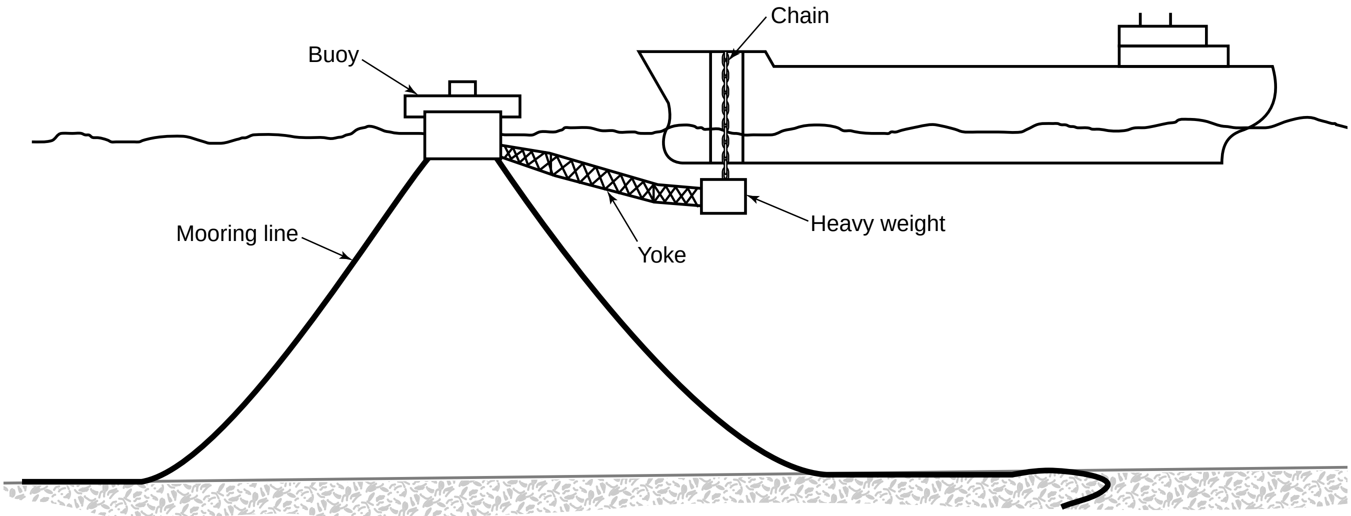



Figure 9—Catenary Anchor Leg Mooring (CALM) with Soft Yoke

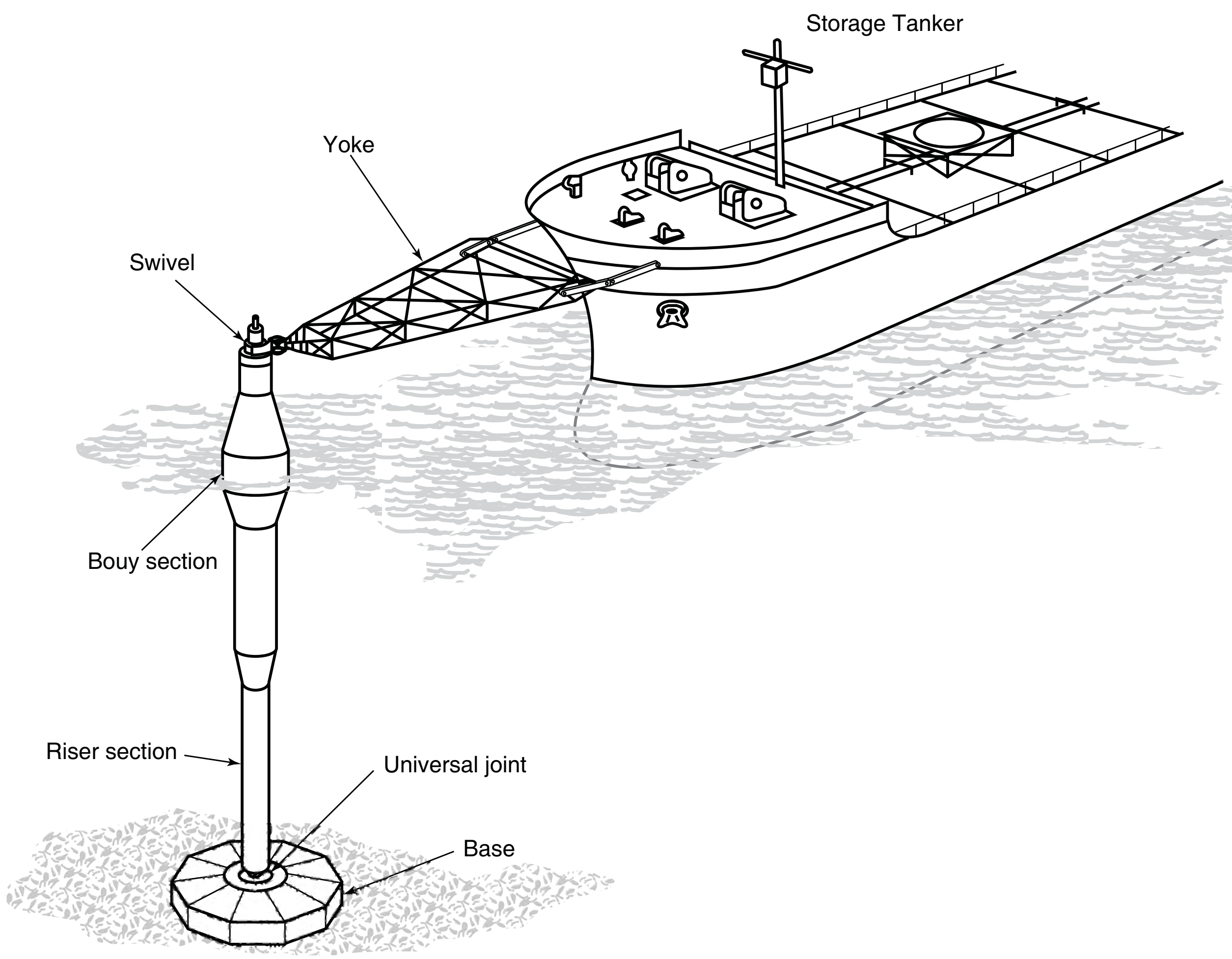



Figure 10—Single Anchor Leg Mooring (SALM) with Tubular Riser and Yoke

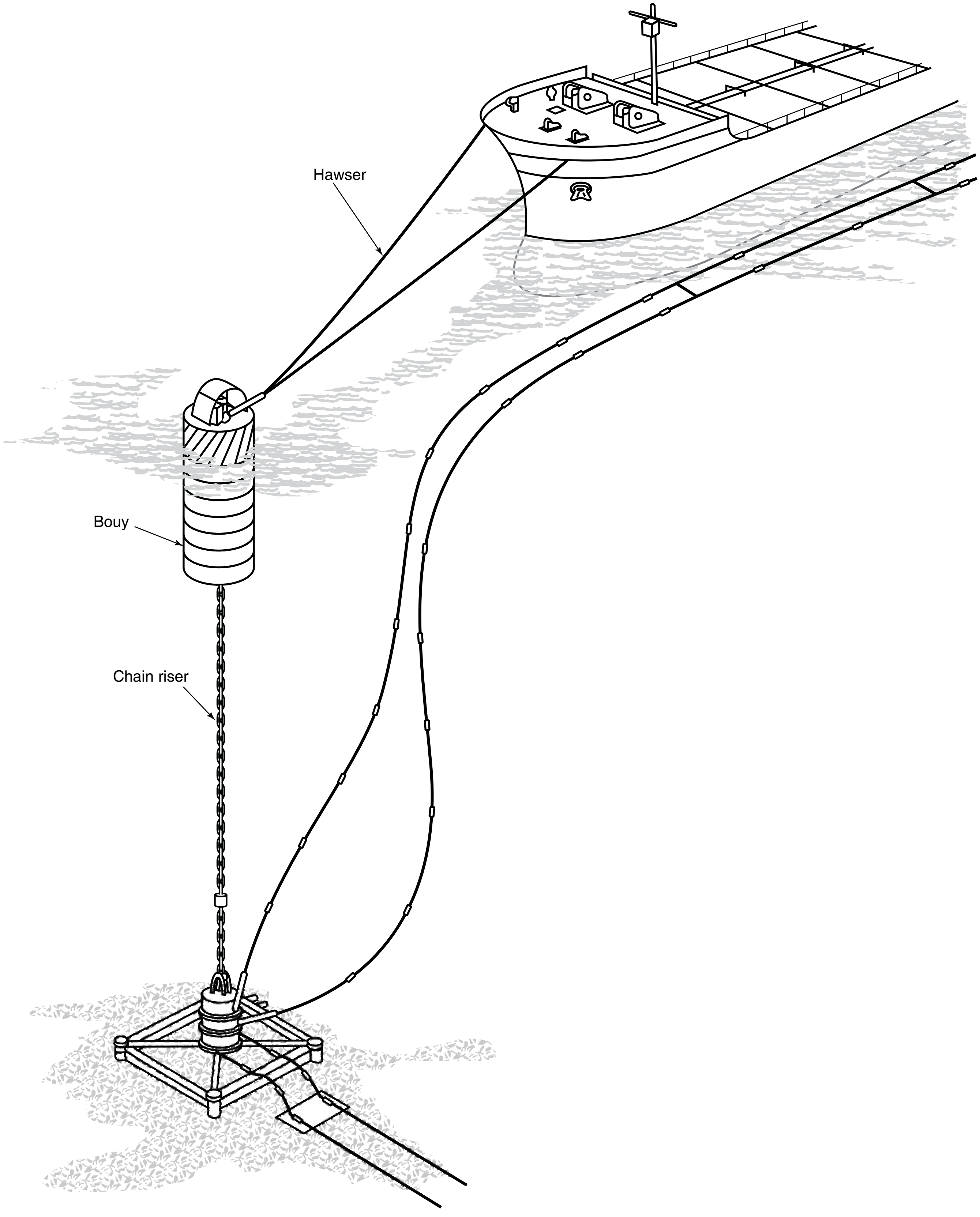



Figure 11—Single Anchor Leg Mooring (SALM) with Chain Riser and Hawser

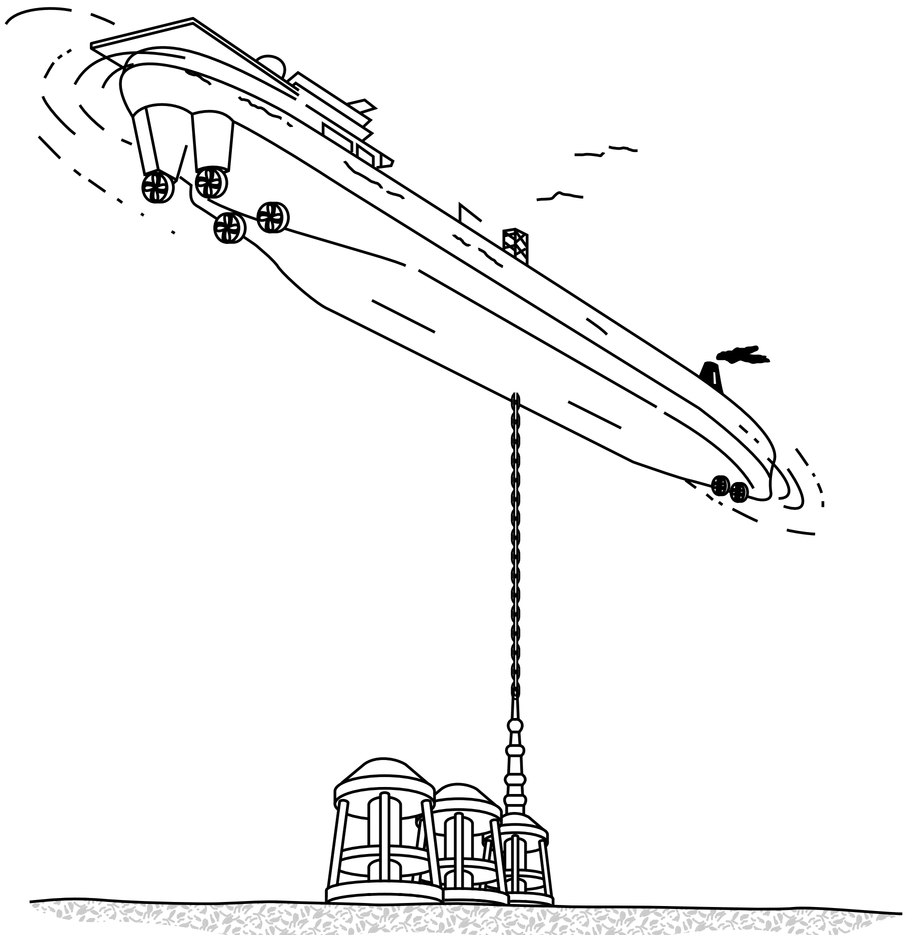


Figure 12—Dynamic Positioning

between permanent and mobile moorings are significant, as discussed below. The discussion can be used as guidelines to determine to what category (permanent or mobile) the floating structure belongs.

### 2.3.1 Type of Mooring

A mobile vessel is normally equipped with a spread mooring, internal turret mooring, or dynamic positioning system. However, a permanent vessel has more choices of mooring design because mobility is normally not required.

### 2.3.2 Environmental Criteria

The design environments for mobile moorings are lower than those for permanent moorings. The lower design environment for mobile moorings is based on the consideration that the consequence of a mooring failure would generally be less severe. This can be illustrated by comparing a MODU with an FPS. In many instances, a MODU can at least disconnect and may even lay down its drilling riser. In the case of tropical storms, it may be possible to move the vessel before the arrival of a storm. By contrast, an FPS is unlikely to be removable from location, and may not even have quickly retrievable risers.

### 2.3.3 Method of Analysis

A quasi-static analysis method is often used for evaluating the performance of a mobile mooring system, and the effects of line dynamics are accommodated through the use of a relatively conservative safety factor. A more rigorous dynamic analysis is required for the final design of a permanent mooring system, and the factor of safety is relaxed to reflect that some uncertainty in line tension prediction is removed. Dynamic analysis can also be performed for mobile moorings.

A fatigue analysis is not required for mobile mooring systems. Because of abuse from frequent deployment and retrieval, many mooring components of a mobile mooring system are replaced before they reach their fatigue limits. However, for permanent installation, fatigue is an important design factor, and a fatigue analysis should be performed.

#### 2.3.4 Mooring Hardware

Mobile moorings use the mooring hardware that can be rapidly deployed and retrieved. This limitation does not apply to permanent moorings. Many mooring components such as anchor pile, linear winch, buoy, and chain jack that may not be suitable for mobile moorings can be used in a permanent mooring. Also permanent moorings often require heavier mooring hardware because of the more stringent design requirements.

#### 2.3.5 Installation

The deployment of a mobile mooring is normally carried out with the assistance of work boats. This operation is simple and usually takes no more than a few days. The deployment of an FPS mooring often requires the assistance of much heavier vessels such as a derrick barge or a purposely built work boat. A portion of the mooring is often preset. Sometimes special design features are incorporated in the mooring design to facilitate deployment.

#### 2.3.6 Inspection and Maintenance

A mobile mooring can often be visually inspected during retrieval or deployment. Retrieving a permanent mooring for inspection can be very expensive. To inspect a permanent mooring, divers or ROVs (Remotely Operated Vehicles) are often used. Also, replacing faulty mooring components is easier for mobile moorings than for permanent moorings.

### 2.4 DESIGN CONSIDERATIONS

#### 2.4.1 Primary Design Considerations

The primary design considerations associated with a mooring system are design criteria, design loads, design life, operation and maintenance considerations, etc. These considerations are addressed in detail in the following sections. In addition, a designer must also pay attention to the riser and subsea equipment considerations.

#### 2.4.2 Riser Considerations

Risers transfer fluids between the seabed and the production or drilling vessel, and constitute one of the primary design constraints of the mooring system. The riser system often places limitations on the allowable vessel offset. In the event of excessive vessel offsets, mooring line adjustments such as slackening the leeward lines are sometimes performed to avoid damage to the riser. An equally important consideration is interference between mooring lines and risers, during both operational and extreme weather conditions. The mooring system and riser system must therefore be designed to accommodate each other, and coordination of these two design efforts is essential.

Design guidelines for riser can be found in API RP 17A/ ISO 13628-1, *Recommended Practice for Design and Operation of Subsea Production Systems* (Reference 3), API RP 17B, *Recommended Practice for Flexible Pipe* (Reference 4), API RP 16Q, *Recommended Practice for Design, Selection, Operation, and Maintenance of Marine Drilling Riser Systems* (Reference 5), and API RP 2RD, *Design of Risers for Floating Production (FPSs) and Tension Leg Platforms (TLPs)* (Reference 6).

#### 2.4.3 Subsea Equipment Considerations

Mooring lines should be clear of subsea equipment such as templates, riser base, satellite wells, and flowlines. Any contact between mooring lines and subsea equipment during installation, operation or maintenance presents a high potential of damage to both the equipment and the mooring lines. If interference, or the potential for interference appears unavoidable, it may be possible to alter the layout and design of the mooring system through the use of an asymmetric arrangement of mooring lines, or the use of clump weights or spring buoys. Coordination of the mooring system design with the subsea equipment layout is essential.

Guidelines for the design of subsea equipment are given in API RP 17A/ISO 13628-1, *Recommended Practice for Design and Operation of Subsea Production Systems* (Reference 3).

### 2.5 FORESEEABLE HAZARDS FOR MOORING SYSTEMS.

The main objective of this section is to outline the causal factors and circumstances surrounding a number of individual and specific hazards relating to potential mooring system failures. It is important for mooring system designers and operators to recognize and minimize these hazards.

#### 2.5.1 Failure of Emergency Mooring Release Mechanism, or Its Premature or Delayed Release

In cases where an emergency mooring release system is adopted, hazards arise if the systems do not activate on demand, or can be activated when not required to do so. Rigorous procedures are therefore essential to ensure correct operation. Planned maintenance and testing of such systems is also essential providing mooring integrity is not compromised in so doing.

#### 2.5.2 Failure of Structures supporting Anchoring Equipment, Fairleads and Winches

Hazards to the structure are caused by poor design and construction, and in service demand resulting in equipment failure from load, vibration, corrosion and wear. In extreme cases, watertight integrity can be affected. Thus, a rigorous Planned Maintenance Schedule should include regular inspections of all mooring component foundations and internal supporting structure. Maintenance should also cover the correct operation of all moveable parts and protection from corrosion.

#### 2.5.3 Manufacturing Defects and Processes

Wire rope and chain, common links, connectors, chasing equipment, pennants etc. are all subject to imperfections in the manufacturing process. Such equipment must be accepted on the basis of quality control procedures, testing and approval, and constructed to recognized industry standards.

#### 2.5.4 Mechanical, Electrical, and Hydraulic Failures relating to Mooring Systems

Hazards from these supply systems can lead to the failure of control, reference or sensors, load response, and loss of manual over-ride. Mechanical failures can occur to windlass brakes, clutches and sleeves, pawls, gears, splines, cracked discs or drums, cooling, striker bar, wire spooling and tensioning devices, and wear to chain windlass/fairlead pockets. Hydraulic failures can occur to pipework, seals, joints, pumps, brake valves, greasing systems, oil contamination, leaks and levels, or emergency release valves. Electrical failures generally affect control and monitoring of position, tension and power systems. Regular inspection and planned maintenance procedures should be set in place to minimize the likelihood of failure of these systems.

#### 2.5.5 Mooring System Overload, Fatigue, and insufficient Anchor Holding Capacity

Overload is defined as any tension which exceeds a predefined limit or which exceeds the capacity of the anchor. Either event may cause a loss of position. Hazards include inadequate use of propulsion, exceptional environmental conditions, inappropriate anchor penetration, excessive tensioning, equipment failure, poor installation and retrieval operations.

The adequacy of the mooring system to resist cyclic loading (fatigue) will be addressed in the mooring analysis. Practical measures can be adopted to move the sections of mooring subject to concentrated fatigue e.g., at the fairleads or at the touch-down point.

Anchor holding failure can be attributed to overloading the soil foundation, inappropriate equipment or anchor design for the soil conditions (e.g., fluke angle), equipment failure (including shackles), inadequate line lengths for extreme conditions. The consequence of these is likely to be anchor drag.

#### 2.5.6 In-service Degradation of Mooring Components due to Corrosion, Metal Fatigue, Abrasion, Deployment and Retrieval

Corrosion, wear, and damage can degrade mooring line integrity. Corrosion is of major concern especially in the splash zone for chain and wire systems. Individual wire breaks are frequently attributed to corrosion. If a high corrosion potential exists between components of a mooring system then hydrogen levels can be sufficient to cause embrittlement in any high-strength materials present.

Abrasion or wear in wire ropes generally occurs at the winch, fairlead and touchdown point. Normal operations will ensure shifting of the contact areas. Abrasion in chain will be concentrated at link contact points and any unforeseen contact with vessel structure.

Damage during deployment and retrieval is common unless careful procedures are followed. For example, wire ropes are particularly subject to crushing damage on winches from high tension spooling. The spiral structure of wire rope can cause a torque build-up if dragged along the seabed resulting in a hock or loop should the rope tension reduce. Also, the improper use of chasers during retrieval of wire ropes may lead to damage to the rope and connectors.

A regular inspection program is essential to monitor the integrity of the moorings.

#### 2.5.7 Inadequate Operating, Maintenance and Handling Procedures

Operating procedures are produced to minimize the risk of mooring failure and injury to personnel and environment. However, they do not always adequately address all potential hazards which may exist during handling, deployment and retrieval. Also, inspection/discard criteria and maintenance requirements are critical elements necessary to assure reliability of any mooring system. Permanent moorings are less likely to suffer from handling damage as they generally remain in place for longer periods. Handling procedures should address normal handling operations with the focus of priorities ensuring the safety of personnel, minimizing service wear, scuffing, abrasion, chaser damage, running operations, winching wire onto drums or chain into a locker.

The condition of all mooring lines will deteriorate with time in service and adequate inspection and maintenance programs will be necessary to ensure continuous integrity.

##### 2.5.7.1 Stored or Trapped Torque Energy in Wire/ Chain Catenary Systems During Retrieval

Stored or trapped torque in mooring chain during retrieval can pose a significant safety hazard to personnel. The chain torque energy is introduced into the system as a result of 6 or 8 strand wire rope dynamic untwist/twist characteristics under increased and decreased tensions. If a marine swivel is not present either at the top or bottom of the chain, torque may become trapped during recovery since the chain chaser position at the anchor shaft prevents the release of torque. This condition could occur whenever a mooring line does not incorporate an operable marine swivel.

Detailed procedures, which thoroughly address best practices for the safe and cautious handling of this potential hazard, should be developed prior to retrieval of any catenary mooring system. Several documented near misses have occurred in the US GOM MODU operations as the direct result of uncontrolled release of trapped chain torque. This potential situation presents a high risk of severe injury or loss of life.

#### 2.5.8 Exceedance of Mooring System Capacity to Perform its Function

Before construction, the design capacity of the mooring lines will have been determined from a mooring analysis. An anchor holding test and winch capacity test will confirm design conditions. It is unlikely however that the winch system will be able to operate in the worst environment when the load is borne by the chain stoppers. Operating out of the proven range will lead to the windlass being unavailable and could lead to overload on the adjacent lines. Similarly, poor anchor holding capacity will lead to anchor drag which can result in a less than optimum mooring spread.

Written operating procedures for managing mooring line tensions should be clearly defined and readily available to those on board. This will include the use of thrusters if available, the redistribution of line tensions to prevent design exceedance of any component, and including the course of action should anything unforeseen occur.

In some areas of the world the practice on receipt of severe weather forecasts is to reduce tensions on all mooring lines and to evacuate personnel. In other areas, personnel stay on board and initiate measures to protect the installation. Clear operating procedures must exist for both scenarios.

All components should be maintained to a satisfactory level achieved by the use of a robust Planned Maintenance System with unambiguous criteria for discard and replacement of key components.

#### 2.5.9 Operator Error

Hazards which arise from operator error include the inability to implement all items described in this Section and as defined in the operating procedures. Operator errors can be minimized with adequate training of responsible persons coupled with a company-wide culture to eliminate errors of all types. This may require mandatory attendance at appropriate training courses, the provision of clear procedural guides and operations manuals, setting out the chain of command, drills and briefings before undertaking work.

# 3 Environmental Criteria

## 3.1 ENVIRONMENTAL CONDITION

The industry recognizes two classifications of environmental condition when evaluating mooring system strength: maximum design condition and maximum operating condition.

### 3.1.1 Maximum Design Condition

The maximum design condition is defined as that combination of wind, waves, and current for which the mooring system is designed. Mooring systems should be designed for the combination of wind, wave, and current conditions causing the extreme load in the design environment. In practice, this is often approximated by the use of multiple sets of design criteria. For example, in the case of a 100-year design environment, three sets of criteria are often investigated:

a. the 100-year waves with associated winds and currents,

b. the 100-year wind with associated waves and currents, and

c. the 100-year current with associated wave and wind.

The most severe directional combination of wind, wave, and current should be specified for the permanent installation being considered, consistent with the site's environmental conditions. Special attention should be given to certain floating structures such as large ship-shaped vessels, which are dominated by low frequency motions. Since low frequency motions increase with decreasing wave periods, the 100-year waves may not yield most severe mooring loads. Lower waves with shorter periods could yield larger low frequency motions and thus higher mooring loads.

#### 3.1.1.1 Maximum Design Conditions for Permanent Moorings

The 100-year return period design criterion should be used for permanent moorings. If the design life of the mooring is substantially lower than 20 years, a shorter recurrence interval may be justified. In this case, the recurrence interval should be determined by a risk analysis taking into account the consequence of mooring failure.

For a permanent operation with a mooring system that permits rapid disconnection of the production vessel from the mooring, the maximum design condition is the maximum environment in which the production vessel remains moored. However, the mooring alone (without the production vessel) should be able to withstand the maximum design environment for permanent moorings.

#### 3.1.1.2 Maximum Design Conditions for Mobile Moorings

##### 3.1.1.2.1 Operations Away From Other Structures

Moorings for mobile floating units such as MODUs operating away from other structures should use a maximum design environment with a return period of at least 5 years. Special attention should be given to operations in an area of tropical cyclone such as Gulf of Mexico (hurricane) and South China Sea offshore China (typhoon). These areas are characterized by generally mild environment combined with severe storms during the cyclone season. For operations out of the cyclone season, the 5-year environment can be determined using the environmental data excluding tropical cyclones. For operations during the cyclone season, the tropical cyclone data should be included.

In some areas of tropical cyclone, the return period can be reduced for operations during the season of tropical cyclone if both of the following conditions are met:

a. A risk analysis is conducted to evaluate the consequence of a mooring failure. Such an analysis examines various scenarios of mooring failure, probability of occurrence of each scenario, and their effect on safety and environment. The risk analysis may be used to determine the appropriate return period, but it should not be less than one year.

b. Operational personnel evacuation is planned and executed before arrival of a tropical cyclone.

##### 3.1.1.2.2 Operations in the Vicinity of Other Structures

The maximum design environment for mobile units operating in the vicinity of other structures should have a return period of at least 10 years. In an area of tropical cyclone, for operations out of the cyclone season, the 10-year environment can be determined using the environmental data excluding tropical cyclones. For operations during the cyclone season, the tropical cyclone data should be included. Operations that belong to this category are characterized by higher risk of collision or contact of structures or equipment. An example of such an operation is a MODU with mooring lines deployed over a pipeline. Damage to the pipeline may occur if the anchors are dragged into the pipeline. Other examples include a drilling tender, a floater, or a service vessel moored next to a platform.

### 3.1.2 Maximum Operating Condition

The maximum operating condition is defined as the combination of maximum wind, waves, and current in which the unit can continue to work, for example, to drill, produce, offload or maintain gangway connection. This condition shall not exceed the maximum design condition.

The operating environmental criteria should be known to the people responsible for the drilling, offloading, or production operations in order that timely plans to suspend operations can be performed.

## 3.2 ENVIRONMENTAL DATA

Models leading to the design responses of interest should consider the jointly distributed environmental phenomena. Environmental data, such as wind, wave, current and tide, have site-specific relationships governing their interaction. When collecting data, various relationships should be included. Of particular importance are the wind/wave, wave height/period, and wave/current relationships and their relative directions. The directions of various environmental phenomena are especially important for single point moorings.

The maximum design environments for mobile moorings discussed in Section 3.1 should generally be determined by annual statistics. However, if the operating season is well defined and seasonal environmental data are sufficient to provide meaningful statistics, these environments can be determined by seasonal statistics. There are areas governed by special weather events, which may not be well represented by typical return period statistics. For example some areas of mild climate may be subject to "sudden storms" such as squalls, and other areas may be subject to very high currents. In these cases, the special weather events should also be taken into consideration in determining the environmental criteria.

The mooring system should be assessed under the most unfavorable wind/wave/current directions that can be reasonably assumed to occur. The ability of the vessel to change heading in response to changing environmental conditions can be considered.

## 3.3 WIND

Two methods are generally used to assess effects of wind for design:

1. Wind is treated as constant in direction and speed, which is taken as the 1-minute average.
2. Fluctuating wind is modeled by a steady component, based on the 1-hour average velocity, plus a time-varying component calculated from a suitable empirical wind gust spectrum. The recommended wind gust spectrum is presented in Appendix B.

For the final design of permanent moorings, Method 2 should be used. However, Method 1 may be used if it can be shown to be more conservative. For mobile moorings, either method may be used. The design wind speed should refer to an elevation of 10 m above still-water level.

### 3.4 WAVES

The wave height versus wave period relationships for the design seastate should be accurately determined from oceanographic data for the area of operation. The period can significantly affect wave drift forces and vessel motions, and therefore a range of wave periods should be examined. For fatigue analysis, the long-term joint distribution of wave heights and periods (scatter diagram) is required, and a single best estimate of the associated wave period can be used for each seastate.

### 3.5 CURRENT

The most common categories of currents are:

a. tidal currents (associated with astronomical tides),

b. circulation currents (loop and eddy currents),

c. storm-generated currents,

d. soliton currents.

The vector sum of the currents applicable to the site is the total current for each associated seastate. The speed and direction of the current at different elevations should be specified. In certain geographic areas, current force can be the governing design load. Consequently, a selection of appropriate current profile requires careful consideration.

### 3.6 WATER DEPTH

The design water depth for the mooring system should account for sea level variations due to tides, storm surges, and seafloor subsidence, if applicable.

### 3.7 SOIL AND SEAFLOOR CONDITIONS

For permanent moorings, bottom soil conditions should be determined for the intended site to provide data for the anchoring system design. Shape of the seafloor should be properly accounted for in the mooring analysis. A bottom hazard survey should be performed. These measures should also be considered for mobile moorings to ensure adequate anchoring.

### 3.8 ATMOSPHERIC ICING

Increased wind area due to superstructure icing should be considered in platform wind force calculations, as appropriate.

### 3.9 MARINE GROWTH

The type and accumulation rate of marine growth at the design site may affect weight, hydrodynamic diameters, and drag coefficients of vessel members and mooring lines. This should be taken into consideration in the design.

## 4 Environmental Forces and Vessel Motions

### 4.1 BASIC CONSIDERATIONS

Environmental loads can be categorized according to the following three distinct frequency bands:

a. Steady loads such as wind, current, and wave drift forces are constant in magnitude and direction for the duration of interest.

b. Low frequency cyclic loads that excite the platform at its natural periods in surge, sway, and yaw. Typical natural periods range from 1 to 10 minutes.

c. Wave frequency cyclic loads with typical periods ranging from 5 to 30 seconds. Wave frequency cyclic loads result in wave frequency motions, which are typically independent of mooring stiffness. The approach of neglecting mooring stiffness in wave frequency motion calculation is always conservative. For small floating structures such as buoys where the first order wave loads are not large, accounting for mooring stiffness will yield more realistic wave frequency motions. Also if the natural period of the moored vessel is close to the wave periods, the wave frequency motions can be dependent on the mooring stiffness. In this case the effect of stiffness should be properly accounted for.

### 4.2 GUIDELINES FOR THE EVALUATION OF ENVIRONMENTAL FORCES AND VESSEL MOTIONS

Environmental forces and vessel motions may be determined either by model testing or calculation. The water depth effects should be included. Guidelines for evaluating these items are provided in API RP 2T (Reference 7). The wind spectrum recommended for the evaluation of low frequency wind forces can be found in Appendix B. For mobile moorings either API RP 2T or the simplified methods presented in Appendix C can be used.

Cylindrical deep draft vessels such as spar under current condition may be subjected to significant VIM (Vortex Induced Motion). Guidelines for mooring analysis under VIM conditions can be found in Appendix H.

### 4.3 SIMPLIFIED METHODS

Design equations and curves for a quick evaluation of environmental forces and vessel motions are provided in Appendix C. These simplified analytical tools were developed primarily for the analysis of mobile moorings. They may be used for preliminary designs of permanent moorings if more accurate information is not available at the early stage of the design process and if the limits for these tools are not exceeded. Simplified methods are available for the following force components:

a. Current forces for ship shaped and semi-submersible hulls.
b. Mean wave drift forces and low frequency motions for ship shaped and semi-submersible drilling vessels.
c. Steady wind force.
d. Wind and current forces for large tankers.
e. Forces due to oblique environment.

# 5 Mooring Strength Analysis

## 5.1 BASIC CONSIDERATIONS

### 5.1.1 Introduction

Mooring analysis shall be performed to predict extreme responses such as line tensions, anchor loads, and vessel offsets under the design environment and other external loads (e.g., riser loads, tandem mooring loads, etc.) The responses are then checked against allowable values to ensure adequate strength of the system against overloading and sufficient clearance to avoid interference with other structures.

Active control of mooring system by mooring line adjustment may be performed for certain operations. However, active mooring line adjustment should not be considered in the mooring analysis for maximum design conditions.

Guidelines for mooring strength analysis are provided in the following sections. A strength analysis example can be found in Appendix J. Strength analysis under VIM conditions is presented in Appendix H. Guidelines for global analysis, which is required for strength and fatigue design, can be found in Appendix I.

### 5.1.2 Simulation of Vessel Dynamics

The following three approaches can be used for simulating vessel dynamics. All three approaches include certain techniques of approximation and therefore may not yield consistent results. Also all three approaches have limitations. The approach selected for the mooring design should be verified by model test data, full-scale test data, or a different analytical approach.

Because of the weather vane nature of the vessel with a single point mooring, the vessel may experience large low frequency yaw motions. These yaw motions may significantly affect vessel and mooring system responses, and therefore should be accounted for in time or frequency domain simulations as well as in model testing.

#### 5.1.2.1 Frequency Domain Approach

In this approach, the general equations of motion describing the response of the vessel are de-coupled and analyzed separately for mean, low, and wave frequency responses. Mean responses are calculated from static equilibrium between the environmental load and the mooring system's restoring force. Wave frequency and low frequency vessel motions are calculated from frequency domain approach that yields standard deviation motion responses. Statistical peak values, such as significant or maximum responses, are then evaluated based on certain peak response distributions. Finally, the wave frequency and low frequency responses are properly combined to yield the maximum combined response for specific storm duration.

To perform analysis for weather vane vessels, using the approach of frequency domain vessel dynamics, the vessel heading must be fixed. The fixed design heading, at which the mooring system responses are calculated, should be determined taking into consideration the mean equilibrium heading and low frequency yaw motions.

#### 5.1.2.2 Time Domain Approach

In this approach, the general equations of motion describing the combined mean, low, and wave frequency response of the vessel are solved in the time domain. The forcing functions include the mean, low frequency, and wave frequency forces due to wave, wind, current, and thrusters. The dynamic equations describing the vessel, mooring lines, risers, and thruster forces are all included in a single time domain simulation. Time histories of all system parameters (vessel displacements, mooring line tensions, and anchor loads, etc.) are available from the simulation, and the resulting time histories are then processed statistically to yield expected extreme values. The time domain simulation should be long enough to establish stable statistical peak values.

#### 5.1.2.3 Combined Time and Frequency Domain Approach

To reduce the complexity and computational effort associated with the full time domain simulation, a combined time and frequency domain approach is often employed. Time and frequency domain solutions for mean loads, wave and low frequency motions can be combined in different ways. In a typical approach, the mean and low frequency responses (vessel displacements, mooring line tensions, and anchor loads, etc.) are simulated in time domain while the wave frequency responses are solved separately in frequency domain. The frequency domain solution for wave frequency responses is processed to yield either statistical peak values or time histories, which are then superimposed on the mean and low frequency responses.

### 5.1.3 Simulation of Mooring Line Response

The responses of a mooring system to mean forces can be predicted by static catenary equations. Generally speaking, the responses to low frequency motions can also be predicted by the same method because of the long periods of these motions. The responses to wave frequency vessel motions can be predicted by either quasi-static or dynamic analysis.

#### 5.1.3.1 Quasi-Static Analysis

In this approach, the dynamic wave loads are taken into account by statically offsetting the vessel by wave induced motions. Vertical motions and dynamic effects associated with mass, damping and fluid acceleration on the mooring line are neglected. Research in mooring line dynamics has shown that the accuracy of tension predictions based on this method can vary widely depending on the vessel type, water depth, and line configuration.

#### 5.1.3.2 Dynamic Analysis

Dynamic analysis accounts for the time varying effects due to mass, damping, and fluid acceleration. In this approach, the time-varying fairlead motions are calculated from the vessel's surge, sway, heave, pitch, roll, and yaw motions. Dynamic models are used to predict mooring line responses to the fairlead motions.

Two methods, frequency domain and time domain analyses, can be used for predicting dynamic mooring loads. In the time domain method, all nonlinear effects including line stretch, line geometry, fluid loading, and sea bottom effects can be modeled. The frequency domain method, on the other hand, is always linear as the linear principle of superposition is used. Methods to approximate non-linear effects in the frequency domain and their limitations should be investigated to ensure acceptable solutions for the intended operation.

There are four primary nonlinear effects that can have an important influence on mooring line behavior:

a. Nonlinear Stretching Behavior of the Line—The strain or tangential stretch of the line is a function of the tension magnitude. Nonlinear behavior of this type typically occurs only in synthetic materials such as polyester. Chain and wire rope can be regarded as linear. In many cases the nonlinearity can be ignored and a linearized behavior assumed, using a representative tangent or secant modulus.

b. Changes in Geometry—The geometric nonlinearity is associated with large changes in shape of the mooring line.

c. Fluid Loading—The Morrison equation is most frequently used to represent fluid loading effects on mooring lines. The drag force on the line is proportional to the square of the relative velocity (between the fluid and the line), hence is nonlinear.

d. Bottom Effects—In many mooring designs, a considerable portion of the line is in contact with the seafloor. The interaction between the line and the seafloor is usually considered to be a frictional process and is hence nonlinear. In addition, the length of grounded line constantly changes, causing an interaction between this nonlinearity and the geometric nonlinearity.

### 5.1.4 Riser Considerations

The riser system interacts with the vessel and the mooring in several aspects. Wave and current loads on the risers increase the environmental loads resisted by the mooring, while the riser system stiffness provides assistance to the mooring. Furthermore damping from the riser system decreases the low frequency motions and in turn reduces the mooring load. The net result of these effects depends on a number of factors such as type and number of risers and water depth, etc. Mooring design should take into consideration the riser loads, stiffness, inertia, and damping unless it can be demonstrated that neglecting some or all riser effects will result in same or more conservative mooring design.

Some of the floating production units are equipped with steel catenary risers or midwater flowlines arranged in asymmetric patterns, which may impose large riser or flowline loads on the mooring system. In this case the riser or flowline loads should be carefully evaluated and properly accounted for.

### 5.1.5 Damping of low frequency motions

Low frequency motion of a moored vessel is narrow banded in frequency since it is dominated by the resonant response at the natural frequency of the moored vessel. The motion amplitude is highly dependent on the stiffness of the mooring system and damping. There is a substantial degree of uncertainty in the estimation of low frequency motions, particularly in the area of damping. There are four sources of damping:

a. Viscous damping of the vessel, including wind, wave, and current drag

b. Wave drift damping of the vessel

c. Mooring system damping

d. Riser system damping

The technology to estimate viscous damping has well been established, and viscous damping is normally included in the low frequency motion calculations. Wave drift damping, mooring system damping, and riser system damping, however, are more complex and are sometimes neglected because of a lack of understanding in these damping components. Recent research indicates that these damping components can be significant. They can even be higher than viscous damping under certain conditions, and neglecting them may lead to significant over-estimation of low frequency motions. In applications where low frequency motions is an important design factor, such as large ship-shaped vessels, it may be warranted to evaluate damping from all these sources either by analytical approach or model testing. A more detailed discussion on damping can be found in Appendix I.

## 5.2 MOORING ANALYSIS CONDITIONS

Mooring Analysis can be carried out for intact, damaged or transient conditions according to Table 1. Definitions of those conditions are given hereunder. Descriptions of analysis conditions for thruster assisted moorings are given in Section 5.9.2.

### 5.2.1 Intact Condition

This is the condition in which all mooring lines are intact.

### 5.2.2 Damaged Condition

This is the condition in which the vessel oscillates around a new mean position after a mooring line breakage or a thruster system failure.

### 5.2.3 Transient Condition

This is the condition in which the vessel is subjected to transient motions (overshooting) after a mooring line breakage or a thruster system failure.

### 5.2.4 Recommended Analysis Methods and Conditions

The analysis methods to be used and conditions to be analyzed for various designs are defined as follows:

Table 1—Recommended Analysis Methods and Conditions

| Type of Mooring | Analysis Method | Conditions to be Analyzed |
|---|---|---|
| Permanent Mooring | | |
| Strength design | Dynamic | Intact/damaged |
| Fatigue Design | Dynamic | Intact |
| Mobile Mooring | | |
| Strength design | Quasi-static or dynamic | Intact/damaged/ transient [a] |
| Fatigue Design | Not required | Not required |

[a] Transient analysis should be performed to check vessel offset (check for tension is not required) for mobile moorings under any one of the following conditions:
- A floating vessel is moored in the vicinity of another structure
- A MODU conducts a deep-water drilling operation where excessive transient motions may cause damage to the drilling riser such as stroke-out of the slip joint or exceeding the flex joint limit.

## 5.3 VESSEL OFFSET

The following definitions of vessel offset apply to any particular points of interest in the vessel.

### 5.3.1 Mean Offset

The mean offset is defined as the vessel displacement due to the combination of the steady components of wind, wave, current, and other external forces.

### 5.3.2 Maximum Offset

When the frequency domain approach is used for the simulation of vessel dynamics (Section 5.1.2.1), the maximum offset is defined as the mean offset plus maximum displacement due to combined wave frequency and low frequency vessel motions. Maximum offset can be determined by the following procedure.

Let:

$S_{mean}$ = Mean vessel offset

$S_{max}$ = Maximum vessel offset

$S_{wfmax}$ = Maximum wave frequency motion

$S_{wfsig}$ = Significant wave frequency motion

$S_{lfmax}$ = Maximum low frequency motion

$S_{lfsig}$ = Significant low frequency motion

The maximum offset is the larger of the values determined by the following equations:

$$S_{max} = S_{mean} + S_{lfmax} + S_{wfsig} \tag{5.1}$$

$$S_{max} = S_{mean} + S_{wfmax} + S_{lfsig} \tag{5.2}$$

The combined offset from different degrees of freedom (for example surge and sway) should be defined as the vector sum of individual degrees of freedom.

Alternatives to this approach are the time domain approach (Section 5.1.2.2) and the combined time and frequency domain approach (Section 5.1.2.3), which involve statistical processing of simulated time history to yield expected extreme offsets.

The above discussion applies to the intact and damaged conditions. For the transient condition, maximum offset is defined in Section 5.10.

## 5.4 LINE TENSION

### 5.4.1 Mean Tension

The mean tension is defined as the line tension corresponding to the mean offset of the vessel.

### 5.4.2 Maximum Tension

When the frequency domain approach is used for the simulation of vessel dynamics (Section 5.1.2.1), the maximum tension is the mean tension plus appropriately combined wave frequency and low frequency tensions. Maximum tension can be determined by the following procedure.

Let:

$T_{max}$ = Maximum tension

$T_{mean}$ = Mean tension

$T_{wfmax}$ = Maximum wave frequency tension

$T_{wfsig}$ = Significant wave frequency tension

$T_{lfmax}$ = Maximum low frequency tension

$T_{lfsig}$ = Significant low frequency tension

The maximum tension is the larger of the values determined by the following equations:

$$T_{max} = T_{mean} + T_{lfmax} + T_{wfsig} \qquad (5.3)$$

$$T_{max} = T_{mean} + T_{wfmax} + T_{lfsig} \qquad (5.4)$$

The above discussion applies to the intact and damaged conditions. For the transient condition, the maximum tension is defined in Section 5.10

Alternatives to this approach are the time domain approach (Section 5.1.2.2) and the combined time and frequency domain approach (Section 5.1.2.3), which involve statistical processing of simulated time history to yield expected extreme tensions.

## 5.5 STATISTICS OF PEAK VALUES

For wave frequency or low frequency responses (motions, tensions, etc.) that can be represented by a narrow band Gaussian process with Rayleigh distributed peaks, statistical peak values used in the frequency domain approach can be calculated from σ (standard deviation) by the following equations:

$$\text{Sig. Value} = 2\sigma \qquad (5.5)$$

$$\text{Max. Value} = \sqrt{2(\ln N)}\,\sigma \qquad (5.6)$$

$$N = T/T_a \qquad (5.7)$$

where

$T$ = the specified storm period (sec),

$T_a$ = the average zero up crossing period (sec) of the response.

A minimum of 3 hours should be specified for the storm period. For low frequency motions, $T_a$ can be taken as the natural period of the vessel/riser/mooring system $T_n$, which can be estimated by the following equation:

$$T_n = 2\pi \sqrt{m/k} \qquad (5.8)$$

where

$m$ = system mass including added mass,

$k$ = system stiffness taken at the vessel's mean position.

Equation 5.6, which is based on a narrow band Gaussian process with Rayleigh distributed peaks, may not always yield conservative predictions of maximum value. For non-Rayleigh peak distributions, alternative approach such as model test or time domain simulation for the specified storm duration is often used. If this approach is used, the simulation or model testing should be of sufficient length to establish reasonable confidence bounds for the expected maximum response in the storm duration. Typically responses in the storm duration should be simulated several times, and statistical fitting techniques should be used to establish the expected maximum response (Refer to Appendix I for more guidance).

Of particular concern is the passive turret moored vessels that will not maintain a constant heading because of low frequency yaw motions. The peak response distribution can be significantly affected by the variation in vessel heading, and the assumption of Rayleigh distribution can substantially under-estimate the maximum value.

## 5.6 STRENGTH ANALYSIS BASED ON FREQUENCY DOMAIN VESSEL DYNAMIC

### 5.6.1 Analysis for Spread Mooring Systems

In a mooring strength analysis based on the approach of frequency domain vessel dynamics, the mean position of the vessel is first determined by the force equilibrium in the surge, sway, and yaw directions. For vessels equipped with a spread mooring system where the yaw moment will not have a significant impact on the vessel heading and line tension, the yaw moment can be neglected. The responses to wave and low frequency excitations are then calculated and added to the mean position. The procedure outlined below is recommended for the strength analysis using a quasi-static or dynamic approach:

1. Determine the environmental criteria such as wind and current velocities, significant wave heights and periods, their relative directions, storm duration, and wind and wave spectrum for both the maximum design, and operating conditions.
2. Determine the mooring pattern, the characteristics of chain, wire, and synthetic rope to be deployed, and the initial tension.

3. Determine the mean environmental loads acting on the hull using either model test data or the procedures described in Section 4.
4. Determine the vessel's mean offset due to the mean environmental loads using static mooring analysis approach which should account for line stretching and friction.
5. Determine the low frequency motions using the data and procedures described in Section 4 or through a hydrodynamic motion analysis. Since calculation of low frequency motions requires the knowledge of mooring stiffness, the mooring stiffness at the mean offset should be determined first using a static mooring analysis approach.
6. Determine the significant and maximum wave frequency vessel motions from a hydrodynamic motion analysis or model test data.
7. Determine the vessel's maximum offset, suspended line length, maximum tension, and anchor load using Equations 5.1 to 5.4. For quasi-static analysis, the wave frequency line tensions are calculated by static catenary equations. For dynamic analysis, the wave frequency line tensions are calculated by frequency domain or time domain line dynamics.
8. Compare the maximum vessel offset, suspended line length, maximum line tension and anchor load from step 7 with the design criteria stated in Section 7. If the criteria are not met, modify the mooring design and repeat the analysis.

### 5.6.2 Analysis for Single Point Mooring Systems

Because of the weather vane nature of the vessel with a single point mooring, the vessel may experience large low frequency yaw motions, and time domain simulation or model testing may be most appropriate for the mooring design. To perform analysis based on the approach of frequency domain vessel dynamics, certain assumption on the vessel heading must be made. The technology is still in a state of development, and a number of approaches have been investigated and used by the industry. An example approach, which is considered to be conservative, is presented below.

In this example approach, the design heading at which the mooring system responses are calculated, is defined as the stable equilibrium heading of the vessel under mean environmental loads plus or minus the significant low frequency yaw motion. The recommended analysis procedure is as follows:

1. Determine the environmental criteria such as wind and current velocities, significant wave heights and periods, their relative directions, storm duration, and wind and wave spectrum for both the maximum design, and operating conditions.
2. Determine the mooring pattern, the characteristics of chain, wire, and synthetic rope to be deployed, and the initial tension.
3. Calculate the combined mean environmental yaw moment about the turret due to wave, wind, and current as a function of vessel heading. These yaw moments may be based on model tests or calculated wind, current, and wave drift force and moment coefficients.
4. From the mean environmental yaw moment, determine equilibrium headings and their stability. Stable vessel headings occur where the total environmental yaw moment is zero and a perturbation of the vessel heading results in a yaw moment opposed to the direction of the perturbation.
5. Determine the yaw moment stiffness at the equilibrium heading. For an unlocked turret the yaw moment stiffness is the rate of change of the mean environmental yaw moment with respect to heading.
6. Determine the standard deviation of the vessel’s low frequency yaw response about the stable equilibrium headings using a hydrodynamic motion analysis program. This requires knowledge of the low frequency yaw moment spectrum, the vessel yaw inertia and added inertia about the turret, the yaw moment stiffness, and the vessel and mooring system yaw damping. All of the above should be determined for the stable mean vessel heading under consideration.

   In the absence of better information, the linearized yaw damping coefficient about the turret can be estimated from the sway damping as follows.

   $$C_{Rz} = \frac{1}{3}C_y(a^3 + b^3)/(a + b) \tag{5.9}$$

   where

   $C_{Rz}$ = linear yaw damping coefficient, Nm/(Rad/sec),

   $C_y$ = linear sway damping, N/(m/sec),

   $a$ = length of vessel forward of turret,

   $b$ = length of vessel aft of turret.

7. Calculate the design heading as the vessel’s stable equilibrium heading plus or minus (which ever is more critical) the significant (two standard deviation) low frequency yaw response.
8. Fix the vessel heading at the design heading determined in step 7 above. Then follow the procedure described in steps 3 through 8 of Section 5.6.1 for calculating the mooring system responses.

### 5.7 STRENGTH ANALYSIS BASED ON TIME DOMAIN VESSEL DYNAMICS

As described in Section 5.1.2.2, time domain methods may be used to perform coupled simulations of mean, low, and wave frequency vessel and mooring system responses. This approach requires a time domain mooring analysis computer program, which solves the general equations of motion for the combined mean, low, and wave frequency responses of the vessel, mooring lines, and risers. A significant advantage of this approach is that low frequency damping from the vessel, mooring lines, and risers are internally generated in the simulation. Also the coupling between the vessel and the mooring/riser system can be fully accounted for. This approach requires, however, much higher computer resources and engineering effort. It also requires that the computer software be validated against either model test results or other analytical solutions. The recommended procedure is as follows:

1. Determine the environmental criteria such as wind and current velocities, significant wave heights and periods, their relative directions, storm duration, and wind and wave spectrum for both the maximum design, and operating conditions.
2. Determine the mooring pattern, the characteristics of chain, wire, and synthetic rope to be deployed, and the initial tension.
3. Determine the vessel's wind and current force coefficients, and hydrodynamic model of the system including vessel, riser, and mooring.
4. Perform a time domain simulation for the storm duration using a time domain vessel/mooring analysis program. Repeat the simulation several times using different seed values for generating the wave and wind time histories.
5. Use statistical analysis techniques to establish the expected extreme values of vessel offset, line tension, anchor loads, and grounded line length.
6. Compare the extreme vessel offset, line tension, anchor loads, and grounded line length from step 5 with the design criteria in Section 7.

The extreme value of a particular response parameter (vessel offset, line tension, anchor loads, grounded line length, etc.) realized in a single time domain simulation will vary about its expected value. Consequently, statistical fitting techniques and repetition of the simulation are required to establish reasonable confidence in the predicted extreme response. The number of repetitions of the simulation that are required will depend upon the extreme value characteristics of the system response parameter and the sophistication of the statistical methods used to predict the expected extreme value. In particular, the scatter of realizations of extreme values from individual storm simulations can be expected to increase as the number of low frequency cycles in the storm duration decreases (low frequency natural periods increase).

For turret moored vessels the low frequency natural period of the vessel's yaw motion will generally be significantly longer than the surge and sway natural periods. When the yaw natural periods are long, a large scatter in the realizations of extreme values from individual storm simulations can be expected. Consequently, a large number of repetitions of the storm simulation may be required to achieve confidence in the prediction of the expected extreme response values. More detailed time domain analysis guidelines can be found in Appendix I.

### 5.8 STRENGTH ANALYSIS BASED ON COMBINED TIME AND FREQUENCY DOMAIN VESSEL DYNAMICS

In this approach, the mean and low frequency responses are typically simulated in time domain which allows for nonlinearities in stiffness of mooring lines and risers, and in vessel forces due to quadratic terms and changes in yaw angle. Constant or variable thruster forces may also be modeled. Transient motions resulting from line breaking or thruster failure may be evaluated by specifying the time of failure in the time domain analysis. Unlike the full time domain approach as described in Section 5.7, evaluation of low frequency damping cannot be included in this simulation because of the absence of wave frequency components. Damping must be evaluated separately and treated as an input parameter.

Wave frequency vessel motions are calculated separately in the frequency domain from the vessel's motion RAOs and the wave spectra. These motions can be combined with the low frequency motions in two ways. In the first method, the frequency domain solution of wave frequency vessel motions is transformed to a time history, which is added to the mean and low frequency vessel displacement to arrive at the combined vessel displacement. In this case the seed values for generating wave frequency and low frequency time histories should be the same to yield consistent results. In the second method, the mean and low frequency motions time histories are statistically analyzed to determine the peak values, which are then combined with the peak values of wave frequency motions to arrive maximum vessel offset, as described in Section 5.3.

### 5.9 THRUSTER ASSISTED MOORING (TAM)

#### 5.9.1 Introduction

In evaluating the capability of the thruster assist systems, it is necessary to consider the equipment that support and control the thrusters, their modes of failure and repair times, and the training and experience of the personnel operating the systems. The determination of thruster forces is a complex multi-disciplinary undertaking, requiring knowledge of thruster design, thruster performance in normal and extreme seastates, thrust allocation logic and optimization, control and monitoring systems, reliability of mechanical and electrical equipment, computer hardware and software, operational practices, operator training, and human factors, etc. This document does not provide detailed recommendations and methods for the design of thruster assisted moorings. Rather, the guidance and background information contained in this section is intended to be used by the mooring analyst, when the thruster systems have been designed and proved to acceptable industry standards.

Thrusters may be used to assist the mooring system by reducing the mean environmental forces, controlling the vessel's heading, damping low frequency motions, or a combination of these functions.

### 5.9.2 Analysis Conditions for Thruster-assisted Moorings

Intact and damaged TAM system definitions are given below.

Table 2—Intact and Damaged TAM Definitions

| TAM Definition and Mooring Factor of Safety | Mooring System Condition | Thruster System Condition |
|---|---|---|
| Intact | Intact | Intact |
| Damaged | Intact | Damaged |
| Damaged | Damaged | Intact |

Table 2 implies that if both the mooring system and the thruster system condition are "intact", then the TAM definition and mooring factor of safety is "intact". If either the mooring system or the thruster system condition is "damaged", then the TAM definition and mooring factor of safety is "damaged".

### 5.9.3 Determination of Allowable Thrust

When thrusters are used to assist the mooring system, it is necessary to quantify the allowable thrust that may be used in the mooring analysis. The recommended procedure is outlined below:

1. Determine the available thrust taking into consideration the efficiency of the thrusters and losses due to vessel motions, current, thruster/hull and thruster/thruster interference effects, and any directional restrictions. Guidance for evaluating available thrust is provided in Appendix F.
2. Determine the worst thruster system failure. Failure modes and effects analyses (FMEA) should be performed to identify the worst single failure. The definition of the worst single failure should allow for thruster system availability (mean time to failure and mean time to repair) over the design life of the installation.
3. Determine the allowable thrust:

   For automatic thruster control systems, the allowable thrust can be determined as follows:

   - For the intact thruster condition, the allowable thrust is equal to the available thrust or effective bollard pull when the thruster system is operating normally.
   - For the damaged thruster condition, the allowable thrust is equal to the available thrust after accounting for the worst failure as determined by FMEA.

   For manual thruster control systems, the allowable thrust as determined above should be multiplied by a reduction factor of 0.7.

   The allowable thrust used in the mooring analysis should be verified during thruster system sea trials. A detailed discussion on thruster assisted turret mooring can be found in IMCA Report No. GM-2096-0600-2561 "Guidance on Thruster Assisted Station Keeping by FPSO and Similar Turret Moored Vessels" (Reference 8).

### 5.9.4 Analysis for Thruster Assisted Mooring

In a thruster-assisted mooring system, the load sharing between the thruster and the mooring systems is complex and can only be fully accounted for with a time domain system dynamic analysis. However, a simple mean load reduction method would yield reasonable results.

#### 5.9.4.1 Mean Load Reduction Method

In this simplified approach the thrusters are assumed to counter only the mean environmental loads in the surge, sway, and yaw directions. Allowable thrusts from thrusters are first evaluated and then subtracted from the mean load. The remainder of the mean load, and the wave and low frequency motions would be taken by the mooring system.

For vessels with a spread mooring where the vessel heading is held stable by the mooring lines, the surge and sway components of the allowable thrust can be subtracted from the mean surge and sway mean environmental forces. The mooring response can then be evaluated using analysis procedure for mooring systems without thruster assist.

For vessels with a single point mooring, the main function of the thrusters is heading control. Reduction of surge and sway mean loads by thrusters should generally not be considered. For vessels with high thruster capacity that significantly exceeds the heading control requirements, mean load reduction can be considered for extreme environments. In this case, a conservative portion of the thruster capacity should first be allocated to heading control. The remaining portion can then be used to reduce the surge and sway mean load.

#### 5.9.4.2 System Dynamic Analysis

A system dynamic analysis is normally performed using a three axis (surge, sway, and yaw) time domain simulator. This simulator generates the mean offset, low frequency vessel motions, and thruster responses corresponding to specific environmental force time records. In this analysis, constant wind, current, steady wave drift forces, and the low frequency wind and wave drift forces are typically included. Wave frequency wave forces, which are not countered by the thruster system, can be excluded in the simulation. The wave frequency motions can be computed separately using a vessel motion program and added to the output from the time domain simulator. To obtain a proper maximum value from the time domain simulation, it may be necessary to generate a number of force and response records for the storm duration and calculate the expected maximum value using a statistical approach.

### 5.10 TRANSIENT ANALYSIS

A moored vessel will experience transient motions after a mooring line breakage or thruster system failure before it settles at a new equilibrium position. Transient analysis checking maximum offset is required for certain mobile mooring operations, as specified in Section 5.2.4. Transient analysis of a moored vessel under wind, wave, current, and thruster loading is complex and may require a time domain solution. To simplify the analysis, a combination of time domain (transient motions) and frequency domain (vessel motions) approaches can be used.

#### 5.10.1 Combination of Time and Frequency Domain Analysis

In this approach, the maximum transient motion is first determined using a time domain approach. Then vessel motions obtained from frequency domain approach are superimposed on the transient motion. The recommended procedure is as follows:

1. Compute the equilibrium position under mean load for an intact mooring.
2. Break a line and compute the maximum transient motion (overshoot) in the time domain with mean load only and with the mooring system stiffness, *K*, updated at each time step. Generally a model with three degrees of freedom (surge, sway, and yaw) is required.
3. Determine maximum vessel offset by the following equation.

$$S_{max} = S_{mean} + S_t + S_{wfsig} + S_{lfsig} \tag{5.10}$$

where

$S_{mean}$ = mean offset as calculated in step 1,

$S_t$ = maximum transient motion (overshoot) with respect to the equilibrium position from step 1 as determined in step 2,

$S_{wfsig}$ = significant wave frequency motion, calculated in the frequency domain,

$S_{lfsig}$ = significant low frequency motion, calculated in the frequency domain using the damaged mooring system stiffness.

#### 5.10.2 Time Domain Analysis

Time domain transient analysis is similar to the time domain analysis for vessel dynamics described in Section 5.7. The only difference is that a mooring line is removed during the simulation to model a line break. The simulation should be repeated for a number of wave force records, and for the break to occur at a number of times during each record. The maximum offset observed during these simulations, or the most probable maximum offset estimated from the results of these simulations, should be used for the design.

# 6 Fatigue Analysis

### 6.1 BASIC CONSIDERATIONS

Fatigue life estimates are made by comparing the long-term cyclic loading in a mooring line component with the resistance of that component to fatigue damage. For mooring systems, a T-N approach is normally used. This approach uses a T-N curve, which gives the number of cycles to failure for a specific mooring component as a function of constant normalized tension range, based on the results of experiments.

The Miner's Rule is used to calculate the annual cumulative fatigue damage ratio D:

$$D = \sum \frac{n_i}{N_i} \tag{6.1}$$

where

$n_i$ = number of cycles per year within the tension range interval *i*.

$N_i$ = number of cycles to failure at normalized tension range i as given by the appropriated T-N curve.

The design fatigue life, which is 1/*D*, should be greater than the field service life multiplied by a factor of safety defined in Section 7.5. For used mooring components, fatigue damage from previous operations should be taken into account.

Quasi-static approach should not be used for calculating tension ranges due to its severe deficiency in estimating wave frequency tensions. Both time and frequency domain dynamic approaches may be used for tension range predictions. Alternatively, tension ranges can be obtained from model testing.

Guidelines for mooring fatigue analysis are provided in the following sections. A fatigue analysis example can be found in Appendix J. Guidelines for mooring fatigue analysis under VIM conditions are presented in Appendix H. Fatigue damage to piles from installation loading is addressed in Appendix E, Section E.7.

## 6.2 FATIGUE RESISTANCE OF MOORING COMPONENTS

T-N curves for various mooring components should be based on fatigue test data for these components and a regression analysis. The component T-N curve typically corresponds to a lower bound defined as the lower bound of a two-sided, 95 percent prediction interval (2.5% probability of fatigue resistance exceedance). However, this practice is not followed precisely for some recommended T-N curves because of uncertainties in the test data such as:

- Insufficient test data.
- Lack of test data in the low tension regime where fatigue damage is most severe.
- Data acquired in unrealistic test environments such as high frequency test in seawater environment.
- Lack of a broad representation of test samples such as testing samples of 1 or 2 sizes from 1 or 2 manufacturers.

Most of the uncertainties are due to difficulties to meet high funding requirement for fatigue testing of mooring components. To address these uncertainties, special considerations were sometimes taken in addition to regression analysis for test data. As a result, some of the recommended T-N curves may represent the “best estimates” of the fatigue resistance of the component instead of the lower bound curves from a simple regression analysis. The special considerations include:

- Fatigue failure probability analysis.
- Experience check such as performing a fatigue analysis using a T-N curve under investigation to see whether the fatigue life prediction severely violates industry experience.
- Other analytical approaches such as finite element for stress concentration or fracture mechanics.
- Comparison with other published T-N curves for the component.

When selecting a T-N curve for fatigue analysis of a component, such as studless chain, the designer should consider the safety margin in all three integral parts of a fatigue life assessment (T-N curves, factor of safety, and analysis method) against experience and available test data.

### 6.2.1 Tension-Tension (T-T) Fatigue

T-N curves presented in Equation 6.2 can be used for calculating nominal tension fatigue lives of mooring components.

$$NR^M = K \tag{6.2}$$

where

$N$ = Number of cycles,

$R$ = Ratio of tension range (double amplitude) to reference breaking strength (RBS). For chain, RBS is taken as MBS (minimum breaking strength) of ORQ common chain link of the same size for ORQ, R3, R4, and R4S common or connecting links. Guidance on increase of chain diameter for corrosion and wear and its effect on fatigue life calculation are given in 7.6. For wire rope, RBS is the same as MBS.

Let $L_m$ = Ratio of mean load to reference breaking strength for wire rope, *M* and *K* values are provided in Table 3.

Table 3—*M* and *K* Values

| Component | *M* | *K* |
|---|---|---|
| Common studlink | 3.0 | 1,000 |
| Common studless link | 3.0 | 316 |
| Baldt and Kenter connecting link | 3.0 | 178 |
| Six/multi strand rope | 4.09 | $10^{(3.20 - 2.79L_m)}$ |
| Spiral strand rope | 5.05 | $10^{(3.25 - 3.43L_m)}$ |

The mooring component fatigue design curves are plotted in Figure 13. Note the wire rope curves are mean load dependent, and a mean load of 30% MBL is assumed in the plot.

The equation for studlink does not applied to links with loose stud. Figure 13 indicates that studless chain has lower fatigue life than studlink chain. However, Studless chain does not have the fatigue issues associated with stud, such as loose stud, stud weld crack, sharp corners at stud foot print, corrosion between stud and link, and defects hidden behind the stud that cannot be detected by inspection. It is important to consider all factors affecting fatigue life in the selection of chain type.

The above T-N curves should be used in conjunction with a factor of safety 3, as specified in Section 7.5. A reliability study indicates that the T-N curves combined with a factor of safety of 3 and Simple Summation method (Section 6.3) would yield fatigue designs with acceptable probability of failure (see Reference 18 for details). Since the slope of the above T-N curve for studless chains lies outside the 95% confidence range from a regression analysis on the available test data, the T-N curves based on regression analysis of the test data presented in 2 chain fatigue test JIP reports (References 19 and 20) may also be considered (Reference 21).

The T-N curves for wire rope are only good for wire ropes protected from corrosion. Elements for corrosion protection include galvanizing, jacketing, blocking compound, and zinc filler wires. Careful investigation considering the design life, inspection, and change-out strategy should be carried out to determine the combination of these elements needed for a specific project.

Mean load has a significant influence on wire rope fatigue life and therefore should be included in the design curve equations. A mean load of 0.3 RBS is considered to be representative for conventional mooring systems. For wire rope fatigue analysis, the following methods can be considered to account for the mean load effect:

1. For each seastate, determine the mean load and the corresponding design curve, which is then used to calculate the fatigue damage for that seastate. This requires using different design curves for different seastates.
2. Determine the average mean load for seastates causing significant fatigue damage and use the design curve for the average mean load for all seastates.
3. Use the design curve for a mean load of 0.3 RBS for conventional mooring systems. For a taut leg mooring or TLP tether system, method (1) or (2) should be used.

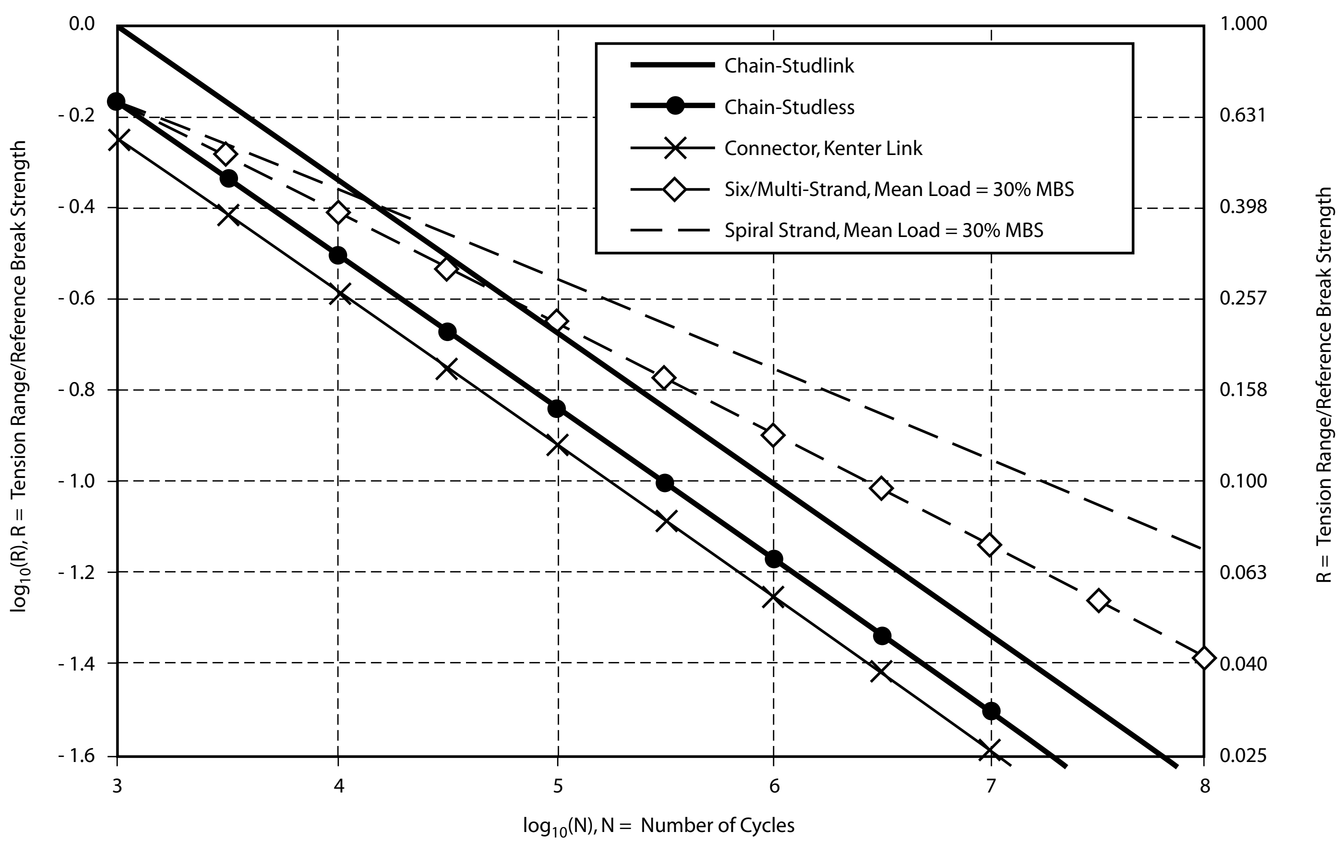



Figure 13—Mooring Fatigue Design Curves

Among the three methods, Method (1) is most accurate but requires more computational effort. If method (2) and (3) are used, a sensitivity study should be performed to ensure these simplified approaches produce conservative predictions.

Data for other types of connecting links are insufficient for generating design curves. Limited test data indicate that the fatigue life of D-shackles is comparable to that of common links of the same size and grade, provided the shackle is machined fit with close tolerance, no locking pin is used penetrating through shackle body.

### 6.2.2 Bending - Tension (B-T) and Free Bending Fatigue

Data for bending-tension fatigue of chain and wire rope are insufficient for generating design curves. In the absence of a fatigue design, precautionary measures should be taken to avoid mooring failure due to bending-tension fatigue. For example, the fairlead to line diameter ratio (*D/d*) should be large enough to avoid excessive bending. The portion of mooring line in direct contact with a fairlead should be regularly inspected. Also this portion should be periodically shifted to avoid constant bending in one area. A study comparing the bending-tension and tension-tension fatigue lives of mooring lines on a semi-submersible under the North Sea environment provided the following data which can be used as reference values for establishing operation policy to avoid excessive bending tension fatigue for wire ropes.

Note that the analysis for bending-tension fatigue is very complex and the values in Table 4 are rough estimates only. Additional more margin of safety is recommended when these data are used.

Free bending at the wire rope terminations can also significantly reduce the wire rope fatigue life. To avoid premature fatigue failure in permanent moorings, a bend limiting device should be incorporated at these locations. Such a device is designed to smoothly transfer the loads from the termination to the rope.

As for tension bending of chain, the portion of mooring line in direct contact with a fairlead should also be regularly inspected and shifted to avoid constant bending in one area. In general, the worst load case is to tension-bend a horizontal link over a shallow groove, which results in very high stress in the stud weld region. Therefore, fairleads must be shaped and sized to avoid this type of unfavorable bending of chain links. Limited fatigue T-N tests of chains over a five-pocket fairlead indicate 5% to 20% of B-T fatigue life in terms of T-T fatigue life. A seven-pocket fairlead design generally gives much improved B-T fatigue life.

Industry experience indicates that chain links in direct contact with fairleads, bending shoes, chain stoppers, or hawser pipes can be subjected to additional stress concentration, which in turn can cause premature fatigue failure. Stress concentration under these conditions should be carefully evaluated by finite element analysis, especially for permanent moorings. Fatigue analysis should account for the additional stress concentration in these areas.

Table 4—Comparison of B-T and T-T Fatigue Life

| Rope Type | D/d Ratio | B-T Fatigue Life in Terms of Percentage of T-T Fatigue Life |
|---|---|---|
| Six strand | 20 | 3 |
| Six strand | 70 | 8 |
| Multi-strand | 20 | 5 |
| Multi-strand | 70 | 15 |
| Spiral strand | 20 | 0.5 |
| Spiral strand | 70 | 1.5 |

## 6.3 FATIGUE ANALYSIS FORMULATION

### 6.3.1 Accumulated Fatigue Damage

The annual fatigue damage, accumulated in a mooring line component as a result of cyclic loading, is summed up from the fatigue damage arising in a set of environmental states chosen to discretise the long-term environment that the mooring system is subjected to:

$$D = \sum_{i=1}^{i=n} D_i \tag{6.3}$$

where

$D_i$ = the annual fatigue damage to the component due to environmental state $i$.

The discretisation into $i = 1,\ldots,n$ environmental states should be sufficiently detailed to avoid any significant error in the total. Each environmental state is defined in terms of the wind, wave, and current parameters and directions required to compute mooring system responses. The probability of occurrence, $P_i$, is required for each environmental state. The calculated fatigue life of the mooring system is:

$$L = 1 / D \text{ (years)} \tag{6.4}$$

The annual fatigue damage accumulated in an individual state may be computed as:

$$D_i = \frac{n_i}{K} E[R_i^M] \tag{6.5}$$

where

$M$ and $K$ *are* defined in Table 3 and:

$n_i$ = the number of tension cycles encountered in state $i$ per year,

$E[R_i^M]$ = the expected value of the normalized tension range $R_i$ raised to the power $M$, in state $i$,

The number of tension cycles per year in each state can be determined as

$$n_i = v_i \cdot T_i = v_i \cdot P_i \cdot 3.15576 \times 10^7 \quad (6.6)$$

where

$v_i$ = the zero up-crossing frequency (hertz) of the tension spectrum in environmental state $i$,

$T_i$ = the time spent in environmental state $i$ per year,

$P_i$ = the probability of occurrence of environmental state $i$.

The normalized tension ranges should be computed taking into account the effects of pretension and the effects of the environmental loads due to wind, waves and current. Although the cumulative effect of the tension cycles is required in the fatigue analysis, rather than the extreme tension, it is still necessary to take care to compute the dynamic response of the mooring line to wave-frequency loads at a representative offset for each environmental state.

### 6.3.2 Methods for Combining Low and Wave Frequency Fatigue Damage

Four methods can be considered, as follows, for combining fatigue damage due to low frequency and wave frequency tensions.

1. Simple Summation—In this approach, low frequency and wave frequency fatigue damages are calculated independently. The total damage is assumed to be the sum of the two.
2. Combined Spectrum—In this approach, the combined low frequency and wave frequency spectrum is first calculated. The total damage is then calculated using the standard deviation of the combined spectrum.
3. Combined Spectrum with Dual Narrow-Banded Correction Factor—In this approach, a correction factor is applied to the result of the combined spectrum method (method 2).
4. Time Domain Cycle Counting—In this approach, fatigue damage is calculated using a cycle counting method, such as the Rainflow method, to estimate the number of tension cycles and the expected value of the tension range from a time history of tensions. The tension time history may be determined directly by a time domain mooring analysis or it may be generated from the combined low and wave frequency tension spectrum.

The time domain cycle counting (Method 4) is generally considered to be the most accurate method of calculating fatigue damage, however the analysis is relatively time consuming. Simple summation (Method 1) will generally give an acceptable estimate of fatigue life if the ratio of standard deviation tensions between wave frequency and low frequency response satisfies the following condition:

$$\sigma_{wf}/\sigma_{lf} \geq 1.5 \quad \text{or} \quad \sigma_{wf}/\sigma_{lf} \leq 0.05$$

Where $\sigma_{wf}$ and $\sigma_{lf}$ are wave frequency and low frequency standard deviation tension, respectively. However, this method (Method 1) may underestimate fatigue damage compared to Method 4 if both low and wave frequency tensions contribute significantly to the total fatigue damage. The combined spectrum method (Method 2) is always conservative and may significantly overestimate the actual fatigue damage. The combined spectrum with dual narrow-banded correction factor method (Method 3) is an improvement, which yields less conservative predictions than Method 2. It is suitable for the cases where both low frequency and wave frequency tensions cause significant fatigue damage. However, when the fatigue damage is dominated by low frequency tensions, this method (Method 3) will overestimate the fatigue damage. Analysis procedures for Methods 1, 2, and 3 are presented below.

#### 6.3.2.1 Simple Summation

Wave frequency and low frequency fatigue damages for environmental state $i$, are estimated by the following equation, which is based on a Rayleigh distribution of tension peaks.

$$D_i = \frac{n_{Wi}}{K}(\sqrt{2}R_{W\sigma i})^M \cdot \Gamma(1+M/2) + \frac{n_{Li}}{K}(\sqrt{2}R_{L\sigma i})^M \cdot \Gamma(1+M/2) \quad (6.7)$$

where

$D_i$ = annual fatigue damage from wave frequency and low frequency tensions in environmental state $i$,

$n_{Wi}$ = number of wave frequency tension cycles per year for environmental state $i$, from Equation 6.6,

$R_{W\sigma i}$ = ratio of standard deviation of wave frequency tension range to RBS. The standard deviation of the tension range should be taken as twice the standard deviation of tension,

$\Gamma$ = Gamma function,

$n_{Li}$ = number of low frequency tension cycles per year for environmental state $i$, from Equation 6.6. The average zero up-crossing frequency may be estimated by $1/T_n$, where $T_n$ is the natural period of the vessel computed at the vessel's mean position,

$R_{L\sigma i}$ = ratio of standard deviation of low frequency tension range to RBS. The standard deviation of the tension range should be taken as twice the standard deviation of tension.

#### 6.3.2.2 Combined Spectrum

In the combined spectrum method fatigue damage for environmental state $i$ is estimated from the following equation, which is based on a Rayleigh distribution of tension peaks.

$$D_i = \frac{n_i}{K}(\sqrt{2}R_{\sigma i})^M \cdot \Gamma(1+M/2) \tag{6.8}$$

In Equation 6.8 the standard deviation of the combined low and wave frequency tension range, $R_{\sigma i}$, is computed from the standard deviations of the low, $R_{L\sigma i}$, and wave, $R_{W\sigma i}$, frequency tension ranges by,

$$R_{\sigma i} = \sqrt{R^2_{W\sigma i} + R^2_{L\sigma i}} \tag{6.9}$$

The number of cycles, $n_i$, in the combined spectrum is calculated from Equation 6.6 with the zero up-crossing frequency (hertz) of the combined spectrum, $\nu_{Ci}$, given by,

$$\nu_{Ci} = \sqrt{\lambda_{Li}\nu^2_{Li} + \lambda_{Wi}\nu^2_{Wi}} \tag{6.10}$$

where

$\nu_{Wi}$ = the zero up-crossing frequency (hertz) of the wave frequency tension spectrum in environmental state $i$,

$\nu_{Li}$ = the zero up-crossing frequency (hertz) of the low frequency tension spectrum in environmental state $i$.

and $\lambda_{Li}$ and $\lambda_{Wi}$ are given by,

$$\lambda_{Li} = \frac{R^2_{Li}}{R^2_{Li} + R^2_{Wi}} \quad , \quad \lambda_{Wi} = \frac{R^2_{Wi}}{R^2_{Li} + R^2_{Wi}} \tag{6.11}$$

#### 6.3.2.3 Combined Spectrum with Dual Narrow-Banded Correction Factor

The combined spectrum with dual narrow-banded correction factor method uses the result of the combined spectrum method and multiplies it by a correction factor, $\rho_i$, based on the two frequency bands that are present in the tension process. The fatigue damage for environmental state $i$ is estimated from Equation 6.12.

$$D_i = \rho_i \frac{n_i}{K}(\sqrt{2}R_{\sigma i})^M \cdot \Gamma(1+M/2) \tag{6.12}$$

The correction factor is given by:

$$\rho_i = \frac{\nu_{ei}}{\nu_{Ci}}\left[(\lambda_{Li})^{\frac{M}{2}+2} \cdot \left(1-\sqrt{\frac{\lambda_{Wi}}{\lambda_{Li}}}\right) + \sqrt{\pi\lambda_{Li}\lambda_{Wi}} \cdot \frac{M\Gamma\left(\frac{1+M}{2}\right)}{\Gamma\left(\frac{2+M}{2}\right)}\right] + \frac{\nu_{wi}}{\nu_{Ci}} \cdot (\lambda_{Wi})^{\frac{M}{2}} \tag{6.13}$$

Where the subscript $e$ refers to the envelope of the combined tension process, and the mean up-crossing frequency (hertz) of the envelope of the normalized tension process, $\nu_{ei}$, is given by,

$$\nu_{ei} = \sqrt{\lambda^2_{Li}\nu^2_{Li} + \lambda_{Li}\lambda_{Wi}\nu^2_{Wi}\delta^2_{Wi}} \tag{6.14}$$

where

$\delta_{Wi}$ = the bandwidth parameter for the wave frequency part of the normalized tension process, which may be taken as equal to 0.1.

### 6.4 FATIGUE ANALYSIS PROCEDURE

The recommended procedure for a detailed fatigue analysis is described below.

1. The long term environmental events can be represented by a number of discrete environmental states. Each environmental state consists of a reference direction and a reference seastate characterized by significant wave height, peak spectral period (or equivalent), spectral shape, current velocity, and wind velocity. The probability of occurrence of each environmental state must be specified. In general, 8 to 12 reference directions provide a good representation of the directional distribution of a long term environment. The required number of reference seastates normally falls in a range of 10 to 50. Fatigue damage prediction can be fairly sensitive to this number for certain mooring systems, and therefore it is best determined by a sensitivity study.
2. Each environmental state can be analyzed analogously to the procedure used for mooring strength analysis as described in Section 5. The wave and low frequency tensions can be computed about the position of the mooring system under mean loading only.
3. Determine the $M$ and $K$ values for Equation 6.2. (Table 3).

4. Compute the annual fatigue damage from one environment (one seastate in one direction) due to both the low frequency and the wave frequency tension according to methods presented in Section 6.3.
5. Repeat Step 4 for all environmental states and compute the total annual fatigue damage *D* and fatigue life *L* according to Equations 6.3 and 6.4.

# 7 Design Criteria

## 7.1 VESSEL OFFSET

Vessel offset limits should be established by clearance requirements and limitation of equipment such as risers and gangways.

## 7.2 LINE TENSION

A tension limit can be expressed as a percentage of the minimum breaking strength (MBS) of the mooring component. MBS is defined as the breaking strength guaranteed by the mooring component manufacturer. The minimum breaking strength of chain may be taken as the break test load (BTL). Guidance on increase of chain diameter for corrosion and wear and its effect on strength calculation are given in Section 7.6.

Tension limits and equivalent factors of safety for various conditions and analysis methods are provided in Table 5.

The criteria in Table 5 are intended for moorings which are properly maintained and inspected, and have connecting hardware with breaking strengths equivalent to the mooring lines.

The same mooring line tension safety factors are applicable for thruster assisted moorings (TAM) assuming the thruster system is reliable and has significant contribution to the station-keeping capability of the floating structure.

Table 5—Tension Limits and Safety Factors

| | **Analysis Method** | **Tension Limit (Percent of MBS)** | **Equivalent Factor of Safety** |
|---|---|---|---|
| Intact | Quasi-static | 50 | 2.0 |
| Intact | Dynamic | 60 | 1.67 |
| Damaged | Quasi-static | 70 | 1.43 |
| Damaged | Dynamic | 80 | 1.25 |

## 7.3 LINE LENGTH

If drag anchors are used, the outboard mooring line length should in general be sufficient to prevent anchor uplift under conditions as specified in Section 5.2.4. This requirement is especially important for anchors in sand and hard soil where anchor penetration is shallow. Uplift of drag anchor may be permitted if it can be demonstrated that the anchor has sufficient vertical load resistance for the soil condition under consideration. Guidelines for the use of drag anchor to resist vertical loads are provided in Appendix D.

Shorter line lengths can be used for moorings with other anchoring systems that can resist substantial vertical pulls such as pile anchors, suction caissons, or vertically loaded anchors.

## 7.4 ANCHORING SYSTEMS

### 7.4.1 Drag Anchor

The holding capacity of a drag anchor in a particular soil condition represents the maximum horizontal steady pull that can be resisted by the anchor at continuous drag. Evaluation of anchor holding capacity is addressed in Appendix D. Factor of safety for drag embedment anchors, which is defined as anchor holding capacity divided by maximum anchor load, is provided in Table 6.

Table 6—Drag Anchor Safety Factors

| | **Quasi Static Analysis** | **Dynamic Analysis** |
|---|---|---|
| Permanent Mooring | | |
| Intact condition | | 1.5 |
| Damaged condition | | 1.0 |
| Mobile Mooring | | |
| Intact condition | 1.0 | 0.8 |
| Damaged condition | Not required | Not required |

### 7.4.2 Pile Anchor, Vertically Loaded Plate Anchor, and Gravity Anchor

Factors of safety, defined as anchor capacity divided by maximum anchor load from dynamic analysis, for anchor piles, plate anchors, and gravity anchors are provided in Table 7.

A discussion on pile and plate anchor design and installation can be found in Appendix E. Further discussions on factor of safety for anchors can be found in Section E.3.1 (suction anchor) and Section E.4.4 (drag embedded plate anchor).

### 7.4.3 Mooring Test Load

After installation, the mooring should be test loaded to ensure adequate holding capacity of the anchoring system, eliminate slack in the grounded portion of the mooring lines, detect damage to the mooring components during installation, and ensure that the mooring line's inverse catenary is sufficiently formed to prevent unacceptable mooring line slacking due to additional inverse catenary cut-in during storm conditions.

For permanent moorings with drag anchors, the mooring lines should be test loaded to at least 80% of the maximum storm load determined by a dynamic mooring analysis for the intact condition. This requirement is mainly based on industry experience with drag anchors in soft clay where deep anchor penetration can be achieved. For drag anchors on hard or sand seafloors where anchor penetration is typically limited to no more than one fluke length, higher anchor test load may be more appropriate. For permanent moorings with pile anchors and suction caissons, a test load should be determined based on the consideration of eliminating the slack in the grounded mooring lines, forming reverse catenary in the mooring line below the seafloor, and detecting damage to the mooring components during installation.

For mobile moorings with drag anchors, test load should be determined by a number of factors such as type of anchor, soil condition, winch pull limit, and anchor retrieval. However, they should meet the following minimum requirements:

The test load at the anchor shank should not be less than 3 times the anchor weight.

The mooring test load at winch should not be less than the mean line tension for an intact mooring under the maximum design condition. This requirement is for close proximity moorings only.

Duration of the test load should be at least 15 minutes for both mobile and permanent moorings. Refer to Appendix E, Section E.6.3 for test loading of plate anchors.

## 7.5 FATIGUE LIFE

The predicted mooring component fatigue life shall be at least 3 times the design service life of the mooring system.

## 7.6 CORROSION AND WEAR

Protection against chain corrosion and wear is normally provided by increase of chain diameter. Current industry practice is to increase the chain diameter by 0.2 mm to 0.4 mm per service year in the splash zone and in the dip or thrash zone on hard bottom. The diameter increase is reduced to 0.1 mm to 0.2 mm per service year in the remaining length. For strength analysis, the diameter of the chain should not include the increase for corrosion and wear. For fatigue analysis, the diameters of the chain for different periods of service life can be established if the corrosion rate can be predicted. In this case the chain diameter for a certain period is the nominal diameter minus the expected corrosion and wear for the time up to that period. It should be noted that corrosion rate depends on type of steel and sea water environment, and is often significantly acclerated in the first few years of service. If the corrosion rate is uncertain, a conservative approach using the chain diameter excluding the increase for corrosion and wear should be considered for the fatigue analysis.

Corrosion of wire rope at connections to sockets can be excessive due to the galvanized wire acting as an anode for adjacent components. For permanent systems it is recommended that either the wire be electrically isolated from the socket or that the socket be isolated from the adjacent component. Additional corrosion protection can be achieved by adding sacrificial anodes to this area.

## 7.7 CLEARANCE

### 7.7.1 Basic Considerations

Contact between a floating vessel, its mooring components, and other marine installations should be avoided with a comfortable margin, especially under severe environments. For floating vessels moored in close proximity, it is common practice to move one of the vessels away before a threshold environment is reached. Under normal operating environment or in areas of mild environment, close proximity mooring can be acceptable, assuming some clearance criteria are met.

The clearances between a floating vessel, its mooring components, and other marine installations should be determined for the conditions specified in Table 1. To determine clearance criteria, many factors should be considered, such as environment, water depth, and risk of injury, asset and environmental damage, etc. Conservative criteria should be established based on these considerations. As minimum, the clearance requirements provided below should be met.

### 7.7.2 Mooring Line Crossing Pipeline

Where a mooring line crosses a pipeline within the elevated part of its catenary, a minimum vertical clearance of 10 m under the intact condition should be maintained. A mooring line may contact a protected pipeline provided this contact remains throughout the full range of predicted intact line tensions thus the contact point must not occur in the thrash zone.

Table 7—Safety Factors for Pile, Plate, and Gravity Anchors (Dynamic Analysis)

| Condition | Suction/Driven Pile and Gravity Anchor | | | | Plate Anchor | |
|---|---|---|---|---|---|---|
| | Permanent | | Mobile | | | |
| | Lateral | Axial | Lateral | Axial | Permanent | Mobile |
| Intact | 1.6 | 2.0 | 1.2 | 1.5 | 2.0 | 1.5 |
| Damaged | 1.2 | 1.5 | 1.0 | 1.2 | 1.5 | 1.2 |

#### 7.7.3 Mooring Line Crossing Mooring Line

Where a mooring line crosses another mooring line, a minimum vertical clearance of 10 m under the intact condition should be maintained if one of the mooring line at the crossing is grounded. The minimum clearance should be increased to 20 m if both lines are suspended at the crossing.

#### 7.7.4 Horizontal Distance Between Installations

A minimum horizontal clearance of 10 m should be maintained between the moored unit (or its mooring lines) and any other installation. This clearance is required for all conditions as defined in Section 5.2.

#### 7.7.5 Clearance Between a Drag Anchor and Other Installations

If a marine installation lies in the dragging path between the anchor and the floating unit, the final anchor position should allow at least 300 m drag before contacting the marine installation. Otherwise the anchor should be at least 100 m from the marine installation.

#### 7.7.6 Clearance Between Mooring Lines and Other Vessel Structures.

Consideration shall be given to the detrimental effects of contact between mooring lines and other vessel structures such as anchor bolsters.

### 7.8 SUPPORTING STRUCTURES

Supporting structures such as chain stopper, fairlead and their foundations should have equal or higher design strength than the mooring line. Special attention should be given to the design of supporting structures such that failure of the supporting structure will not result in multiple line failures.

## 8 Mooring hardware

### 8.1 BASIC CONSIDERATIONS

Specifications and conditions of the mooring components should be in accordance with those assumed or required by the mooring analysis.

### 8.2 MOORING LINE COMPONENTS

Requirements for mooring line components such as wire rope, chain, connecting link, and buoy are given in this section. Requirements for synthetic fiber rope can be found in API RP 2SM, *Recommended Practice for Design, Manufacturing, and Maintenance of Synthetic Fiber Ropes for Offshore Mooring* (Reference 2).

#### 8.2.1 Mooring Wire Rope

Mooring wire rope should have no fiber core. Blocking compound of good quality should be used to fill the spaces between the wires. The ends of each rope section should be terminated with resin or zinc poured sockets. Mooring wire ropes and end sockets should meet the material, design, manufacture, and testing requirements specified in certain classification rules such as those provided in Reference 9.

For non-torque-balanced mooring wire ropes, such as six-strand and eight-strand wire ropes, the torque or twisting characteristics of a wire rope should be considered in the design of mooring line configuration and in mooring line handling procedure to ensure proper mooring application and safety of handling crews. Wire rope manufacturers should provide users torque/twist data for the allowable tension range as part of the wire rope basic properties.

Contact of wire rope in the dip or thrash zone may cause excessive wear in the rope/jacket or excessive free bending at the socket. This condition should be avoided for permanent moorings under normal operating environments. This condition may be acceptable under extreme environments.

#### 8.2.2 Mooring Chain

Mooring chain should be manufactured according to one of the following specifications:

- API Spec 2F, *Specification for Mooring Chain* (Reference 10)
- RCS (Recognized Classification Society) Rules for Offshore Mooring Chain (Reference 11)

#### 8.2.3 Connecting Link

Connecting links such as shackles and detachable links should be made of forged or cast material. They should be fully inspected by non-destructive testing (magnetic particle, die penetrant, eddy current, etc.) according to recognized standards. Cast connecting links should also be examined by x-ray or ultrasonic testing to detect internal casting defects. In addition, forgings and castings should satisfy a Charpy V-notch energy requirement of 40 J at –20° C. Kenter connecting links should meet the requirements specified in API Specification 2F (Reference 10). Inspection, mechanical, proof and break testing of other types of connecting links should meet similar requirements or other recognized standards.

#### 8.2.4 Mooring Buoy

Mid-line buoys are typically constructed from steel or synthetic materials. They should be rated for the maximum buoy submergence depth derived for the mooring system intact and 1-line damage analysis. The maximum safe working depth for mid-line buoys should be based on analyses and/or testing using applicable and recognized design and manufacturing codes.

Surface mid-line buoys should be designed to meet the following conditions:

- Buoys should be designed to remain at the surface in all intact and one-line damaged conditions, unless the buoy is rated for the maximum submergence that could occur in any of these conditions.
- Buoys should be designed to remain at the surface in case of one compartment flooding in all intact mooring conditions, if applicable such as for hollow steel buoys, unless the buoy and the internal compartment boundaries are rated to withstand the submergence resulting from the flooding of any compartment.
- Filling of floodable compartments with foam preventing the ingress of water in case of compartment damage may be considered to eliminate the 1 compartment flooded design, provided that the foam is capable of withstanding the hydrostatic pressure at the maximum design submergence of the buoy.
- Certain low-density foams used to fill buoy compartments, may take on water slowly over time. In this case, some percentage flooding of the compartment should be considered in the design of the buoy.
- Buoys should be rated to withstand overtopping by storm waves in the maximum design environment.
- Buoys should be fitted with draft marks and/or other suitable means for monitoring flooding of the buoy.

Any mid-line buoy used to maintain a safe separation between the mooring line and other critical systems, such as pipelines, other mooring legs, etc. shall be designed to maintain adequate clearance in all applicable intact, 1-line damage and 1 buoy compartment flooded conditions, and shall be rated for the maximum buoy submergence in any of these conditions.

For mid-line hollow buoys that are not close to other structures, the consequences of buoy collapse should be investigated, if applicable. For example, if the flooding of a compartment in such a hollow mid-line buoy may result in the buoy submerging below its rated depth, the consequences of a complete collapse of the buoy should be considered and analyzed in the mooring system design. The consequences to be analyzed should include, but may not be limited to: dynamic mooring line and mooring equipment loads due to buoy implosion and rapid descent, impact of rapid changes in vessel position resulting from buoy collapse, recovery of collapsed buoy and mooring line, etc.

The connecting hardware of mid line buoy to the mooring line should be designed for the maximum hydrostatic and dynamic loads using applicable and recognized design and manufacturing codes. Also the design should consider the wear or loosening of the connecting hardware and its locking devices due to buoy motions during the intended service life. Mid-line buoys subject to high motions should be inspected on a regular basis to ensure the integrity of the buoy connections to the mooring lines. If there is concern about the failure of the buoy's connection to the mooring line during the intended service life or period between inspections, the consequences of a loss of the buoy should be investigated in the mooring system design as a damaged condition. In this case the criteria for the damaged condition should be met.

## 8.3 WINCHING EQUIPMENT

Winches should meet the requirements specified in ISO 9089 "Marine Structures—Mobile Offshore Units—Anchor Winches," First Edition, 1989-12-01 (Reference 12) and Section 4.11 of *Code for the Construction and Equipment of Mobile Offshore Drilling Units*, 1989, International Maritime Organization (Reference 13).

## 8.4 MONITORING EQUIPMENT

### 8.4.1 Line Tension

Moored floating units should be equipped with a calibrated system for measuring mooring line tensions if the operation requires mooring line adjustment, and line tensions should be continuously displayed at each winch. For units that do not require a tension measurement device, a device for detecting mooring failure should be considered.

For units with thrusters that are intended for mooring line tension reduction, a means of indicating line tension and/or vessel offset should be provided. This means should be suitably redundant to cover the single failure requirement.

### 8.4.2 Line Payout

Moored floating units should be equipped with a system for measuring mooring line payout if the operation requires mooring line adjustment.

### 8.4.3 Vessel Position

Moored floating units should be equipped with a system for monitoring the position of the vessel if the operation requires restraining the vessel offset. If available, a semi-rigid link to a fixed object (for example, link bridge from tender to platform) may be used to monitor the floating unit's position. For MODU operations, a position system should be available to provide the unit's bearing and distance from the wellhead or point of riser attachment. For units with automatic thruster assist system, the measurement of position should be by at least two different means. For units with manual thruster assist system, the measurement of position should be by at least one means.

#### 8.4.4 Vessel Heading

Floating units with a single point mooring should be equipped with a device for measuring heading. If the headings are to be controlled, at least two different heading measurements are required. If the heading control is automatic, the accuracy and frequency of both measurements should be adequate to meet automatic control requirements.

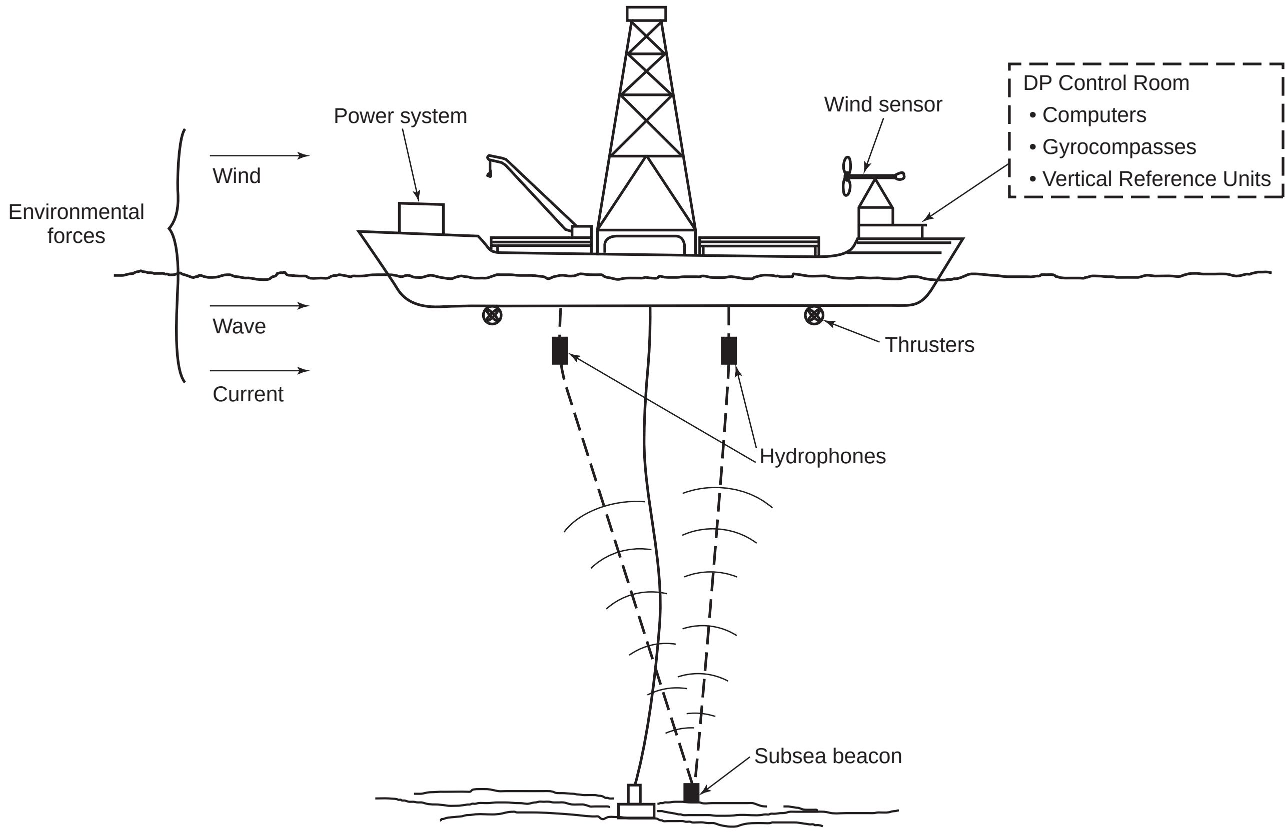



Figure 14—Major Elements of a Dynamic Positioning System

# 9 Dynamic Positioning

## 9.1 BASIC CONSIDERATIONS

Dynamic Positioning (DP) is a technique of automatically maintaining the position of a floating vessel within a specified tolerance by controlling onboard thrusters which generate thrust vectors to counter the wind, wave and current forces. As shown in Figure 14, the major elements of a DP system include:

- Power system: Prime movers and auxiliary equipment, generators, switchboards, cabling, etc.
- Thruster system: Thrusters and auxiliary equipment, including main propellers and rudders, associated cabling and thruster control equipment, etc.
- Control system: The DP computers and associated software, including position references, vessel sensors, operator interface, power management, etc.

A more detailed description of DP system can be found in Reference 14.

## 9.2 DESIGN AND ANALYSIS

### 9.2.1 Basic Design Philosophy

A DP system should be able to keep a vessel in position within certain excursion limits under the design environment. Since the consequences of losing station can be serious, DP systems should be designed to have high reliability and a certain amount of built-in redundancy.

### 9.2.2 Failure Mode and Effect Analysis.

Failure modes and effects analysis (FMEA) should be conducted for floating vessels with a DP system. Failure modes to be considered in the FMEA should include the following:

- The sudden loss of major items of equipment
- The sudden or sequential loss of several items of equipment with a common link.
- Control and monitoring instabilities and failures, and methods of detection and isolation
- Faults that can be hidden until another fault occurs

DP systems should be designed so that, as far as is reasonably possible, there are no common single-point failures. The DP system FMEA should be proved in sea trials, as far as is reasonably practicable, to demonstrate the effects of the various failure modes and to ensure that both equipment and procedures are in place to safely cope with failures.

### 9.2.3 Guidelines for Design, Test, and Maintenance

Detailed guidelines for design, test, and maintenance of DP systems can be found in the IMO document MSC Circular 645, *Guidelines for vessels with Dynamic Positioning Systems* (Reference 15). It defines vessel redundancy into three equipment classes, where Equipment class 1 vessels have the least redundancy, and vessels complying with Equipment class 3 have the most redundancy. In this context 'equipment' refers to all the equipment (power, control, and references), together with its location/layout on the vessel, that goes to define the degree of redundancy. Equipment Class definitions are:

- **Class 1:** Loss of position may occur in the event of a single fault.
- **Class 2:** Loss of position should not occur from a single fault of an active component or system such as generators, thrusters, switchboards, remote controlled valves etc. Static components such as cables, pipes, manual valves etc. should be adequately protected against accidental damage.
- **Class 3:** Loss of position should not occur from any single failure including a completely burnt fire subdivision or flooded watertight compartment.

Using these classifications and the results obtained from the FMEA, it is possible to allocate the vessel with an equipment class notation. Selection of a DP vessel with its inherent redundancy or class should be based on a risk analysis for the particular type of DP operation. The risk analysis should take into account the risks involved with specific operations such as drilling, diving, flotel services, heavy lifting, pipe laying, floating production, shuttle tanker, etc. The particular risk analysis is likely to take into account some of the following:

- The time to reach a safe situation, or recover from the immediate danger,
- Speed of loss of position (drift-off, drive-off, or a large excursion),
- Environmental limitations,
- Operational procedures,
- Human factors etc.

The risk analysis can be general and cover different working situations and types of work. However generic assumptions and principles should be considered for each project, location, and procedure to ensure the analysis is valid and/or changes are made to maintain its validity and applicability.

### 9.2.4 Determination of Station-Keeping Capability

#### 9.2.4.1 Basic Considerations

A holding capability analysis should be performed to determine whether a DP system can maintain the position of a floating vessel within an acceptable watch circle under the operating environment. This analysis should be performed for new designs as well as for individual operations. Two methods can be used to analyze the holding capability of a DP system. A time domain system dynamic analysis is normally performed for new system designs and critical operations, especially those in shallow water. For routine operations in deepwater, a simplified method addressing only the mean environmental forces can be used.

#### 9.2.4.2 Environmental Loads and Vessel Motions

A DP system counters steady loads and damps out low frequency motions. Wave frequency motions are not affected by the DP system and therefore can be excluded from the holding capacity analysis. However, they should be included in the determination of maximum offset.

#### 9.2.4.3 Available Thrust

Guidelines for determining available thrust are provided in Appendix F, which deals with typical propulsion devices and installation scenarios for DP vessels supporting offshore operations.

#### 9.2.4.4 System Dynamic Analysis

System dynamic analysis for a DP vessel is similar to that for a vessel with a thruster assisted mooring (Section 5.9.4.2). The major difference is that the mooring stiffness is not included in the analysis.

#### 9.2.4.5 Simplified Method

In this approach, we assumed the DP holding capability is satisfactory if the DP capability is greater than the mean environmental load. The procedure is as follows:

1. Establish an operating environment and a vessel heading relative to the operating environment.
2. Calculate the mean surge and sway forces and the yaw moment due to wind, waves, and current.
3. Determine the required output of each individual thruster based on the DP system algorithm for thrust allocation.
4. Determine the available thrust for each thruster.
5. Calculate the DP capability according to IMCA M 140 *Specification for DP Capability Plots*, August 1997 (Reference 16).
6. Repeat the previous steps for different headings, operating environments, and thruster failure cases.
7. Produce plots of calculated DP holding capability versus vessel heading for different environments and thruster failure scenarios. Examples of DP holding capability rosettes for a semi-submersible and a drill ship are presented in Figure 15.

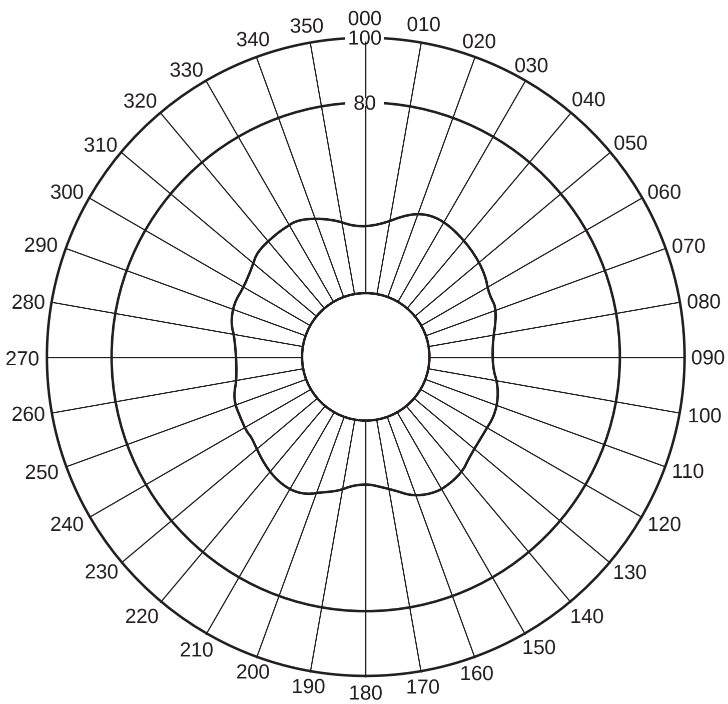
### 9.3 OPERATING PERSONNEL

Specially trained personnel are required to operate the DP system with its sophisticated electronic equipment. Guidelines for training DP operators can be found in the IMO document MSC Cir. 738 "The Training and Experience of Key DP Personnel" (Reference 17).

## 10 References

1. API RP 2I, *Recommended Practice for In-Service Inspection of Mooring Hardware for Floating Drilling Units*, 2nd Edition, Nov. 1996.
2. API RP 2SM, *Recommended Practice for Design, Manufacturing, and Maintenance of Synthetic Fiber Ropes for Offshore Mooring*, 1st Edition, March 2001.
3. API RP 17A/ISO 13628-1, *Recommended Practice for Design and Operation of Subsea Production Systems*, 3rd Edition, September 2002.
4. API RP 17B, *Recommended Practice for Flexible Pipe*, 3rd Edition, March 2002.
5. API RP 16Q, *Recommended Practice for Design, Selection, Operation, and Maintenance of Marine Drilling Riser Systems*, 1st Edition, August 2001.
6. API RP 2RD, *Design of Risers for Floating production (FPSs) and Tension Leg Platforms (TLPs)*, 1st Edition, June 1998.
7. API RP 2T, *Planning, Designing, and Constructing Tension Leg Platforms*, 2nd Edition, August 1997.
8. IMCA Report No. GM-2096-0600-2561 "Guidance on Thruster Assisted Station Keeping by FPSO and Similar Turret Moored Vessels."
9. DNV Certification Notes No. 2.5 "Certification of Offshore Wire Ropes," Det norske Veritas, 1995.
10. API Spec 2F, *Specification for Mooring Chain*, 6th Edition, June 1997.
11. RCS (Recognized Classification Society) *Rules for Offshore Mooring Chain*
12. ISO 9089, *Marine Structures—Mobile Offshore Units - Anchor Winches*, First Edition, 1989-12-01.
13. IMO, *Code for the Construction and Equipment of Mobile Offshore Drilling Units*, 1989, International Maritime Organization.
14. OPL *Oilfield Seamanship Series Volume 9, Dynamic Positioning*, ISBN 1 902157 02 8.
15. IMO document MSC Circular 645 "Guidelines for vessels with Dynamic Positioning Systems."
16. IMCA M 140, *Specification for DP Capability Plots*, August 1997.
17. IMO document MSC Cir. 738 "The Training and Experience of Key DP Personnel."
18. Memo to API RP 2SK Work Group, "Studlink and Studless Fatigue Curves for Mooring Lines," ExxonMobil Upstream Research Company, March 2003.
19. Joint Industry Study Report, "Fatigue Testing of Large Diameter Anchor Chains," Noble Denton & Associates, Inc., August 19, 1994.
20. Joint Industry Study Report, "Corrosion Fatigue Testing of 76mm Grade R3 & R4 Studless Mooring Chain," Noble Denton & Associates, Inc., May 15, 2002.
21. Memo to API RP 2SK Work Group, "T-N curve for Studless Chain Proposed in API RP 2SK draft," Shell, April 2004.

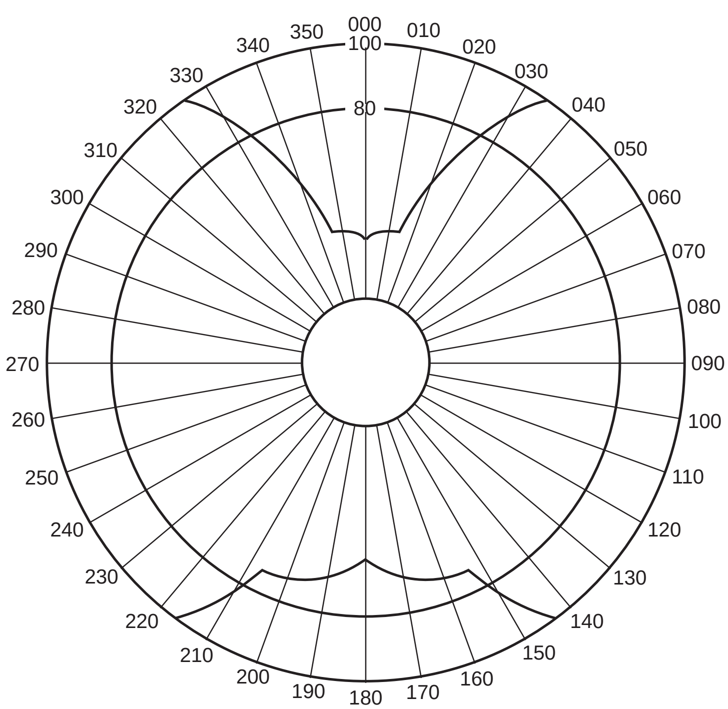
**SEMISUBMERSIBLE**




| | | |
|---|---|---|
| Number of thrusters available: | | 8 |
| Vessel drilling draft: | | 70.0 ft |
| Wind | 54.0 knots from | 000 deg |
| Current | 0.90 knots from | 000 deg |
| Waves | 21.0 ft from | 000 deg |
| Riser restoring force | | 0 kips |

Percent of available thruster loading versus vessel heading

**DRILL SHIP**




| | | |
|---|---|---|
| Number of thrusters available: | | F-6 A-6 M-6 |
| Vessel drilling draft: | | 22.0 ft |
| Wind | 54.0 knots from | 000 deg |
| Current | 0.90 knots from | 000 deg |
| Waves | 21.0 ft from | 000 deg |
| Riser restoring force | | 0 kips |

Percent of available thruster loading versus vessel heading

Figure 15—Dynamic Positioning Capacity Rosettes

# APPENDIX A—INTRODUCTION TO MOORING COMPONENTS

## A.1 Mooring Line

### A.1.1 CLASSIFICATION

Mooring lines for moored vessels may be made up of chain, wire rope, synthetic rope, or a combination of them. There are many possible combinations of line type, size, and location and size of clump weights or buoys that can be used to achieve given mooring performance requirements. Following are typical systems used by the industry:

1. **All Wire Rope System**

   Because wire rope is much lighter than chain, wire rope provides a greater restoring force for a given pretension. This becomes increasingly important as water depth increases. However, to prevent anchor uplift with an all wire system, much longer line length is required. A disadvantage of an all wire rope mooring system is wear due to long term abrasion where it contacts the seabed. For these reasons, all wire rope mooring systems are seldom used for mobile or permanent moorings.

2. **All Chain System**

   Chain has shown durability in offshore operations. It has better resistance to bottom abrasion and contributes significantly to anchor holding capacity. However, in deep water an all chain system imposes an increasing weight penalty on the vessel's load carrying capacity by its own self weight and high initial tension requirements.

3. **Combination System**

   In this system, a mooring line may be a combination of chain, wire rope, and fiber rope. In a chain/wire rope mooring, a length of chain is typically connected to the anchor. This provides good abrasion resistance where the mooring line contacts the seabed and its weight contributes to anchor holding capacity. The choice of chain or wire rope at the vessel end and the type of termination also depends on the requirements for adjustment of line tensions during operations. By proper selection of the lengths of wire rope and chain, a combination system offers the advantages of reduced pretension requirements with higher restoring force, improved anchor holding capacity, and good resistance to bottom abrasion. These advantages make combination system attractive for deep water mooring.

An alternative to the above system is the wire rope/chain/wire rope combination system where wire rope segments are connected to both the vessel and the anchor. A length of chain is used in the dip zone where the mooring line is in dynamic contact with the seafloor. This minimizes the amount of chain which is costly and difficult to deploy at deepwater sites. A chain/wire rope/chain combination system is sometimes used. For example, a wire rope is often inserted in an all chain mooring line on a MUDU to increase the water depth capability of the drilling vessel.

When fiber rope is used, it is normally placed in the catenary portion of the mooring line to avoid contact with the seafloor and the winching equipment. Chain or wire rope segments can be used to connect the fiber rope to both the vessel and the anchor.

### A.1.2 CHAIN

The choice of material and fabrication of large diameter chain for a moored vessel requires careful evaluation. It is desirable to have chain used for this application manufactured in continuous lengths for each mooring leg. This eliminates the need for chain connection links and the associated problems with fatigue. Otherwise, connecting links with sufficient fatigue life should be used.

Chain can be obtained in several grades with Grade 4 or R4 being the highest strength. Oil Rig Quality (ORQ) or R3 chain has been sold in large quantities over the years and has generally performed well. Grade 2 chain is not recommended for major mooring operations.

A grade of chain somewhere between R3 and R4, for example R3S, is preferred by some designers since it is easier to manufacture than R4 chain. In any case it is recommended that considerable care is taken in establishing correct chemical composition of the bar stock, manufacturing techniques which incorporate precise quality control and finally, comprehensive testing of samples of the final manufactured product.

Recently a new grade, R4S, chain was introduced by some chain manufacturers. The R4S studlink chain is stronger than the R4 chain, with the increase in strength achieved by using a larger diameter bar stock within the overall dimensions of a specific chain link size and by improving the ultimate tensile strength of the base metal. The outside length and width of the link are virtually unchanged, making it possible to use existing chain windlasses and fairleads, thus enabling a significant increase in mooring chain breaking strength with minimal changes in rig mooring equipment. The disadvantage of this chain is its relatively short service experience.

Stud chain (Figure A.1) has been used by the offshore industry for more than 30 years. For R3 grade, studs are often welded on the side opposite to the flash weld. Studs are normally not welded for higher grades. The industry has experienced significant problems associated with studs, including loose stud, fatigue crack and fracture at the stud weld or stud footprint. In the 1990s a new chain, studless chain (Figure A.1), gained wide acceptance in the application of permanent moorings. Studless chain is about 10% lighter than the stud

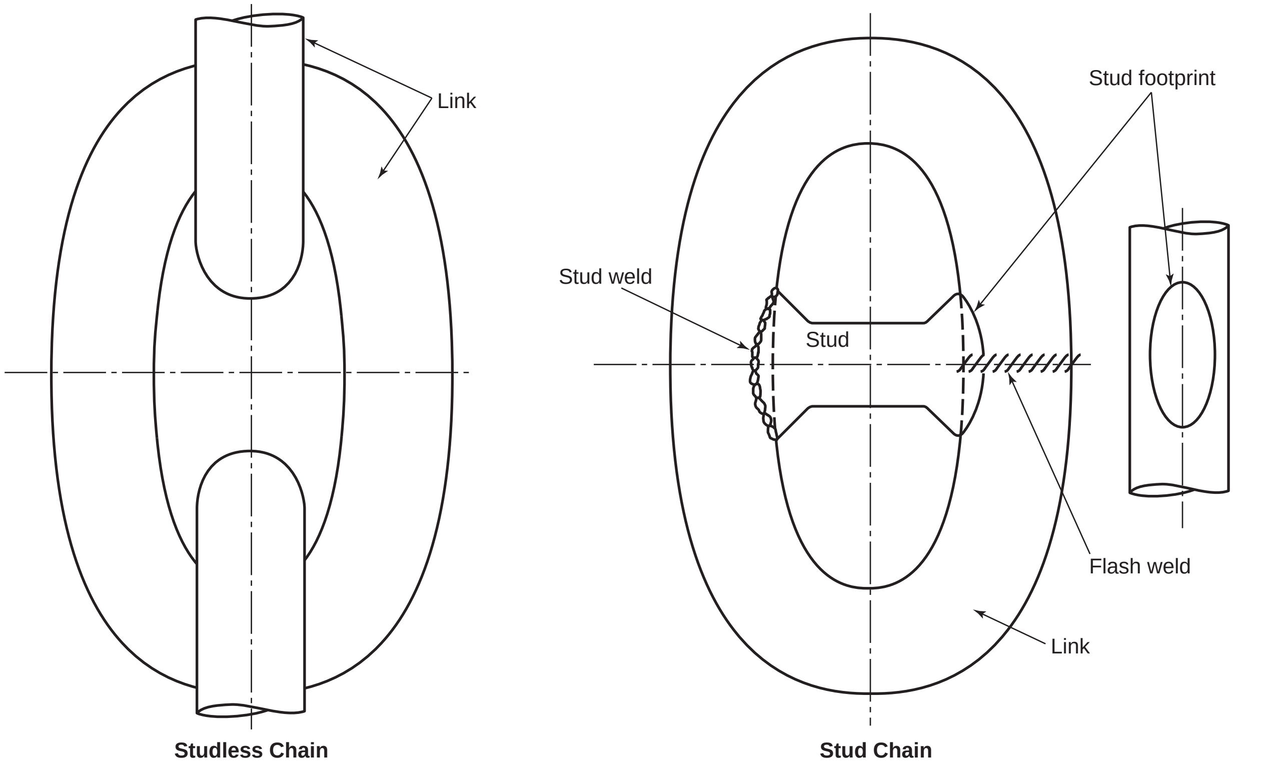



Figure A.1—Stud and Studless Chain

chain, but has the same breaking strength. Limited fatigue testing indicates that studless chain may have lower fatigue life, but the industry's experience with studless chain has been favorable so far. In fact most of the permanent moorings recently installed or designed use studless chain instead of stud chain.

### A.1.3 WIRE MOORING LINE

The wire rope sections of the moorings can be of various constructions as shown in Figure A.2. The wire rope construction type includes a number of strands wound in the same rotational direction around a center core to form the rope. The number of strands and wires in each strand (i.e., 6x36, 6x42, 6x54), core design and lay of strands are governed by required strength and bending fatigue considerations for the rope. This construction generates torque as tension increases.

The spin-resistant strand type constructions (spiral strand and multi-strand) are attractive for use with permanent moorings since they do not generate significant torque with tension changes. Both constructions use layers of wires (or bundles of wires) wound in opposing directions to obtain the spin resistance characteristics.

For corrosion resistance in permanent moorings, typically a polyethylene or polyurethane jacketing is employed. The jacketing material should be a high density type. Also all wires should be galvanized. Zinc filler wires are sometimes incorporated to provide additional corrosion protection. A filler material is used to block the inside spaces between the wires to minimize the spread of corrosion with ingress of salt water.

Life expectancy of different types of wire rope is provided below:

- Galvanized 6- strand 6-8 years
- Galvanized unjacketed spiral strand 10-12 years
- Galvanized unjacketed spiral strand with zinc filler wires 15-17 years
- Galvanized jacketed spiral strand 20-25 years
- Galvanized jacketed spiral strand with zinc filler wires 30-35 years

The ends of each mooring line section should be terminated with sockets. A resin material is preferred over zinc for pouring the sockets. For permanent moorings, the sockets are typically provided with flex relieving boots (bend stiffener) joined to the socket in a manner to seal out the ingress of water and limit free benching fatigue. Zinc anodes are attached to protect the socket from corrosion, and isolation washers are used to electrically separate the two connected segments (Figure A.3).

Careful quality control and testing should be exercised prior to and during the fabrication of the rope to ensure that the rope meets design specifications and the final product produces guaranteed minimum break strength as specified.

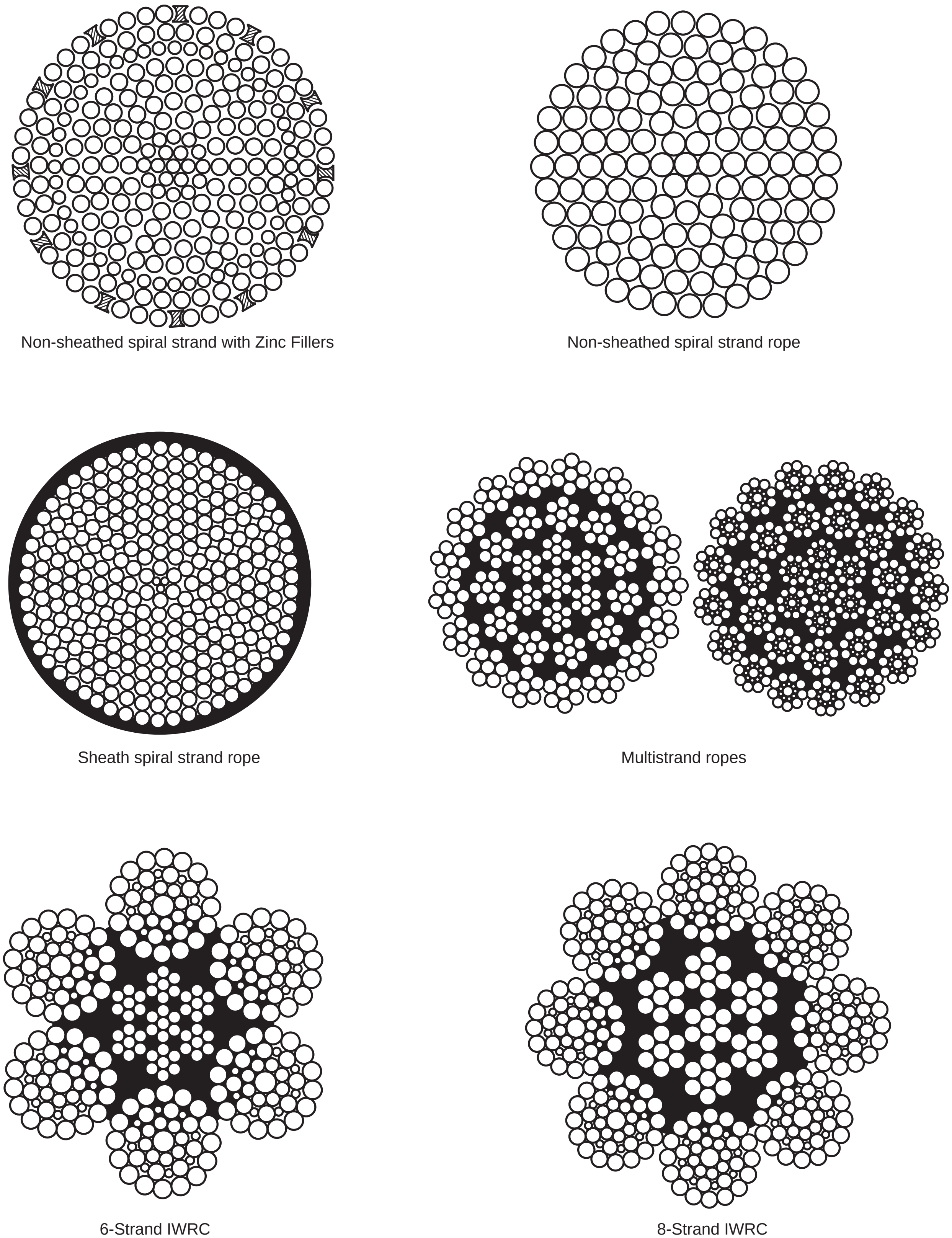



Figure A.2—Typical Wire Rope Constructions

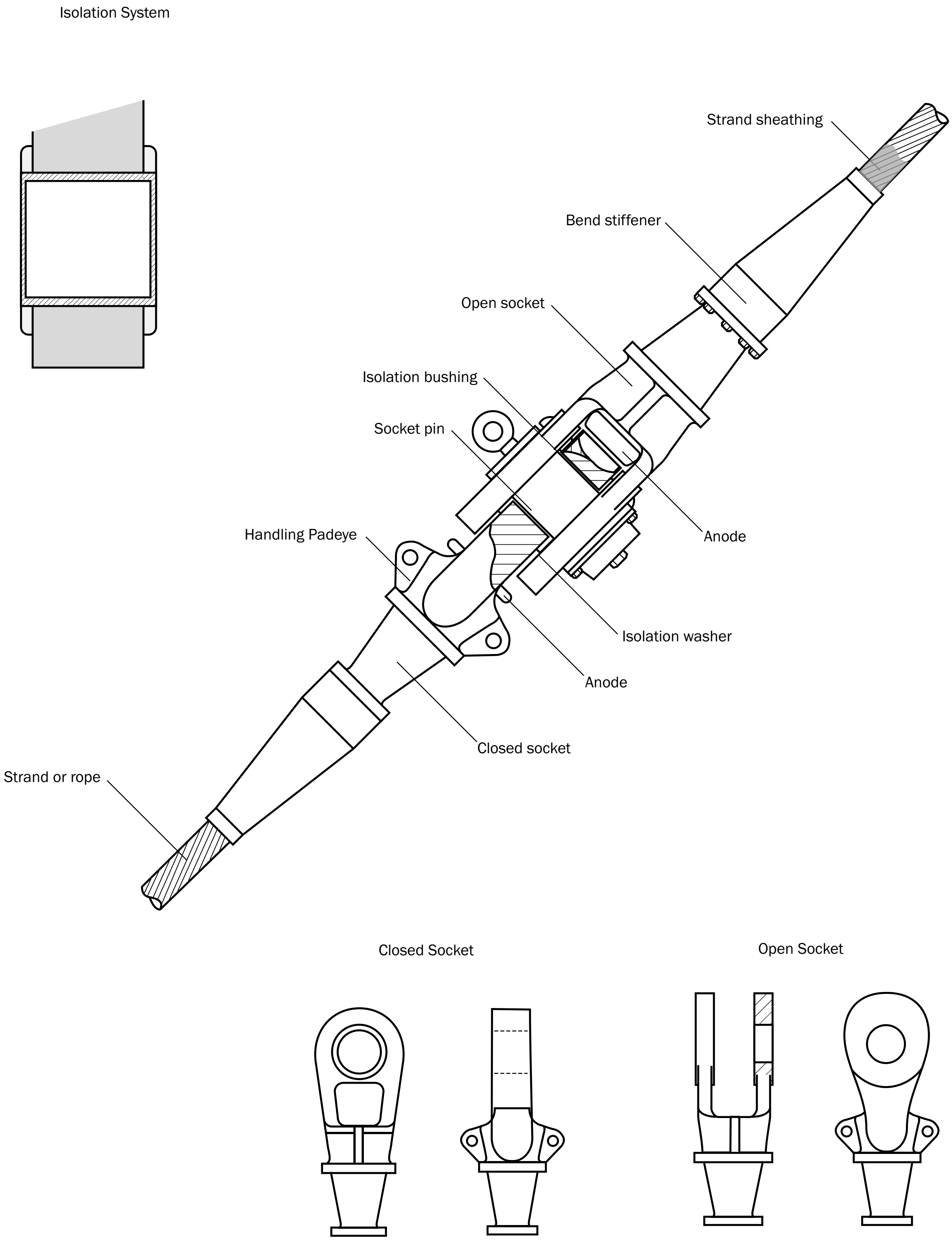



Figure A.3—Wire Rope Socket for Permanent Mooring

### A.1.4 SYNTHETIC FIBER ROPE

A short introduction to synthetic fiber rope is given in this section. Detailed design guidance can be found in API RP 2SM, *Recommended Practice for Design, Manufacturing, and Maintenance of Synthetic Fiber Ropes for Offshore Mooring*.

Fiber ropes may be used as segments in steel catenary systems, or in taut leg systems. The subtle differences from that of the steel wire rope/chain mooring systems include the non-linear stiffness, minimum tension requirements, location of fiber rope segment to be away from fairlead and seafloor, creep phenomenon, and different handling procedures.

#### A.1.4.1 Fiber Type

The fibers currently being considered for use in permanent or temporary deepwater moorings are polyester (polyethylene terephthalate), aramid (aromatic polyamide), HMPE (high modulus polyethylene), and nylon (polyamide). Currently, polyester is considered to be a good candidate for the offshore mooring application due to its low cost, low stiffness which induces less dynamic tension, good resistance to axial compression fatigue, good fatigue properties, good strength to weight ratio, and good creep resistance. Polyester rope is the only fiber rope that has been installed in permanent moorings. Other fibers such as HMPE and aramid may be more suitable for applications where a smaller rope diameter is required (e.g., for frequent handling) or for ultra-deep water mooring applications. Nylon rope has been used for hawsers in CALM systems where high elasticity is an important property. These hawsers can be inspected frequently and replaced. Also in shallow water locations, a length of nylon rope is often inserted in the mooring line to absorb the energy from vessel dynamics.

#### A.1.4.2 Rope Construction

Note: Figure A.4 was intentionally omittted.

There are many different rope construction types. Two types of rope construction, “wire-rope construction” (WRC, as used in steel wires) and “parallel strand”, are most commonly used. Jacketing should be used on fiber ropes where external abrasion is expected to occur while in service and during installation and recovery. Certain rope constructions must have a protective jacket to keep the strength core strands together, such as parallel strands. A jacket may also provide some protection to soil ingress, marine growth, and fish bite. A sand filter is often placed between the jacket and load bearing fibers to give additional protection against soil ingress.

Fiber ropes are typically constructed to be torque balanced. A torque matched rope is sometimes used when it is connected with a mooring component that is not torque free, such as a six-strand wire rope.

#### A.1.4.3 Rope Properties

Unlike steel components, fiber rope stiffness increases with mean load, and decreases with cyclic load range and with load relaxation over time. After the rope has been tensioned to allow bedding-in, and cyclic load and relaxation has occurred for some time, the stiffness of fiber ropes tends toward a linear function of mean load and load range.

Little data is currently available on creep in ropes. A general indication of the effects of creep is given by yarn data. Continuing elongation under load may lead to a need to retension of mooring lines. HMPE fibers show significant creep, which can lead to creep rupture failure. Creep of polyester and aramid fibers is much lower than HMPE.

Factors which may limit the life of synthetic fiber ropes for deepwater mooring and which should be checked include hydrolysis, heating and internal abrasion, tension-tension fatigue, axial compressive fatigue, and creep rupture.

### A.1.5 CLUMP WEIGHT

Clump weights are sometimes incorporated in mooring legs to improve performance or reduce cost. By providing a concentrated weight to the mooring leg at a point close to the seabed, a clump weight can be used to replace a portion of chain, and increase the restoring force of a mooring leg. Using clump weights in a mooring design requires careful consideration of potentially adverse effects, such as increased use of connecting hardware and installation complexity, undesirable dynamic response of the mooring line, and embedment of the clump weight in the seabed. In some mooring designs, heavy chain segments are used in place of clump weight for easy installation and lower cost.

### A.1.6 SPRING BUOY

Spring buoys are surface or subsurface buoys that are connected to a mooring line. Benefits of spring buoys are:

- Reduced weight of mooring lines that must be supported by the vessel hull; this is particularly advantageous to floating vessels moored in deep water.
- Reduced vessel offset for a given line size and pretension.
- Increased vertical clearance between the mooring line and the equipment below.

Adverse effects of spring buoys are:

- Increased use of connecting hardware and installation complexity.
- Potential increased design loads on the mooring lines due to dynamic response of the buoy in heavy seas.

Spring buoys used with permanent moorings are typically constructed from steel or a combination of synthetic material surrounding a steel structure. A high density foam material (glass spheres encased in a high density foam) has been successfully used to provide buoyancy for deepwater drilling and production riser and mooring operations. Steel buoys have been found to provide a cost competitive solution. The buoys can be built either spherical in shape, using unstiffened dished ends welded together, or with ring stiffened cylindrical bodies and ends. Buoys can be placed in line with the mooring (with a strength member through the buoy) or attached separately to the mooring through a tri-plate as shown in Figure A.5. When using the in-line buoy approach, care must be taken to allow for rotation in the end connections.

The buoys should be so designed to have adequate strength for maximum operating depth. During fabrication of the buoys all welding should be tested with appropriate non-destructive testing. Also, corrosion protection should be adequately provided.

### A.1.7 CONNECTING HARDWARE

Connecting hardware such as shackles, swivels, fishplates and detachable links (Figure A.6) are used to connect together the main mooring line components. Inspection and replacement of connecting hardware in a permanent mooring are difficult, therefore fatigue life and corrosion protection become important considerations. The design of all connecting hardware to be used in permanent mooring lines should be thoroughly evaluated to ensure that stress concentration factors are correctly identified, and that fatigue life and corrosion protection is adequate. Manufacturing of connecting hardware should be subject to an appropriate level of quality assurance.

Connecting links such as Kenter and Baldt links are often used in mobile moorings. They can pass through chain fairleads and windlasses and can be periodically inspected and replaced.

Recently subsea connectors were developed to allow connect and disconnect of two mooring line segments under water. These connectors typically have a male part and a female receptacle, which are attached to two different line segments to be connected. The under water operation of connecting or disconnecting the male and the female parts are performed by an ROV (Figure A.7).

## A.2 Winching Equipment

The type and design of winching equipment required in a particular mooring system depends on the type of mooring line to be handled, and whether or not the floating vessel itself must initially tension the mooring lines or test load anchors. A floating vessel often has the means of adjusting mooring line tension, retensioning after anchor drag, and disconnecting individual mooring lines. Besides, a floating vessel is often used for combined drilling and production. This will require the capability for finite surface positioning for maneuvering the risers. This positioning can be achieved by paying-out and heaving-in mooring lines.

### A.2.1 WINDLASS

The most common method of handling and tensioning chain is through the use of a windlass. The windlass consists of a slotted “wildcat” which is driven by a power source through a gear-reduction system. As the wildcat rotates, the chain meshes with the wildcat, is drawn over the top of the wildcat, and lowered into the chain locker. Once the chain is hauled in and tensioned, a chain stopper or brake is engaged to hold the chain. Windlass has proven to be a fast and reliable method for handling and tensioning chain (Figure A.8).

### A.2.2 CHAIN JACK

Chain jack is a device which reciprocates linearly to haul-in and tension chain. Usually powered by one or more hydraulic cylinders, chain jack engages the chain, pulls in a short amount of the chain, engages a stop, retracts, and repeats the process. Although chain jack can be a powerful means for tensioning chain, it is very slow and is recommended for applications not requiring frequent line manipulation (Figure A.9).

### A.2.3 DRUM-TYPE WINCH

Conventional drum-type winch is the most common method used for handling wire rope. Operation of drum-type winch is fast and smooth. Drum-type winch consists of a large drum on which the wire rope is wrapped. The base of the drum is often fitted with special grooves sized specifically to the size of wire rope being handled. The groves control the positioning of the bottom layer of wire rope on the drum. For subsequent layers of wire rope, an external guidance mechanism such as a level-wind is often used to control positioning of the wire rope on the drum. The tensioning capacity of the winch is a function of number of wraps on the drum (Figure A.10).

Drum-type winch can be a cumbersome method of handling wire rope for deepwater or high strength mooring systems. As the requirement for line sizes and lengths increases, the size of the winch can become impractical. In addition, when wire rope is under tension at an outer layer on the drum, spreading of preceding layers can occur causing damage to the wire rope.

### A.2.4 LINEAR WINCH

Linear winch is similar in principal to chain jack. Two sets of grippers, one stationary and one translating, are used to haul-in and tension the wire rope. Linear winch is available in a single-acting form in which case the wire rope moves intermittently as the gripper is retracted to begin another stroke, and in a continuous double-acting form in which case two translating grippers are used alternately for continuous smooth motion of the wire rope. Linear winch is most applicable in a permanent application when high tension and

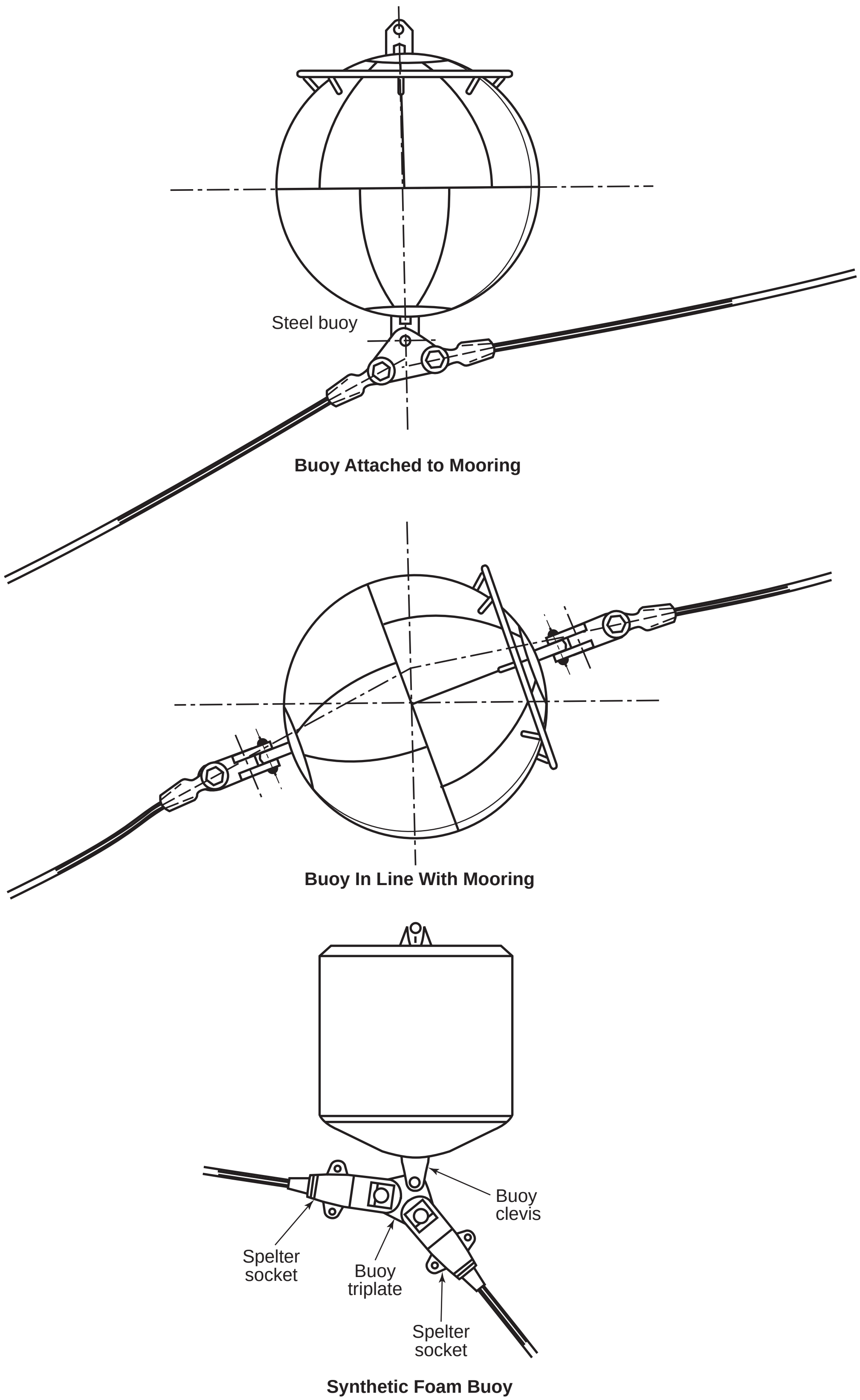



Figure A.5—Submersible Buoy Configurations

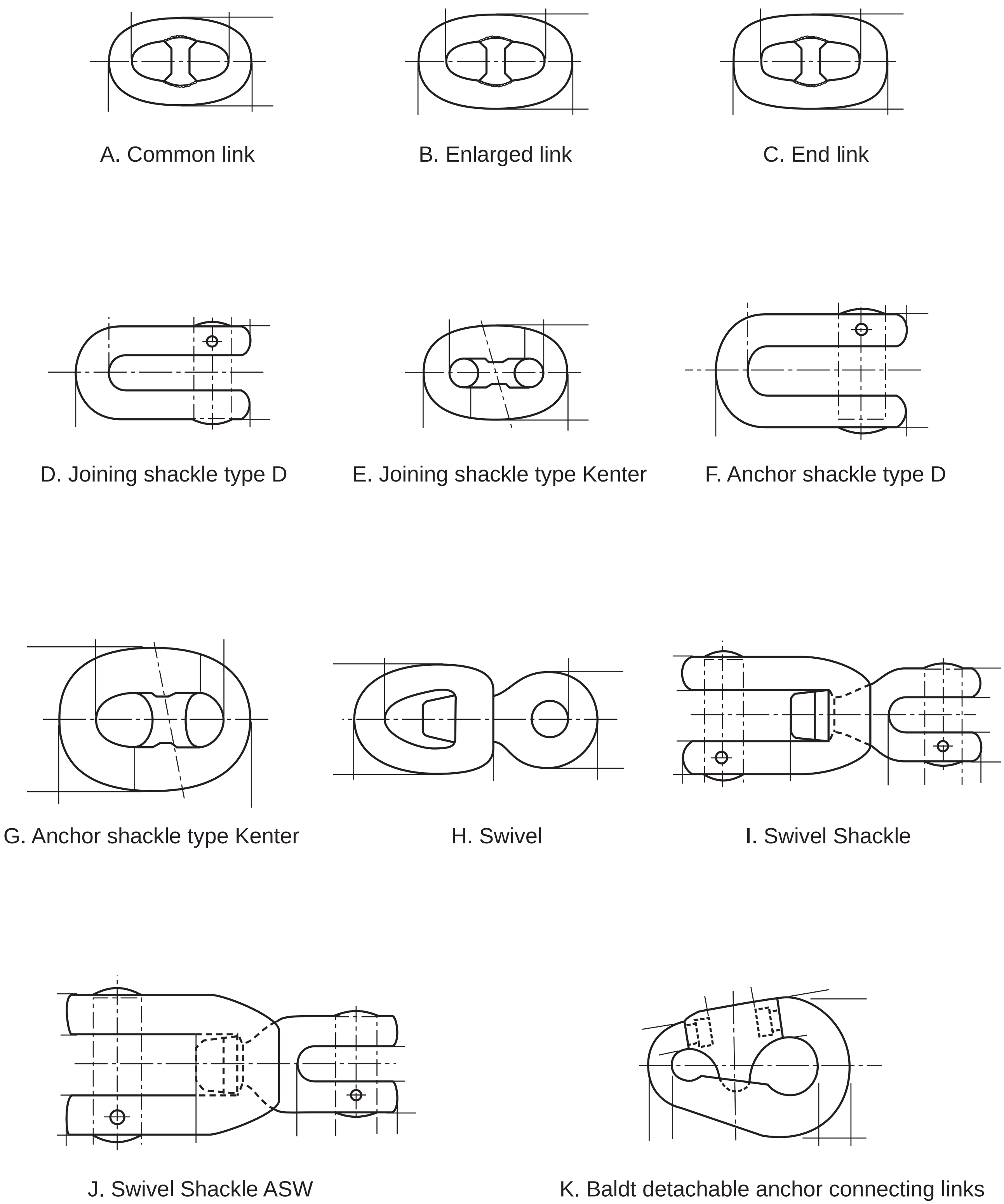
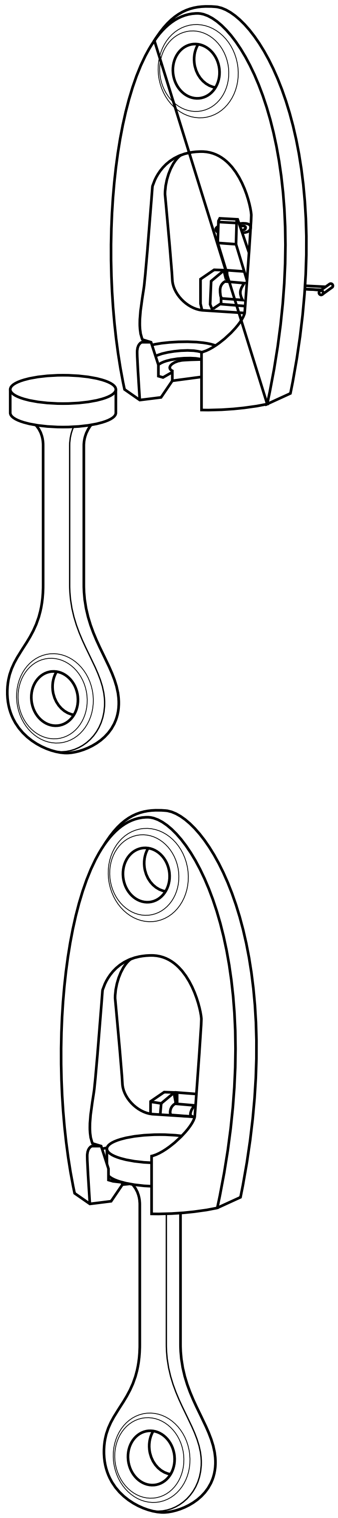
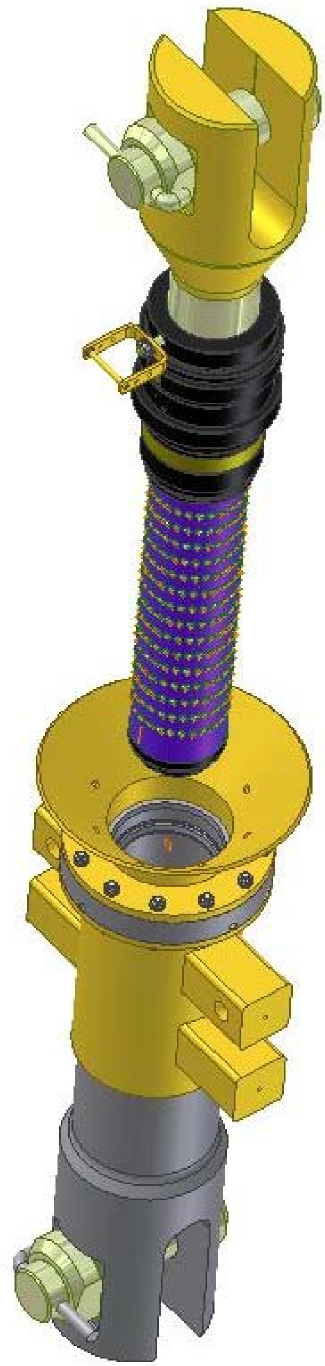



Figure A.6—Typical Mooring Connectors



**Delmar Connector**



**Ballgrab Connector**

Figure A.7—Typical Subsea Mooring Connectors

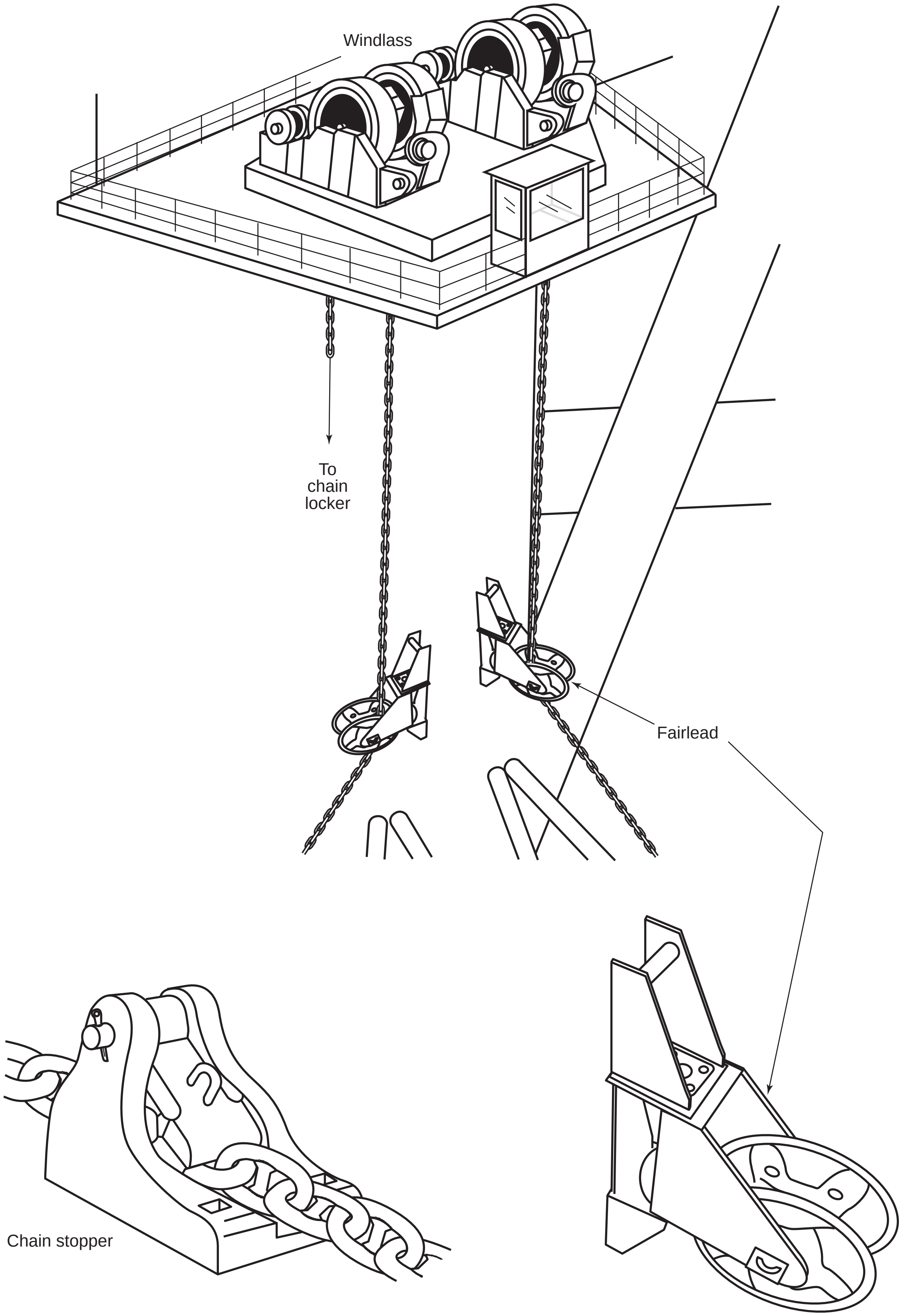



Figure A.8—Winching Equipment for Chain

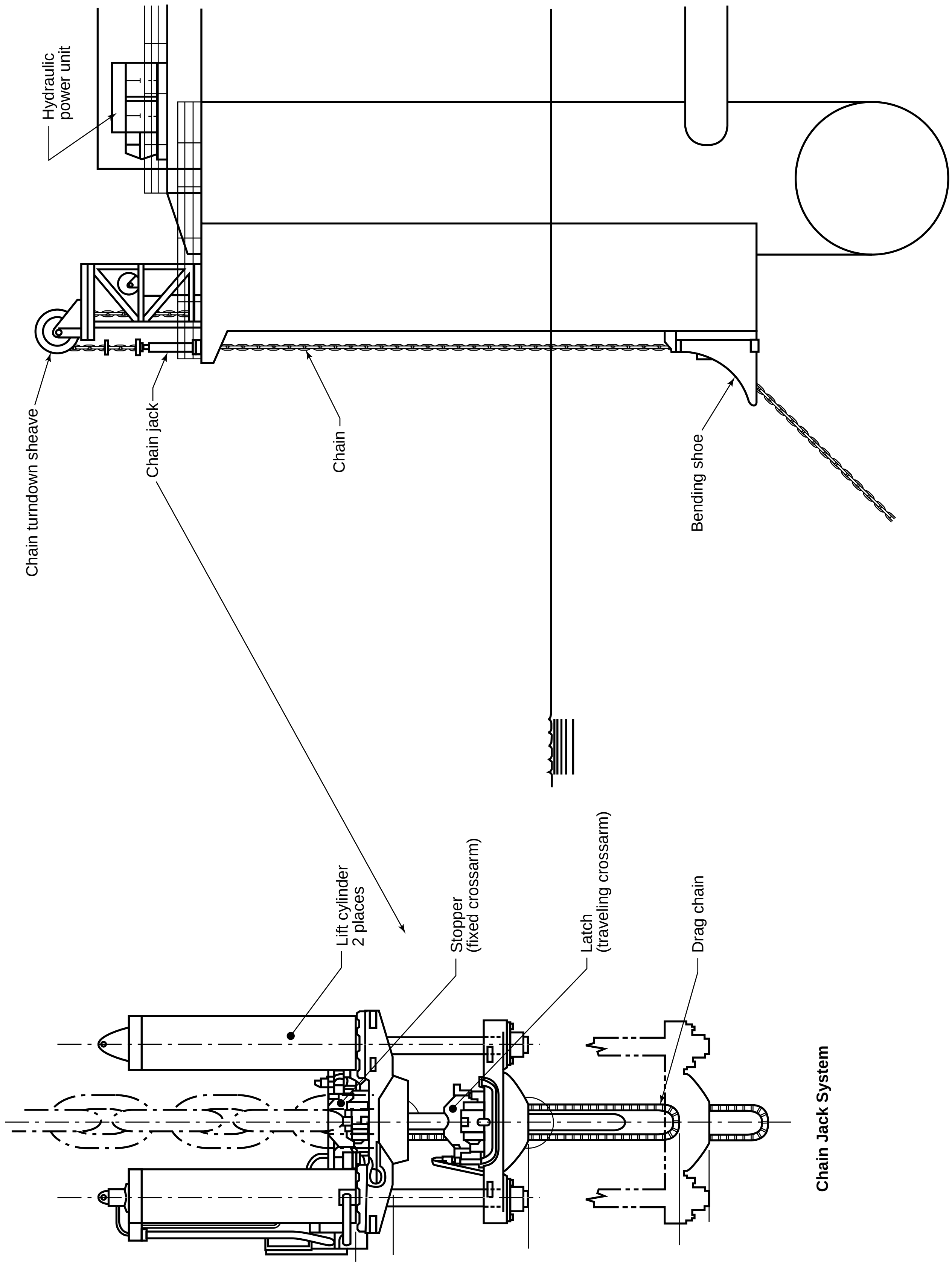



Figure A.9—Chain Jack and Chain Bending Shoe Fairlead

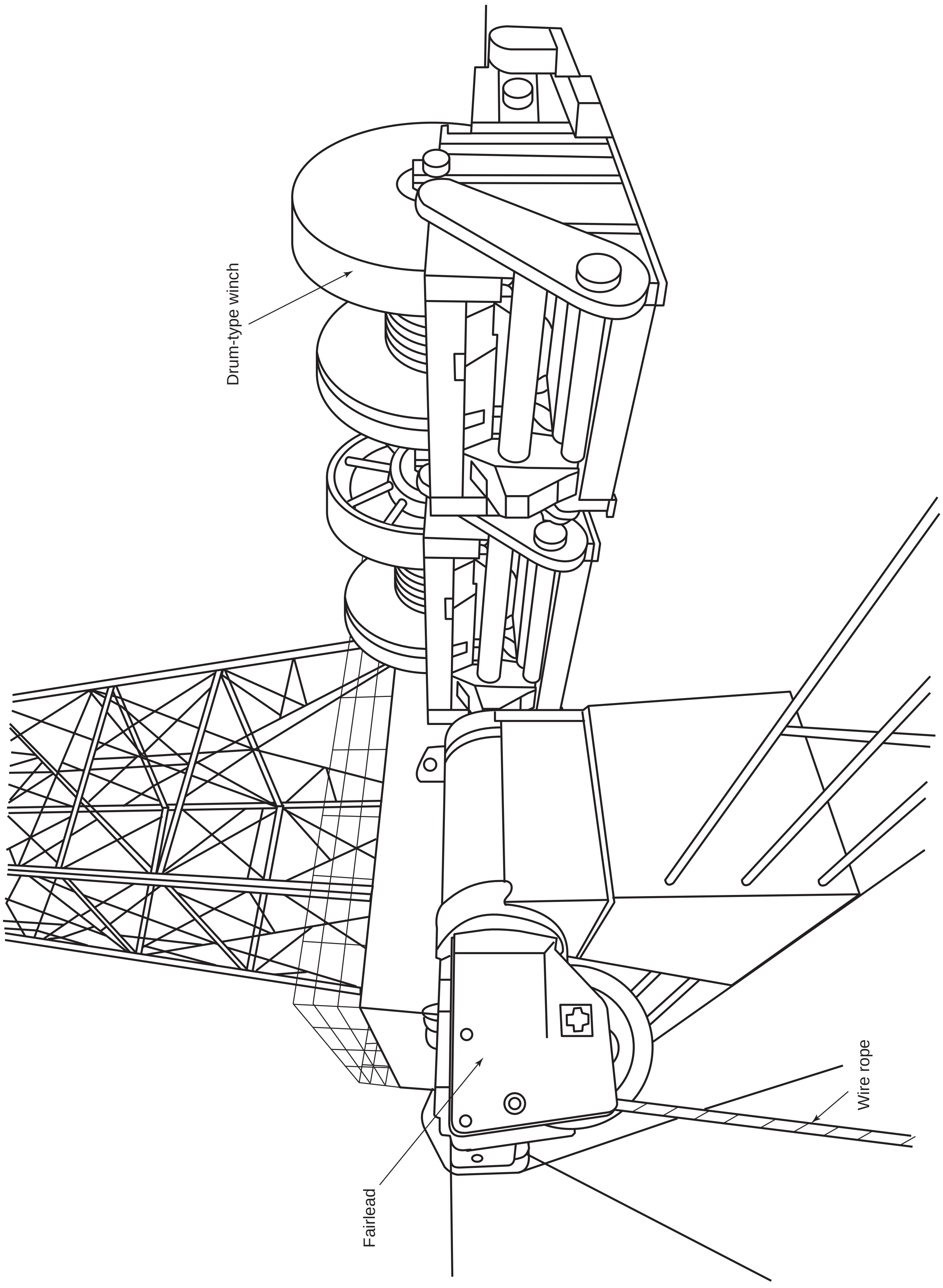



Figure A.10—Drum Type Winch and Fairlead for Wire Rope large-diameter wire rope is required. A take-up reel is necessary in this case to coil the wire rope after it passes through the linear winch. A winching system using linear winches is illustrated in Figure A.11.

### A.2.5 TRACTION WINCH

Traction winch has been developed for high tension mooring applications as well as for handling combination mooring systems. It consists of two closely spaced parallel mounted powered drums, which are typically grooved. The wire rope makes several wraps (typically 6 to 8) around the parallel drum assembly. The friction between the wire rope and the drums provides the gripping force for the wire rope. The wire rope is coiled on a take-up reel which is required to maintain a nominal level of tension in the wire rope (typically 3% to 5% of working tension) to ensure the proper level of friction is maintained between the wire rope and the traction winch. This system has been favored for use in high tension applications due to the compact size, capability to provide constant torque, and ability to handle very long wire rope without reduced pull capacity.

### A.2.6 FAIRLEAD AND STOPPER

Mooring lines are subjected to high wear and stress at the fairlead and stopper arrangements. The long term service of a mooring system requires that fairlead and stopper arrangements be carefully designed to minimize wear and fatigue.

Mooring chain and wire rope are often stopped off at the vessel in order to take direct mooring loads off the winch. Chain stoppers and wire rope grips used for permanent mooring systems must be designed so that the stress concentrations and wear within the chain or wire rope are kept at acceptable levels.

Fairleads should provide sufficient sheave to rope diameter ratio to minimize tension-bending fatigue. Typically 7 to 9 pocket wildcat sheaves are used for chain. Sheaves for wire rope have diameter (*D/d*) ratios of 16-25 for mobile moorings, and 40–60 for permanent moorings. There are other devices which provide attractive alternatives for fairleading large diameter mooring lines. An example is the underwater swivelling bending shoe shown in Figure A.11. This device, which is used initially with wire ropes, incorporates a shoe to rope diameter ratio of more than 70 and a special high density nylon bearing material secured to the bearing surface on the shoe. Replacement of the material is possible by slacking down the mooring line and removing the bearing material which is bolted to the bearing surface in sections. Industry experience indicated that this device can also be used for chain (Figure A.9). However, operating with high chain tension and frequent vessel move should be avoided because it can cause serious damage to the bearing material.

## A.3 Anchoring System

The options that are available for anchoring floating vessels include:

- Drag Embedment Anchors
- Pile Anchors (driven, jetted, drilled and grouted)
- Suction pile and Suction Caisson
- Gravity Anchor
- Plate Anchor (drag embedded and direct embedded)

In selecting anchor options, consideration must be given to required system performance, soil conditions, reliability, installation, and proof loading.

### A.3.1 DRAG EMBEDMENT ANCHOR

Traditional drag embedment anchors (Figure A.12) were initially used for mobile mooring operations. Drag embedment anchor technology has advanced considerably in recent years. Engineering and testing indicate that the new generation of fixed fluke drag embedment anchors develop high holding power even in the soft soil conditions. High efficiency drag embedment anchor is generally considered to be an attractive option for mooring applications because of its easy installation and proven performance. In fact, many existing permanent and mobile moorings use drag embedment anchors. The anchor section of a mooring line can be pre-installed and test loaded prior to platform installation.

### A.3.2 PILE ANCHOR

A pile anchor's resistance to uplift and lateral loading is primarily a function of pile dimensions, the manner in which the pile is installed and loaded, and the type, stiffness, and strength of the soil adjacent to the pile. Horizontal capacity can be increased considerably by adding special elements such as skirts or wings to the pile top. Pile anchors can be designed to develop high lateral and vertical resistance, and be very stable over time. Piles are generally installed using driving hammers although other methods such as jetting and drilling and grouting techniques have been used. Installation of jetted or drilled and grouted piles can be handled by a conventional drilling rig without major modifications. However, disturbance of soil during jetting and drilling operations should be carefully evaluated.

### A.3.3 SUCTION PILE AND SUCTION CAISSON

#### A.3.3.1 Suction Pile

Suction piles can be used for large deepwater mooring systems and can be designed for very high mooring line loads. They are typically tall steel cylindrical structures with or without internal stiffener systems. The cylinder unit is open at the bottom and normally closed at the top (Figure A.13). A

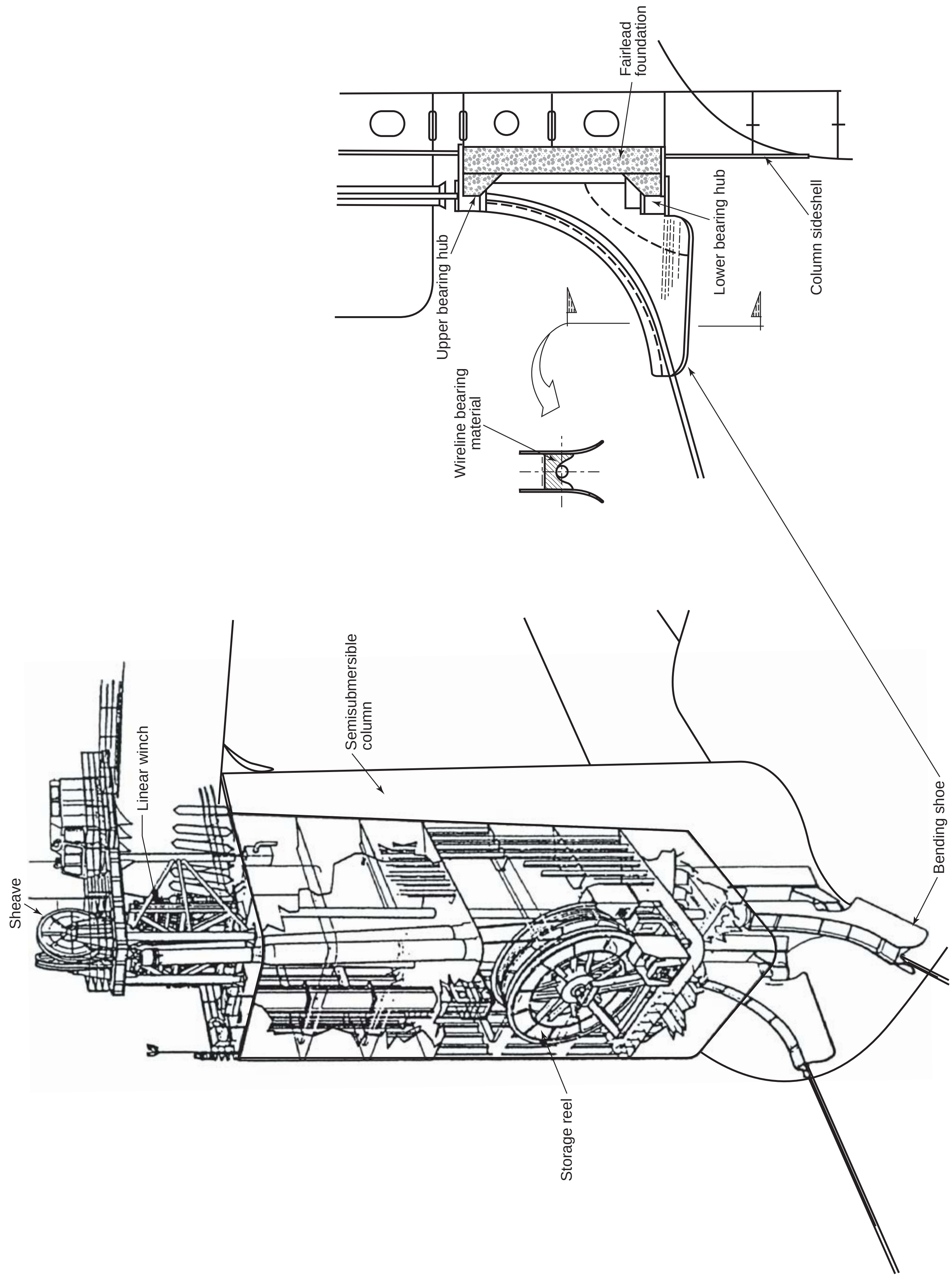



Figure A.11—Linear Winch and Bending Shoe Fairlead for Wire Rope

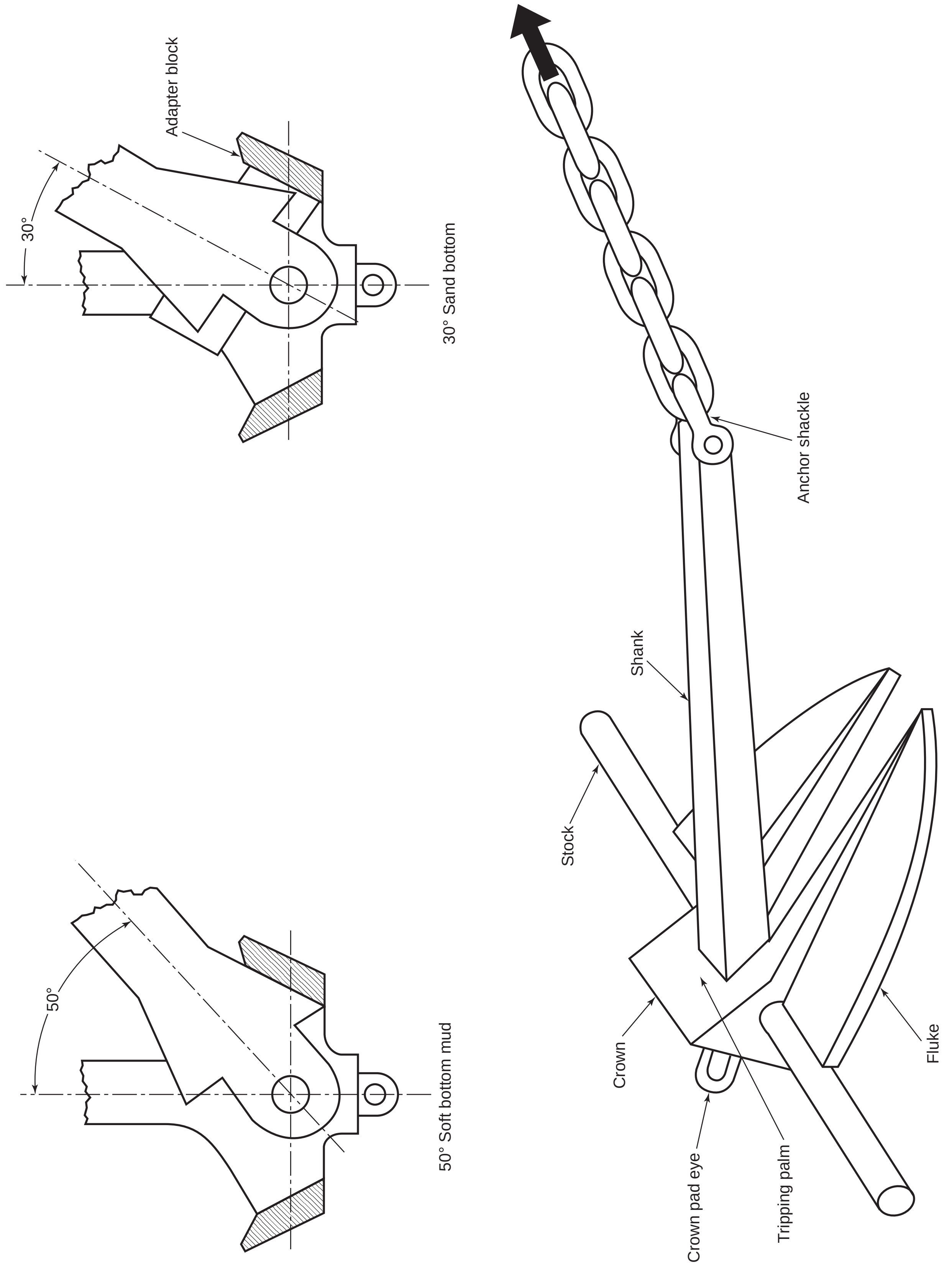



Figure A.12—Traditional Drag Embedment Anchor

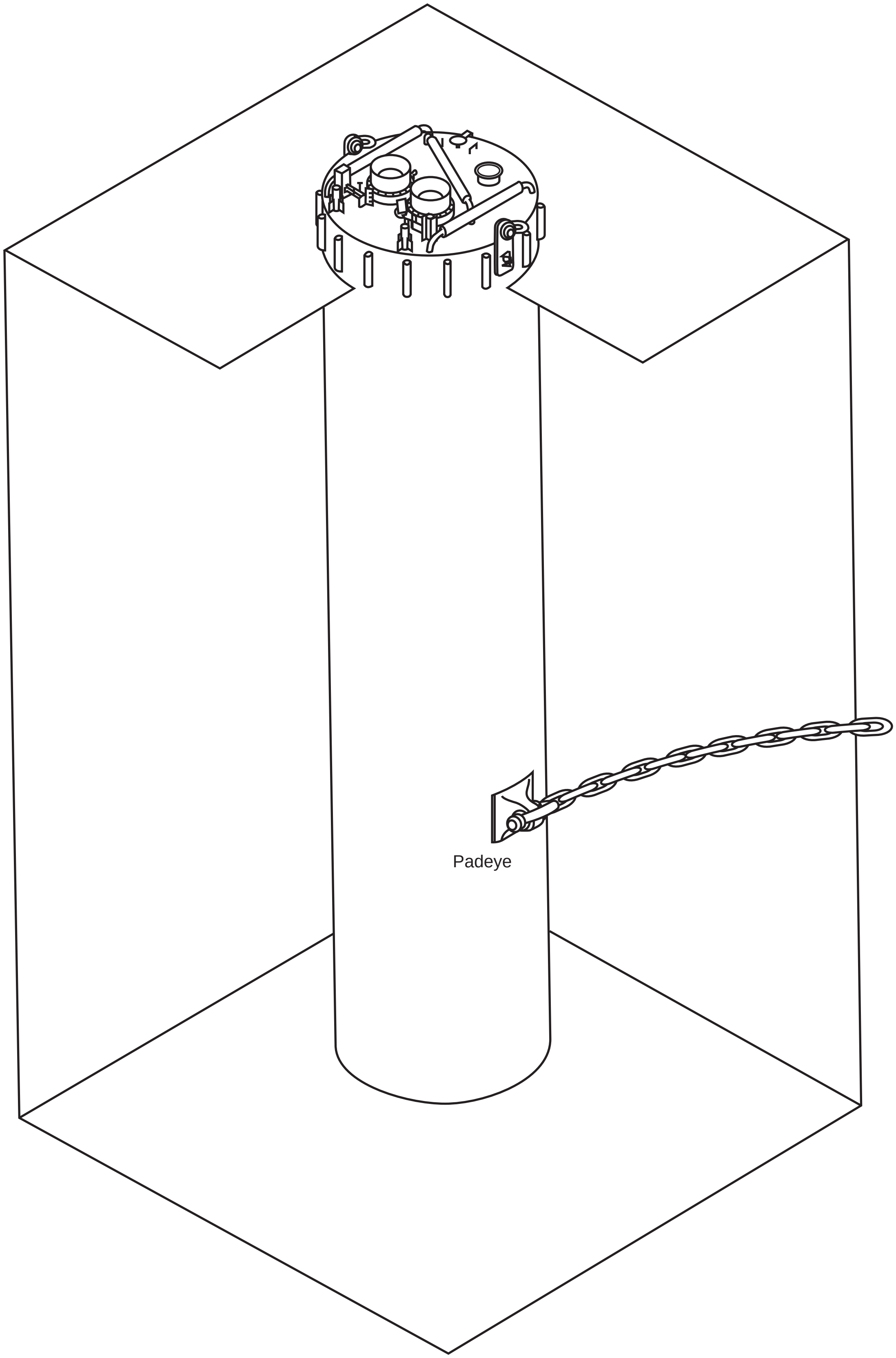



Figure A.13—Suction Pile

suction pile is installed by first lowering it into the soil to self-penetration depth (i.e., penetration due to submerged pile weight). The remainder of the required penetration is achieved by pumping the trapped water from the inside of the suction pile. The pressure differential thus created will result in an additional driving force on the anchor top, which will drive the pile into the soil. As the penetration increases, the driving force needed normally increases, requiring a gradually increasing differential pressure.

After penetration, the water outlet is normally closed, and a suction pile may achieve substantial capacity to resist vertical downward loads, horizontal loads, vertical uplift loads, moments, and combinations of these loads. For suction piles embedded in clay and with a closed outlet, the capacity to resist environmental loads is governed by an undrained shear failure in the soil around and beneath the pile. The capacity depends on depth of skirt penetration, cylinder diameter, shear strength in the clay, shear strength at the clay/wall interface, the load inclination, and the location of the load attachment point. In the case where the top part is left open or retrieved, or for long-term uplift load components, pull-out of the skirts may also be a possible failure mechanism.

The holding capacity is generally greater if the pile is prevented from tilting. A translational failure mode without tilting can be achieved by lowering the load attachment point from the top of the anchor to the anchor wall at an optimal depth below the seabed. The location of the optimal load attachment point depends on the shear strength profile, the shear strength at the clay/wall interface, the load inclination, the submerged anchor weight, and the depth/diameter ratio of the pile. The optimal location is typically two-thirds of the length of the pile downwards from the pile top.

As suction piles are shallow structures compared to driven piles, deep soil borings are not required, but more detailed soil data are needed at shallow depths than for driven piles. Suction piles have mainly been applied in cohesive clay type soils. Suction embedment penetration through thin sand or granular layers may be feasible, provided the suction pile design takes this into account. Penetration in non-cohesive granular type soils requires special considerations, which are not covered in this document.

Suction pile length to diameter ratios may range from 2:1 for stiff clay soils to as much as 7:1 for very soft clay soil. Suction piles are often designed with large depth/diameter ratios in soft clays, since the upper part of soft clay deposits provide limited bearing capacity and skin friction. A suction pile can consist of two sections, an upper driving section and a lower pile section, which are connected during installation. Once full penetration is reached, the two sections are disengaged from each other and the upper section is recovered, leaving the lower section in the soil. The upper section is then reused to drive other anchor sections.

#### A.3.3.2 Suction Caisson

Suction caisson is a suction embedded anchor that is relatively shallow in height and is designed for relatively small penetration. The suction caisson's submerged weight makes up a large part of the anchor’s vertical holding capacity. A multi-cell concrete structure with a large footprint and a shallow skirt penetration would be an example of a suction caisson (Figure A.14). The vertical load capacity is mainly from its own weight plus possibly some skin friction and internal suction. Horizontal load capacity is generated by skirt penetration and friction between the soil layers being sheared.

### A.3.4 GRAVITY ANCHOR

Gravity anchors are deadweight anchors which commonly consist of concrete or steel blocks, scrap metal or other materials of high density. Skirt penetration is obtained through self-weight penetration, and the design uplift capacity is dependent on the submerged weight of the anchor. Horizontal capacity is a function of the friction between the anchor and the soil and shear strength of the soil beneath the anchor. Gravity anchors can be used for small mooring systems, but typically are not used for large deepwater mooring systems.

### A.3.5 PLATE ANCHOR

Plate anchors were initially used by the US Navy for anchoring of fleet mooring buoys. They are installed at deep penetration beneath the seafloor where the generally higher soil strength allows the use of relatively small plate anchors for high mooring loads. Plate anchors typically have significant vertical holding capacity. This allows the use of taut leg mooring systems where the anchor line can intersect the seafloor at significant inclinations. Plate anchors can be placed in two broad categories: drag embedded and direct embedded.

#### A.3.5.1 Drag Embedded Plate Anchor

Drag embedded plate anchors are embedded to deep penetration in a manner similar to drag anchors. During installation, the anchor is first placed on the seafloor, and as the anchor is pulled along the bottom, it penetrates the soil. Initially, the anchor dives more or less parallel to the fluke, eventually rotating such that the target penetration depth is achieved. Following the embedment, the anchor is “set or keyed”, i.e., the anchor fluke is oriented such that it becomes nearly perpendicular to the anchor line, thus providing high horizontal and vertical holding capacity. These drag embedded anchors are often referred to as VLA, which stands for <u>V</u>ertically <u>L</u>oaded <u>A</u>nchor. Two VLAs are commonly used by the offshore industry: Stevmanta and Denla. The Stevmanta anchor uses a bridle system to convert from its installation configuration to its plate anchor operational orientation, whereas the Denla anchor uses an articulated shank (Figure A.15).

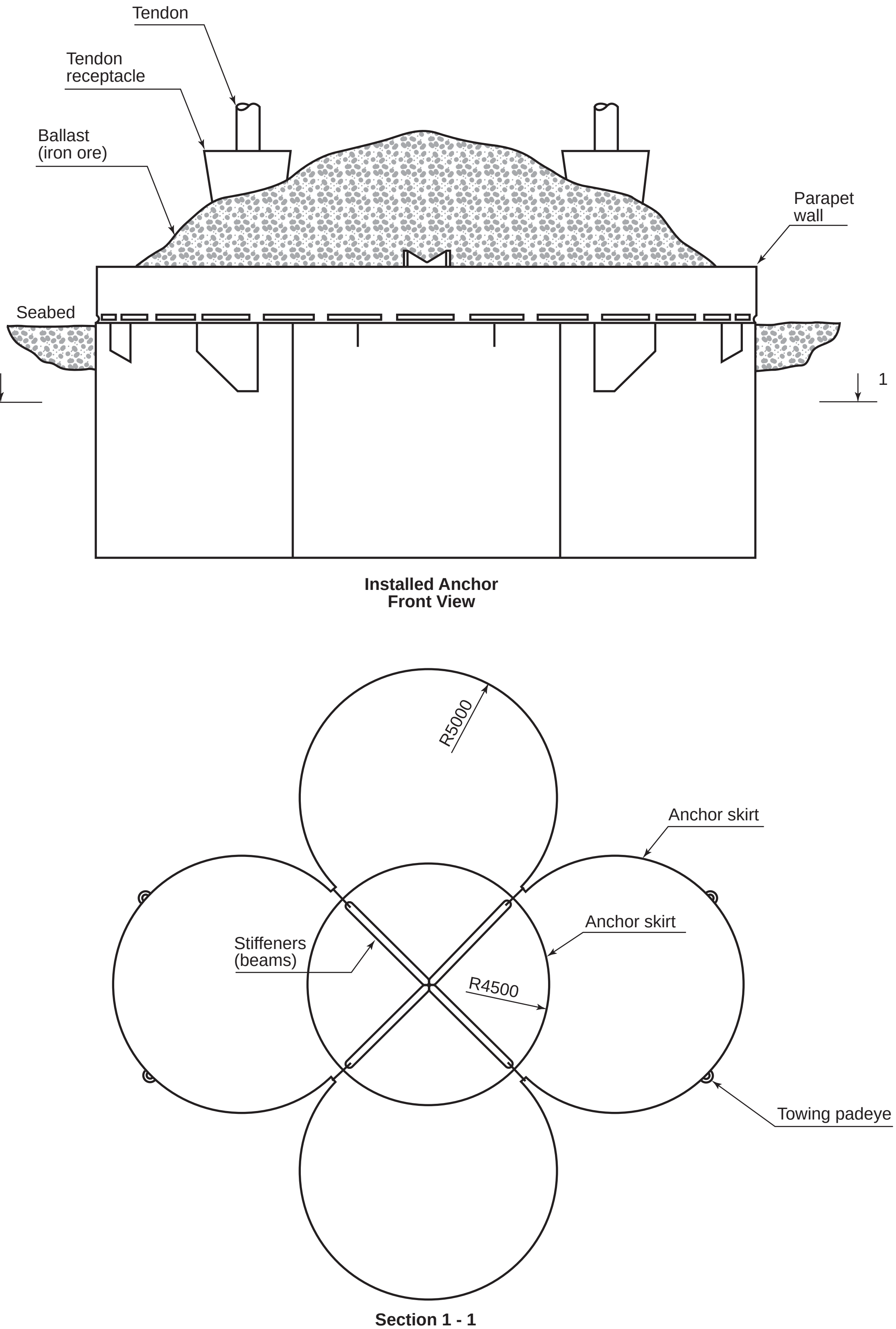



Figure A.14—Suction Caisson

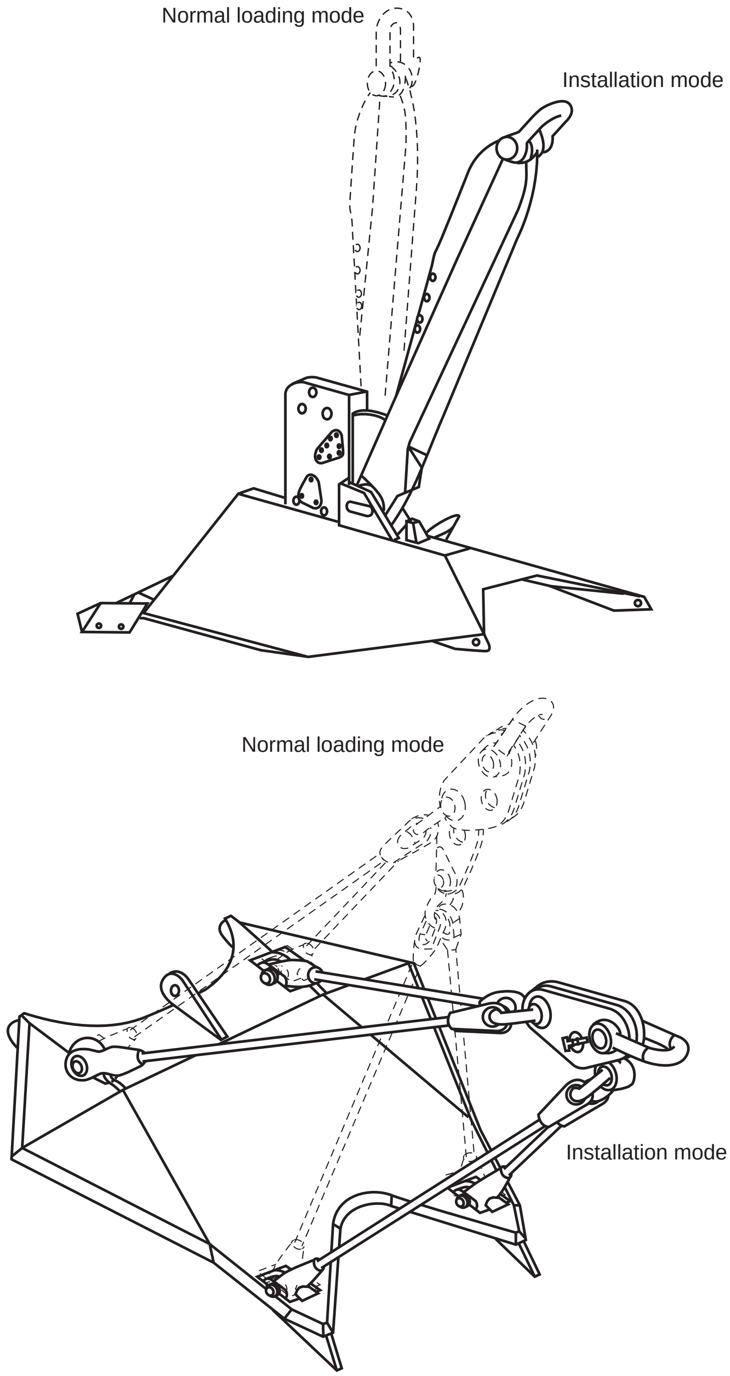



Figure A.15—Drag Embedded Plate Anchor (VLA)

### A.3.5.2 Direct Embedded Plate Anchor

Direct embedment of plate anchors can be achieved by suction, impact or vibratory hammer, propellant, or hydraulic ram. The suction embedded plate anchor has been used for major offshore mooring operations. As an example, the SEPLA (Suction Embedded PLate Anchor) uses a so-called suction follower, which is essentially a reusable suction anchor with its tip slotted for insertion of a plate anchor. The suction follower is immediately retracted by reversing the pumping action once the plate anchor is brought to the design depth, and can be used to install additional plate anchors (Figure A.16). In the SEPLA concept, the plate anchor's fluke is embedded in vertical position, and adequate fluke rotation is achieved during a keying process by pulling on the mooring line.

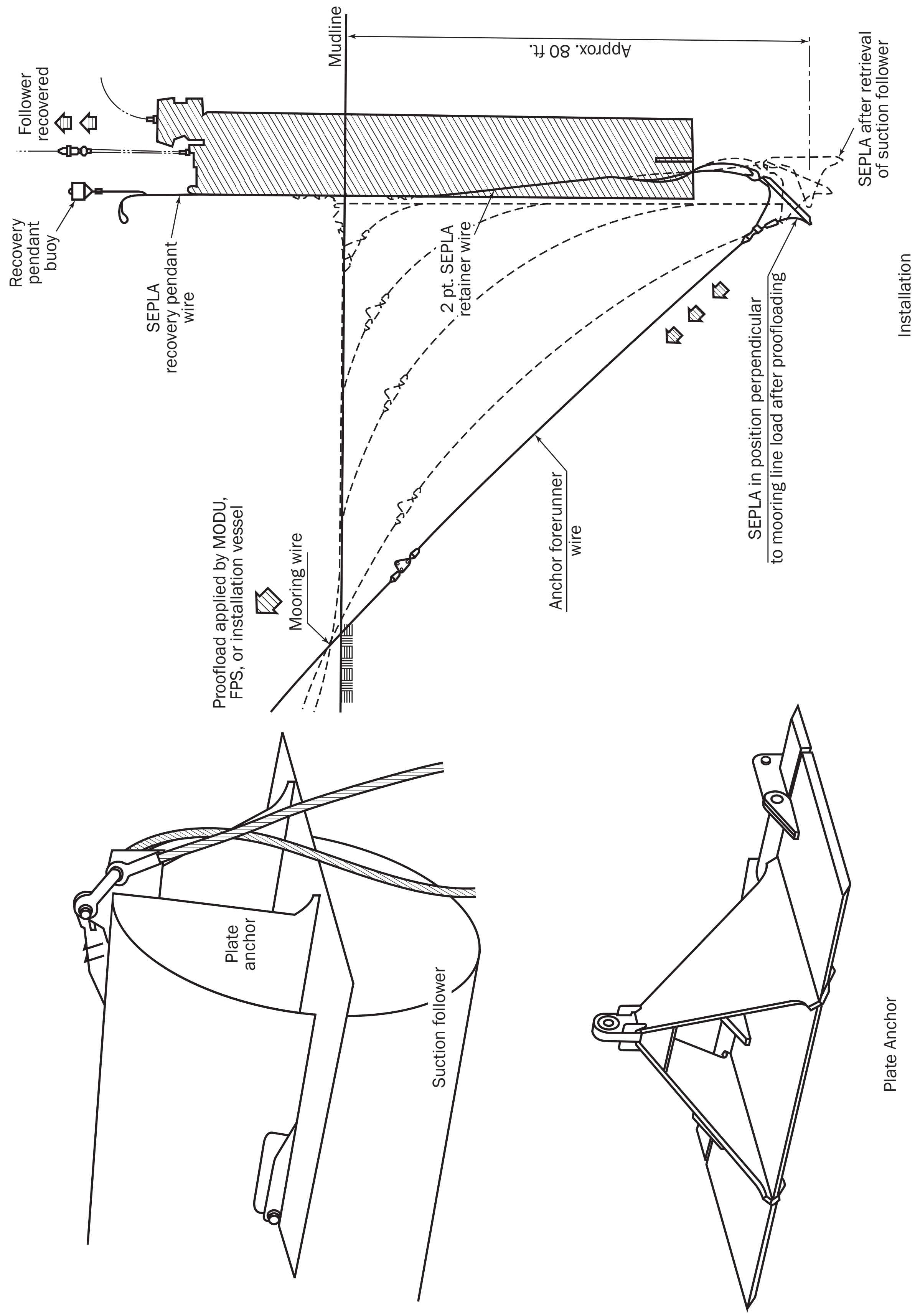



Figure A.16—Suction Embedded Plate Anchor (SEPLA)

# APPENDIX B—RECOMMENDED WIND SPECTRUM

## B.1 Basic Considerations

As discussed in Section 3.3, fluctuating wind can be modeled by a steady component, based on the 1-hour average velocity, plus a time-varying component calculated from a suitable empirical wind gust spectrum. A number of wind spectra have been developed from various resources, such as the Ochi, Davenport, Harris, API, and NPD spectrum. Currently only the API and NPD (Reference B.1) spectrum are commonly used by the offshore industry. The API spectrum, which was published in earlier editions of API RP 2A, has much smaller empirical database than the NPD spectrum. The uncertainty of the API spectrum is addressed through specifying a range instead of a single value for the dimensionless peak frequency. This results in a spectrum defined by upper and lower bound values. In the latest edition of API RP 2A, the API spectrum was replaced by the NPD spectrum, which was also specified by the draft ISO standard.

The recommended wind spectrum for this document is the NPD spectrum. However, this spectrum may have significant uncertainty in the region of long periods, say over 500 seconds. For responses with a natural period longer than 500 seconds, the API upper bound spectrum can be considered. A sensitivity study on the effects of API and NPD wind spectrum to global responses for spar, TLP, semi-submersible, and FPSO in various water depths can be found in Appendix I, Section I.7.

It should be noted that the NPD spectrum formulas published in the 21st Edition of API RP 2A (Reference B.2) and its Supplement 1 published in 2002 contain errors, which have been corrected in this document.

## B.2 Equations for NPD Wind Spectrum

The Norwegian Petroleum Directorate, NPD, [B.1] wind profiles, gust factors, and spectra are all defined by a single parameter. The defining or characteristic parameter is the 1-hour mean wind speed at 10m above sea level, $U_0$ (m/s). Note that the equations provided below for the NPD wind profiles, gust factors, and spectra assume metric units.

### B.2.1 WIND PROFILES AND WIND GUST SPEEDS

The maximum wind speed, in 1 hour, averaged over $t$ seconds ($t$ < 3600s) at a height of $z$ meters above sea level is given by,

$$u(z,t) = U(z)[1 - 0.41 I_u(z) ln(t/t_0)] \qquad \text{(B.1)}$$

where

$u(z,t)$ = maximum $t$-second averaged wind speed in 1 hour at elevation $z$ above sea level [m/s],

$t_0$ = 3600 [s],

$z$ = elevation above sea level [m],

$t$ = wind speed averaging time period, $t < 3600$s [s].

and the 1-hour mean wind speed $U(z)$ at elevation $z$ is given by,

$$U(z) = U_0\left[1 + C\ln\left(\frac{z}{10}\right)\right] \qquad \text{(B.2)}$$

with

$$C = 0.0573\sqrt{1 + 0.15U_0} \qquad \text{(B.3)}$$

and

$$I_u(z) = 0.06[1 + 0.043U_0]\left(\frac{z}{10}\right)^{-0.22} \qquad \text{(B.4)}$$

where

$U(z)$ = 1-hour mean wind speed at elevation $z$ above sea level [m/s],

$U_0$ = 1-hour mean wind speed at elevation of 10m above sea level [m/s].

### B.2.2 WIND SPECTRUM

The NPD wind spectrum describes the energy density of the longitudinal wind speed fluctuations at a point. The 1-point energy density is given by,

$$S_{NPD}(f) = \frac{320\left(\frac{U_0}{10}\right)^2\left(\frac{z}{10}\right)^{0.45}}{(1 + \tilde{f}^{\,0.468})^{3.561}} \qquad \text{(B.5)}$$

where

$S_{NPD}(f)$ = is the spectral energy density at frequency $f$ [$(m/s)^2$/Hz],

$f$ = frequency [Hz].

and

$$\tilde{f} = \frac{172f\left(\frac{z}{10}\right)^{2/3}}{\left(\frac{U_0}{10}\right)^{3/4}} \qquad \text{(B.6)}$$

### B.2.3 COHERENCE SPECTRUM

The 2-point coherence spectrum describes the squared correlation between the spectral energy densities *S(f)* of the longitudinal wind speed fluctuations of frequency *f* between two points in space.

The coherence spectrum between two points at levels $z_1$ and $z_2$ above sea level, with across-wind positions $y_1$ and $y_2$, and along-wind positions $x_1$ and $x_2$ is given by,

$$Coh_{NPD}(f) = \exp\left(-\frac{1}{U_0}\sqrt{\sum_{i=1}^{3} A_i^2}\right) \qquad \text{(B.7)}$$

where

$Coh_{NPD}(f)$ = the squared correlation between spectral energy densities *S(f)* for two points

with

$$A_i = \alpha_i f^{r_i} \Delta_i^{q_i} z_g^{-p_i} \qquad \text{(B.8)}$$

and

$$z_g = \frac{\sqrt{z_1 z_2}}{10} \qquad \text{(B.9)}$$

and with the coefficient α, *p*, *q*, and *r* and the distances ý are given in Table B.1 below.

Table B.1—Coefficients for NPD 2-Point Coherence Spectrum

| *i* | $\acute{y}_i$ | $q_i$ | $p_i$ | $r_i$ | *i* |
|---|---|---|---|---|---|
| 1 | $\lvert x_2 - x_1\rvert$ | 1.00 | 0.40 | 0.92 | 2.90 |
| 2 | $\lvert y_2 - y_1\rvert$ | 1.00 | 0.40 | 0.92 | 45.00 |
| 3 | $\lvert z_2 - z_1\rvert$ | 1.25 | 0.50 | 0.85 | 13.00 |

The two points are

- at levels $z_1$ and $z_2$ above sea level [m],
- with across-wind positions $y_1$ and $y_2$ [m],
- and with along-wind positions $x_1$ and $x_2$ [m].

## B.3 Equations for API Wind Spectrum

The American Petroleum Institute, API, [B.2] wind profiles and gust factors are defined by a single parameter, the 1-hour mean wind speed $U_0$ at 10m (33ft) above sea level. However two parameters are required to define the API wind spectrum, $U_0$ and α.

### B.3.1 WIND PROFILES AND WIND GUST SPEEDS

The average 1-hour wind speed at a height *z* above sea level (the 1-hour mean wind profile) is given by,

$$U(z) = U_0\left(\frac{z}{z_R}\right)^{0.125} \qquad \text{(B.10)}$$

where

$U(z)$ = 1-hour mean wind speed at elevation *z* above sea level [m/s, ft/s],

$U_0$ = 1-hour mean wind speed at elevation of 10m (33ft) above sea level [m/s, ft/s],

$z$ = elevation above sea level [m, ft],

$z_R$ = 10m (33ft) = reference elevation above sea level.

The wind gust speed averaged over *t* seconds ($t < 60$s) at a height of *z* meters above sea level is given by,

$$u(z,t) = U(z)[1 + g(t)I(z)] \qquad \text{(B.11)}$$

where

$u(z,t)$ = the *t*-second averaged wind gust speed at elevation *z* above sea level [m/s, ft/s],

$t$ = wind speed averaging time period, $t < 60$s [s],

with

$$g(t) = 3 + \ln\left[\left(\frac{3}{t}\right)^{0.6}\right] \text{ for } t \le 60s \qquad \text{(B.12)}$$

and

$$I(z) = \begin{cases} 0.15\left(\frac{z}{z_s}\right)^{-0.125} & \text{for } z \le z_s \\ 0.15\left(\frac{z}{z_s}\right)^{-0.275} & \text{for } z \le z_s \end{cases} \qquad \text{(B.13)}$$

where

$z_S$ = 20m (66ft) = thickness of the “surface layer”.

### B.3.2 WIND SPECTRUM

The API wind spectrum describes the energy density of the longitudinal wind speed fluctuations at a point. The 1-point energy density is given by,

$$S_{API}(f) = \frac{\sigma(z)^2}{f_p\left(1 + 1.5\frac{f}{f_p}\right)^{5/3}} \qquad \text{(B.14)}$$

where

$S_{API}(f)$ = is the spectral energy density at frequency $f$ [(m/s)$^2$/Hz, (ft/s)$^2$/Hz],

$f$ = frequency [Hz],

with

$$\sigma(z) = I(z)U(z) \tag{B.15}$$

and

$$f_p = \frac{a}{z}U(z) \text{ with } 0.01 \le \alpha \le 0.1 \tag{B.16}$$

For measured wind spectra the average value of $f_p$ is given by $\alpha = 0.025$.

## B.4 Comparison Plots of NPD and API Wind Spectra

See Figures B.1, B.2, and B.3.

## B.5 References

**B.1.** NORSOK Standard: *Actions and Effects*: N-003, Rev.1, Norwegian Technology Standards Institution, Oslo, 1999.

**B.2.** API RP 2A-WSD, *Planning Designing and Constructing Fixed Offshore Platforms-Working Stress Design*, 21st Edition, December 2000.

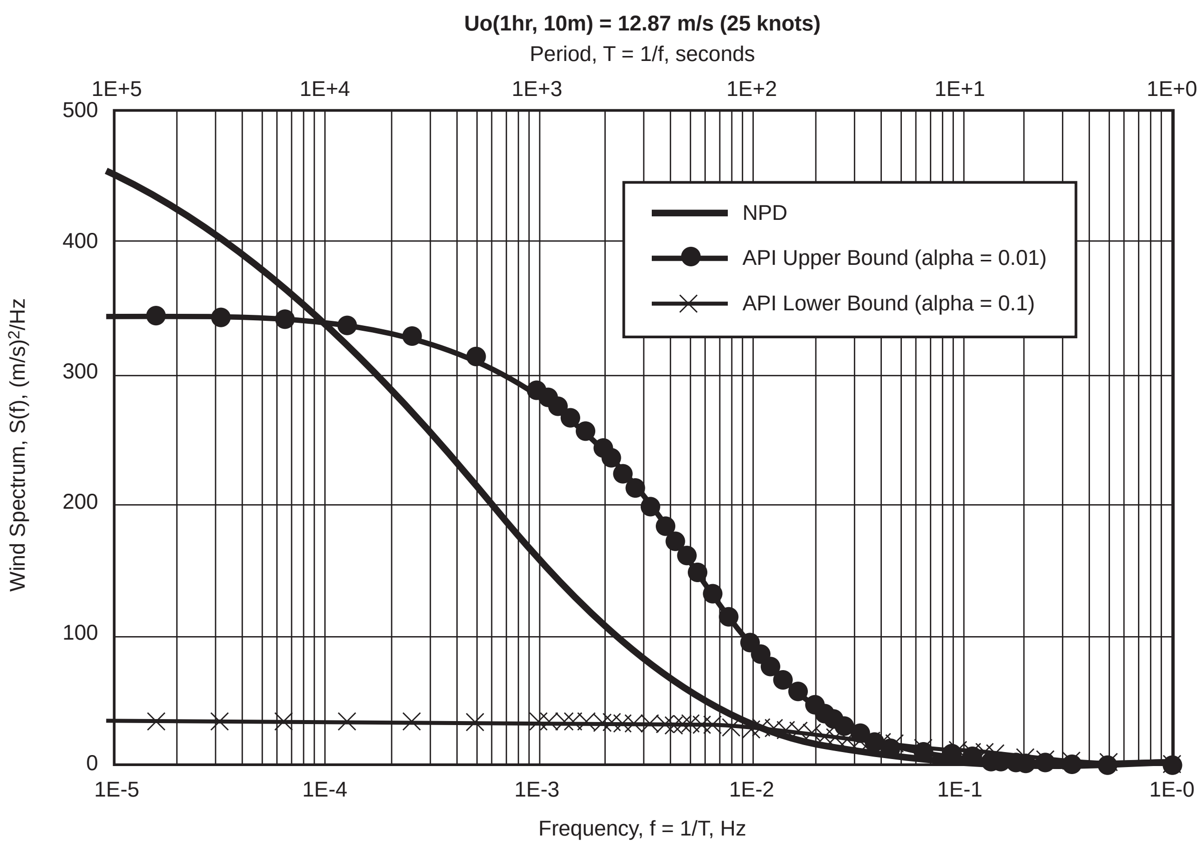



Figure B.1—Comparison of API and NPD Spectrum for 25-Knot Wind

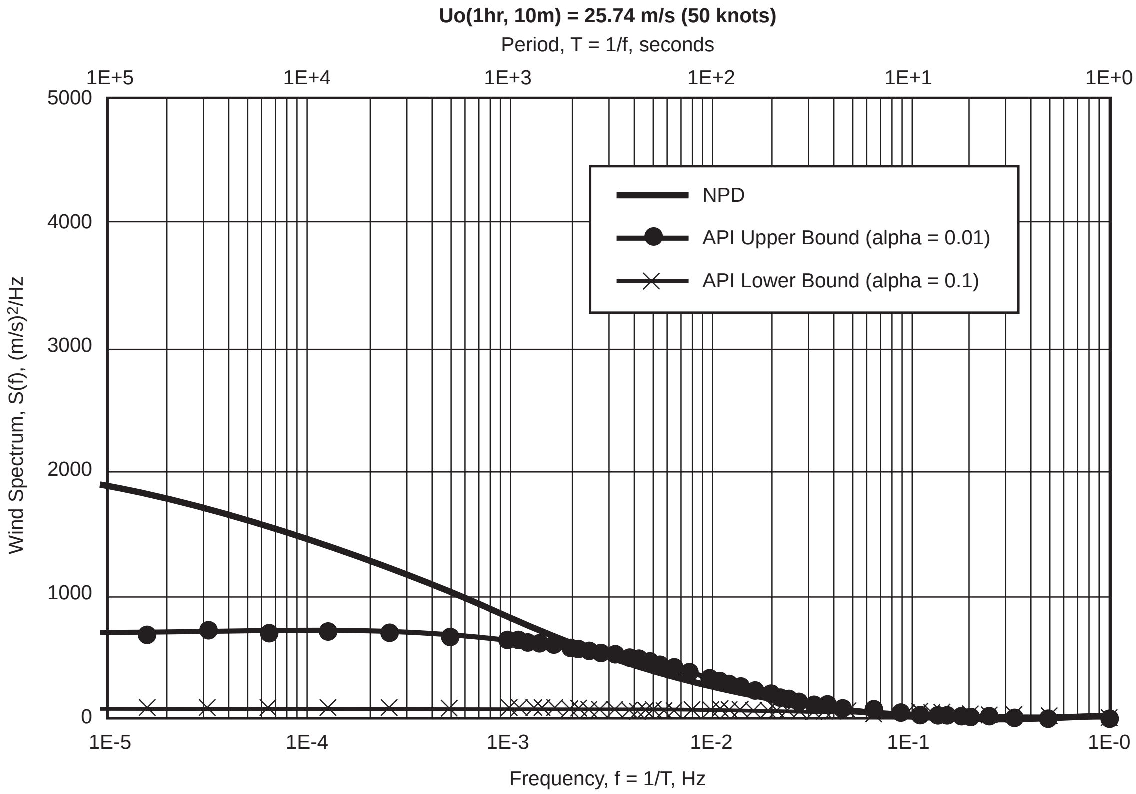



Figure B.2—Comparison of API and NPD Spectrum for 50-Knot Wind

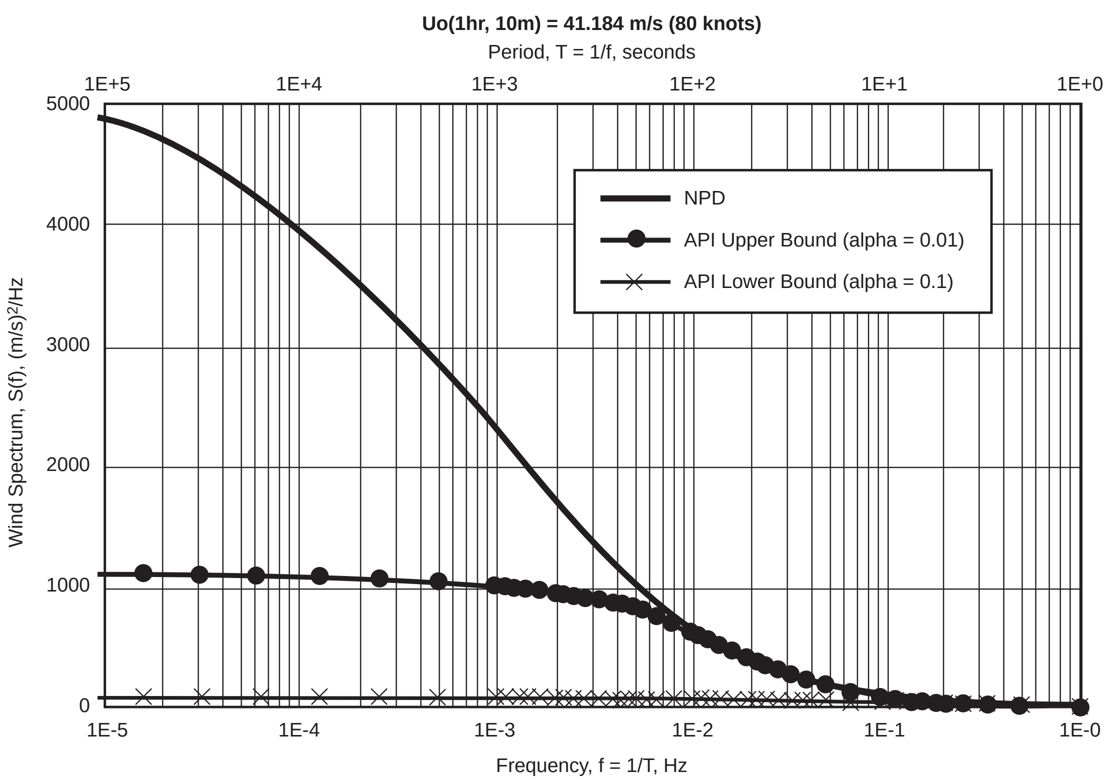



Figure B.3—Comparison of API and NPD Spectrum for 80-Knot Wind

# APPENDIX C—SIMPLIFIED METHODS FOR THE EVALUATION OF ENVIRONMENTAL FORCES AND VESSEL MOTIONS

## C.1 Basic Considerations

Design equations and curves for a quick evaluation of environmental forces and vessel motions are provided in this appendix. These simplified analytical tools were developed primarily for the analysis of mobile moorings. They may be used for preliminary designs of permanent moorings if more accurate information is not available at the early stage of the design process and if the limits for these tools are not exceeded. For the final design of permanent moorings, however, the more rigorous approaches as outlined in 4.2 are recommended.

## C.2 Current

Current forces are normally treated as steady state forces in a mooring analysis. They can be estimated by model tests or calculations.

### C.2.1 MODEL TESTS

Model test data from towing tank or wind tunnel tests may be used to predict current loads for mooring system design provided that a representative underwater model for the unit is tested and that the contribution to current load made by thrusters, anchor bolsters, bilge keels, and other appendages is accounted for. Care should be taken to assure that the character of the flow in the model test is the same as the character of the flow for the full-scale unit.

### C.2.2 CURRENT FORCE CALCULATIONS

If current forces are to be calculated, the following equations should be used:

a. Current force due to bow or stern current on ship shaped hulls:

$$F_{cx} = C_{cx} S V_c^2 \qquad \text{(C.1)}$$

where

$F_{cx}$ = current force on the bow, lb (N),

$C_{cx}$ = current force coefficient on the bow,

= 0.016 lb/(ft$^2$ • kt$^2$) (2.89 Nsec$^2$/m$^4$),

$S$ = wetted surface area of the hull including appendages, ft$^2$(m$^2$),

$V_c$ = design current speed, kts (m/sec).

b. Current force due to beam current on ship-shaped hulls:

$$F_{cy} = C_{cy} S V_c^2 \qquad \text{(C.2)}$$

where

$F_{cy}$ = current force on the beam, lb (N),

$C_{cy}$ = current force coefficient on the beam,

= 0.40 lb/(ft$^2$ • kt$^2$) (72.37 Nsec$^2$/m$^4$).

Note: Equations C.1 and C.2 were developed for estimating current forces on drillships. They are applicable only to production vessels with similar hull form and size.

c. Current and wind forces for large tankers: Current and wind forces for large tankers can be estimated using the report Prediction of Wind and Current Loads on VLCCs published by Oil Company International Marine Forum [C.1]. This report presents coefficients and procedures for computing wind and current loads on very large crude carriers (VLCCs), namely, tankers in the 150,000 to 500,000 dwt class. Wind/current force and moment coefficients are presented in nondimensional form for a moored vessel in various draft and under keel clearance conditions. While the analysis of mooring restraint has not been addressed, coefficients are provided for use with either computer oriented or hand calculation techniques for design of tanker/terminal mooring equipment.

d. Current force on semi-submersible hulls:

$$F_{cs} = C_{SS}(C_d A_c + C_d A_f)\, V_c^2 \qquad \text{(C.3)}$$

where

$F_{cs}$ = current force, lb (N),

$C_{ss}$ = current force coefficient for semi-submersible hulls,

= 2.85 lb/(ft$^2$ • kt$^2$) (515.62 Nsec$^2$/m$^4$),

$C_d$ = drag coefficient (dimensionless),

= 0.50 for circular members (see Figure C.1 for members having flat surfaces),

$A_c$ = summation of total projected areas of all cylindrical members below the waterline, ft$^2$ (m$^2$),

$A_f$ = summation of projected areas of all members having flat surfaces below the waterline, ft$^2$ (m$^2$).

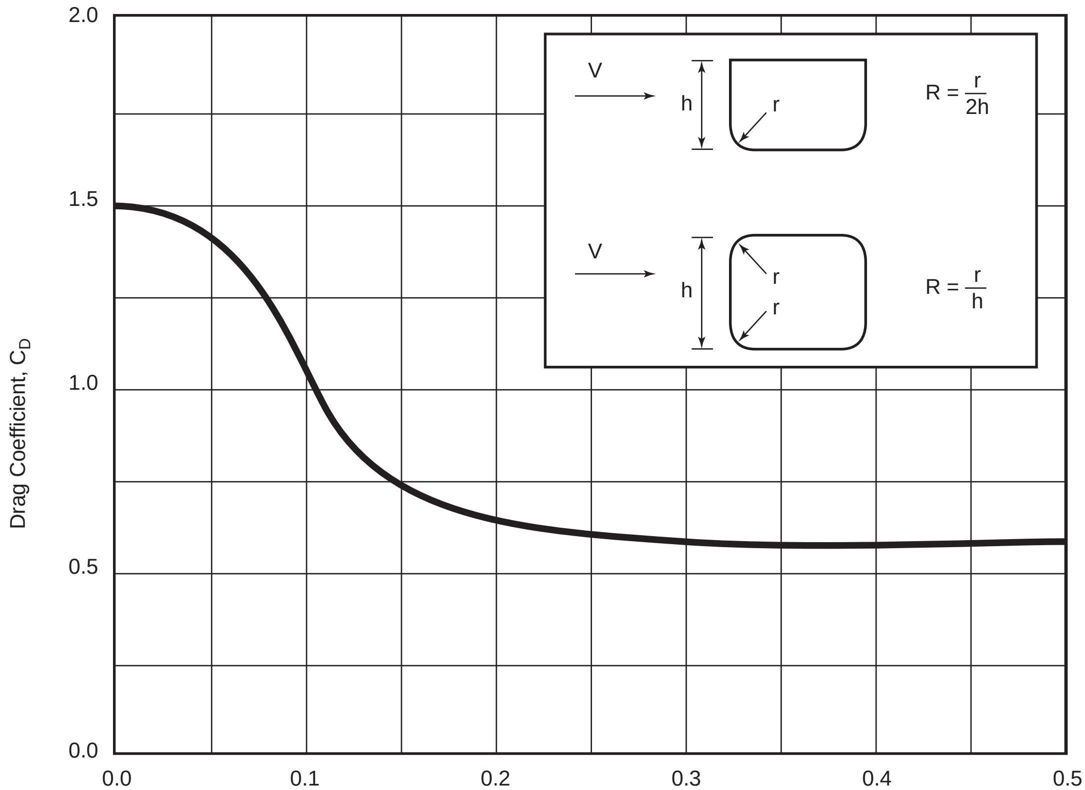



Figure C.1—Semi-submersible Current Drag Coefficient for Members Having Flat Surfaces

e. Current force on mooring lines and risers: The effect of current loads on mooring lines and risers on the overall mooring design should be evaluated. This is particularly important for deepwater locations with high currents. Current loads on mooring lines and risers can be calculated using appropriate current profiles and drag coefficients. In high currents, drag coefficients should be adjusted for the presence of vortex-induced vibrations.

## C.3 Waves

Interactions between ocean waves and a floating vessel results in forces acting on the vessel that can be conveniently split into three categories (Figure C.2): (a) first order forces that oscillate at the wave frequencies inducing first order motions known as high frequency or wave frequency motions; (b) second order forces with frequencies below wave frequencies inducing second order motions known as low frequency motions; and (c) steady component of the second order forces known as mean wave drift forces which can be estimated by model test or calculation.

### C.3.1 MODEL TESTS

Model test data may be used to predict wave forces for mooring system design provided that a representative underwater model of the unit is tested. Care should be taken to assure that the character of the flow in the model test is the same as the character of the flow for the full-scale unit.

### C.3.2 WAVE FREQUENCY VESSEL MOTIONS

The motions of the vessel at the frequency of the waves is an important contribution to the total mooring system loads, particularly in shallow water. These wave frequency motions can be obtained from regular or random wave model test data or computer analysis using either time or frequency domain techniques.

Wave frequency motions have six degrees of freedom: surge, sway, heave, pitch, roll, and yaw. They are normally considered to be independent of mooring stiffness except for floating systems with natural periods less than 30 seconds.

### C.3.3 MEAN WAVE DRIFT FORCE

The mean wave drift force is induced by the steady component of the second order wave forces. The determination of mean drift force requires motions analysis computer programs or model tests. Design curves for estimating mean wave drift forces for drillships and semi-submersibles are provided in Figures C.3 through C.17. The curves are applicable to typical MODU type vessels. However, for large drilling and production semi-submersibles (with displacements

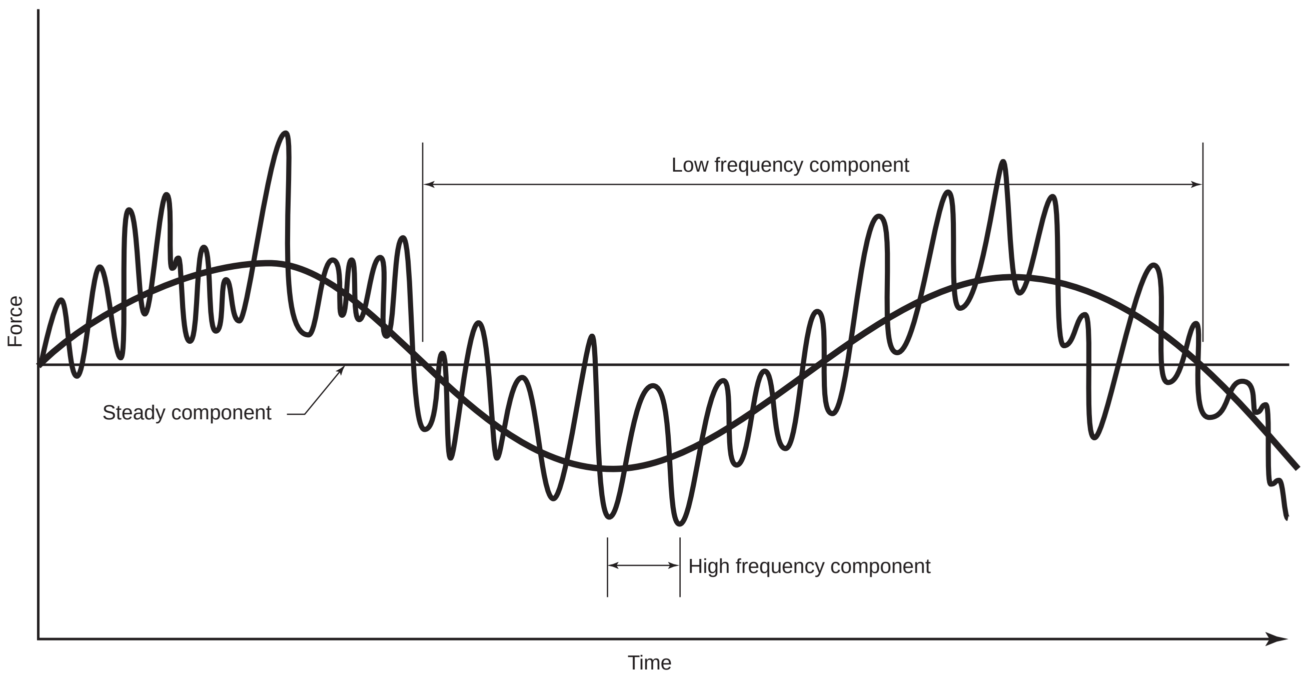



Figure C.2—Wave Force Components

over 30,000 short tons) and large tankers, the use of these curves is not recommended.

The curves for semi-submersibles represent the upper bound of the mean wave drift forces generated by a motions analysis computer program for four semi-submersible designs including typical 4, 6, and 8 circular column twin hull designs and the pentagon design. The curves for drillships were generated for ship lengths of 400 ft to 540 ft. For drillships which are outside this length range, the mean drift force can be estimated by extrapolation. However, extrapolation for ship lengths below 350 ft or above 600 ft is not recommended.

The data presented are appropriate for ship-shape vessels with normal hull form. Care should be used in applying this data to vessels with blunt bows or sterns or other unusual hull features.

### C.3.4 LOW FREQUENCY VESSEL MOTIONS

Low frequency motions are induced by the low frequency component of the second order wave forces, which in general are quite small compared to the first order forces. Because of this, the low frequency forces do not play a significant role in the motions in the vertical plane (i.e., roll, pitch, and heave motions) where large hydrostatic restoring forces are present. However, in the horizontal plane (i.e., surge, sway, and yaw motions), where the only restoring forces present are due to mooring or dynamic positioning systems and production risers, the motions produced by the low frequency forces can be substantial. This is particularly true at frequencies near the natural frequency of the mooring. Therefore, in general, only low frequency surge, sway, and yaw motions are included in a mooring analysis.

Low frequency motion of a moored vessel is narrow banded in frequency since it is dominated by the resonant response at the natural frequency of the moored vessel. The motion amplitude is highly dependent on the stiffness of the mooring system. The motion amplitude is also highly dependent on the system damping so that a good estimate of damping is critical in computing low frequency motions. Methods for predicting the low frequency motions are still in a state of development. There is a substantial degree of uncertainty in the estimation, particularly in damping.

There are three sources of damping:

a. Viscous damping of the vessel.

b. Wave drift damping of the vessel.

c. Mooring system damping.

The technology to estimate viscous damping has been well established, and viscous damping is normally included in the low frequency motion calculations. Wave drift damping and mooring system damping, however, are more complex and are often neglected because of a lack of understanding in these damping components. Recent research indicates that wave drift damping and mooring system damping can be significant. They can even be higher than viscous damping under certain conditions, and neglecting them may lead to significant over-estimation of low frequency motions. In applications where low frequency motions are an important design factor, such as for large tankers, it may be warranted to evaluate damping from all these sources either by analytical approach or model testing.

The determination of low frequency motions requires motions analysis computer programs or model tests. Design curves for estimating low frequency motions for drill ships and semi-submersibles are also provided in Figures C.3 through C.17. These curves are applicable to typical MODU type vessels. However, for large production and drilling semi-submersibles (with displacements over 30,000 short tons) and large tankers, the use of these curves is not recommended.

The curves presented are appropriate for mooring spring stiffness of 18 kips per ft of vessel offset. For other mooring stiffnesses, the results from Figures C.3 through C.17 should be adjusted by Equations C.4 and C.5:

$$X_s = (X_s)_{REF}\left(\frac{18}{k}\right)^{1/2} \quad \text{(C.4)}$$

$$Y_s = (Y_s)_{REF}\left(\frac{18}{k}\right)^{1/2} \quad \text{(C.5)}$$

where

$(X_S)_{REF}$ = rms single amplitude low frequency surge from Figures C.3 to C.17,

k = mooring system spring stiffness in kips/ft taken at the vessel's mean position,

$(Y_S)_{REF}$ = rms single amplitude low frequency sway from Figures C.3 to C.17.

The drillship curves in these figures are for drillships of 400 ft to 540 ft in length. For drillships that are outside this length range, the low frequency motions can be estimated by extrapolation. However, extrapolation for ship lengths below 350 ft or above 600 ft is not recommended.

## C.4 Wind

The force due to wind may be determined by using wind tunnel or towing tank model test data or Equation C.6. The wind speed used is defined in 3.3.

### C.4.1 MODEL TESTS

Model test data may be used to predict wind loads for mooring system design provided that a representative model of the unit is tested, and that the condition of the model in the tests, such as draft and deck cargo arrangement, closely matches the expected conditions that the unit will see in service. Care should also be taken to assure that the character of the flow in the model test is the same as the character of flow for the full scale unit.

### C.4.2 WIND FORCE CALCULATION

#### C.4.2.1 Constant Wind Force

The steady state force due to wind acting on a moored floating unit can be determined using Equation C.6.

$$F_w = C_W \Sigma (C_s C_h A) V_w^2 \quad \text{(C.6)}$$

where

$F_w$ = wind force, lbs (N),

$C_w$ = 0.0034 lb/(ft$^2$ • kt$^2$) (0.615 Nsec$^2$/m$^4$),

$C_s$ = shape coefficient,

$C_h$ = height coefficient,

$A$ = vertical projected area of each surface exposed to the wind, ft$^2$(m$^2$),

$V_w$ = design wind speed, knots (m/sec).

The projected area exposed to the wind should include all columns, deck members, deck houses, trusses, crane booms, derrick substructure, and drilling derrick as well as that portion of the hull above the waterline. Wind shielding in accordance with acceptable methods may be considered.

In calculating wind areas, the following procedures can be followed:

a. The projected area of all columns should be included.

b. The blocked-in projected area of several deck houses may be used instead of calculating the area of each individual unit. However, when this is done, a shape factor, $C_S$, of 1.10 should be used.

c. Isolated structures such as derricks and cranes should be calculated individually.

d. Open truss work commonly used for derrick mast and booms may be approximated by taking 60 percent of the projected block area of one face.

e. Areas should be calculated for the appropriate hull draft for the given operating condition.

f. The shape coefficients, $C_S$, of Table C.1 can be used.

g. Wind velocity increases with height above the water. In order to account for this change, a wind force height coefficient, $C_h$, is included. The height coefficients, $C_h$, of Table C.2 can be used. This table applies to the approach using 1-minute constant wind (see 3.3).

h. Equation C.7 may be used to adjust the wind velocities of various average time intervals.

$$V_t = \alpha V_{hr} \quad \text{(C.7)}$$

where

$V_t$ = wind velocity for the average time internal $t$,

$a$ = time factor from Table C.3,

$V_{hr}$ = 1 hour average wind velocity.

Table C.1—Wind Force Shape Coefficients

| Exposed Area | $C_s$ |
|---|---|
| Cylindrical shapes | 0.50 |
| Hull (surface above waterline) | 1.00 |
| Deck house | 1.00 |
| Isolated structural shapes (cranes, channels, beams, angles) | 1.50 |
| Under deck areas (smooth surfaces) | 1.00 |
| Under deck areas (exposed beams and girders) | 1.30 |
| Rig derrick | 1.25 |

Table C.2—Wind Force Height Coefficients (for 1-Minute Wind)

| Height of Area Centroid Above Water Level | | | | |
|---|---|---|---|---|
| Feet Over Not Exceeding | | Meters Over Not Exceeding | | $C_h$ |
| 0 | 50 | 0 | 15.3 | 1.00 |
| 50 | 100 | 15.3 | 30.5 | 1.18 |
| 100 | 150 | 30.5 | 46.0 | 1.31 |
| 150 | 200 | 46.0 | 61.1 | 1.40 |
| 200 | 250 | 61.0 | 76.0 | 1.47 |

Note: This table applies to the approach using 1-minute constant wind (3.3). It is based on the following equation for wind velocity:

$$\frac{V_z}{V_{10}} = \left(\frac{z}{10}\right)^{1/10}$$

where

$Z$ = Height of area centroid above water level (m),

$V_z$ = Wind velocity at $z$,

$V_{10}$ = Wind velocity at 10 m height.

Table C.3—Wind Velocity Time Factor

| Average Time Period $t$ | Time Factor $a$ |
|---|---|
| 1 hour | 1.000 |
| 10 min. | 1.060 |
| 1 min. | 1.180 |
| 15 sec. | 1.260 |
| 5 sec. | 1.310 |
| 3 sec. | 1.330 |

#### C.4.2.2 Low Frequency Wind Force

As stated in 3.3, wind force can be treated as constant or a combination of a steady component and a time varying component. The time varying component is also known as low-frequency wind force. Similar to the low frequency second order wave forces, low-frequency wind force also induces low frequency resonant surge, sway, and yaw motions. Low-frequency wind forces are normally computed from an empirical wind energy spectrum. Low-frequency wind and wave forces are normally combined to yield low frequency vessel motions due to both effects.

The recommended wind spectrum is presented in Appendix B.

#### C.4.2.3 Wind Force for Large Tankers

Steady wind forces for large tankers can be estimated using the report, *Prediction of Wind and Current Loads on VLCCs*, published by Oil Company International Marine Forum [C.1], as discussed in C.2.2.

## C.5 Oblique Environment

The equations presented are convenient for calculating wind and current forces for bow and beam environments. For environments approaching from an oblique direction, Equation C.8 can be used to evaluate wind and current forces if more accurate predictions are not available.

$$F_\phi = F_x\left[\frac{2\cos^2\phi}{1+\cos^2\phi}\right] + F_y\left[\frac{2\sin^2\phi}{1+\sin^2\phi}\right] \qquad \text{(C.8)}$$

where

$F_f$ = force due to oblique environment, lbs (N),

$F_x$ = force on the bow due to a bow environment, lbs (N),

$F_y$ = force on the beam due to a beam environment, lbs (N),

$f$ = direction of approaching environment (degrees off bow).

## C.6 References

**C.1** Prediction of Wind and Current Loads on VLCCs, Oil Companies International Marine Forum, Second Edition, 1994.

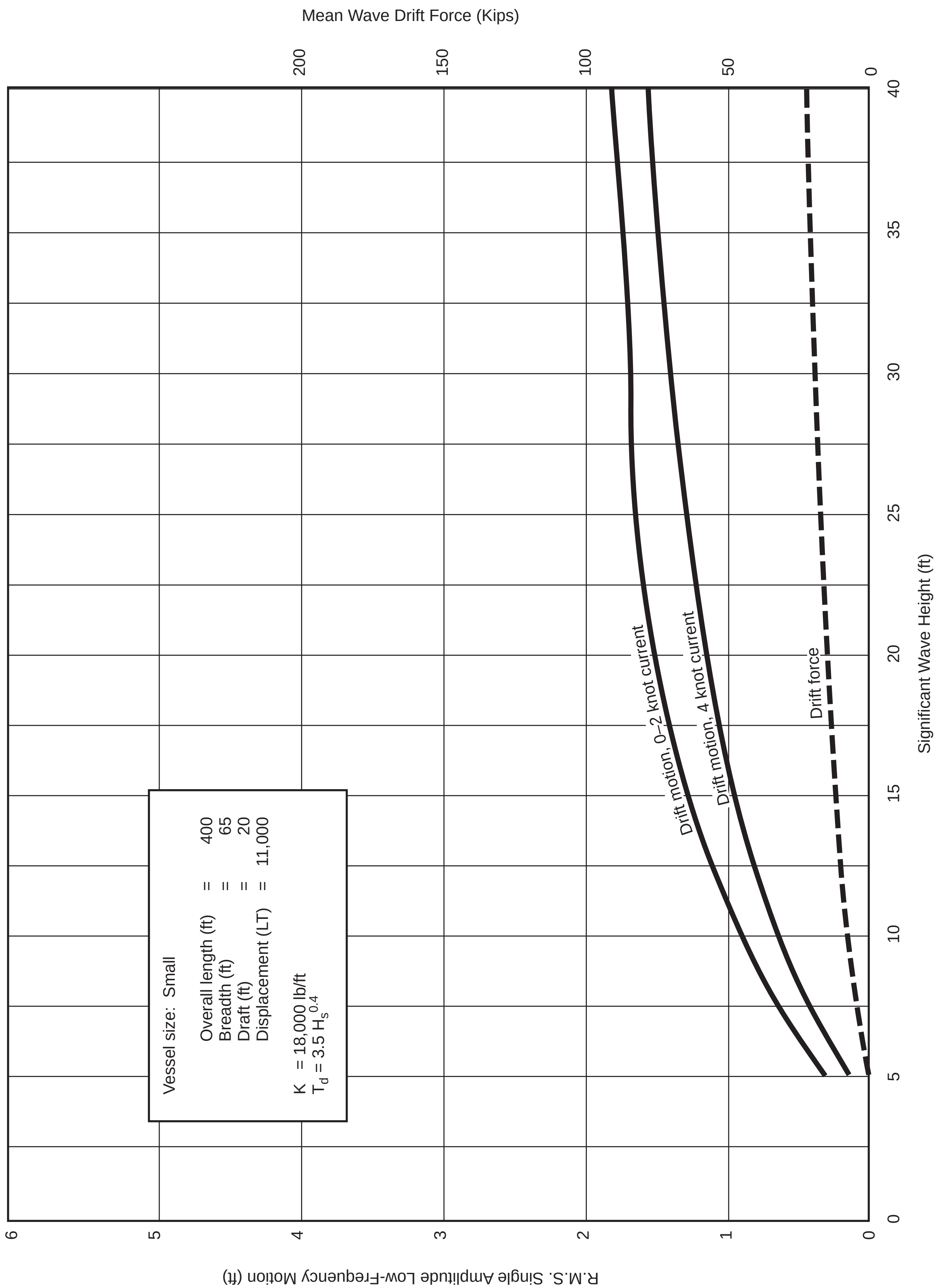



Figure C.3—Wave Drift Force and Motion for Drillships Bow Sea

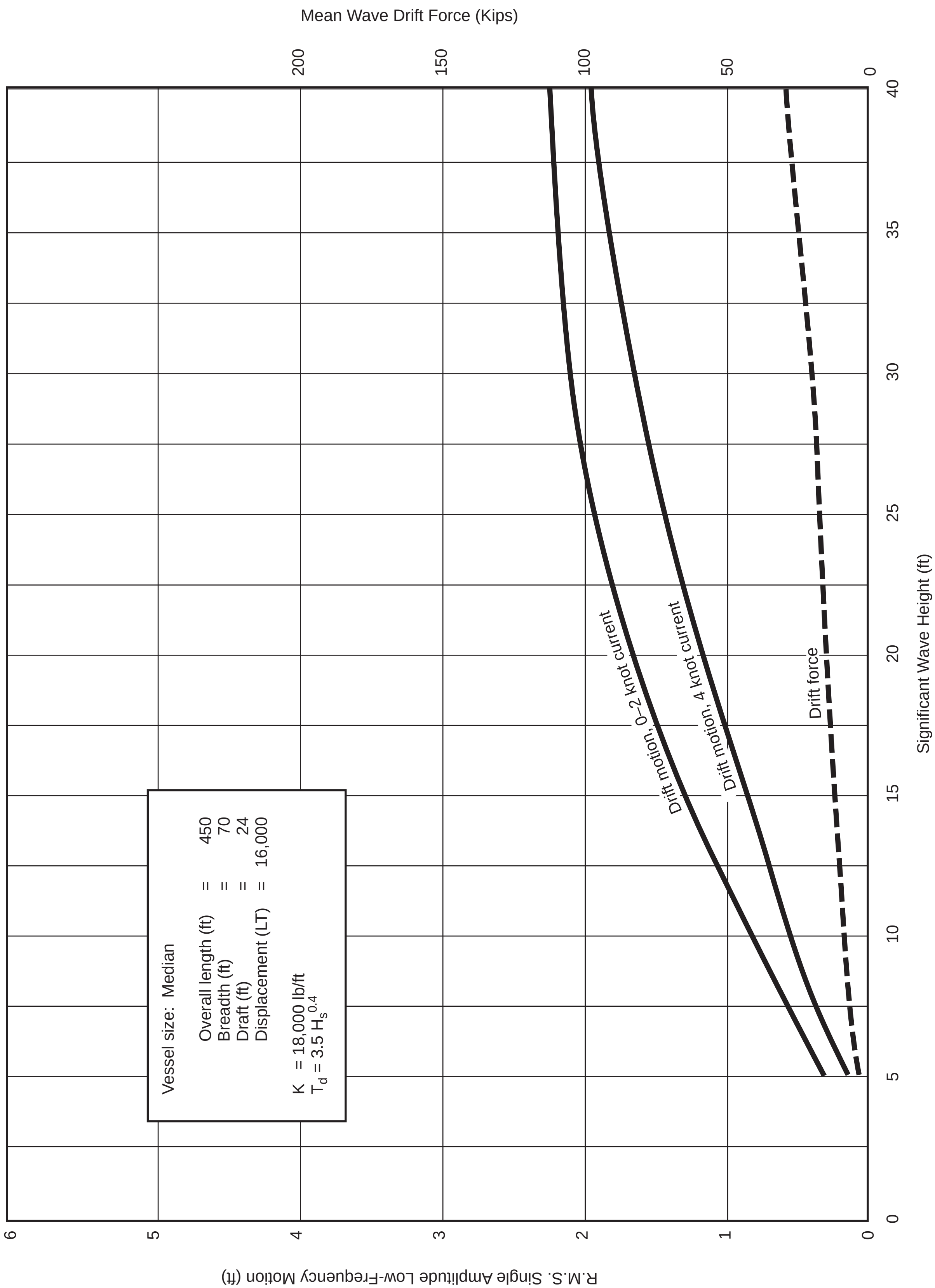



Figure C.4—Wave Drift Force and Motion for Drillships Bow Seas

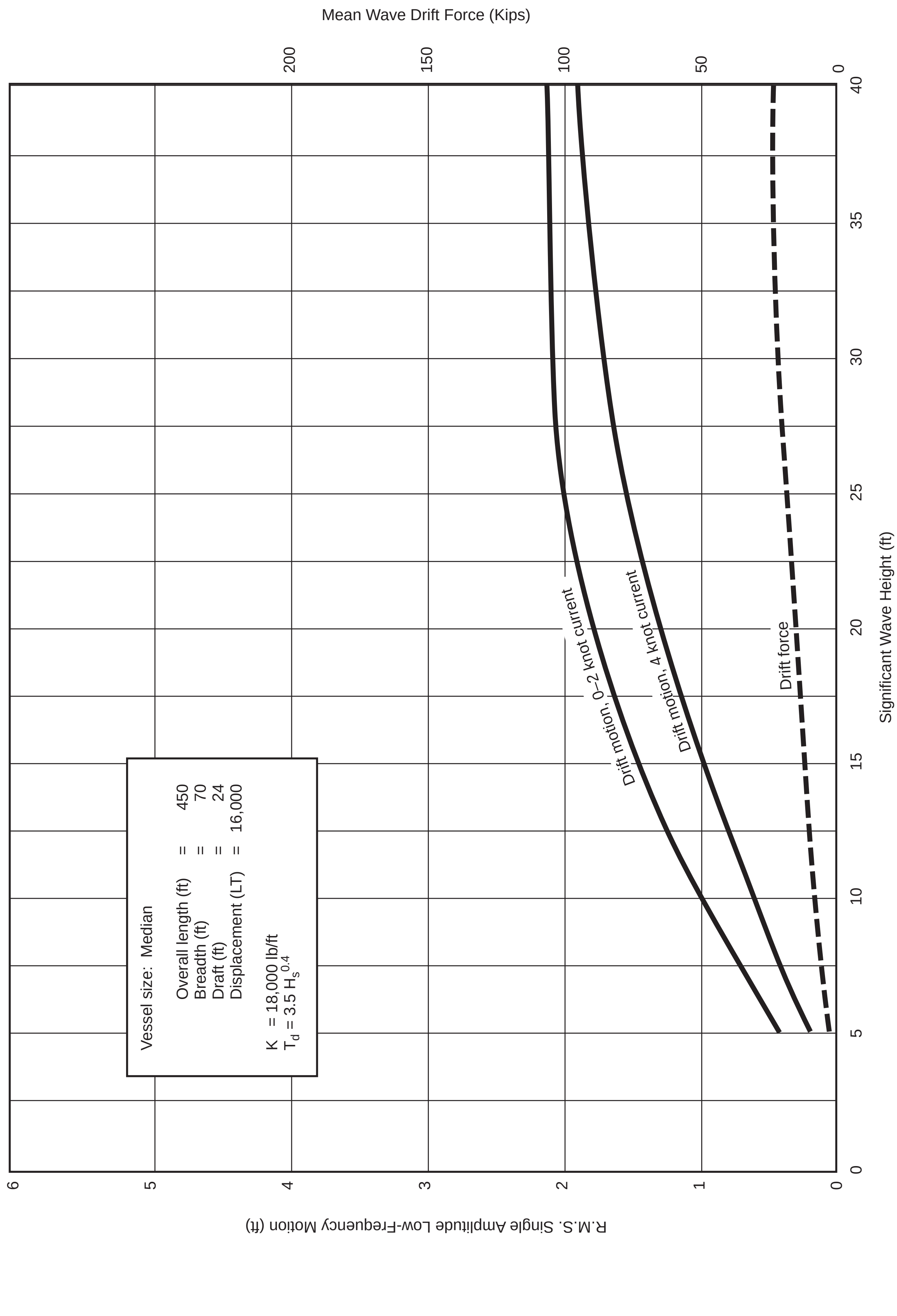



Figure C.5—Wave Drift Force and Motion for Drillships Bow Seas

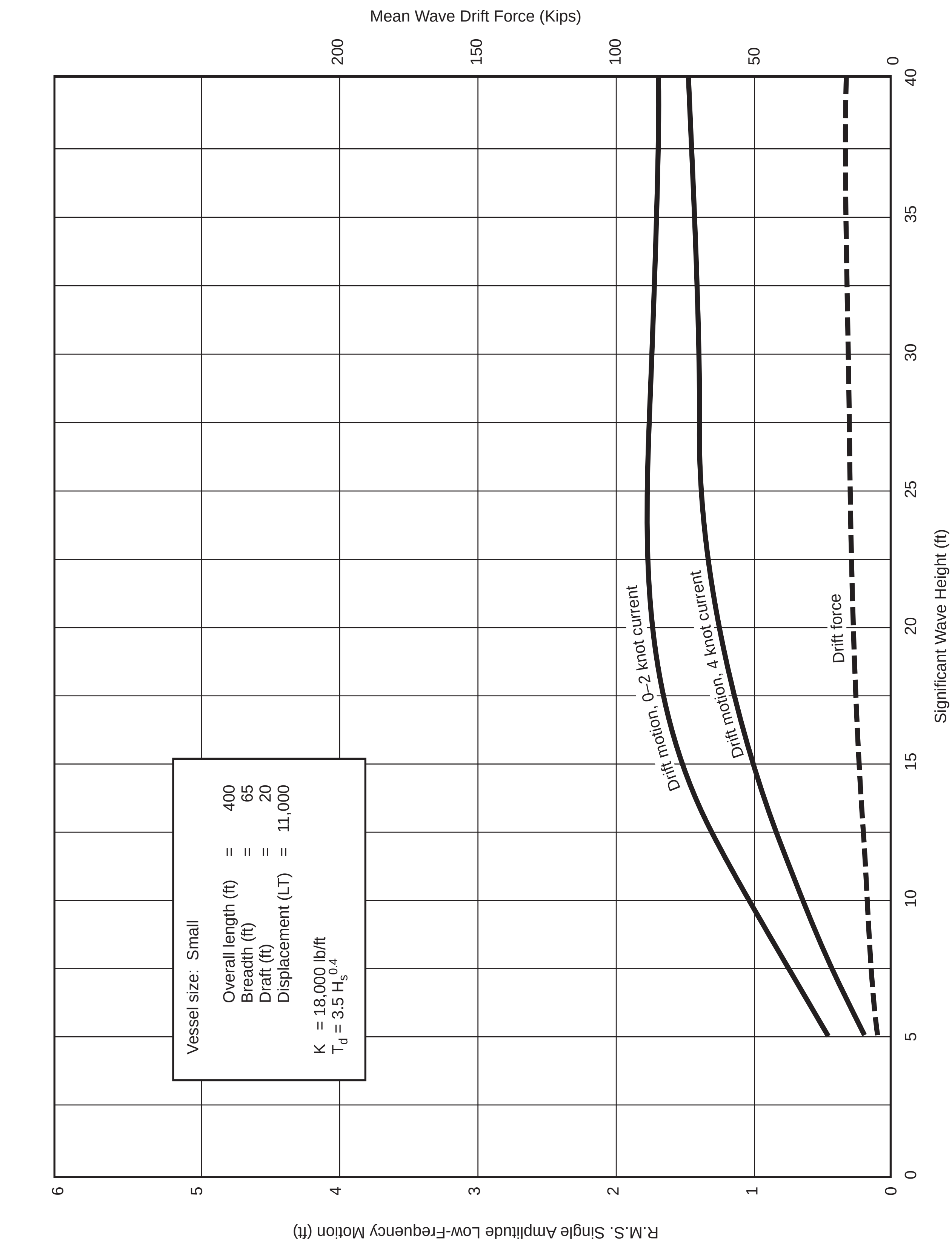



Figure C.6—Wave Drift Force and Motion for Drillships Quartering Seas, Surge

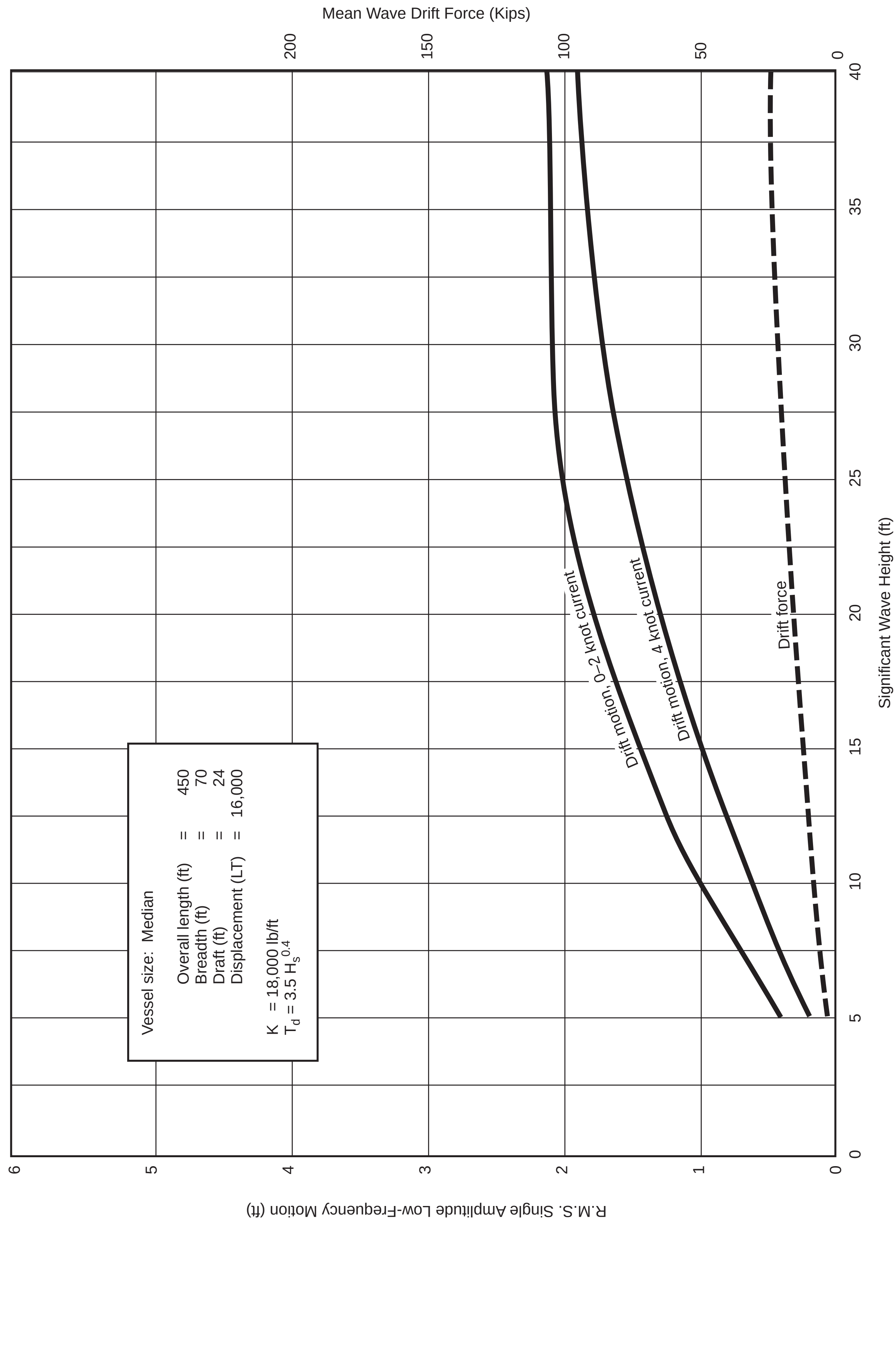



Figure C.7—Wave Drift Force and Motion for Drillships Quartering Seas, Surge

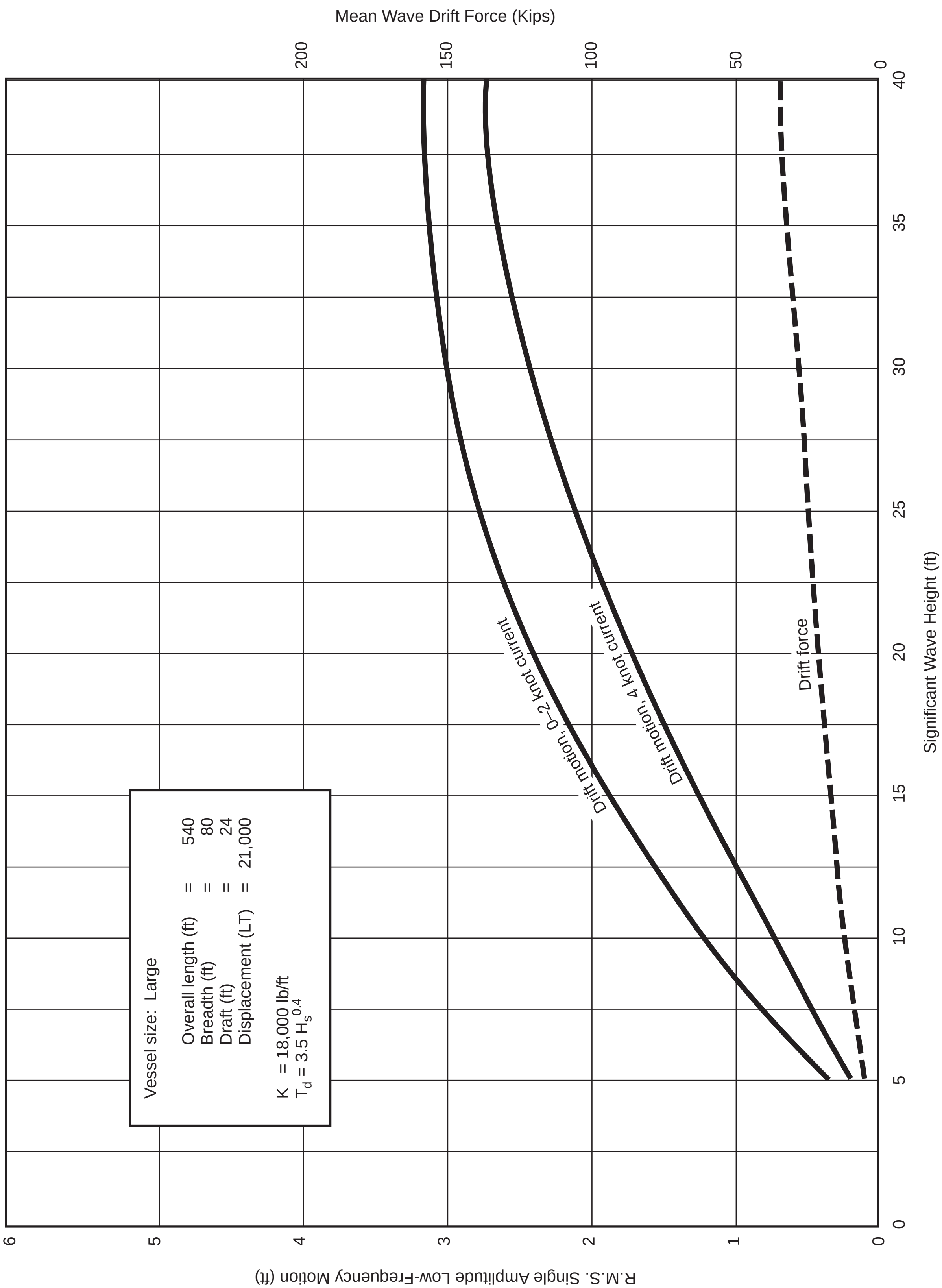



Figure C.8—Wave Drift Force and Motion for Drillships Quartering Sea, Surge

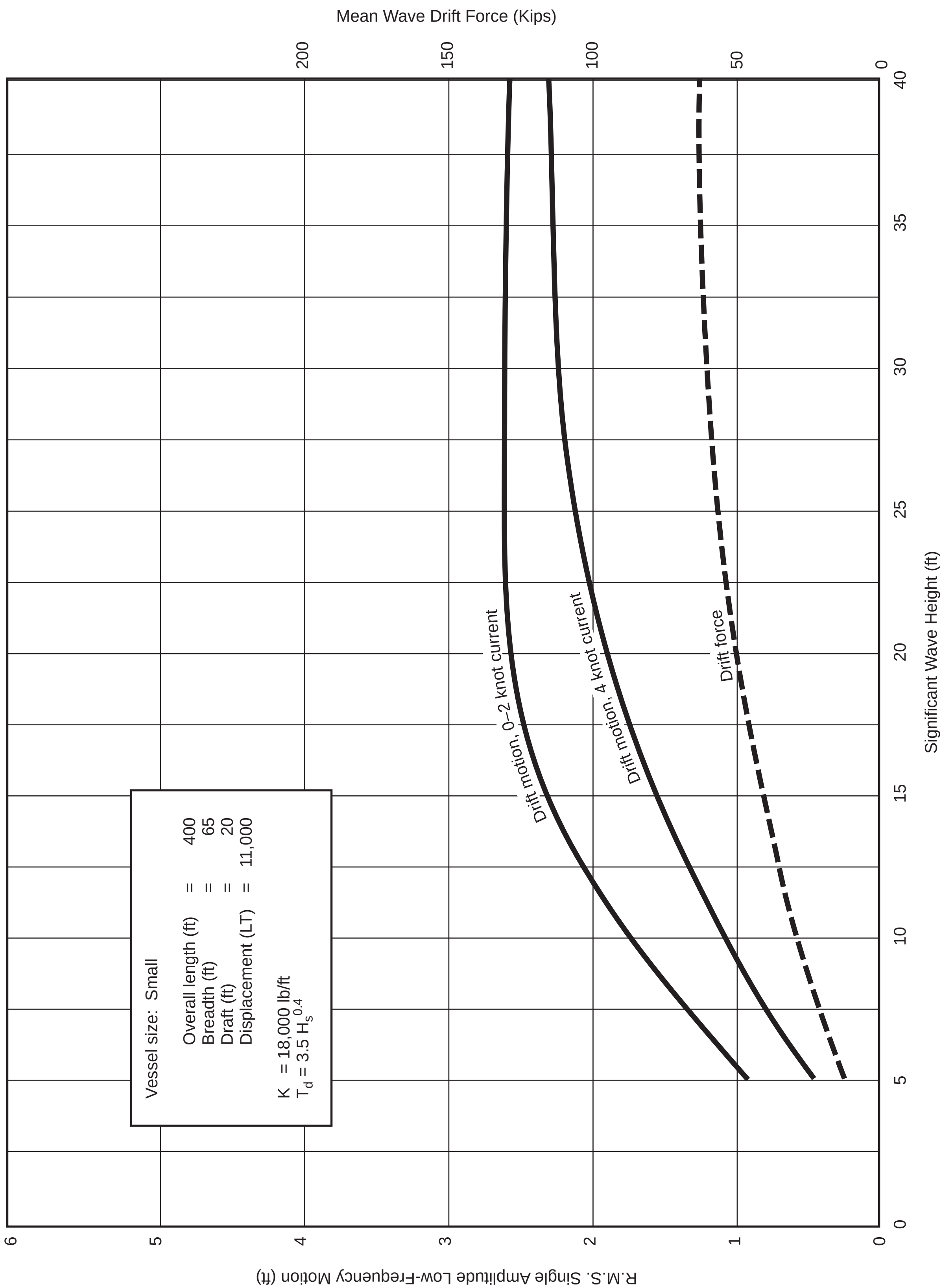



Figure C.9—Wave Drift Force and Motion for Drillships Quartering Seas, Sway

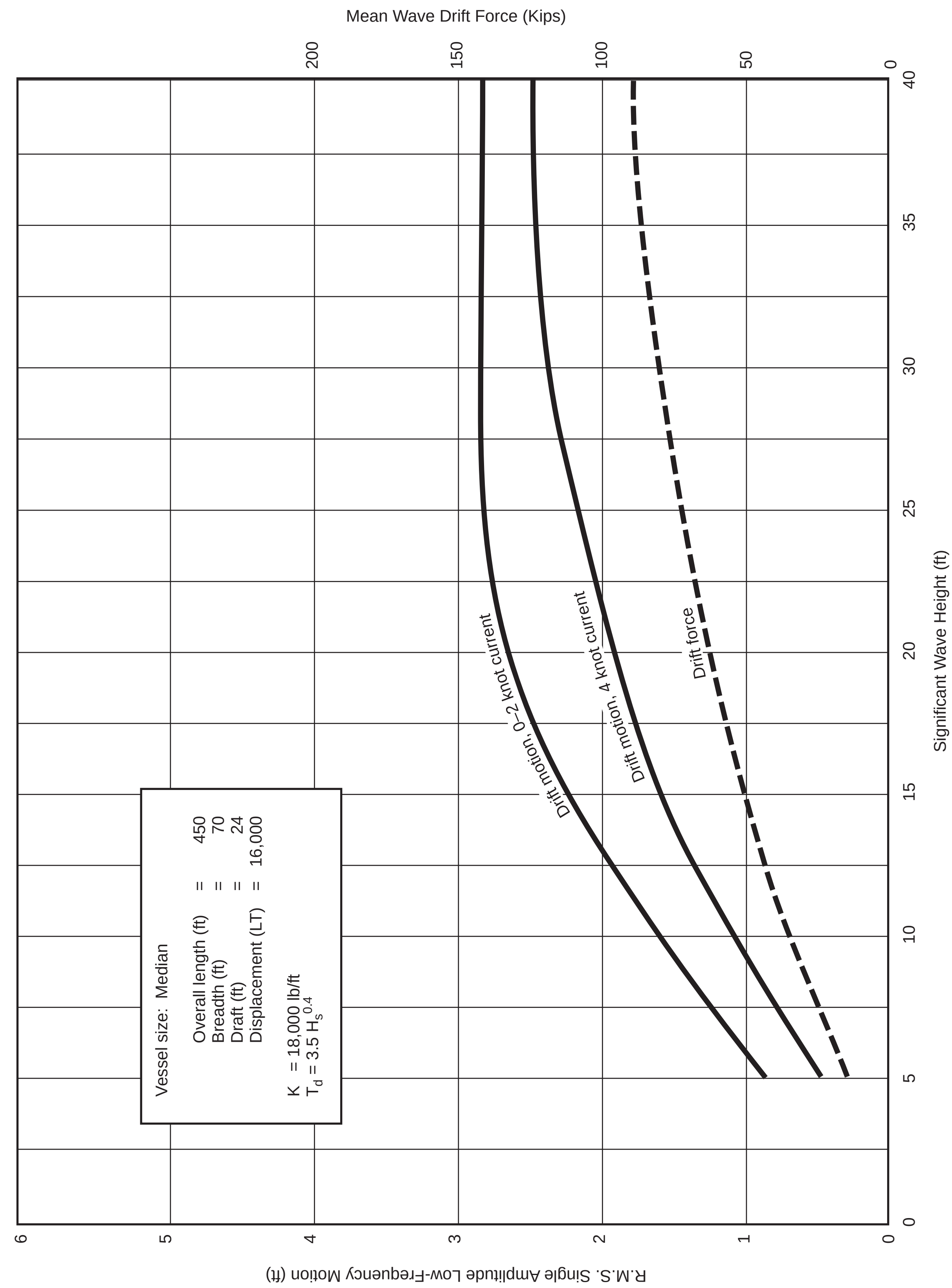



Figure C.10—Wave Drift Force and Motion for Drillships Quartering Seas, Sway

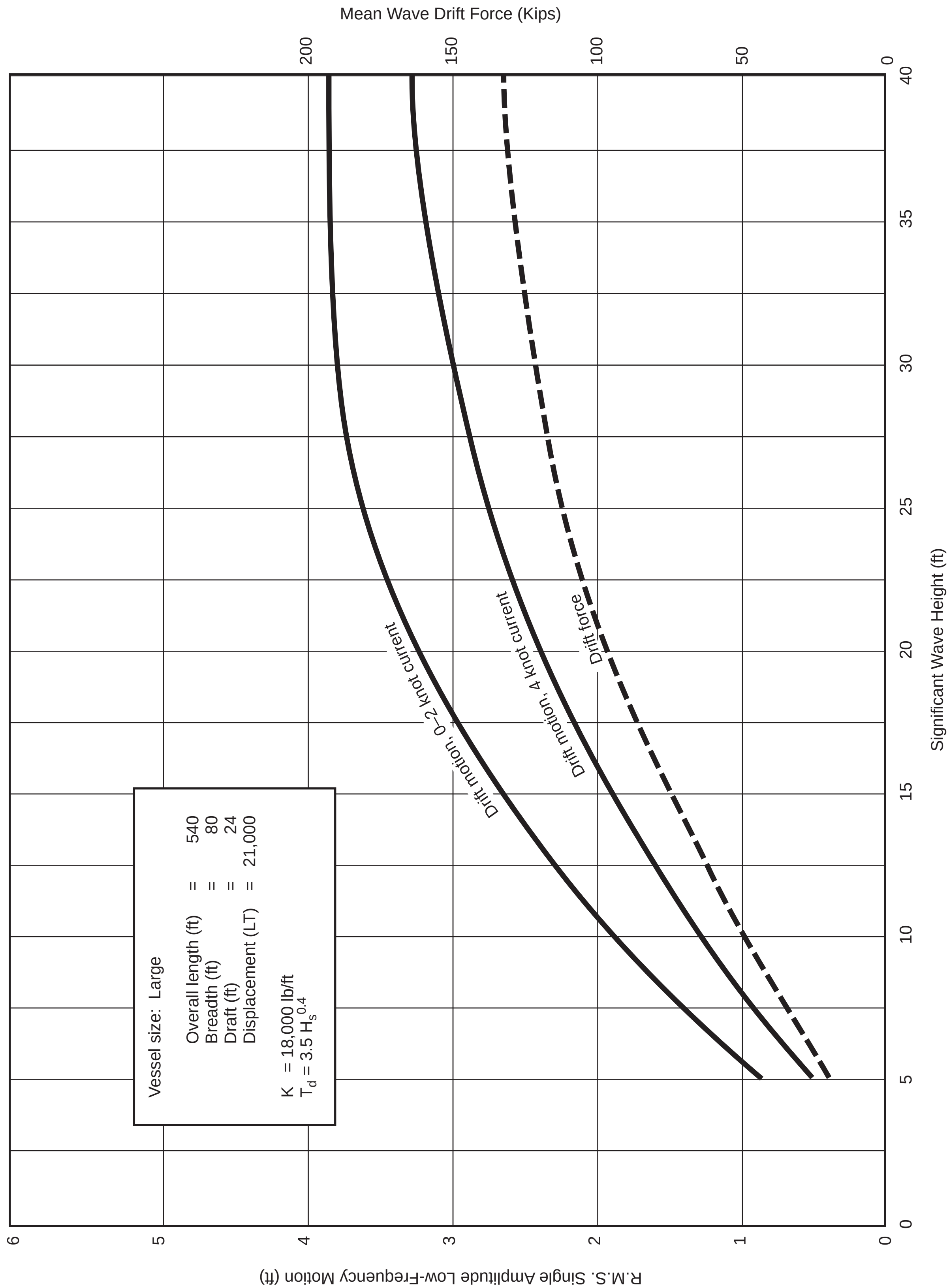



Figure C.11—Wave Drift Force and Motion for Drillships Quartering Seas, Sway

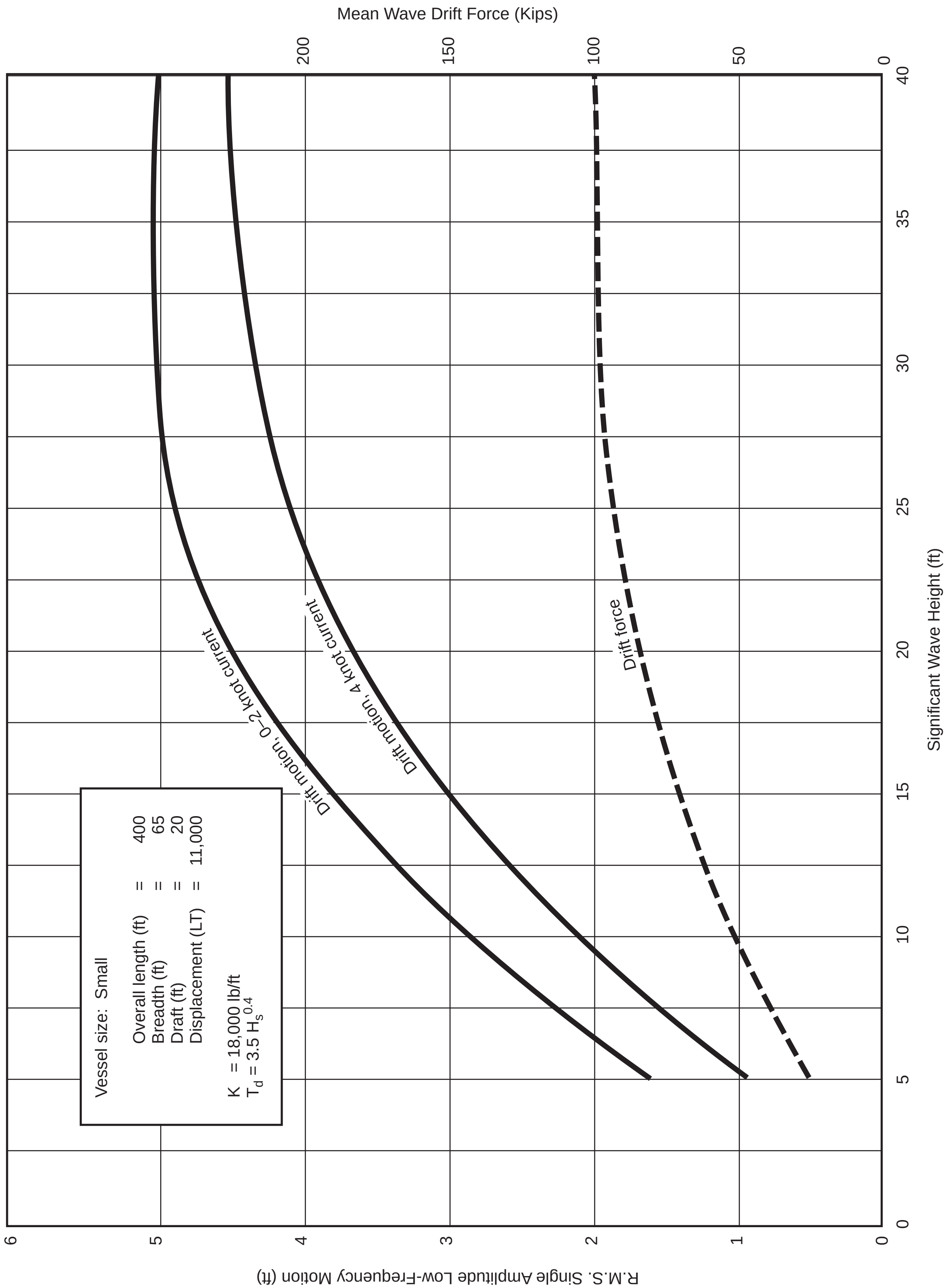



Figure C.12—Wave Drift Force and Motion for Drillships Beam Seas

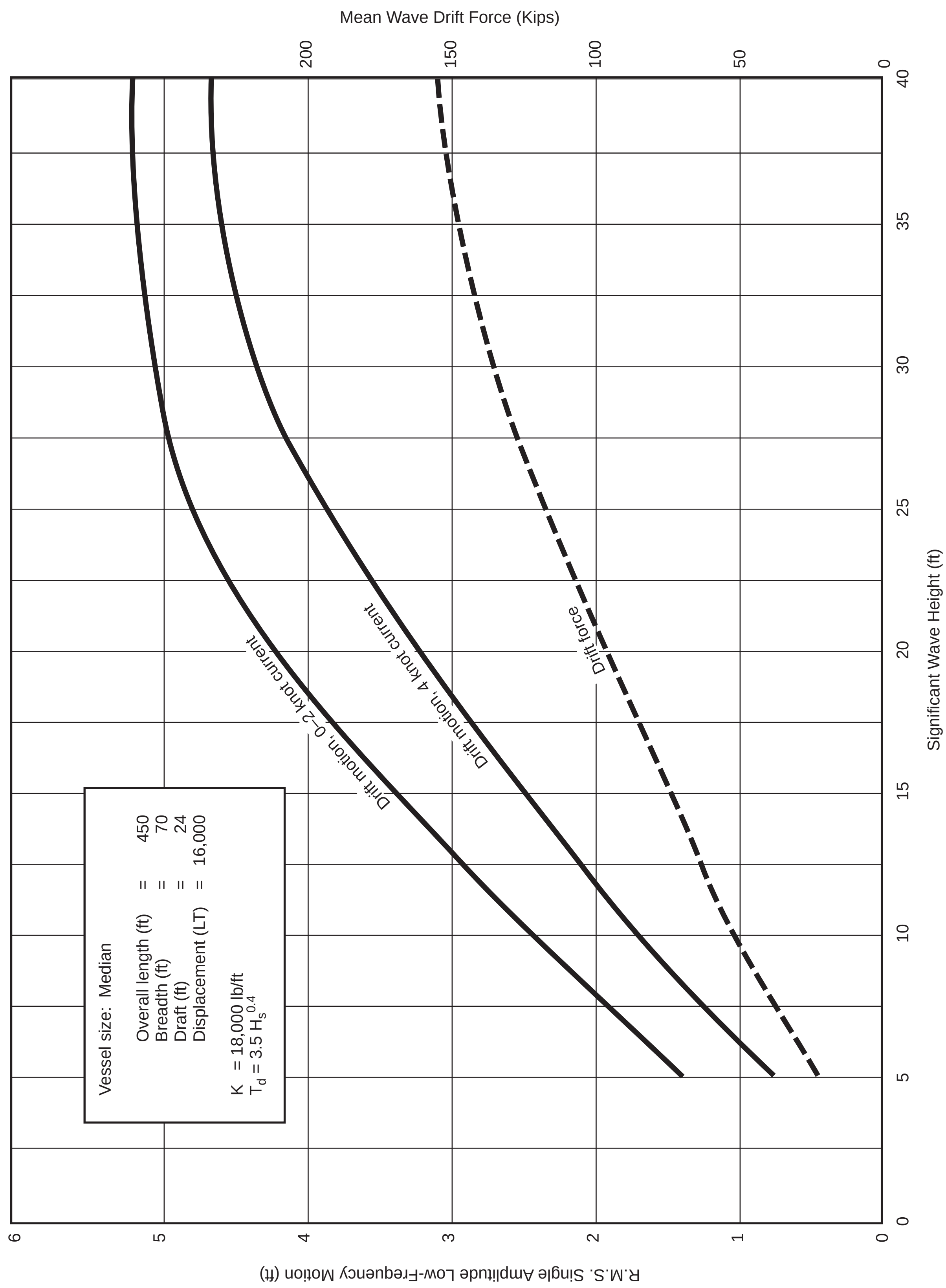



Figure C.13—Wave Drift Force and Motion for Drillships Beam Seas

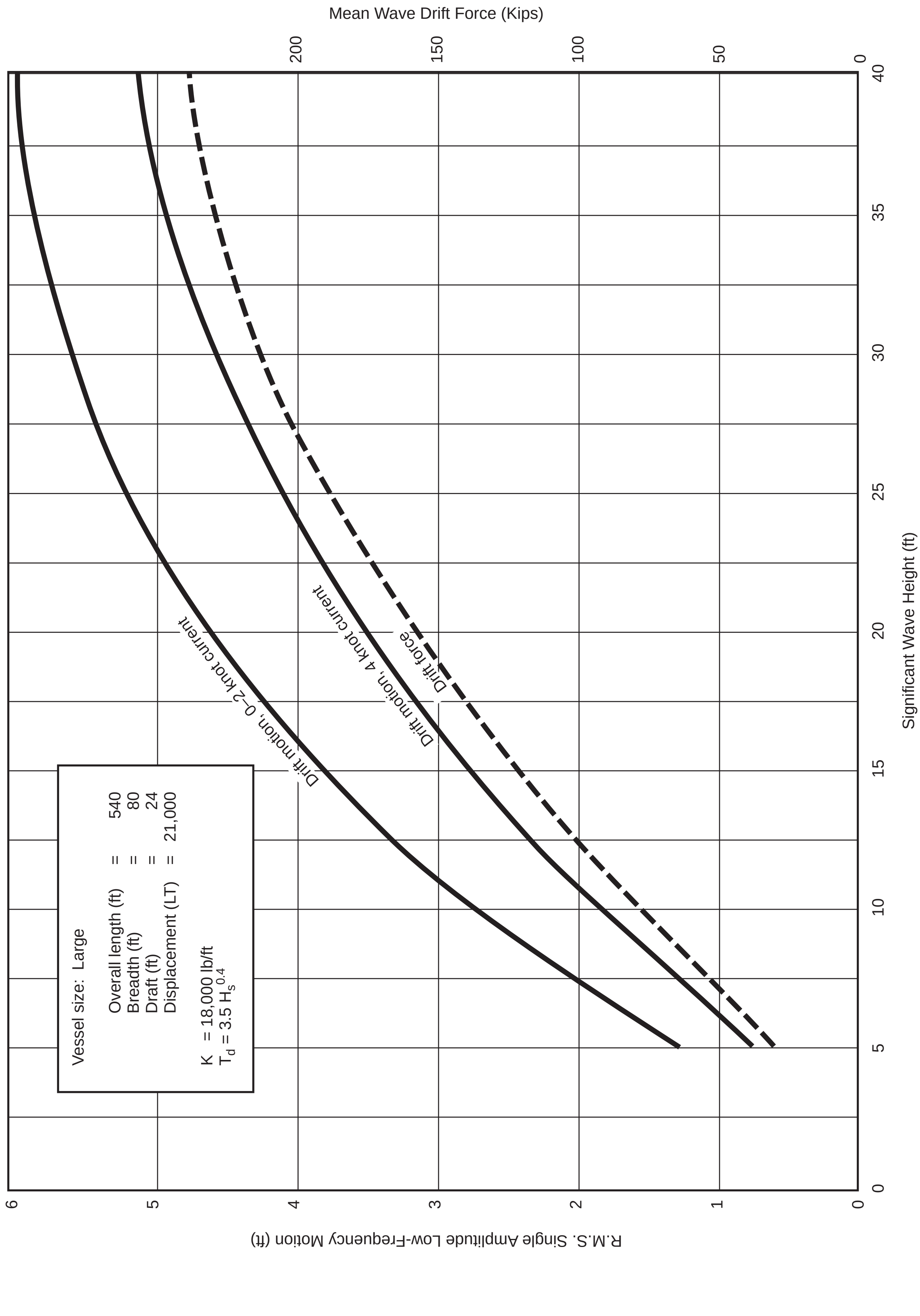



Figure C.14—Wave Drift Force and Motion for Drillships Beam Seas

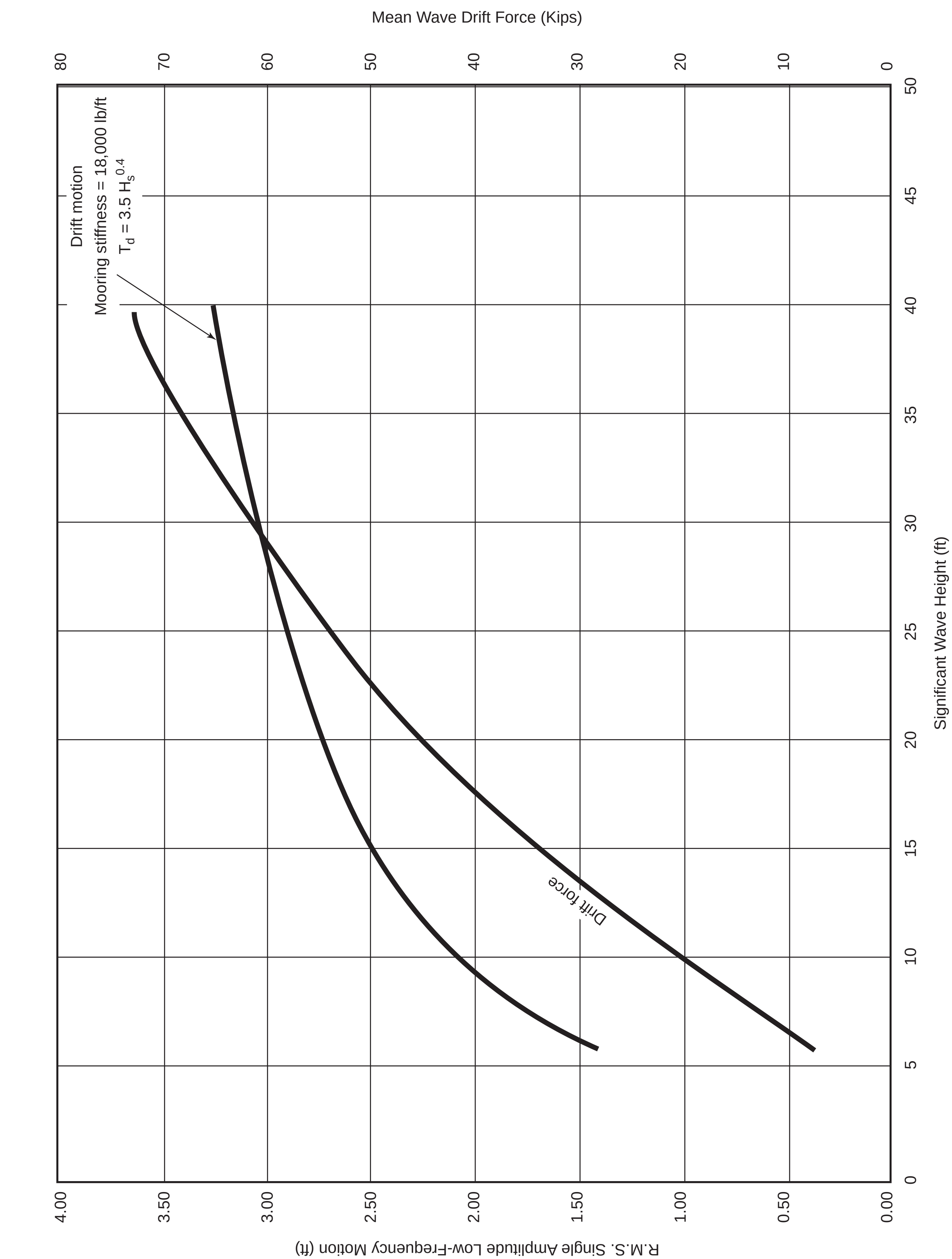



Figure C.15—Wave Drift Force and Motion for Semi-submersibles—Bow Seas

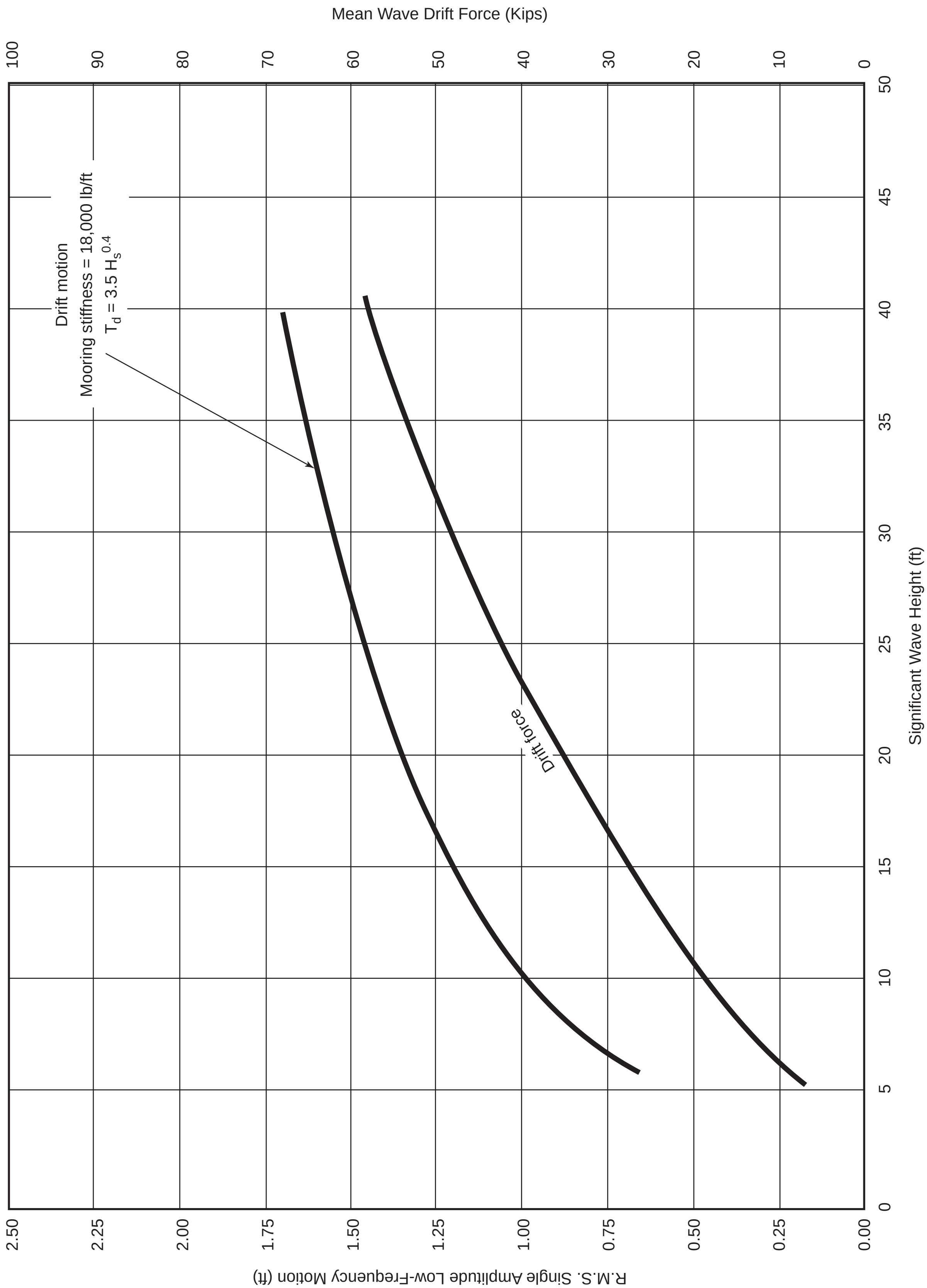



Figure C.16—Wave Drift Force and Motion for Semi-submersibles—Quartering Seas

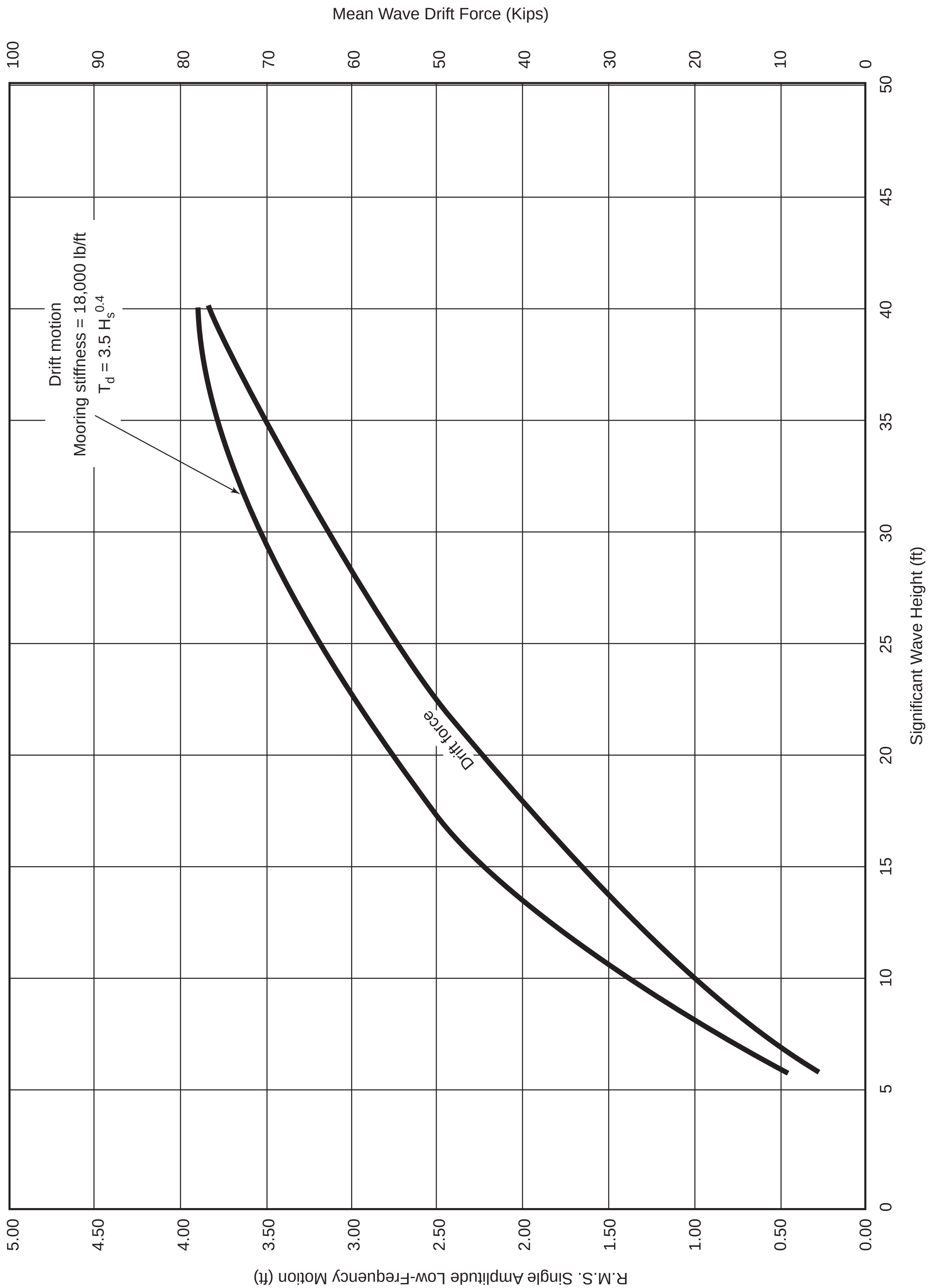



Figure C.17—Wave Drift Force and Motion for Semi-submersibles—Beam Seas

# APPENDIX D—DRAG EMBEDMENT ANCHOR DESIGN

## D.1 Basic Considerations

The holding capacity of a drag embedment anchor in a particular soil condition represents the maximum horizontal steady pull which can be resisted by the anchor at continuous drag. This load includes the resistance to the chain or wire rope in the soil for an embedded anchor, but excludes the friction of the chain or wire rope on the seabed. Drag embedment anchor holding capacity is a function of several factors, including the following:

1. Anchor type—fluke area, fluke angle, fluke shape, anchor weight, tripping palms, stabilizer bars, etc. Figure D.1 shows drag embedment anchors commonly used by the offshore industry.
2. Anchor behavior during deployment—Opening of the flukes, penetration of the flukes, depth of burial of the anchor, stability of the anchor during dragging, soil behavior over the flukes, etc.

Due to the wide variation of these factors, the prediction of an anchor's holding power is difficult. Estimates of anchor holding capacity are normally achieved through empirical approaches, as discussed in the following section. Analytical tools based on limit equilibrium principles for anchor embedment and capacity calculation in soft clay are now available. However, exact holding power can only be determined after the anchor is deployed and test loaded.

## D.2 Anchor Holding Capacity in Typical Soft Clay and Sand

Anchor performance data for the specific anchor type and soil condition should be obtained if possible. In the absence of credible anchor performance data, Figures D.2 and D.3 may be used to estimate the holding power of anchors commonly used to moor floating vessels.

Figures D.2 and D.3 are primarily based on Techdata sheet 83-08R, “Drag Embedment Anchors for Navy Moorings,” Naval Civil Engineering Laboratory, 1987 (Reference D.1), combined with industry anchor test data and field experience. The design curves presented in these two figures represent in general the lower bounds of the test data. The tests used to develop the curves were performed at a limited number of sites. As a result, the curves are for use in generic soil types such as “soft clay” and “sand”. Industry studies indicate, however, that several parameters such as soil strength profile, lead line type (wire rope versus chain), cyclic loading, and anchor soaking may significantly influence anchor performance in soft clay. Also some high efficiency anchors have demonstrated substantial resistance to vertical load in soft clay. Furthermore, there are new versions of high efficiency anchors which are not covered by these two Figures. These issues are addressed in the following sections which are based on centrifuge and full scale tests, analytical investigations, and field experience. A significant portion of the information is from Reference D.2 which presents the results of a centrifuge test program and an analytical investigation using limit equilibrium analysis. The study was conducted for two high efficiency anchors—Bruce FFTS Mark III and Stevpris Mark III. It is uncertain whether the results are applicable to conventional anchors such as Moorfast or LWT.

## D.3 Effect of Soil Shear Strength Gradient in Soft Clay

According to U.S. Navy’s publication [Reference D.1], the generic design curves in Figure D.2 are valid for soft clays with shear strength gradients ranging from 8 psf/ft. to 16 psf/ft. A study [Reference D.2] indicates that soil shear strength gradient may influence anchor holding capacity. Adjustment of anchor holding capacity may be justified if the soil under consideration is significantly different from the soils in which the anchor tests were performed. The U.S. Navy’s data were obtained from anchor tests carried out in soft clays with shear strength gradients of about 9 to 13 psf/ft. If the soil under consideration has a similar shear strength gradient, no adjustment is necessary. Otherwise, adjustment of anchor holding capacity according to the shear strength gradient should be considered.

Limited analytical and centrifuge test data indicate that anchor holding capacity tends to vary more or less linearly with the shear strength gradient value. However, this has not been fully verified by field tests and deviations from the linear distribution have been observed in the centrifuge test data, depending on anchor size and soil strength profile. Therefore the adjustment of anchor holding capability is not a simple exercise. It requires judgment from the designer. Reference D.2 provides some data and a discussion on limit equilibrium analysis which may be useful for this exercise.

## D.4 Effect of Lead Line Type in Soft Clay

The design curves in Figure D.2 are based on testing of anchors connected to chain. Field tests and analytical studies indicate that when the lead line is wire rope, an anchor may yield significantly higher holding capacity in soft clay. The anchor was found to penetrate deeper into stronger soil. For the limited cases studied, anchor connected to wire rope provides 15% to 40% higher holding capacity than the same anchor connected to chain. This is in good agreement with the results from a full scale test program. It should be noted that the studies have been limited to high efficiency anchors in soft clay with a fairly constant shear strength gradient.

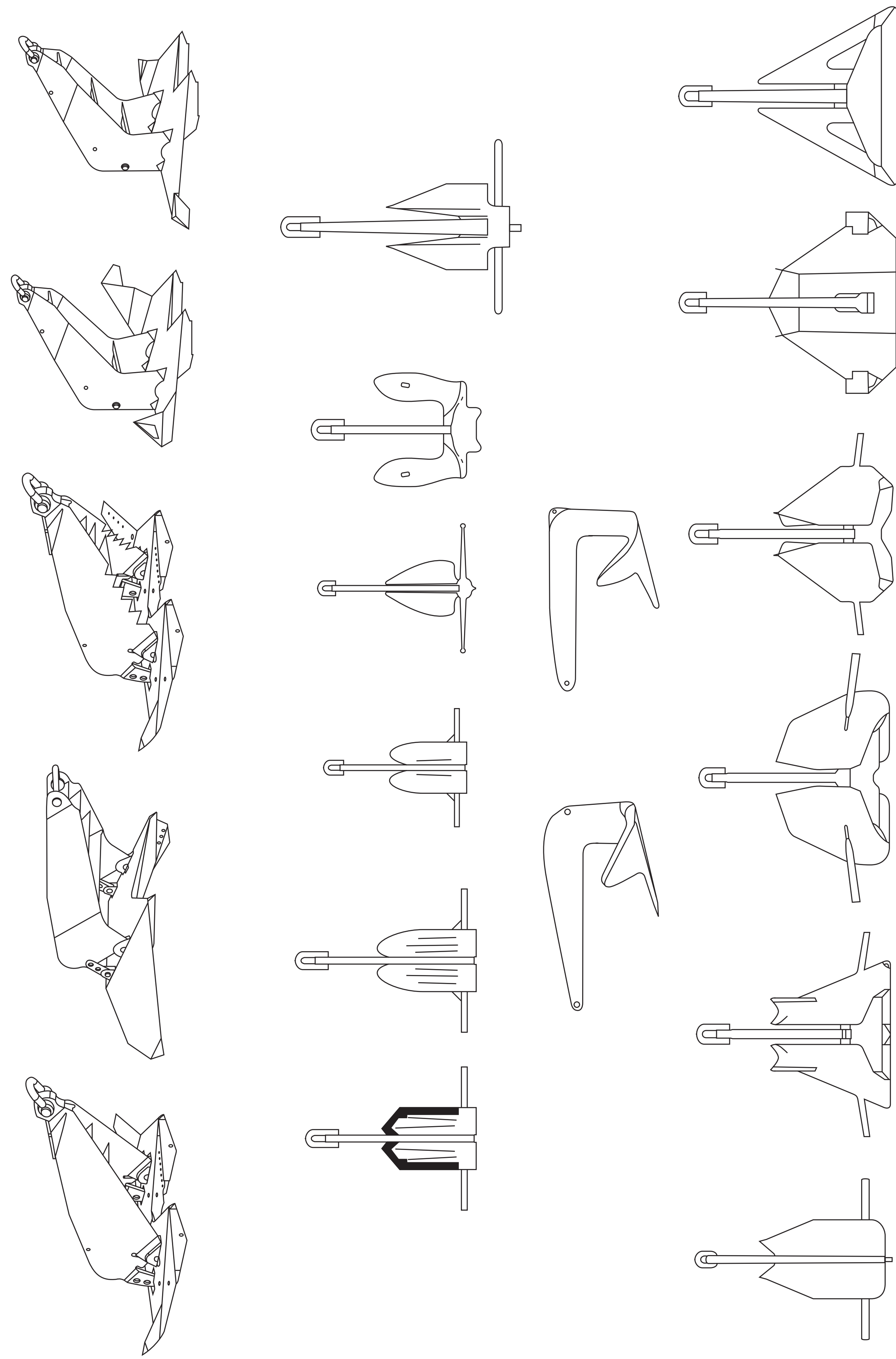
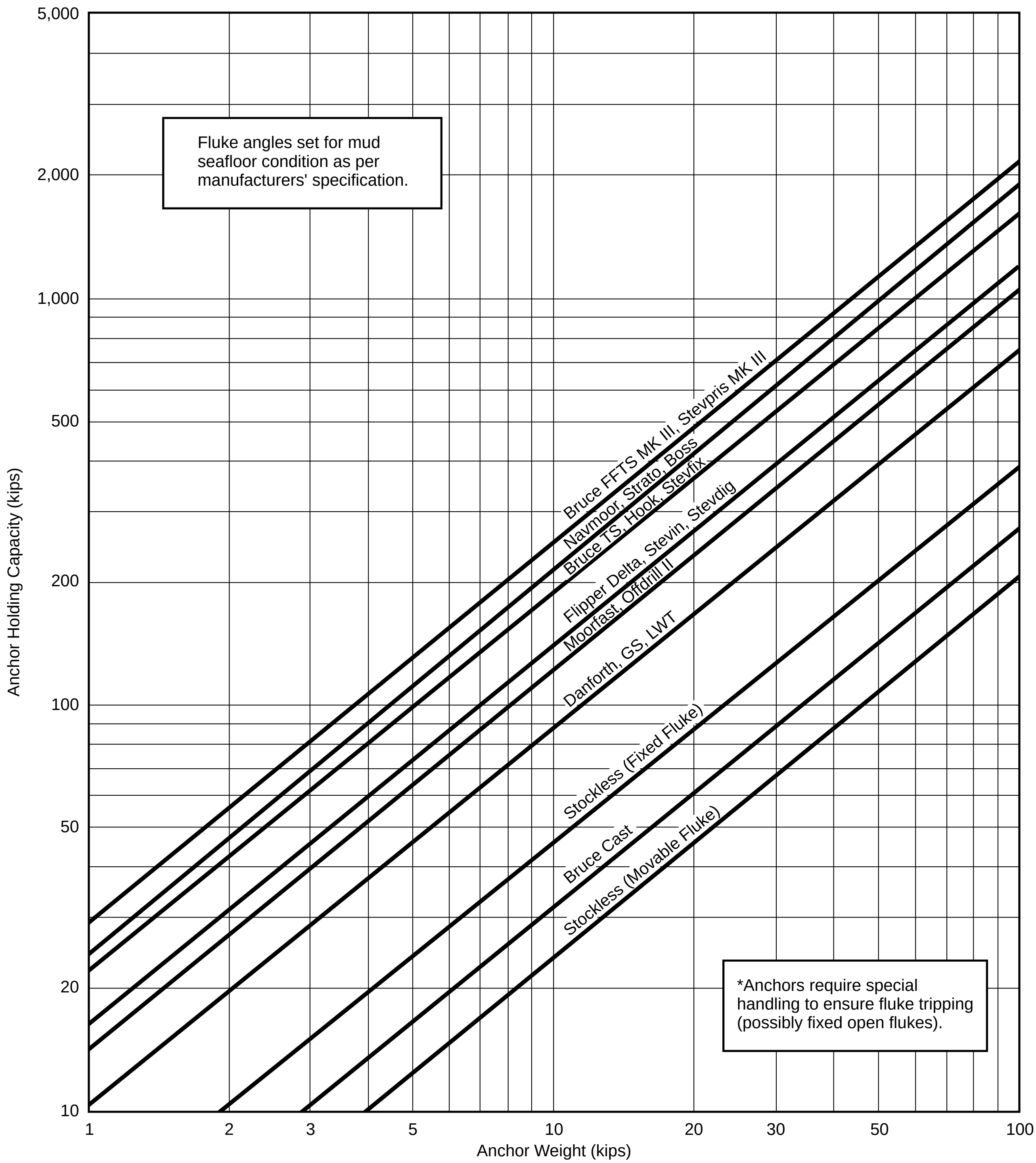
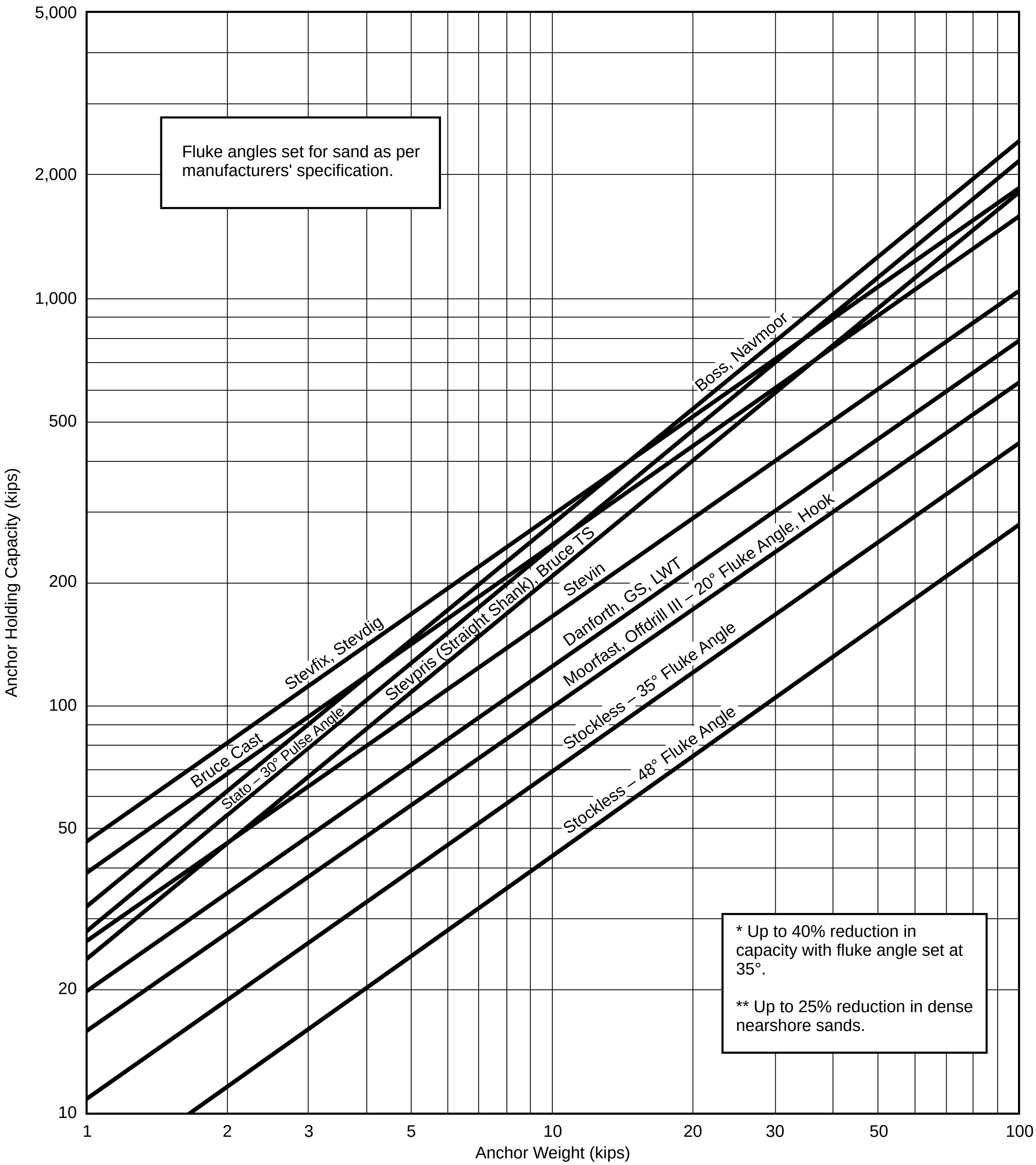


Figure D.1—Typical Drag Embedment Anchors




Note: This figure was reproduced from Techdata Sheet 83, "Drag Embedment Anchors for Navy Moorings," Naval Civil Engineering Laboratory, 1987, except that the holding capacity curves for the Moorfast (or Offdrill II) and the Stevpris anchor were upgraded. The upgrading of these two curves was based on model and field test data and field experience acquired in recent years. The design curves in the figure represent in general the lower bounds of the test data. They reflect data valid for anchor designs as of 1987. New anchor designs have since been developed. However, performance data for these new designs were insufficient and therefore their design curves were not included. Some guidelines for the performance evaluation of late anchor models are provided in D.9. The design curves do not include a factor of safety.

Figure D.2—Anchor Holding Capacity in Soft Clay




Note: This figure was reproduced from Techdata Sheet 83, "Drag Embedment Anchors for Navy Moorings," Naval Civil Engineering Laboratory, 1987. The design curves in the figure represent in general the lower bounds of the test data. They reflect data valid for anchor designs as of 1987. New anchor designs have since been developed. However, performance data for these new designs were insufficient and therefore their design curves were not included. The design curves do not include a factor of safety.

Figure D.3—Anchor Holding Capacity in Sand

## D.5 Effect of Cyclic Loading in Soft Clay

The effect of cyclic loading was studied by centrifuge testing [Reference D.2]. Three tests were performed at 32.2 g's simulating 1.1-kip prototype Stevpris anchors. The first test was a steady pull test performed to establish a baseline static anchor capacity. The subsequent cyclic tests began with a steady pull that approached the anchors' capacity, followed up with a series of loading cycles, and ended with a steady pull to establish the post-cyclic capacity. A sample load-displacement curve is shown in Figure D.4. It was found that cyclic loading did not damage the anchor's capacity. Cycling actually improved anchor capacity by about 25 to 50 percent, by causing the anchors to dig deeper and become embedded in stronger soil.

Also there are studies indicating that anchors can resist higher maximum cyclic loads than static loads. Although the studies were conducted for pile anchors, we expect similar conclusions for drag embedment anchors.

Based on above discussion and other industry experience for anchors in soft clay, the mooring test load requirement for permanent moorings (Section 7.4.3) should be test loaded to at least 80% of the maximum storm load determined by a dynamic analysis for the intact condition.

## D.6 Effect of Anchor Soaking in Soft Clay

A "soaked anchor" is one which has been embedded for some time. In normally consolidated clays, the effect of anchor soaking presumably is to increase the anchor's capacity. This occurs because the disturbed soil that surrounds the anchor consolidates with time and thereby gets stronger.

Anchor soaking was also studied by centrifuge testing. The tests were performed for 1.1-kip and 15-kip prototype Stevpris anchors. These tests typically were performed by pulling the anchor to capacity permitting it to rest in the soil for about an hour, and then resuming the pull until the ultimate capacity was achieved. In this case, the one-hour wait in the centrifuge is equivalent to a wait in the field of about 247 days. A sample load-displacement curve for the 15-kip prototype Stevpris anchor is shown in Figure D.5. As is seen, the anchor soaking produced a substantial increase in capacity upon re-initiation of movement, but this benefit quickly disappeared as movement continued. The anchor soaking effect therefore appears to be localized to points near where the anchor initially rested within the soil, and is not dependable if there is any possibility that the anchor could be loaded beyond its capacity resulting in dragging during subsequent loading.

## D.7 Capacity in Soft Clay Under Inclined Line Loading

In the centrifuge test program, the Stevpris Mark III and Bruce FFTS Mark III anchors were pulled such that, at the end of the tests, the pulling line penetrated the mudline at an angle of about 30°. The maximum holding capacity reached earlier was maintained at this angle. This leads to the conclusion that these anchors are able to sustain significant vertical

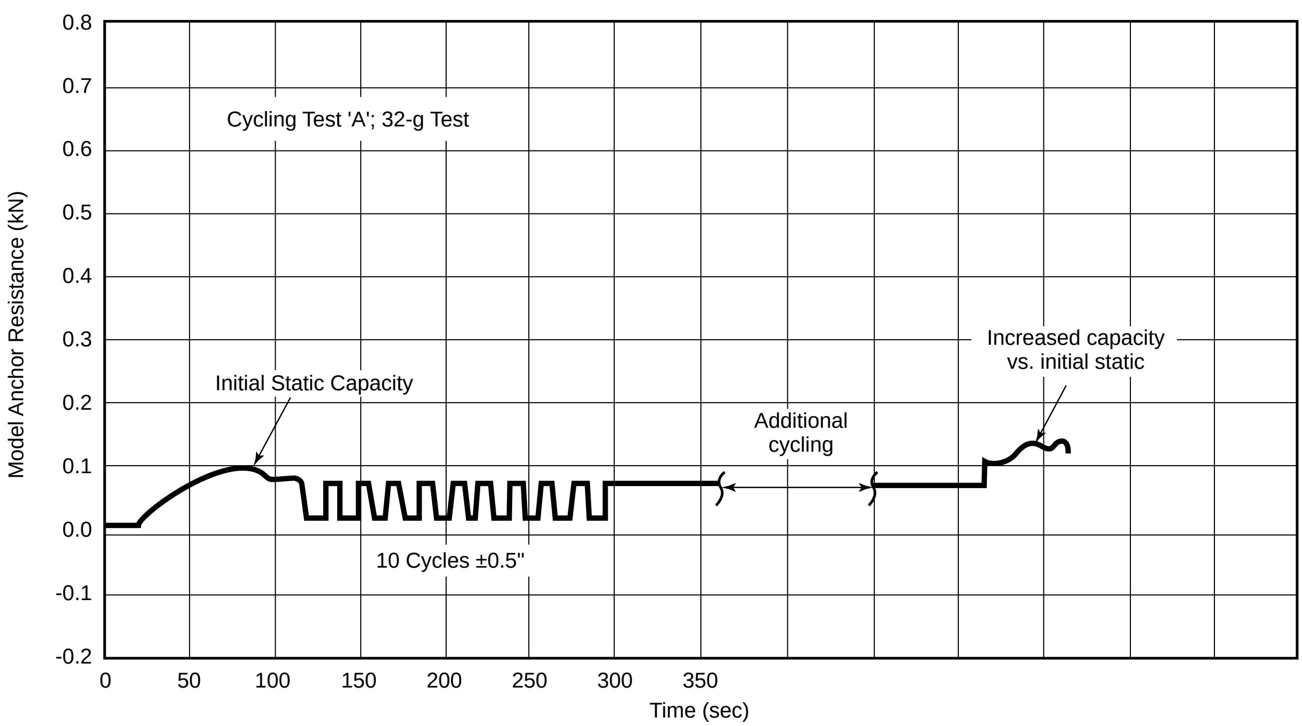



Figure D.4—Effect of Cyclic Loading

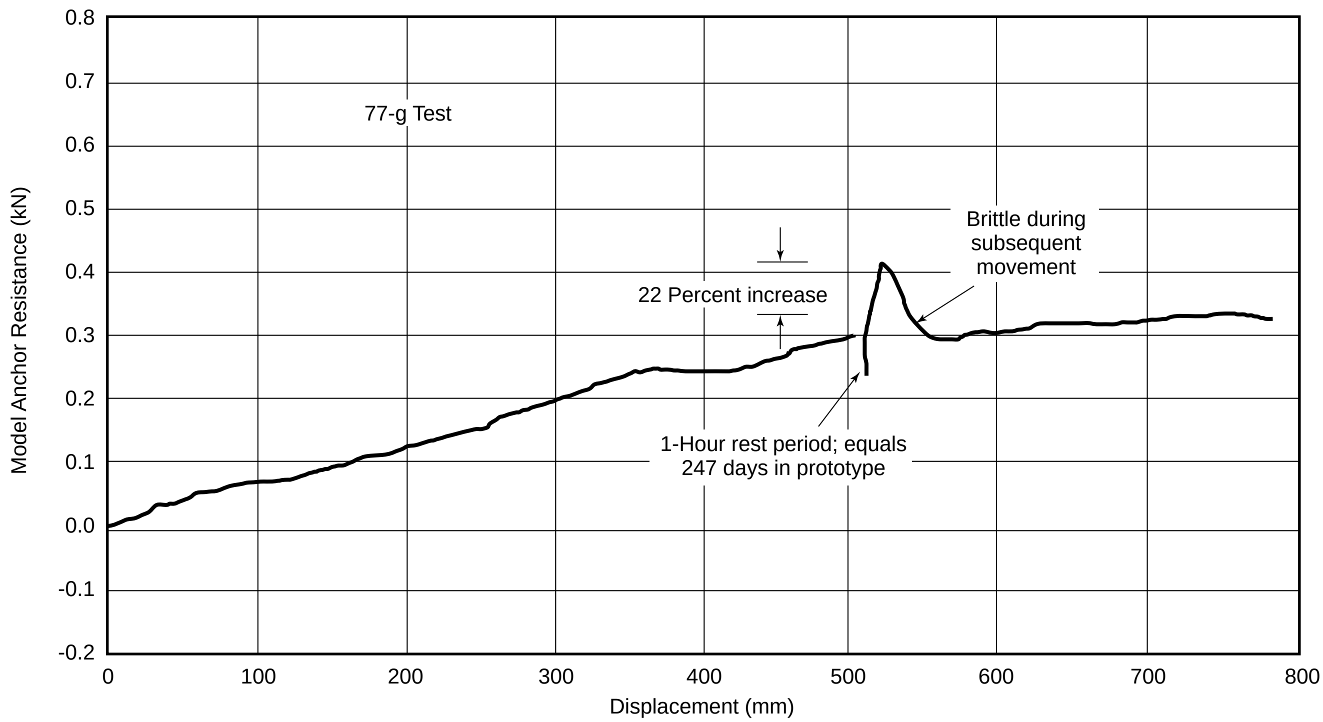



Figure D.5—Effect of Anchor Soaking

load in soft clay. Full scale tests and field experience also support this conclusion. There were reports that the operation personnel had difficulties to retrieve a high efficiency anchor after the completion of a drilling operation. In some cases the pendant line or the chaser line for anchor retrieval broke. In other cases the anchors were retrieved by vertical force from the mooring line in excess of 25 times the anchor weight. In a full scale anchor test program carried out in the Gulf of Mexico, the pull out forces in the pendant line for the test anchors were close to their holding capacities.

Significant evidence is present to support the allowance of vertical loads on some drag embedment anchors. Following are the guidelines for drag embedment anchors subject to vertical loads.

1. Vertical loads are applied to anchors under extreme environment only. Drag embedment anchors should not be subjected to vertical loads under normal operating environments.
2. It is applicable only to certain high efficiency anchors for which sufficient research has been conducted and much field experience has been gained.
3. The anchors are deployed in soft clay where deep penetration is expected. This may exclude certain operations with mobile moorings where the soil conditions have not been thoroughly investigated or the anchor test load is insufficient to ensure deep penetration.
4. The maximum line angle at the mud line (including the effect of wave and low frequency vessel motions) should be less than 20° under the maximum design environment for the intact and damaged condition. This angle should be zero at the early stage of test loading to ensure anchor penetration. Furthermore, the holding capacity should be reduced by a factor *R*, which is a function of the angle at the mudline and takes into account the reduced friction due to shorter embedded line length. For example the following *R* values may be applicable for Bruce FFTS Mark IV and Stevpris Mark V anchors:

| Mudline Angle (Deg.) | 0 | 5 | 10 | 15 | 20 |
|---|---|---|---|---|---|
| R | 1.0 | 0.98 | 0.95 | 0.89 | 0.81 |

The factor of safety should still be satisfied after reduction of holding capacity.

There are cases for permanent moorings where a taut-leg mooring spread may be utilized. In such a configuration, the mooring lines will have an initial angle to the seabed and will always have vertical and horizontal loads imposed at the anchor. It has been common practice to use pile or plate anchors to resist these loading conditions, and design guidelines for these anchors are provided in Appendix E.

## D.8 Anchor Drag Distance and Penetration Depths in Soft Clay

Anchor drag distance and penetration depth estimates from Reference D.1 are presented in Figure D.6 and Table D.1. Again, this information is valid for chain lead line and a shear strength gradient of 9 to 13 psf/ft. Deviation from this may affect these values, especially the penetration depth estimates.

Table D.1—Estimated Maximum Fluke Tip Penetration

| Anchor Type | Normalized Fluke Tip Penetration (Fluke Lengths): Sands/Stiff Clays | Normalized Fluke Tip Penetration (Fluke Lengths): Mud (e.g., Soft Silts and Clays) |
|---|---|---|
| Stockless | 1 | 3* |
| Moorfast<br>Offdrill II | 1 | 4 |
| Boss<br>Danforth<br>Flipper Delta<br>GS (Type 2)<br>LWT<br>Stato<br>Stevfix | 1 | $4^1/_2$ |
| Stevpris MK III<br>Bruce FFTS MK III<br>Bruce TS<br>Hook<br>Stevmud | 1 | 5 |

*Fixed fluke stockless.

## D.9 New Anchor Design

Driven by economics and frontier development, new anchor designs and improvements to existing anchors continue to evolve. For example, Bruce FFTS Mark IV and Stevpris Mark V are advanced versions of the Bruce FFTS Mark III and Stevpris Mark III anchors, for which design curves are presented in Figures D.2 and D.3. Also other new anchor designs have become available. Anchor test data and service experience are insufficient for these new designs and therefore design curves cannot be presented at this point. Studies have shown, however, that the efficiency of the new version is improved through improving the fluke area to weight ratio. Based on this and in the absence of better information, the holding capacities of these new versions can be estimated using the following equation:

$$H_n = H_s \cdot (A_n/A_s)^n \qquad \text{(D.1)}$$

where

$H_n$ = holding capacity of new version,

$H_s$ = holding capacity of reference (for example Bruce FFTS Mark III or Stevpris Mark III in Figures D.2 and D.3) anchor of same weight,

$A_n$ = fluke area of new version,

$A_s$ = fluke area of reference anchor of same weight,

$n$ = 1.4 for commonly used high efficiency anchors.

The fluke area ratio $A_n/A_s$ can be obtained from anchor manufacturers.

## D.10 Analytical Tools for Anchor Performance Evaluation in Soft Clay.

Analytical tools based on limit equilibrium principles for anchor embedment and capacity calculation in soft clay are now available. These tools allow modeling of different anchor designs and provide detailed anchor performance information such as anchor movement trajectory, anchor rotation, mooring line profile below seafloor, and ultimate anchor capacity, etc. However, there are certain requirements for these tools to yield reliable predictions:

1. The analytical tool needs to be calibrated by field or centrifuge test data for the anchor of interest.
2. The analytical tools require that the soil properties are well known, which may not be the case for many drag embedment anchor applications. If there is uncertainty in the soil properties, then suitable upper and lower bound soil parameters should be established, and the anchor design should be based on the more conservative predictions.
3. The user of these tools should be aware of the limitations of the tools and the mooring operation. For example some tools typically show that the anchor continues to penetrate gradually to a very deep penetration, picking up more and more anchor holding capacity. In this case the user should consider limiting the drag distance for calculating the anchor holding capacity to a distance that will not result in unacceptable vessel excursion.
4. Empirical formulas or field experience, if available, should be used to check the analytical predictions.

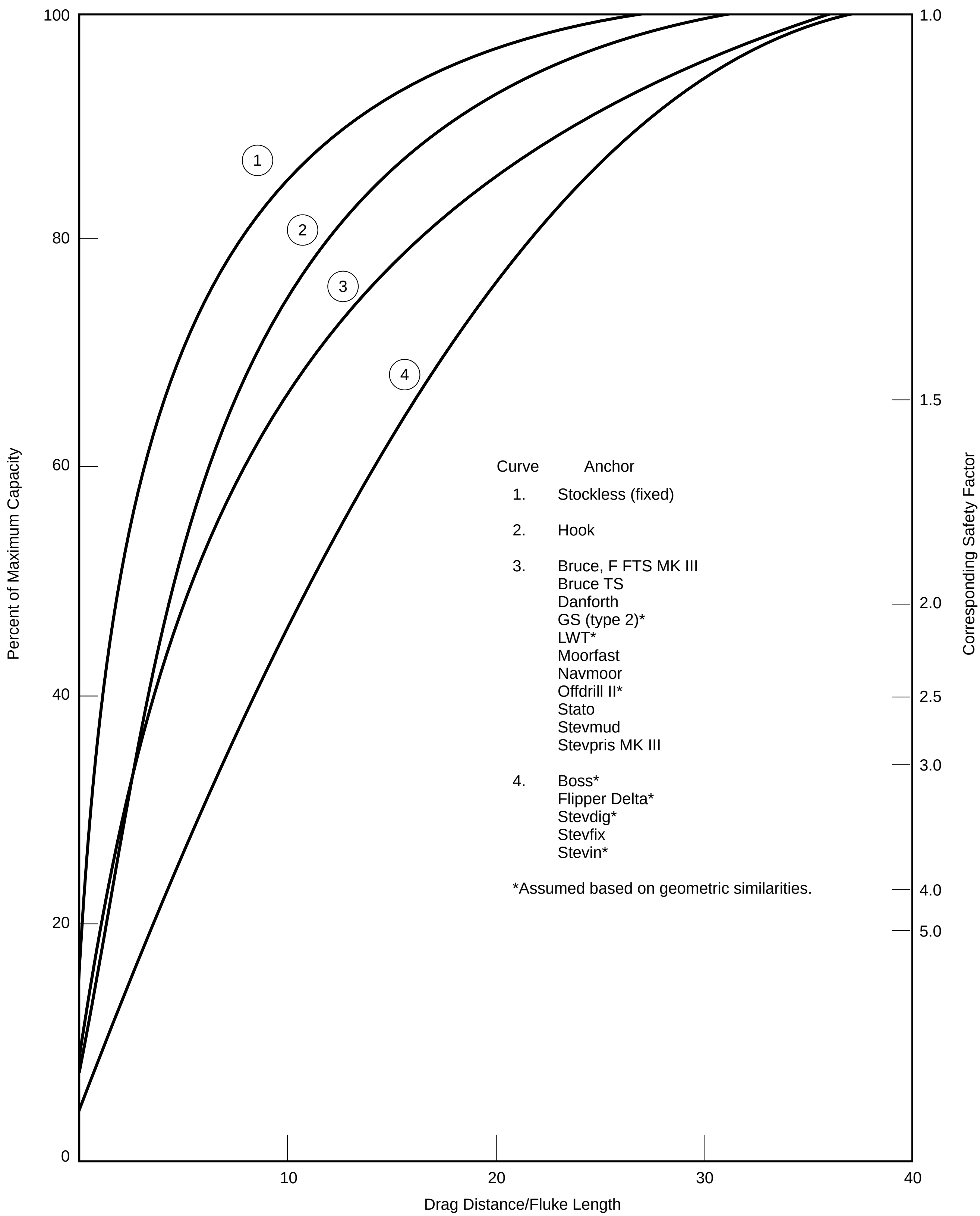



Figure D.6—Percent Holding Capacity versus Drag Distance in Soft Clay

5. The analytical tools may not be suitable for layered soil profiles.

## D.11 Anchor Holding Capacity in Sand

No significant study on the behavior of drag embedment anchors in sand has been carried out since the U.S. Navy's Study. Therefore, the design curves presented in Figure D.3 still represent the best available third party information on anchor holding capacity in sand. A nnchors do not penetrate deep in sand and therefore will not provide substantial uplift resistance (Table D.1). Design for vertical loads, in this case, is not recommended.

## D.12 Anchor Holding in Soils Other than Soft Clay and Sand

The above discussions address anchor holding capacities in soft clay and sand. There are other soil conditions such as hard clay, calcareous sand, coral or rock seafloor and layered soil profile. Predicting anchor holding capacity under these conditions is more complex, and anchor design guidelines are not available at this point. We hope that more research for these conditions will be carried out and some guidelines can be developed in the future.

## D.13 Holding Capacity from Friction of Chain and Wire Rope

The holding capacity from friction of chain and wire rope on the seafloor may be estimated using Equation D.2.

$$P_{cw} = fL_{cw}W_{cw} \tag{D.2}$$

where

$P_{cw}$ = chain or wire rope holding capacity, lb,

$F$ = coefficient of friction between chain or wire rope and the ocean bottom, dimensionless,

$L_{cw}$ = length of chain or wire rope in contact with the ocean bottom, ft,

$W_{cw}$ = submerged unit weight of chain or wire rope, lb/ft.

The coefficient of friction, $f$, depends upon the actual ocean bottom at the anchoring location and type of mooring line. Generalized friction coefficients for chain and wire rope are given in the following table. The static (starting) friction coefficients are normally used to compute the holding power of the line and the sliding coefficients are normally used to compute the friction forces on the line during mooring deployment. These generalized coefficients can be used for various bottom conditions such as soft mud, sand, and clay if more specific data are not available. Industry experience indicates that $f$ can vary significantly for different soil conditions, and much higher sliding $f$ values have been recorded.

| | Coefficient of Friction (f) | |
|---|---|---|
| | Static | Sliding |
| Chain | 1.0 | 0.7 |
| Wire Rope | 0.6 | 0.25 |

## D.14 References

**D.1** "Drag Embedment Anchors for Navy Moorings," Naval Civil Engineering Laboratory Tech Data Sheet 83-08R, June 1987.

**D.2** Dunnavant, T. W. and Kwan, C. T., "Centrifuge Modeling and Parametric Analyses of Drag Anchor Behavior," Paper No. 7202, Offshore Technology Conference, Houston, May 1993.

# APPENDIX E—PILE AND PLATE ANCHOR DESIGN AND INSTALLATION

## E.1 Basic Considerations

### E.1.1 PURPOSE AND SCOPE

This appendix addresses a number of design and installation issues for driven piles, suction piles, and plate anchors, all of which are capable of resisting vertical loads. The issues include soil investigation requirements, anchor capacity evaluation, structural design, fabrication, handling and transportation, installation, and pull testing. A general description of these anchors can be found in Appendix A, Section A.3, and the geotechnical factors of safety for these anchors are presented in Section 7.4 in the main body of this recommended practice.

Some of the technological aspects of the design of suction piles and plate anchors are still under development. Specific and detailed recommendations are given in this appendix to the extent currently possible. General statements are also used to indicate that considerations should be given to some particular points, and references are given for further guidance. Designers are encouraged to utilize all research advances available to them. Designers can find an additional discussion on plate anchor design in [E.35].

### E.1.2 FOUNDATION REQUIREMENTS

The design of the foundation structure should ensure that, in the anchor and the soil surrounding it, permissible limits of stress, displacement, and fatigue are not exceeded during and after installation. The foundation system above the mudline should include provisions for inspection and maintenance. The extent of inspection, timing of the inspection, and maintenance should be commensurate with the redundancy relative to overall safety and performance.

## E.2 Site Investigation

### E.2.1 PERMANENT MOORING

#### E.2.1.1 General Consideration

The areal extent of the foundation system for floaters greatly exceeds that of fixed structures and TLPs. Requirements for site investigations should be guided primarily by the type of platform to be installed, the availability and quality of data from prior site surveys, and the consequences that would result from a partial or complete foundation failure.

It is recommended that a high-quality, high-resolution geophysical survey be performed over the entire areal extent of the foundation. This survey should then have a realistic geological interpretation and be integrated with the possibly existing geotechnical data to assess restraints imposed on the design by geological features. Such an integrated study can then serve as a guide to develop a scope of work for the vertical and horizontal extent of the final geotechnical investigation (i.e., number, depth, and location of soils borings and/or in-situ tests such as PCPTs) and to aid in the interpretation of the acquired geotechnical data. Previous site investigations and experience may permit a less extensive site investigation. Some examples of these integrated geoscience studies are given in [E.22] and [E.23].

#### E.2.1.2 Soil Sampling and Laboratory Testing

Should the designer choose to rely on soil sampling and laboratory testing instead of in-situ testing during design, the designer should be aware that the measured properties of soil samples retrieved from deep waters may be different from in-situ values. Without special precautions, the relief of hydrostatic pore pressure and its resulting effect on any dissolved gases can yield soil properties significantly different from in-situ conditions. Because of these effects, in-situ or special laboratory testing to determine soil properties is warranted. Some of the geotechnical tools available when rotary drilling techniques are employed for deepwater investigations are discussed in [E.24]. Coring with "jumbo" or "long" coring devices has also been shown to provide shear strengths equivalent to those obtained by rotary drilling methods and holds promise as an alternative coring method [E.25] and [E.26].

#### E.2.1.3 In-Situ Testing

In-situ testing may allow a more reliable estimate of soil parameters and alleviate issues with sample disturbance. Typical tools used include: the remote vane (either seabed or downhole units), the piezoprobe (to obtain estimates of in-situ pore pressure and permeability), PCPT (Cone Penetrometer Tests equipped with pore pressure transducers). Advantages of the PCPT include obtaining a continuous profile of soil resistance that allows for detailed stratigraphy. A detailed discussion of PCPT data interpretation can be found in [E.32]. Other promising tools include the T-bar penetrometer [E.48].

#### E.2.1.4 Recommended Sequence for Site Characterization

A site investigation program should be accomplished for each platform location. The program should, as a minimum and preferably in the order listed, consist of the following:

**a. Background geophysical survey**

Regional geological data should first be obtained to provide information of a regional character which may affect the analysis, design and siting of the foundation. Such data should be used in planning the high-resolution surveys and geotechnical site investigation, and to ensure that the findings of the subsurface investigation are consistent with known geological conditions. Site-specific background data should include a re-examination of the 3-D, multichannel data obtained for exploratory purposes and a review of the "geo-hazard" study used to site the exploratory wells. The 3-D data set may be re-processed to enhance its high frequency content. Suggested reading for further information is given in [E.27].

**b. Seafloor and sub-bottom survey**

A site-specific, high-resolution geophysical information should be obtained relating to the conditions existing at and near the surface of the seafloor. The survey should include the mapping and description of all seafloor and sub-bottom features that may affect the foundation system. Such features may include: seafloor contours, seabed slope angles, shallow stratigraphy, position of bottom shapes which might affect scour, boulders, obstructions, and small craters, fluid expulsion features, pockmarks, shallow faults, slump blocks, drill cuttings plume, previous usage of seafloor, and gas hydrates.

The survey should use geophysical equipment and practices appropriate to the water depth of interest and provide high-resolution imaging of the seafloor as well as detailed stratigraphic information to a reasonable penetration below the zone of influence of the structure.

**c. Geotechnical investigation**

The sampling and in-situ testing intervals should ensure that each significant stratigraphic layer is properly characterized. The design soil parameters in various soil strata should be determined from a field program that tests the soil in as nearly an undisturbed state as feasible. Because the quality of soil samples can be expected to decrease with increasing water depth, the use of in-situ testing techniques is encouraged for deepwater sites. In addition, soil samples may be required to provide advanced engineering soil properties.

The scope of the geotechnical site investigation (i.e., number, location, depth of borings and/or long cores and/or PCPT, etc…) will depend on the mooring system, and the quality and interpretation of the high-resolution geophysical study (i.e. inferred lateral variations in soil properties). Typically, soil characterization (i.e., boring, long cores, or in-situ tests) is performed at each anchor (or at each anchor group for a group mooring pattern), or at least at two locations over the anchor pattern if the interpretation of the high resolution survey indicates little variation in soil properties across the pattern.

However, if high-quality geotechnical data already exist in the general vicinity of the anchor pattern and little variation of soil properties is inferred over the areal extent of the foundation, or if extensive experience with the chosen foundation concept in the area can be drawn upon, the above recommendations may be modified as appropriate [E.33], [E.34].

The minimum vertical extent of the site investigation should be related to the expected zone of influence of the loads imposed by the base of the foundation and must exceed the anticipated design penetration by at least the anchor diameter or anchor fluke width, B (see Figure E.7). If Reverse End Bearing (REB) at the suction anchor tip is to be taken advantage of in the vertical capacity analysis, soil characterization up to three diameters for suction piles or three fluke widths for plate anchors below the design penetration depth may be more appropriate. It is critical to ensure that no high-permeability layers are present within the zone influenced by the mobilization of the REB, particularly if the anchor is to resist long-duration loads such as those imposed by loop currents.

If the soil investigation is performed primarily using PCPT, it is recommended that at least one boring and/or long core be taken to properly calibrate the PCPT results. This boring/core should be taken at one of the PCPT locations.

The site investigation should also consider that during the detailed platform and mooring design process, the seabed location of the anchors may change due to changes in mooring lines lengths and/or headings, field layout, platform properties, and mooring leg properties.

Some examples of the scope of deepwater investigations are given in [E.2] and [E.33] and examples of data interpretation are given in [E.28] and [E.29].

**d. Soil testing program**

The main goal of the laboratory testing program should be to properly evaluate all input parameters required for geotechnical and structural design, for all significant strata. When applicable, testing should be performed in accordance with recognized standards (i.e., ASTM or others).

Additional testing should be performed to define the creep and cyclic behavior of the soil to allow prediction of soil structure interaction due to sustained and cyclic loading. Consideration should be given to the performance of permeability and consolidation tests in order to understand set-up effects for driven piled structures and capacity consideration for suction piles and suction caissons.

In all-clay profiles, the site investigation and laboratory testing program should consider providing the following information needed for the reliable design of pile and plate anchors, as applicable for the type of anchor, size, and anchor loading:

- General soil description, classification, and index testing.
- Soil stress history and over-consolidation ratio (OCR), soil compressibility (i.e., unload and reload moduli), as measured in Constant Rate of Strain (CRS) tests or constant load tests.
- Soil permeability.
- Remolded shear strength and soil sensitivity.

- Monotonic and cyclic shear strength under appropriate average and cyclic stresses for triaxial compression, extension, and DSS stress paths; Samples should preferably be anisotropically consolidated and cyclic tests should preferably be performed at the expected load period.
- Creep data to define possible loss of shear strength under sustained load (in cases where large sustained loads, e.g., loop currents, are important). Cyclic stresses should be superimposed on the sustained stresses if relevant for the actual load conditions.
- Remolded soil consolidation characteristics (compressibility and permeability).
- Reconsolidated remolded soil strength characteristics.
- Soil thixotropy.
- Parameters needed for generation of P-y curves (i.e., 50% strain factor, $\varepsilon_{50}$).

Database for cyclic soil properties are available in [E.2] and [E.30] for Gulf of Mexico clays. Such database should be used to interpret tests results and reduce the number of site-specific cyclic tests.

**e. Additional studies**

As applicable, additional analytical studies or scaled tests should be performed to assess the following aspects:

- Scouring potential
- Earthquake ground response studies or analysis
- Seafloor instabilities in the area where the foundation system is to be placed
- Set-up effects

For drag embedded plate anchors, an alternate design philosophy, currently used mainly in Brazil and South East Asia, has consisted of performing a reduced site investigation, gaining experience with the site specific performance of plate anchors through extensive load testing, and performing proof load tests after the anchor installation to at least 80% of the maximum storm load determined by a dynamic mooring analysis for the intact condition.

### E.2.2 MOBILE MOORINGS

If detailed site-specific soil data are available for a mobile mooring location, this information should be used for the design and selection of the mooring anchors. However, this information may not be available for some mobile mooring locations such as drilling locations in new exploration areas. The only geophysical data available may be the 3-D multi-channel data obtained for exploration purposes. Evaluation of geohazards and constraints to the mooring system should still be performed, based on a re-examination of the above 3-D data. The design and selection of pile and plate anchors should then be based on the best available soil data from nearby surrounding areas. Such information should be interpreted by recognized geotechnical experts whenever possible.

## E.3 Geotechnical Design of Suction Piles

### E.3.1 BASIC CONSIDERATIONS

The design of suction anchors for floating systems includes the following aspects: penetration and removal, capacity, and soil reactions or soil structure interaction analyses for structural design. In areas, such as the Gulf of Mexico, where tropical cyclonic storms may exceed the capacity of the mobile mooring or mobile anchoring system, the design of suction piles should consider an anchor failure mode that reduces the chance of anchor pullout.

The calculation of the geotechnical holding capacity of the anchor should be based on best-estimate soil properties. Anchor adequacy with respect to installation should be checked against upper bound soil strength properties.

If the uncertainty in the geotechnical data is greater than typically encountered (i.e., unusually large scatter in shear strength measurements), consideration should be given to increasing the safety factors given in Table 7 of the main text.

The impact of the mooring line geometry in the soil on the anchor loads should be considered, since this geometry may change the relationship between the horizontal and vertical anchor loads. The inverse catenary of the mooring line in the soil may make the mooring line angle steeper at the anchor padeye than at the mudline. This steeper angle could result in a reduced horizontal load but an increased vertical load at the anchor padeye. Both an upper and lower bound inverse catenary should be checked to ensure the worst–case anchor loading is established.

### E.3.2 ANALYSIS METHOD

#### E.3.2.1 Penetration Analyses

A typical penetration analysis includes the calculation of three quantities, for all penetration depths. These are:

- The penetration resistance exerted on the anchor by the soil.
- The required underpressure to allow anchor embedment.
- The critical pressure that will cause the soil plug to fail.

It is of paramount importance to properly estimate the underpressure required to penetrate the pile to its design depth. Required underpressure is a critical input parameter to the structural design of the anchor and it must also be verified that the predicted under pre sure can actually be generated by the pumps to be used during installation.

#### E.3.2.1.1 Penetration Resistance

The penetration resistance can be calculated as the sum of the side shear and end bearing on the side wall and any other protuberances. Protuberances that might be present include mooring and lifting padeyes, longitudinal or ring stiffeners, changes in wall thickness, mooring chain, launching skids, and others.

For an anchor penetrated <u>in clay</u> without protuberance and with a flat tip, the installation resistance, at a given penetration depth, z, can be calculated by Eq. E.1

$$\begin{aligned} Q_{tot} &= Q_{side} + Q_{tip} \\ Q_{side} &= A_{wall} \cdot (\alpha_{ins} \ Su_{DSS})_{AVE} \\ Q_{tip} &= (N_c \cdot Su_{tip}^{AVE} + \gamma' \cdot z) \cdot A_{tip} \end{aligned} \tag{E.1}$$

where

$Q_{tot}$ = Total penetration resistance,

$Q_{side}$ = Resistance along the sides of the pile,

$Q_{tip}$ = Resistance at the pile tip,

$A_{wall}$ = Sum of inside and outside wall area embedded into soil,

$A_{tip}$ = Pile tip cross sectional area,

$\alpha_{ins}$ = Adhesion factor during installation (see Section E.3.2.1.1.1),

$Su_{DSS}$ = Direct simple shear strength,

$\alpha_{ins}\ Su_{DSS}$ = Side friction,

$(\alpha_{ins} \cdot Su_{DSS})_{AVE}$ = Average side friction from mudline to depth $z$,

$N_c$ = Bearing capacity factor (see Section E.3.2.1.1.2),

$Su_{tip}^{AVE}$ = Average of triaxial compression, triaxial extension, and *DSS* undrained shear strength at anchor tip level,

$\gamma'$ = Effective unit weight of soil,

$z$ =Tip penetration depth.

##### E.3.2.1.1.1 Adhesion Factor During Installation, $\alpha_{ins}$

The adhesion factor during installation, $\alpha_{ins}$, is usually defined as the ratio of remolded shear strength over undisturbed shear strength, that is as the inverse of the soil sensitivity. The adhesion factor can be determined by various methods but fall cone, UU triaxial, and miniature vanes are presently the most common. The typical range of $\alpha_{ins}$ for Gulf of Mexico deepwater clays is 0.2 to 0.5.

There may be uncertainty in the sensitivity since it is influenced by the quality of the intact strength that it is related to. Alternatively, the side friction, $\alpha_{ins} \cdot Su_{DSS}$, can be equated to the direct measurement of the remolded shear strength, through fall cone, UU triaxial, or minivane tests. The remolded strength used in design should reflect both the directly measured value and the value derived from the intact strength divided by the sensitivity.

Some installation records have, however, shown that the interface shear strength mobilized during installation can, at a given depth, be less than $\alpha_{ins} \cdot Su_{DSS}$. In cases where the full interface shear strength, $\alpha_{ins}$, may not be mobilized along the anchor wall, such as when the anchor is painted or subjected to unusual surface treatment, a correction factor may need to be applied to the factor to properly predict the penetration resistance [E.9] [E.52]. Ring shear tests, with the actual wall surface modeled in the tests, may be used to measure the actual interface shear strength.

##### E.3.2.1.1.2 Bearing Capacity Factor, $N_c$

The values of the bearing capacity factor $N_c$ to be used to calculate the penetration resistance of the anchor tip or of a given protuberance depends on the shape of the protuberance and the ratio of the width of the protuberance over the embedment depth of the protuberance. Values of $N_c$ ranging from 5.1 to 9.0 for round and strip footings are recommended in [E. 31].

Because the anchor wall thickness is usually small compared to the anchor diameter and the embedment depth, the pile tip area is usually considered to be a deeply embedded strip footing with an associated $N_c$ equal to 7.5. The values of $N_c$ to be used in Eq. E.1 are summarized in Table E.1

A detailed example of $N_c$ calculation is given in [E.11]. Values of $N_c$ different than those of Table E.1 are acceptable provided that they can be documented by appropriate modeling and test results.

##### E.3.2.1.1.3 Changes In Penetration Resistance Due To Protuberances

Equation E.1 should be modified if protuberances are present. The change in penetration resistance due to the presence of mooring and lifting padeyes, longitudinal or ring stiffeners, mooring chain, launching skids, pile tip other than flat (i.e., beveled) or any other internal or external protuberance should be considered carefully and require assessing the changes in friction and end bearing resistance caused by the protuberances. Most protuberances will cause an increase in penetration resistance, except for internal ring stiffeners, which may cause a decrease in internal side friction if they are closely spaced [E.36], [E.54].

Table E.1—Recommended $N_c$ Factor

| Purpose | Shape of area | $N_c$ |
|---|---|---|
| Calculation of pile tip penetration resistance | Strip | 7.5 |
| Calculation of critical underpressure causing soil plug failure (Section E.3.2.1.3) | Circular | 6.2 to 9.0 depending on embedment ratio [E.31] |
| Calculation of penetration resistance of protuberances (Section E.3.2.1.1.3) | Varies | 5 to 13.5 [E.36] |

#### E.3.2.1.2 Required Underpressure

The required underpressure, $\Delta U_{req}$, to embed the anchor can be calculated as follows:

$$\Delta U_{req} = \frac{Q_{tot} - W'}{A_{in}} \tag{E.2}$$

where

$Q_{tot}$ = Total penetration resistance,

$W'$ = Submerged weight during installation,

$A_{in}$ = Plan view inside area where underpressure is applied.

#### E.3.2.1.3 Critical and Allowable Underpressures

The critical underpressure at a given depth, $\Delta U_{crit}$, defined as the underpressure that will cause a general reverse bearing failure at the anchor tip and large soil heave within the anchor, can be calculated at a given depth as follows:

$$\Delta U_{crit} = N_c \cdot Su_{tip}^{AVE} + \frac{A_{inside} \cdot (\alpha_{ins} \cdot Su_{DSS})_{AVE}}{A_{in}} \tag{E.3}$$

where

$A_{inside}$ = Inside lateral area of anchor wall.

In shallow water one must check that the critical underpressure does not exceed the water cavitation pressure.

The recommended allowable underpressure, $\Delta U_{allow}$, defined as the maximum underpressure that should be applied to the anchor, can be calculated as the critical underpressure divided by an appropriate factor of safety. This safety factor is typically a minimum of 1.5. Lower values may be acceptable provided that, during installation, the plug behavior is monitored and it is confirmed that no plug failure occurred.

#### E.3.2.1.4 Soil Plug Heave Inside Anchor

The soil heave inside the anchor during installation may be estimated by assuming that a percentage of the clay volume displaced by the cross sectional area of the anchor goes inside the anchor. This percentage may depend on: anchor tip geometry, mode of penetration (i.e., self-weight penetration vs. penetration by underpressure) [E.37]. It is commonly assumed the 50% of the soil displaced by the cross sectional area of the anchor will go inside the anchor during self-weight penetration if the tip of the anchor is flat.

The final elevation of the internal plug surface may also depend on the wall thickness variations, internal soil plug stability, spacing and type of internal stiffeners [E.37].

Soil heave should be accounted for in calculating the required pile stick-up and total length.

#### E.3.2.1.5 Items of Special Consideration

Sand layers: If sand layers are present, they should be given special attention. The penetration resistance in layered profiles consisting of interbedded sands and clays may be significantly higher than through clay, depending on the density, degree of cementation, grain size distribution, thickness, spacing, and depth of the sand layers. The penetration rate through sand layers will need to be high enough to prevent excessive flow of water through the sand layers ahead of the anchor tip, as this may cause large plug heave.

### E.3.2.2 Removal Analyses

The geotechnical analysis should also consider anchor retrieval for the following cases:

- Mobile moorings where anchor removal is needed for reuse of the anchor or to clear the seabed. The suction pile retrieval procedures and analysis should account for the estimated maximum setup time;
- Permanent moorings where it is required by authorities that the anchors be removed after the system service life. The suction pile retrieval procedures and analysis should be based on full soil set-up.
- Mobile or permanent moorings: installation tolerances are exceeded, a mooring line is damaged during installation and for other contingencies.

The overpressure required to retrieve the anchor, $(\Delta U_{req})_{retr}$, can be calculated by Eq. E.4.

$$(\Delta U_{req})_{retr} = \frac{Q_{tot}(t = t_r) + W'}{A_{in}} \tag{E.4}$$

where

$Q_{tot}(t = t_r)$ = Total soil resistance at time of retrieval, $t_r$. Time $t = 0$ is defined as the time at the end of penetration,

$W'$ =Submerged weight during retrieval,

$A_{in}$ = Plan view inside area where overpressure is applied.

When calculating the total soil resistance during retrieval, $Q_{tot}(t = t_r)$, Eq. E.1 can be used with some modifications. It should be noted that the interface shear strength might be higher than its values during installation, due to soil set-up. Paragraph E.3.3 gives guidance on assessing the increase in adhesion factor with time. The designer should also be mindful of possible differences between end bearing resistance in tension and compression for the protuberances that may be present. In addition, the maximum extraction pressure used should not be higher than the pressure causing soil plug failure.

The vessel removing the anchor is often capable of applying a lifting force on the anchor with the recovery line. This assistance can significantly reduce the required extraction pressure and should be included in the removal analysis. Therefore, if a load is taken by the lifting wire during retrieval, that load can be subtracted from the numerator in Eq. E.4.

The effect of the maximum extraction pressure on the steel structure of the suction pile should be considered (see Section 5.2.3 and 5.2.5).

### E.3.2.3 Holding Capacity

Analysis and design tools to determine the capacity of suction anchors can be classified as one of three general methods [E.36]. These are, in order of detail:

- The Finite Element Method (FEM) advanced numerical analysis,
- Limit equilibrium or plastic limit analysis methods (models involving soil failure mechanisms),
- Semi-empirical methods (highly simplified models of soil resistance including beam column models).

For the analysis and design of suction anchors for anchoring deepwater floaters, the central focus is the ultimate capacity of the suction anchor and not the load deflection behavior.

It is recommended that suction pile design for permanent moorings use FEM, limit equilibrium techniques or limit analyses (see Sections E.3.2.3.1 and E.3.2.3.2). For mobile moorings with mainly horizontal loads, semi-empirical methods such as beam-column analyses (see Section E.3.2.3.3) using load transfer-displacement curves (i.e., P-y, T-z, Q-z) described in API RP 2A are also considered adequate if suitably modified. A method to modify P-y curves to account for the larger diameter of suction piles and to ensure lateral deflection is not overestimated can be found in [E.5]. The merits and shortcomings of each method are discussed below.

#### E.3.2.3.1 The Finite Element Method (FEM)

As discussed in [E.36], the finite element method is the most rigorous general method of analysis available for complex structural systems (including soil continua and soil-structure interaction). The FEM will find the critical failure mechanism without prior user assumptions, provided an appropriate constitutive model is used. The FEM also has many advantages including the ability to include complex geometries, spatially varying soil properties, and non-linear constitutive behavior with failure criterion, to name a few. Major disadvantages include the required specialty knowledge of advanced numerical analysis and the large time investment to set up a model.

In ductile plastic systems (foundations on soft clays are usually in this category) the ultimate capacity of the system is independent of the sub-failure properties (e.g., Young's modulus, Poisson's ratio) [E.38]. It has been shown that carefully formulated and executed analyses give system load carrying capacities that compare favorably with the few exact, analytical solutions available [E.39].

FEM programs are widely available and have been used to advantage for assessing specific suction piles configurations, matching the few experimental results available, and providing ground truth calibration for simpler models. As mentioned above, such analyses require special expertise and a significant investment in time and are therefore not yet well suited to parametric studies or conventional design iteration (such as are required for finding the optimum anchor line attachment point).

FEM analyses may however be warranted for complex load and/or soil conditions where little experience is available, or to gain insight on specific behavioral aspects of the foundation (i.e., assessment of pore pressure changes and effective stress path at any point within the soil mass).

#### E.3.2.3.2 Limit Equilibrium or Plastic Limit Analysis Methods

As discussed in [E.36], these models are more approximate than FEM models but are generally much easier to use than general FEM programs. The methods involve estimating the ultimate capacity of plastic systems using assumed failure mechanisms. These mechanisms are typically based on a combination of experimental observation, more rigorous numerical or analytical studies, and engineering judgment. These methods may also include the ability to incorporate complex geometry and soil strength variability and do not require characterizing sub-failure behavior.

Disadvantages of these methods are the approximate nature of the analysis and the difficulty of generalizing results, i.e., the need to calibrate the models to experiment or more rigorous analysis for specific structural configurations and soil profiles. For example, changing either the soil strength profile, the anchor geometry, the load inclination, the load attachment point, the load type (i.e., duration, frequency, ratio of cyclic to mean load component, etc.) may require a change in the basic geometry of the assumed failure mechanism.

In general there are two approaches that can be taken using assumed mechanisms; the limit equilibrium method and the plastic limit analysis method. In the limit equilibrium method, a failure mechanism is assumed, usually described in terms of one or more geometric parameters [E.3], [E.14]. The body force distribution, stress boundary conditions, and the stress or force distribution on failure surfaces are estimated, and a search is conducted to find the geometry that is closest to equilibrium conditions. The plastic limit analysis method also uses an assumed failure mechanism with the added requirement that the mechanism satisfies kinematic constraints (i.e., incompressibility for a purely cohesive material, displacements continuity, etc.) [E.4], [E.41].

A possible failure mechanism is shown on Figure E.1. Other proposed mechanisms can be found in [E.4], [E.50], and [E.51]. Depending on the failure mechanism, the anchor is shown to resist vertical uplift loads by self-weight, skin friction, reverse end bearing (REB) and/or shear and/or rotational failure at the pile tip, passive and active earth pressure, and soil flow around the pile.

In some limit equilibrium methods, the circular area is transformed to a rectangle of the same area and the width equal to the diameter, and 3D effects accounted for by side shear factors [E.3].

In general both limit equilibrium and limit analysis methods give upper bound estimates of ultimate loads such that minimizing the ultimate load with respect to the geometric parameters gives the "best" answer for the particular mechanism. However, the "best" answer may or may not be close to the exact answer depending on the assumed mechanism. In the limit equilibrium method the result will not be a true upper bound if the mechanism does not satisfy kinematic constraints. A discussion of these methods is provided in [E.38].

There are a number of existing computer programs and spreadsheets that implement these methods, but they are generally of a one-off variety and are primarily used inhouse by various consultants, contractors, or research groups. A few of these programs are publicly available or available for purchase. While some programs have been rather widely used, there is no single general, industry accepted, program or procedure. Selected models have been shown to compare favorably with more rigorous FEM results for soft clay profiles and various anchor geometry and load attachment points [E.53].

This is, however, an active area of development. Automated solutions (programs, spreadsheets, etc) using these approaches generally require much less input description and are much easier to use than general FEM programs. As a result they are well suited for conducting parametric studies and design iterations. However, as mentioned above these solutions do not necessarily converge to correct capacity estimates even with great care and analyst skill, and results from different formulations may give significantly different answers. Thus, obtaining accurate results is very dependent on the analysts understanding of the methodology and his/her engineering judgment.

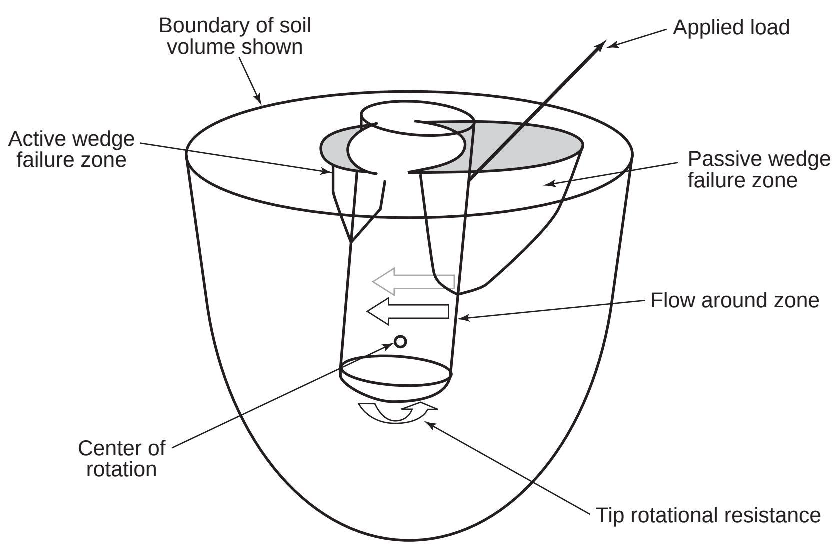



Figure E.1—Three-Dimensional View of a Possible Failure Mechanism

#### E.3.2.3.3 Semi-Empirical Methods: Beam-Column Analyses

As discussed in [E.36], these models are the most approximate, but may be the easiest to use if computer programs or spreadsheets with FEM, limit equilibrium or plastic limit analysis methods are not available. They are labeled semi-empirical to suggest that they incorporate the basic mechanics of a suction pile loaded to failure, but depend on a set of empirical rules to represent the soil resistance. These rules are typically less general than the methods discussed in Sections E.3.2.3.1 and E.3.2.3.2. For example, they do not explicitly incorporate soil failure mechanisms, but instead represent the soil resistance as a load distribution varying along the boundary at the soil-pile interface. It is difficult to generalize such a load distribution for a wide range of soil profile types so a particular solution may apply say, only to a normally consolidated strength profile. Rules for constructing these distributions are typically based on a combination of experimental and analytical results. In the so-called beam-column model, the soil is represented by uncoupled, non-linear, soil springs along the pile boundary. The beam column method can provide estimates of the load displacement history up to and including the full capacity of the soil-pile system.

In the beam-column model the soil resistance is represented by uncoupled, non-linear soil springs (P-y curves) which describe the sub-failure behavior of the local soil resistance as well as the peak capacity [E.42] and [E.43]. In the API RP2A P-y formulations for piles, the curves exhibit softening behavior (reduced resistance with continued displacement) to account for the effects of cyclic loading [E.44]. It has been argued in [E.45], however, that ultimate capacity estimates for piles, and thus presumably for suction piles as well, should be based on non-softening (static) P-y curves. In this model the governing equations of a beam on an (non-linear) elastic foundation are solved iteratively until an equilibrium solution is found for a given value of the applied load. The user can gradually increase the load in subsequent steps until the solution no longer converges, a point which is interpreted as failure.

The beam-column model has been used by geotechnical engineers for almost 50 years for the analysis of laterally loaded piles. Hence it has the decided advantage of being a familiar tool. There are many versions of beam-column programs in use, including general purpose programs where loads as well as non-linear springs can be prescribed at virtually any point on the pile, as well as special versions where non-linear spring construction is automated based on minimal soil property input. Thus, there might be an understandable tendency for engineers to select these programs for suction pile analysis. However, the user should be aware that these programs have significant limitations. As detailed in [E.36], among the limitations, the conventional beam column models:

1. Ignore that the resistance elements depend on the deformation mode and ignore the coupling between the resistance elements. This can lead to large errors, particularly for relatively short piles.
2. Do not include independent side shear resistance components on active and passive sides to model different relative shear displacements between the soil and the pile on the two sides.
3. Do not include the coupling between the horizontal and vertical soil resistance components along the pile sides and thus do not show the effect of inclined anchor load. It is possible in principle to couple these elements (P-y and T-z curves), but this has only been done in special cases [E.46].
4. Require input that is not essential to the capacity assessment such as pile bending stiffness and sub-failure soil response and produce output that is of little interest for the analysis such as moment and shear profiles and load deformation response that is probably not very accurate. Because most piles are stiffened shells, the beam equations are of doubtful validity and are largely irrelevant with regard to stresses in the pile. A better pile model for these purposes is actually a rigid body that can be approximated by setting the pile EI to an arbitrarily large value (see Section E.5.3 for recommendations on structural design).
5. Require user intervention to determine the pile capacity. In most beam column programs the ultimate capacity is determined by trial and error, gradually increasing or decreasing applied loads until the minimum load that produces numerical instability (interpreted to be the failure load) is found.
6. Require special elements for rotational, vertical and horizontal tip resistance.
7. Do not explicitly include effects such as soil-pile interface roughness and loss of soil contact on the back side of the pile.

It is possible to formulate and implement a beam-column program that overcomes most of the above limitations. There seems to be little incentive to do so however, as other methods are available that are simpler to implement and can be especially tailored to suction pile analysis.

### E.3.3 INCREASE OF SIDE FRICTION WITH TIME

As described in Section E.3.2.1.1, the side friction at a given depth can be calculated as $a_{ins} \cdot Su_{DSS}$, with $a_{ins}$ ranging in value from 0.2 to 0.5 for Gulf of Mexico deepwater clays during installation. With the passage of time after installation, the side friction increases through soil thixotropic effects and pore pressure redistribution at the pile interface. This phenomenon is often referred to as “set-up”. Set-up effects are often addressed by estimating the change in the adhesion factor, $\alpha$, with time. Set-up mainly influences the vertical capacity, and to a lesser extent, the horizontal capacities of a suction pile [E.7].

The set-up process may be different for the part of the anchor penetrated by weight and the part penetrated by underpressure. For suction piles in highly plastic clays, the set-up time can be long and there can be a permanent loss of shear strength, whereby the ultimate side friction after full set-up is less than the original undisturbed shear strength (i.e, the adhesion factor, $\alpha$, is less than 1.0 after full set-up), both for the portion of the anchor penetrated by weight and the portion penetrated by underpressure.

Some researchers [E.6] have reported that the portion of the anchor penetrated by underpressure is expected to typically have a shorter set-up time and a lower ultimate side friction after full set-up than the portion penetrated by weight. Figure E.2 shows a typical soil set-up prediction graph for a large diameter suction anchor in typical Gulf of Mexico soils and illustrates the current uncertainty in calculating soil set-up. The methods in [E.7] and [E.20] are shown to illustrate the potential differences in set-up time and ultimate friction for different parts along the side of the anchor. The set-up along the part of the anchor penetrated by underpressure may occur much faster, but the permanent reduction may be larger. The method in [E.20] was developed for driven piles with ratio of diameter over wall thickness less than 40. The method in [E.7] was proposed for penetration by underpressure. Both methods should be applied with caution outside the range of data used in their development. Other methods developed for driven piles include the one described in [E.49]. There is no single industry-wide accepted set-up curve.

As with other piled foundation systems, the calculated anchor ultimate capacity should be reduced if soil set-up will not be complete before significant loads are imposed on the anchor pile.

Set-up can be addressed in various ways during design. The designer can ensure adequate anchor capacity if:

- The suction pile is designed with partial soil set-up;
- The suction pile is installed well in advance of platform hook-up and platform first-oil to ensure adequate soil set-up when the mooring may experience design loads.

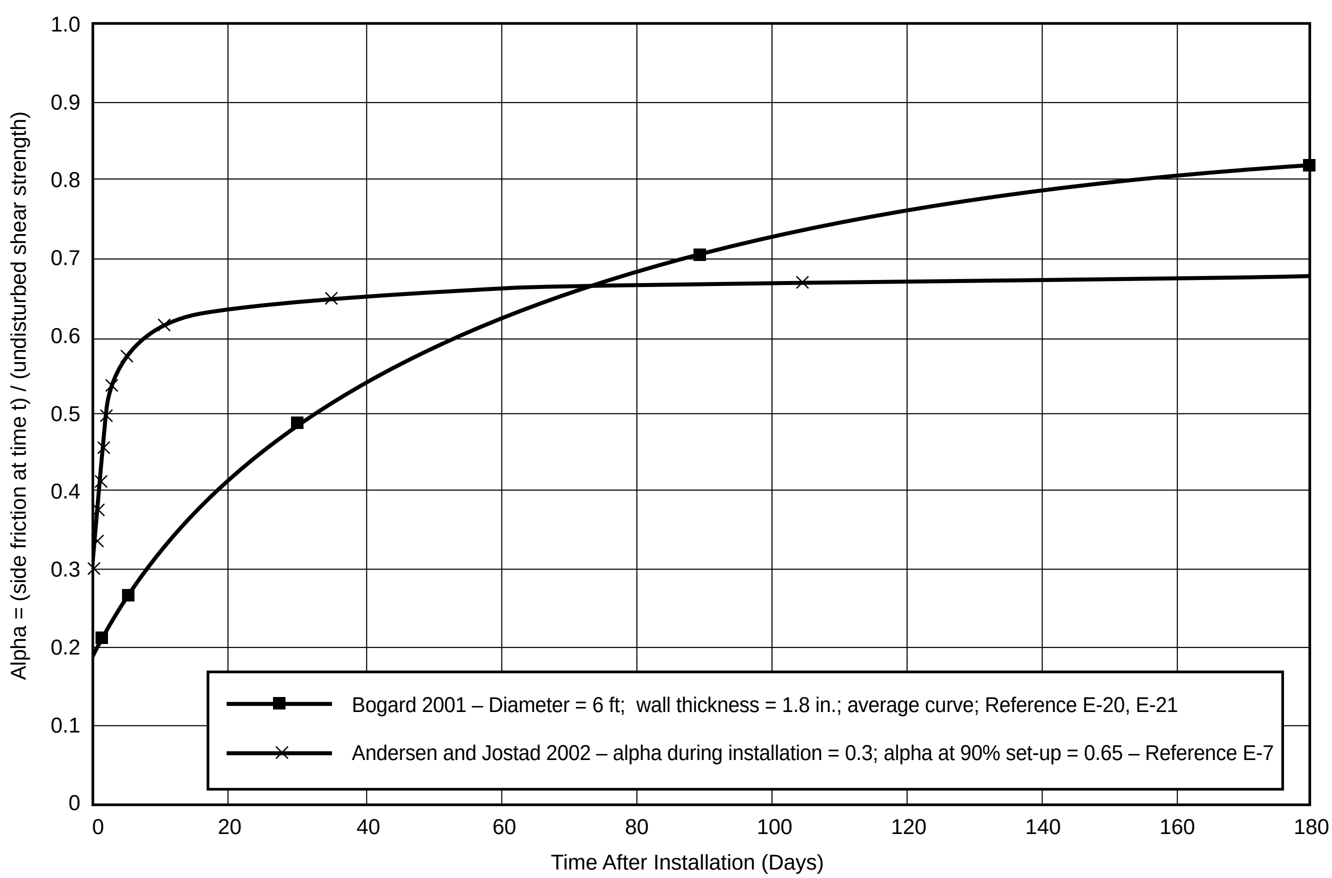



Figure E.2—Example of Increase in Adhesion Factor with Time

- For a limited amount of time between installation and first-oil, reduced extreme load criteria are assumed, based on suitable risk analysis.

### E.3.4 COUPLING BETWEEN HORIZONTAL AND VERTICAL CAPACITY

When a suction anchor resists the design loads, the vertical and horizontal components of the anchor capacity are not mobilized independently. Coupling between vertical and horizontal capacities may be important in some cases. Studies have shown that for mooring line angles at the padeye between 15° and 45° (as measured from the horizontal), it may be non-conservative to neglect this coupling [E.3] and [E.8].

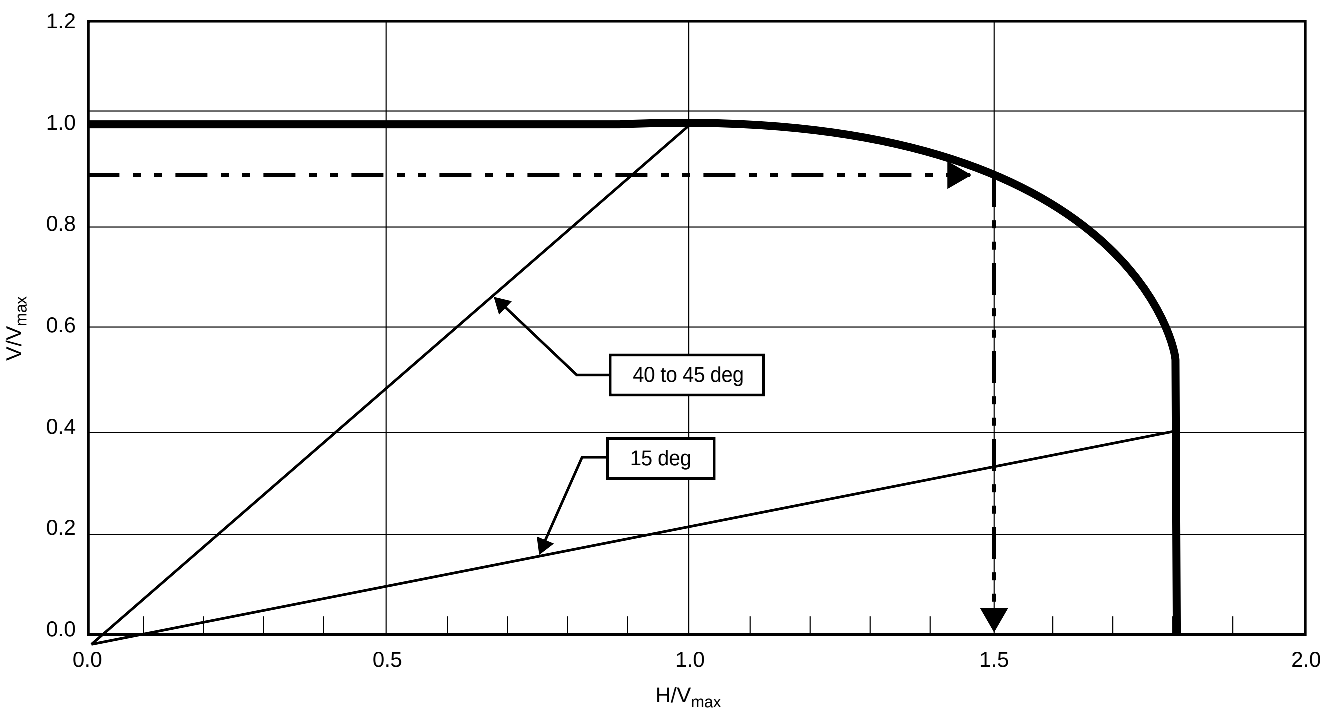
The following discussion is for illustration only and therefore should not be used for design. The sample failure interaction diagram shown in Figure E.3 is typical for suction piles with a length to diameter ratio of 5, in a linear increasing shear strength profile with low shear strength at the seabed. The mooring padeye is located on the pile shell, about $^{2}/_{3}$rds of the way down from the pile top. In this example, the diagram shows that if the load is primarily vertical, with padeye angles from 40° or 45° to 90°, the failure mode is controlled by vertical pullout and 100% of the vertical capacity is available. In a similar manner, if the load is primarily horizontal, with padeye angles from zero to 15°, the failure mode is controlled by horizontal pullout and 100% of the horizontal pile capacity is available. In this case, the maximum horizontal capacity is equal to 1.8 times the vertical capacity. If, however, the load angle at the padeye is between 15° and 40°, less than maximum vertical and horizontal capacities are available. In the example shown, only 90% of the vertical capacity is available; and the available horizontal capacity is reduced to 150% of the vertical capacity, from the original 180%.

Examples of failure interaction diagrams can be found in [E.55] and [E.56].

### E.3.5 FACTORS OF SAFETY

Factors of safety for holding capacity, defined as the calculated capacity divided by the maximum anchor load from dynamic analysis, are provided in Table 7 (ref. main body of this recommended practice) for axial and lateral loads. Infor-

**Interaction Diagram**
**(Illustration Only—Do Not Use for Design)**




Mooring padeye angles measured from horizontal.

with:

V: Vertical load component.
H: Horizontal load component.
$V_{max}$: Vertical ultimate capacity of the anchor, for purely vertical loads.

Figure E.3—Example of Failure Interaction Diagram mation on coupling between vertical and horizontal capacities can be found in Section E.3.4. Axial safety factors consider that the pile is primarily loaded in tension, and are therefore higher than for piles loaded in compression.

As the lateral failure mode for piles is considered to be less catastrophic than the vertical one, lower factors of safety have been recommended in Table 7 for lateral pile capacity. Use of separate factors of safety for vertical and lateral pile capacities may be straightforward for simple beam-column analysis of, for example, mobile moorings (ref. Section E.3.2), but more complex methodologies do not differentiate between vertical and lateral pile resistance.

The safety factor used in design should be based on the failure mechanism controlling the capacity and not only on the load angle. Although load angle and failure mechanisms are related, other parameters such as soil profile, anchor geometry, and load attachment points are also important in determining failure mechanisms. For cases where axial pullout controls, the minimum safety factor should be as per Table 7, regardless of load angle. For cases where lateral pullout controls, the minimum safety factor should be as per Table 7, regardless of load angle. Eq. 5 is proposed in order to provide a combined factor of safety for situations where neither the axial nor the lateral capacity control the design:

For a given geometry, load attachment, and soil profile, the combined safety factor can therefore be calculated as follows:

$$\left|\begin{array}{l} \text{If } \theta \le \theta_{lateral}\text{, } FOS_{combined} = FOS_{lateral} \\ \text{If } \theta \le \theta_{axial}\text{, } FOS_{combined} = FOS_{axial} \\ \text{If } \theta_{axial} \le \theta \le \theta_{axial}\text{,} \end{array}\right. \qquad \text{E.8}$$

$$FOS_{combined} = FOS_{lateral} + \frac{\theta \le \theta_{lateral}}{\theta_{axial} - \theta_{lateral}} \cdot (FOS_{axial} - FOS_{lateral})$$

where

$FOS_{combined}$ = Combined factor of safety (*FOS*),

$FOS_{lateral}$ = Lateral *FOS* from Table 7,

$FOS_{axial}$ = Axial *FOS* from Table 7,

$\theta$ = Angle of mooring line from horizontal at pile attachment point,

$\theta_{lateral}$ = Load angle, measured from horizontal, below which the ultimate capacity is controlled by the lateral capacity. The lateral capacity is defined as the capacity under purely horizontal loads,

$\theta_{axial}$ = Load angle, measured from horizontal, above which the ultimate capacity is controlled by the axial capacity. The axial capacity is defined as the capacity under purely vertical loads.

For a given failure interaction diagram, Figure E.4 illustrates how to calculate the required safety factor.

### E.3.6 OTHER SPECIAL CONSIDERATIONS

1. *Closed vs. open top*: The top of the anchor should remain sealed throughout the life of the field, if the REB at the anchor tip is to be relied upon in design. Note that, with increased soil set-up and side friction, the need for REB decreases and, thus, the requirement to maintain a sealed top cap. For anchors with essentially horizontal loading, a sealed top is not essential for the capacity, and the top part can be removed after installation (Reference E.47).
2. *Strength anisotropy*: Capacity calculations should be performed with anisotropic shear strength, including effect of combined static and cyclic loading history.
3. *Internal ring stiffeners*: For large long-term loads and for suction piles that are not sealed at the top, the skin friction along the inside skirt wall is an important contribution to the capacity. The inside wall friction may be significantly lower than the original shear strength due to the disturbance during installation, especially if the anchor has internal stiffeners. In cases with series of ring stiffeners, clay from the upper part of the profile, and also water, may be trapped between the stiffeners and give low capacity at larger depth. In such cases, the compartment between the ring stiffeners may also act as a drainage channel [E.36].
4. *Gapping:* A gap may form on the outside at the active side (i.e., backside) of the anchor. There are uncertainties on how to predict gap formation, unless the clay is soft and with essentially zero strength intercept, in which case a gap is not expected to form. Therefore, one should make conservative assumptions with respect to whether there will be a gap or not. One should consider conservatively placing the load attachment point far enough below the optimal load attachment depth for the suction anchor top to move "backwards" (i.e., away from the direction of the mooring line) during loading to prevent gap formation.
5. *Installation tolerances*: The allowable installation tolerances (e.g., tilt and orientation) shall be included in the capacity calculations, as tilt and out of plane loading may reduce the holding capacity of the pile.
6. *Change in outer diameters*: It is considered that variations in the outer diameter with depth could reduce the outside interface strength and it is recommended that, in general, designs with variations in outside diameters should be avoided.

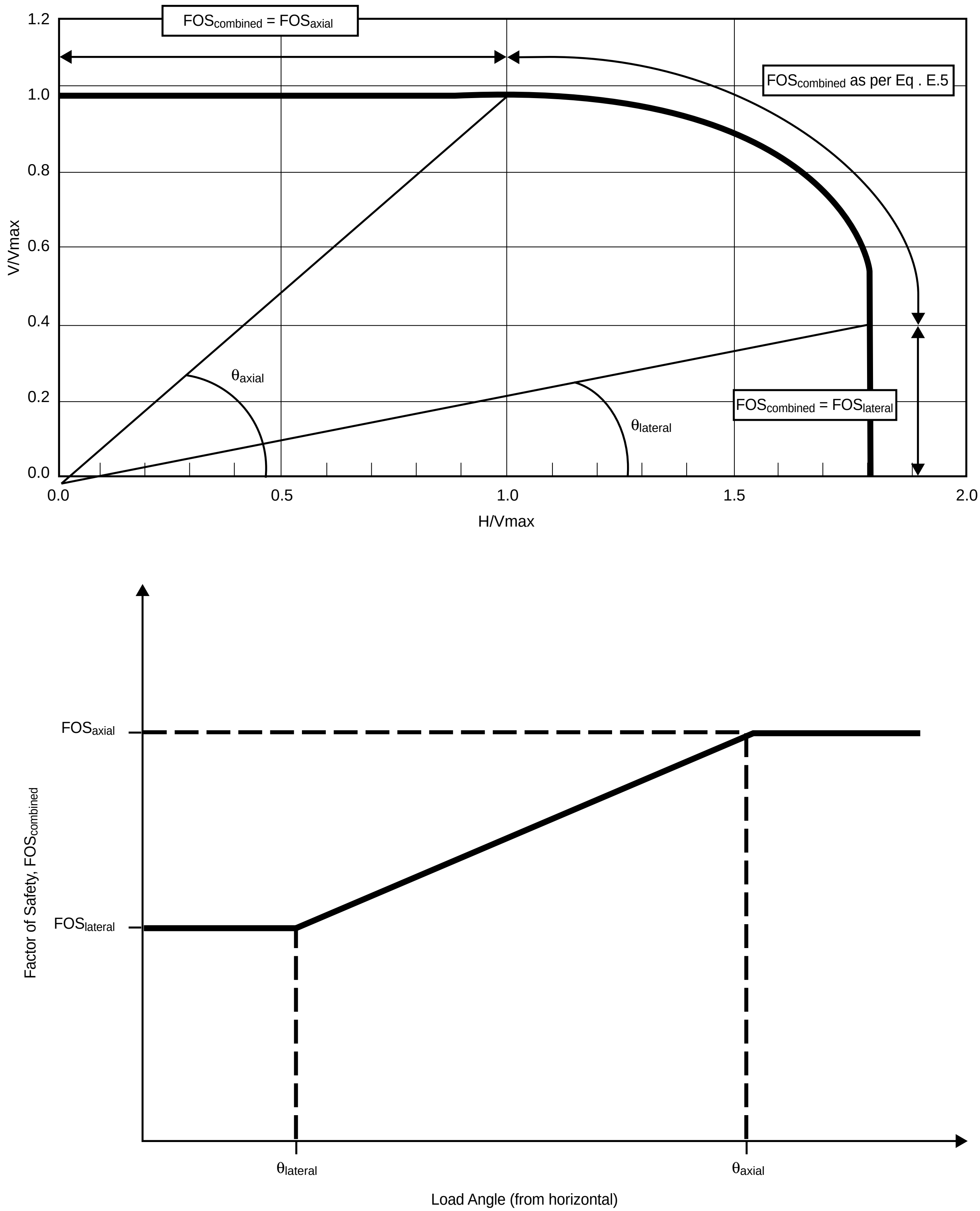



Figure E.4—Calculation of Required Safety Factor as a Function of Failure Mode

7. *Sand layers*: sand layers, if present may have a significant effect on the holding capacity. It should be ensured that the sand layers will not cause excessive drainage and pore pressure redistribution that could negatively affect the REB, particularly if the anchor is to resist long-duration loads.
8. *Distance between installation locations*: In the event that an anchor needs to be retrieved and re-installed, the determination of the minimum distance between the first location and the subsequent location should ensure that the volume of soil disturbed during the first installation will not be mobilized when the anchor resists the design load at the subsequent location.
9. *Sustained Load*: The duration of sustained loads (e.g., creep under loop current load) and the period of cyclic loading should be considered and the anchor capacities should be adjusted to account for these effects. Examples of capacity reduction as a function of load hold time can be found in [E.57] for vertically loaded anchors in Gulf of Mexico clays.

A combination of these considerations may be used to arrive at a suitable suction pile design. Due to the complexity of analyzing the load capacities of large permanent suction piles, a recognized geotechnical expert should be consulted.

## E.4 Geotechnical Design of Plate Anchors

### E.4.1 BASIC CONSIDERATIONS

For plate anchors, the ultimate holding capacity is often defined as the ultimate pull-out capacity (UPC), which is the load for the soil around the anchor reaching failure mode. At UPC, the plate anchor starts moving through the soil in generally the direction of the applied anchor load with no further increase in resistance or the resistance starts to decline. The ultimate pull-out capacity of a plate anchor is a function of the soil undrained shear strength at the anchor fluke, the projected area of the fluke, the fluke shape, the bearing capacity factor, and the depth of penetration. When analyzing the plate anchor ultimate pull-out capacity, the disturbance of the soil due to the soil failure mode should be considered. This mode is generally accounted for in the form of a disturbance factor or capacity reduction factor. The bearing capacity factor and disturbance factor should be based on reliable test data, studies, and references for such type of anchors. Typically, the plate anchor's penetration is in a range of 2 to 5 times the fluke width, *B* (see Figure E.7), depending on the undrained shear strength of the soil, in order to generate a deep failure mode [E.10]. If the final depth does not generate a deep failure mode, a suitable reduction in bearing capacity factor should be used.

Plate anchors get their high holding capacity from their embedment into more competent soil. Therefore, it is important that the anchor's penetration depth can be established during the installation process. Furthermore, a plate anchor gets its high ultimate pull-out capacity by having its fluke oriented nearly perpendicular to the applied load. To ensure that the fluke will rotate to achieve a maximum projected bearing area, the plate anchor design and installation procedure should:

- Facilitate rotation of the fluke when loaded by environmental loads or during installation or both;
- Ensure that no significant or unpredicted penetration is lost during anchor rotation, which may move the fluke into weaker soil;
- Have the structural integrity to allow such fluke rotation to take place during installation and keying operations or while subject to the ultimate pull-out capacity load. Depending on the type of plate anchor and its installation orientation, this item may also apply to fluke rotation about both horizontal and vertical axis.

As appropriate, the anchor capacities should be reduced to account for anchor creep under long-term static loading and cyclic degradation.

Factors of safety for holding capacity, defined as the calculated soil resistance divided by the maximum anchor load from dynamic analysis, are provided in Table 7

### E.4.2 PREDICTION METHOD FOR DRAG EMBEDDED PLATE ANCHOR

Three aspects of drag embedded plate anchor performance require prediction methods:

- Anchor line mechanics;
- Installation performance; and
- Holding capacity performance.

All three mechanisms are closely linked and influence one another, as explained below.

#### E.4.2.1 Anchor Line Mechanics

As discussed in [E.58], [E.59], [E60] and [E61], the anchor line mechanics influence strongly the prediction of the drag embedded plate anchor's final orientation and depth below the seabed, which in return governs the holding capacity of the anchor system. Figure E.5 is a schematic of an anchor line configuration showing the reverse curvature of the line as it cuts through the soil. As the load in the anchor line increases, the inclination of the line with the horizontal at the anchor attachment point decreases, giving rise to an interaction between the anchor line and the holding capacity of the anchor.

In general, this problem is approached in the same manner as that for predicting the displaced shape of a catenary, fixed

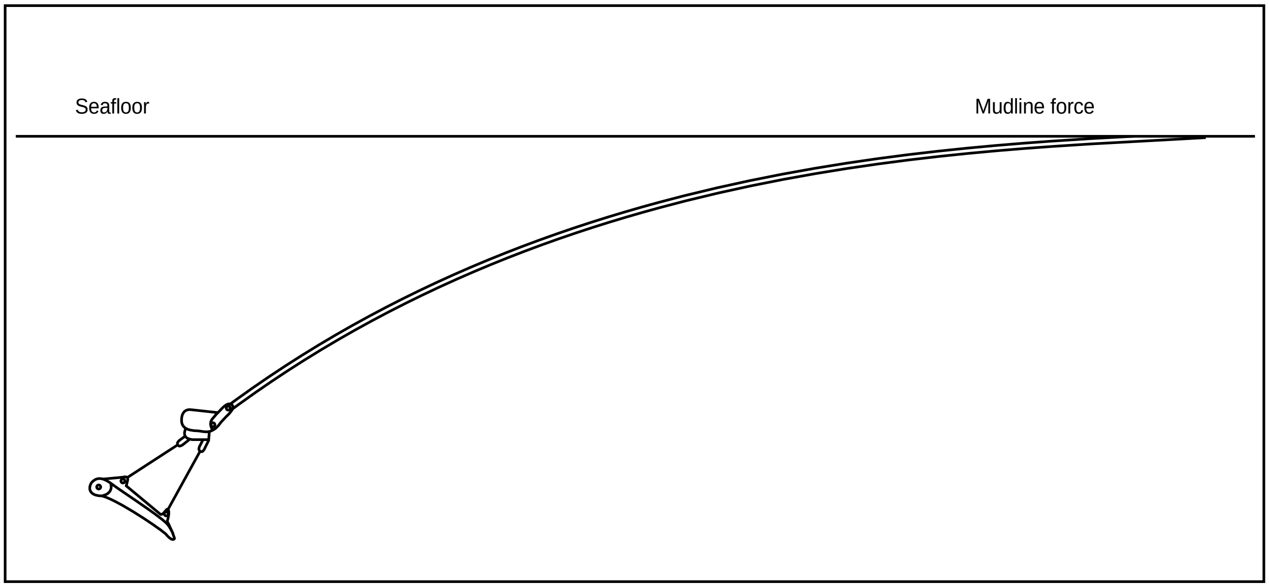



Figure E.5—Schematic of Anchor Line Configuration During Embedment

at both ends, and deformed only by its own weight plus bearing pressures exerted from the soil normal to the line and shear resistance tangent to the line. The governing differential equations for this system of forces are nonlinear and require an iterative numerical solution.

### E.4.2.2 Installation Performance

As discussed in [E.58] and [E.59], the capacity of a drag embedded plate anchor depends strongly on its final orientation and depth below the seabed, hence prediction of the anchor trajectory during installation is a critical issue. Figure E.6 is a schematic diagram showing a typical anchor trajectory and sequence of anchor orientations as the anchor line is dragged along the seabed.

Methods for predicting this scenario generally fall into four groups:

1. **Empirical Methods** are typically based on correlations with observed anchor performance and dependent on anchor characteristics (weight) and an approximated measure of soil resistance. However, many of those field studies remain proprietary, and are therefore not readily available.
2. **Limit Equilibrium Methods** take into account a more detailed description of soil and anchor geometry/weight. The method is based on an estimated soil force distribution on the anchor at failure condition; site specific soil and anchor information can be incorporated in more detail. This approach is most commonly used, and commercial software based on this approach is available.
3. **Plastic Limit Analysis** is in many ways very similar to the Limit Equilibrium Methods. Virtual work princi-

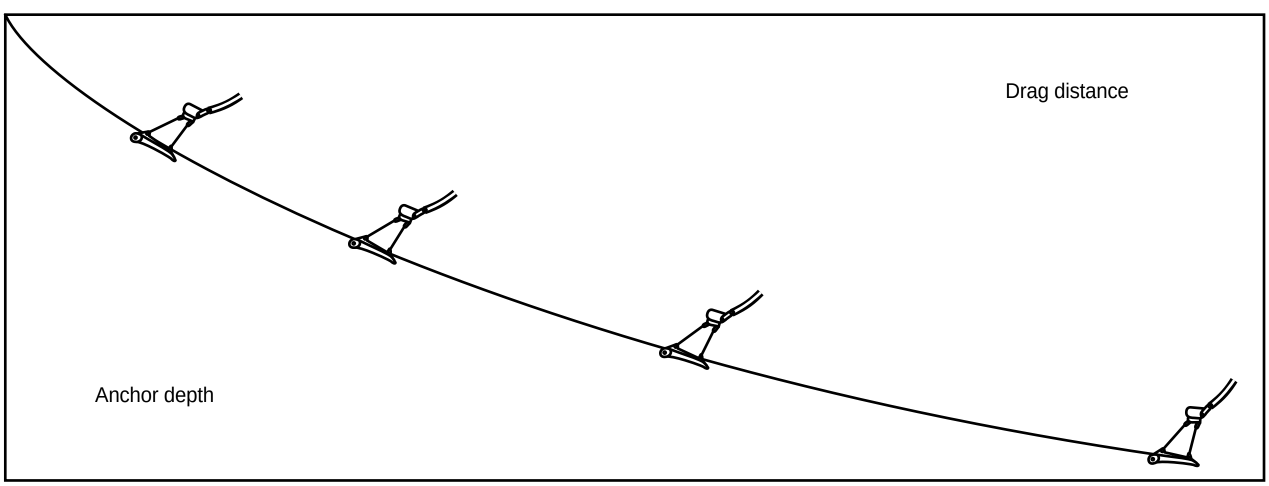



Figure E.6—Anchor Trajectory a nd Fluke Orientation During Installation ples are used to minimize the calculated failure load with respect to the geometric parameters defining the failure mechanism at any anchor depth, anchor orientation, and anchor line conditions.

4. **Advanced Numerical Methods** include usually the finite element (FE) method. It has the potential of obtaining a rigorous solution for all aspects of anchor behavior. In practice, however, it has considerable limitations. A complete solution would require a FE model defining nonlinear material behavior, nonlinear boundary conditions, large strain and large deformation theory. Hence a simple anchor trajectory prediction would require a prodigious effort to formulate, set up, and solve. However, FE models can be easily used to check calculations or enhance other prediction methods.

#### E.4.2.3 Holding Capacity Performance

As discussed in [E.58] and [E.59], anchor holding capacity, as previously mentioned, is only a special case of the installation sequence and, hence, the methods underlying installation prediction described above are directly applicable. This problem is considerably simpler than the installation problem since the ultimate holding capacity for a single location and orientation is of interest only. The ultimate holding capacity can, therefore, be expressed on the basis of conventional bearing capacity theory in conjunction with the anchor line solution:

$$F_{max} = N_c A_{eff} \eta S_u \qquad \text{(E.6)}$$

where

$F_{max}$ = Ultimate holding capacity,

$A_{eff}$ = Effective area of the anchor accounting for shape and projected area,

$N_c$ = Bearing capacity factor determined for example from method of characteristics or finite element solutions,

$\eta$ = Reduction for soil disturbance due to penetration and keying,

$S_u$ = Measure of the local undrained shear strength at the design penetration depth.

Overall, considerable judgment and experience is required to evaluate the input parameters for any of the predictive methods. An example of anchor selection can be found in [E.40].

### E.4.3 PREDICTION METHOD FOR DIRECT EMBEDDED PLATE ANCHOR

Anchor capacity determination for direct embedded plate anchors is identical to that shown for drag embedded anchors with the following exceptions:

- Final penetration depth is accurately known;
- Nominal penetration loss during keying should be included (usually taken as 0.25 to 1.0 times the fluke's vertical dimension, or $B$ in Figure E.7, depending on shank and keying flap configuration);
- Calculation of effective fluke area should use appropriate shape factor and projected area of fluke with keying flap in its set position.

### E.4.4 SPECIAL CONSIDERATION ON FACTOR OF SAFETY FOR DRAG EMBEDDED PLATE ANCHORS

Factors of safety for drag embedded plate anchors are higher than for drag anchors because overloading of anchor normally results in pullout of the anchor, while drag anchor may drag horizontally or penetrate deeper, developing constant or higher holding capacity under similar situation (see Tables 6 and 7 in Section 7.4 of main text). For plate anchors that exhibit similar overloading behavior as drag anchor, consideration may be given to using drag anchor factors of safety, assuming the behavior can be verified by significant field tests and experience.

## E.5 Structural Design of Suction Piles

### E.5.1 BASIC CONSIDERATIONS

The purpose of this section is to provide guidance and criteria for the structural design of suction piles. Some of the guidance and criteria are also applicable to driven piles. Structural design for plate anchors is not addressed because it is typically performed by anchor manufacturers.

#### E.5.1.1 Fabrication Considerations

The structural design criteria given in the following sections assume the suction pile has been fabricated to be within certain dimensions and tolerances. Although, API Specification 2B offers guidance in this area, it not considered sufficient for the fabrication of suction piles. At a minimum, the following dimensions and tolerances should be specified in the suction pile fabrication specification in addition to the pile diameter and wall thickness schedule:

1. **Pile length:** Total length of the pile should be specified with a suitable tolerance. The minimum acceptable length so specified should be acceptable with respect to the geotechnical design.

Note: Specification 2B allows a 1.5 in. tolerance per 10 ft of length; thus if the pile were 100 ft in length, it would be theoretically accept-

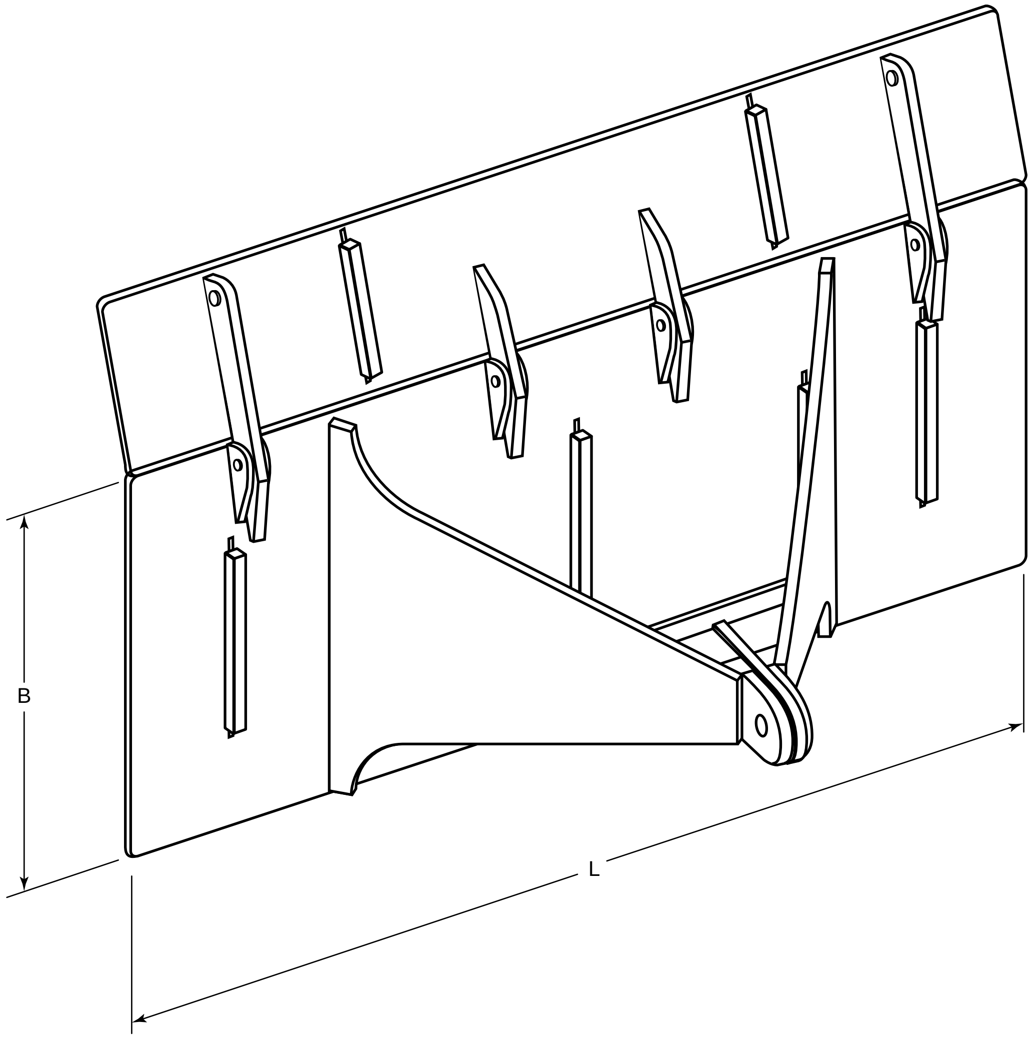



Figure E.7—Definition of Anchor Fluke Width, *B*, and Length, *L*

able if the pile were 15 in. short. This may not be acceptable if the pile design were highly optimized.

2. **Out-of-Roundness (OOR):** Out-of-roundness, the difference between the major and minor outside (or inside) diameters at any point along the length of the pile should not exceed 1% of the nominal outside (or inside) diameter. The 1% roundness value is the maximum OOR assumed in the buckling formulations given API RP 2A, Bulletin 2U and other codes.

   For each cross-section checked, a minimum of two sets of two diameters each should be checked (i.e., eight points along the circumference of the pile). In Figure E.8 that would be A-E and G-C for the first set and F-B and D-H for the second set. Note that Figure E.8 also outlines a procedure to measure OOR on cans with their longitudinal axes horizontal. This technique, to a large extent, will remove the effect of ovalization of the can due to gravity in the OOR calculations. Alternatively, OOR measurements can be made with the axis of the can vertical during diameter measurements.

3. **Circularity:** Circularity is a measure of the pile wall's local deviation from the theoretical shape; in this case, the theoretical shape is an arc with same radius as the pile. It is measured using a sweep gauge that has one edge cut to the theoretical inside or outside radius, as appropriate. The recommended sweep gauge arc length is $^{1}/_{10}$th of the circumference of the pile. Measuring circularity ensures that dents, flat spots or other geometrical imperfections do not adversely affect the buckling resistance of the cylindrical pile wall during suction embedment.

   As the sweep is moved around the circumference of the pile in a plane perpendicular to the axis of the pile, the gap between the sweep gauge and pile wall is measured. The acceptance tolerance can be determined from non-linear buckling analysis of the pile wall; a

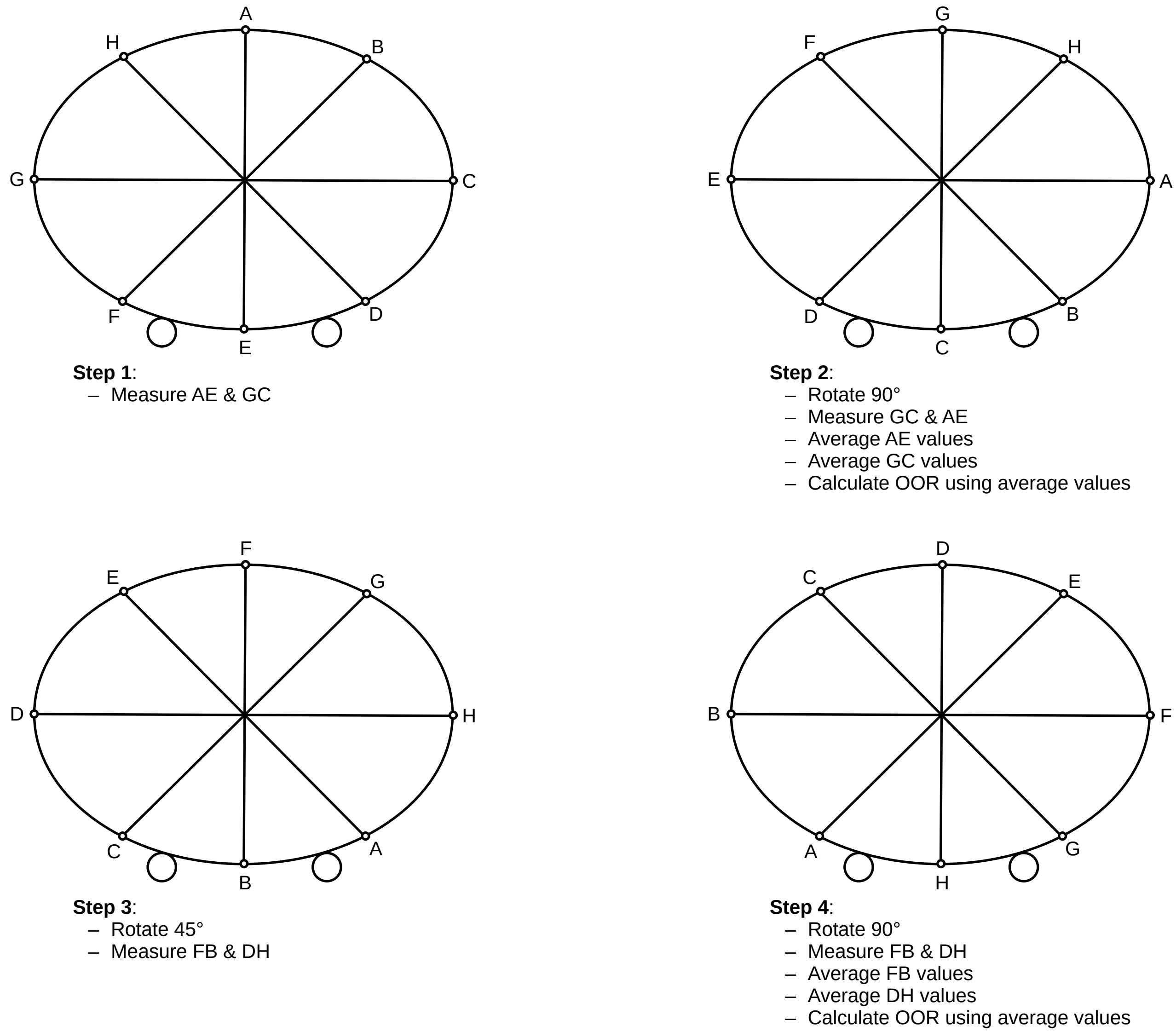



Figure E.8—Procedure to Measure Pile Out-of-Roundness

typical acceptable gap value would be 0.5 in. The number of circumferences checked along the length of the pile should be sufficient to capture all potential dents or flat spots in the pile. A straightedge, held parallel to the longitudinal axis of the pile, should be used to survey the extent of any dents found using the sweep gauge.

It is recommended that the dimensional control program include OOR and circularity checks as part of the can forming process. Individual cans should not proceed to pile assembly until passing OOR and circularity requirements. The completed pile should also be subject to final OOR and circularity checks in addition to the survey of straightness and length.

4. **Straightness:** Generally, the requirements of Specification 2B are sufficient with respect to straightness. This is due to relatively large diameter to buckling length ratio of suction piles. Thus, suction piles are usually not subject to a column buckling mode of failure that lack-of-straightness would exacerbate, but only to a local buckling failure in which global straightness is not factor.

#### E.5.1.2 Handling and Transportation Considerations

In order to achieve economic and weight control goals, it is not uncommon for suction piles to have large sections of thin walled steel. These thin pile walls are especially vulnerable to damage by inadequate temporary support during handling operations in the fabrication yard. It is recommended that temporary supports be pre-engineered prior to handling and that rigging personnel be briefed on proper suction pile handling techniques.

Another potential source of pile damage is during loadout, transportation and offloading operations. Care should be taken when loading and unloading suction piles from their cradles to minimize side or vertical impact. Cradle design and fitment of the pile into its cradle should not invalidate the transportation design assumptions. For example, if nearly full cradle contact is necessary to keep the pile stresses generated during the design environment below allowable values, the actual pile fitment should match the assumptions or the cradle design modified accordingly.

## E.5.2 DESIGN CONDITIONS

The suction pile structure should be designed to withstand the maximum loads applied by the mooring line, the maximum negative pressure required for anchor embedment, the maximum internal pressure required for anchor extraction, and the maximum loads imposed on the anchor during lifting, handling, launching, lowering and recovery. Fatigue lives of critical components and highly stressed areas of the anchor should be determined and checked against the required minimum fatigue life.

### E.5.2.1 Mooring Loads on Global Anchor Structure

The load case that provides the maximum horizontal and vertical loads at the mooring padeye should be used for the global structural design of the anchor. The soil reactions generated by the geotechnical analysis will be used in these calculations. Sensitivity checks should be performed to ensure that a load case with less than the maximum load, but applied at a more onerous angle at the padeye, does not control the design.

### E.5.2.2 Mooring Loads on Anchor Attachment

The mooring line attachment padeye or lug is a critical structural component. In order to meet fatigue resistance criteria, the padeye is often an integral cast lug and base structure. This avoids the use of heavy weldments, which can result in a lower fatigue life. The attachment padeye should be designed to satisfy both strength and fatigue requirements. The padeye should be designed for the controlling design load with an appropriate factor of safety. Designing the padeye for a maximum load equal to a factor times the break strength of the mooring line may lead to a significantly overdesigned padeye, which may not integrate well with the anchor shell and back-up structures. The mooring line padeye should be designed for the controlling load case, and sensitivity checks should be performed to ensure that a load case with less than the maximum load but applied at a more onerous angle does not control the design. The orientation of the applied load at the padeye will be affected by the inverse catenary of the mooring line, vertical misalignment due to anchor tilt, and rotational misalignment due to deviation from the target orientation. These factors should be properly accounted for.

### E.5.2.3 Embedment Loads

For anchor embedment, the estimated upper bound suction pressure required to embed the anchor to its design penetration should be used for the design of anchor wall and anchor cap structure. However, the maximum suction pressure used should not be higher than the suction at which internal plug uplift occurs.

### E.5.2.4 Extraction Loads

With respect to anchor extraction, there are two conditions that require evaluation:

1. **Temporary condition:** Extraction of a suction pile may be required for permanent moorings. For example, after all suction piles have been preinstalled along with the mooring lines, one of the mooring lines is accidentally dropped to the seafloor and damaged during the hookup operation with the vessel. At this time, a decision to extract the suction pile and recover the mooring leg may be made. Typically, such situations may occur 30 to 60 days after the first suction pile has been installed.

   For mobile moorings, the suction piles are often extracted at the end of the current drilling or testing operation and reused in other locations.

2. **Terminal condition:** The suction piles for a permanent mooring may be extracted at the end of its service life. The estimated maximum internal pressure required to extract the anchor for these two situations should be used for the design of anchor wall and anchor cap structure.

### E.5.2.5 Transportation and Handling Loads

The suction pile structure and its installation appurtenances should be designed for the maximum loads generated during suction pile handling, transportation, lifting, upending, lowering, and recovery. The suction pile designer should interface closely with the installation contractor when determining these load cases. Design of appurtenances for these load cases are typically performed using the installation contractor's in-house design guidelines or other recognized codes. Nevertheless, all lifting appurtenances and their supporting structures should meet the minimum requirements of API RP 2A.

### E.5.3 STRUCTURAL ANALYSIS METHOD

Pile analysis in accordance with Section 3 of API RP 2A is appropriate for piles with diameter to thickness ratios (D/t) of less than approximately 100 to 120. For cylindrical piles with *D/t* ratios exceeding 100 to 120, it is recommended that a detailed structural finite element model be developed for the global structural anchor analysis to ensure that the anchor wall structure and appurtenances have adequate strength in highly loaded areas. Supplementary manual calculations may be appropriate for members or appurtenances subjected to local loading.

#### E.5.3.1 Space Frame Model

A space frame model generally consists of beam elements plus other elements needed to model specific structural characteristics. This is appropriate for piles with *D/t* ratios less than approximately 100 to 120 and for preliminary design of the top cap or padeye backup structures on large diameter piles (i.e., *D/t* > 120).

#### E.5.3.2 Finite Element Model

Finite element analysis is recommended for the global shell structure, top cap plate and supporting members and the padeye backup structure for piles with D/t greater than approximately 100 to 120. Complex shapes such as the padeye casting or welding should also be analyzed by finite element methods.

#### E.5.3.3 Manual Calculations

Manual calculations using empirical formulas and basic engineering principles may be performed where detailed finite element analysis is not needed.

#### E.5.3.4 Stress Concentration Factors

Stress concentration factors can be determined by detailed finite element analysis, physical models, and other rational methods or published formulas.

#### E.5.3.5 Stability Analysis

Formulas for the calculation of the buckling strength of structural elements are presented in API Recommended Practice 2A, API Bulletin 2U, *Stability Design of Cylindrical Shells*, and API Bulletin 2V, *Design of Flat Plate Structures*. As an alternative, buckling and post-buckling analysis or model tests of specific shell or plate structures may be performed to determine buckling and ultimate strength.

#### E.5.3.6 Dynamic Response

Significant dynamic response is not expected for the anchor in its in-place condition, therefore anchor structures are often analyzed statically. Transportation analysis, however, will typically include dynamic loads generated by harmonic motions of a simple single-degree-of-freedom model.

### E.5.4 STRUCTURAL DESIGN CRITERIA

#### E.5.4.1 Design Codes

The design method adopted in this document is the working stress design method, where stresses in all components of the structure are kept within specified values. In general, cylindrical shell elements should be designed in accordance with API RP 2A for diameter to thickness ratios (*D/t*) less than 300 or API Bulletin 2U when *D/t* exceeds 300, flat plate elements in accordance with API Bulletin 2V, and all other structural elements in accordance with API RP 2A or AISC, as applicable. In cases where the structure's configurations or loading conditions are not specifically addressed by these codes, other accepted codes of practice can be used. In this case, the designer must ensure that the safety levels and design philosophy implied in the API Recommended Practice 2SK are adequately met.

In API RP 2A and AISC, allowable stress values are expressed, in most cases, as a fraction of the yield or buckling stress. In API Bulletin 2U, allowable stress values are expressed in terms of critical buckling stresses. In API Bulletin 2V, the allowable stresses are classified in two basic limit states: ultimate limit states and serviceability limit states. Ultimate limit states are associated with the failure of the structure whereas serviceability limit states are associated with adequacy of the design to meet its functional requirements. For the purpose of suction anchor design, only the ultimate limit state is considered.

#### E.5.4.2 Safety Categories

There are two safety categories: Category A safety criteria are intended for normal design conditions, and Category B safety criteria are intended for rarely occurring design conditions. The criteria in Table E.2 are recommended.

#### E.5.4.3 Allowable Stresses

For structural elements designed in accordance with API RP 2A or AISC, the allowable stresses recommended in these codes should be used for normal design conditions associated with safety criteria A. For extreme design conditions associated with safety criteria B, the allowable stresses may be increased by one-third if the working stress design method is utilized (e.g., API RP 2A-WSD).

Table E.2—Suction Pile Safety Criteria

| Load Condition | Safety Criteria |
|---|---|
| Maximum intact | A |
| Maximum one-line damaged | B |
| Anchor embedment | A |
| Anchor extraction (temporary) | A |
| Anchor extraction (terminal) | B |
| Handling / lifting / lowering / recovery | A |
| Transportation | B |

For shell structures designed in accordance with API Bulletin 2U, a factor of safety equal to 1.67 Ψ is recommended for buckling modes for safety criteria A. For safety criteria B, the corresponding factor of safety is equal to 1.25 Ψ. The parameter Ψ varies with buckling stress and is defined in API Bulletin 2U. It is equal to 1.2 for elastic buckling stresses at the proportional limit, and reduces linearly for inelastic buckling to 1.0 when the buckling stress is equal to the yield stress

For flat plate structures designed in accordance with API Bulletin 2V, the allowable stress is obtained by dividing the ultimate limit state stress by an appropriate factor of safety, which is 2.0 for safety criteria A and 1.5 for safety category B.

For cylindrical elements with $D/t$ ratios exceeding approximately 100 to 120, it is recommended that global strength be analyzed using finite element techniques. Local buckling formulations for axial compression, bending and hydrostatic pressure given in API RP 2A (for $D/t$ <300) and API Bulletin 2U ($D/t$ Š300) are considered valid if due consideration is made for variable wall thicknesses (when it occurs) and buckling length (which may extend below the mudline when performing suction embedment analysis).

The nominal Von Mises (equivalent) stress at the element's extreme fiber should not exceed the maximum permissible stress as calculated below:

$$\sigma_A = \eta_i \sigma_y \tag{E.7}$$

where

$\sigma_A$ = Allowable Von Mises stress,

$\eta_i$ = Design factor for specified load condition,

$\sigma_y$ = Specified minimum yield stress of anchor material.

Design factors for the listed load conditions are given in the table E.3.

The design factors recommended in Table E.3 above are limits on the structure's primary stresses generated by the applied loads. Note that primary stresses are not self-limiting; i.e. primary stresses that exceed the yield strength of the material in question can result in failure. Secondary stresses, on the other hand, can be developed by local structural discontinuities or by constraint of adjacent parts; such stresses are self-limiting. In some cases, it may be acceptable to exceed material yield for secondary stresses in elastic design. If the designer allows local yielding, he should ensure the material has sufficient ductility and that load will be able to redistribute to adjacent areas of the structure.

Table E.3—Design Factors for Finite Element Analysis

| Load Condition | Design Factor $\eta_i$ |
|---|---|
| Maximum Intact | 0.67 |
| Maximum Damaged | 0.90 |
| Anchor embedment | 0.67 |
| Anchor extraction (temporary) | 0.67 |
| Anchor extraction (terminal) | 0.90 |
| Handling / lifting / lowering / recovery | 0.67 |
| Transportation | 0.90 |

### E.5.5 FATIGUE DESIGN

#### E.5.5.1 Fatigue Analysis

In-place fatigue of the anchor structure is caused by the tension-tension cyclic loading of the mooring line attached to the anchor. The fatigue analysis for suction anchor is similar to that for the mooring system, which is discussed in Section 6 in the main body of this recommended practice. The major differences are:

1. The S-N (stress range versus number of cycles to failure) approach is recommended for suction anchor fatigue analysis instead of the T-N (normalized tension range versus number of cycles to failure) approach used for the mooring system. Appropriate S-N curve formulations that include the effect of member thickness should be utilized.
2. Normalized tension ranges from mooring fatigue analysis are converted to stress ranges for suction anchor using highly refined finite element models.

Fatigue should be checked not only at joints, but also at any details with high stress concentrations; e.g., the padeye casting at the base of the lug and in the eye.

#### E.5.5.2 Fatigue Life Requirement

The predicted minimum fatigue life shall be at least 3 times the design service life of the suction anchor.

## E.6 Installation of Suction Piles, Suction Caissons and Plate Anchors

### E.6.1 SUCTION PILES AND SUCTION CAISSONS

In order to verify that the suction pile installation is successful and in agreement with the assumptions in design, the following data should be monitored and recorded during the installation of suction piles, for permanent and mobile moorings:

- Distance from intended seabed location,
- Underpressure,
- Penetration depth,
- Penetration rate,
- Verticality,
- Orientation.

For permanent mooring systems, other parameters usually monitored include plug stability at all depths and plug heave at final penetration.

### E.6.2 PLATE ANCHORS

#### E.6.2.1 Direct Embedded Plate Anchors

Direct embedment of plate anchors can be achieved by suction, impact or vibratory hammer, propellant, hydraulic ram, or gravity. The suction embedded plate anchor has been used for major offshore mooring operations. As an example, the SEPLA (Suction Embedded PLate Anchor) uses a so-called suction follower, which is essentially a reusable suction anchor with its tip slotted for insertion of a plate anchor. The suction follower is immediately retracted by reversing the pumping action once the plate anchor is brought to the design depth, and can be used to install additional plate anchors. In the SEPLA concept, the plate anchor's fluke is embedded in vertical position and necessary fluke rotation is achieved during a keying process by pulling on the mooring line.

Installation procedures should be developed and installation analyses should be performed for direct embedded plate anchors to verify that the anchors can be penetrated to the design depth. The installation analysis should also consider plate anchor retrieval if applicable.

For the embedment analysis, the risk of causing uplift of the soil plug inside the suction embedment tool should be considered. The allowable underpressure to avoid uplift should exceed the required embedment pressure by a factor of 1.5 (see Section E.3.2.1.3). Plate anchor installation tolerances should be established and should be considered in the anchor's geotechnical, structural, and installation design. Typical tolerances to be considered are:

- Allowable deviation from target heading of the mooring line attachment to limit padeye side loads and rotational moments on the anchor padeye,
- Minimum penetration required before keying or test loading to achieve the required holding capacity,
- Allowable loss of anchor penetration during plate anchor keying or test loading.

In order to verify that the plate anchor installation is successful and in agreement with the assumptions in design, the parameters listed in Section 6.1 should also be recorded.

#### E.6.2.2 Drag Embedded Plate Anchors

For drag embedded plate anchors used in permanent moorings, the installation process should provide adequate information to ensure that the anchor reaches the target penetration, and that the drag embedment loads are within the expected load range for the design soil conditions. Typical information to be monitored and verified is:

- Line load in drag installation line;
- Catenary shape of installation line based on line tension and line length to verify that uplift at the seabed during embedment is within allowable ranges and to verify anchor position;
- Direction of anchor embedment;
- Anchor penetration.

### E.6.3 TEST LOADING OF ANCHORS

For suction piles, suction caissons, and plate anchors, the installation records should demonstrate that the anchor penetration is within the range of upper and lower bound penetration predictions developed during the anchor geotechnical design. In addition, the installation records should confirm the installation behavior, i.e. self weight penetration, embedment pressures, drag embedment loads, and that the anchor orientation is consistent with the anchor design analysis. Under these conditions, test loading of the anchor, as per Section 7.4.3 of main text, should not be required. However, the mooring and anchor design should define a minimum acceptable level of test loading. This test loading should ensure that the mooring line's inverse catenary is sufficiently formed to prevent unacceptable mooring line slacking due to additional inverse catenary cut-in during storm conditions.

Plate anchors should be subjected to adequate keying loads to ensure that sufficient anchor fluke rotation will take place without further loss of anchor penetration. The keying load required and amount of estimated fluke rotation should be based on reliable geotechnical analysis and verified by prototype or scale model testing. The keying analysis used to establish the keying load should also include analysis of the anchor's rotation when subjected to the maximum intact and

1-line damage survival loads. If the calculated anchor rotation during keying differs from the anchor rotation in survival conditions, then the anchor's structure should be designed for any resulting out-of-line loading to ensure that the anchor's structural integrity will not be compromised.

In cases where the installation records show significant deviation from the predicted values and these deviations indicate that the anchor holding capacity may be compromised, test loading of anchor, as per Section 7.4.3 of main text, may be required and may be an acceptable option to prove holding capacity for temporary moorings. However, testing anchors to the maximum intact load does not necessarily prove that required anchor holding safety factors are met, which is of special concern for permanent mooring systems. Consequently, if the installation records show that the anchor holding capacity is significantly smaller than calculated and factors of safety are not met, then other measures, as listed below, to ensure adequate factors of safety should be considered.

- Additional soil investigation at the anchor location to establish and/or confirm soil properties at the anchor site,
- Retrieval of the anchor and re-installation at a new undisturbed location,
- Retrieval of the anchor, redesign and reconstruction of the anchor to meet design requirements and re-installation at new undisturbed location,
- Delay of vessel hookup to provide additional soil consolidation.

Drag embedded plate anchors should be test loaded, as per Section 7.4.3 of main text, unless one of the following conditions are complied with:

- The anchor installation load (drag-in load) is equal or higher than the anchor required test load, as per as per Section 7.4.3 of main text, and the anchor is not keyed in the opposite direction,
- Soil properties at the anchor locations have been established in accordance with E.2, the depth of the anchor after keying is known with reasonable accuracy and is not less than the minimum depth used for the design of the anchor.

## E.7 Driven Pile Anchor

### E.7.1 BASIC CONSIDERATIONS

Driven pile anchors provide a large vertical load capacity for taut mooring systems. The design of driven pile anchors builds on a strong industry background in the evaluation of geotechnical properties and the axial and lateral capacity prediction for driven piles. The calculation of driven pile capacities, as developed for fixed offshore structures, is well documented in API RP 2A. The recommended criteria in API RP 2A should be applied for the design of driven anchor piles, but with some modifications to reflect the differences between mooring anchor piles and fixed platform piles. Some of the guidance provided in Section E5 for structural capacity of suction piles may also be applicable for driven piles. The design of a driven pile anchor should consider four potential failure modes:

1. Pull-out due to axial load.
2. Overstress of the pile and padeye due to lateral bending.
3. Lateral rotation and/or translation.
4. Fatigue due to environmental and installation loads.

Factors of safety for holding capacity, defined as the calculated soil resistance divided by the maximum anchor load from dynamic analysis, are provided in Table 7. Information on coupling between vertical and horizontal capacities can be found in Section E.3.4. Axial safety factors consider that the pile is primarily loaded in tension, and are therefore higher than for piles loaded in compression. As with other piled foundation systems, the calculated ultimate axial soil resistance should be reduced if soil set-up, which is a function of time after pile installation, will not be complete before significant loads are imposed on the anchor pile.

As the lateral failure mode for piles is considered to be less catastrophic than the vertical, lower factors of safety have been recommended in Table 7 (see Section 7.4) for lateral pile capacity. Use of separate factors of safety for vertical and lateral pile capacities may be straightforward for simple beam-column analysis of, for example, mobile moorings (ref. Section E.3.2.3), but more complex methodologies do not differentiate between vertical and lateral pile resistance. The safety factor should be in accordance with the guidelines of Section E.3.5.

### E.7.2 GEOTECHNICAL AND STRUCTURAL STRENGTH DESIGN

In most anchor pile designs, the mooring line is attached to the pile below the seafloor, to enhance the lateral capacity. As a result, the design should consider the mooring line angle at padeye connection resulting from the reverse catenary through the upper soil layers. Calculation of the soil resistance above the padeye location should also consider remolding effects due to this trenching of the mooring line through the upper soil layers.

Driven pile anchors in soft clay typically have aspect ratios (penetration/diameter) of 25–30. Piles having such an aspect ratio would be fixed in position about the pile tip, and consequently would deflect laterally and fail in bending before translating laterally as a unit. Driven pile anchors are typically analyzed using a beam-column method with a lateral load-deflection model (P-y curves) for the soil, with an awareness of the limitations of such models as described in Section E.3.2.3.3. These computations should include the axial loading in the pile, as well as the mooring line attachment point, which will influence the deflection, shear, and bending moment profiles along the pile. Pile stresses should be limited to the basic allowable values in API RP 2A under intact condition. Basic allowable stresses may be increased by one-third for rarely occurring design conditions such as one-line damaged condition (see Sections E.5.4.2 and E.5.4.3 for more detailed discussion).

As argued in [E.45], "static" P-y curves may be considered for calculation of lateral soil resistance. "Cyclic" P-y curves may be more appropriate for fatigue calculations. A modification to the API RP 2A P-y curves has been proposed in [E.5], to ensure that lateral deflections are not over-predicted. Consideration should be given to degrading the P-y curves for deflections greater than 10% of the pile diameter. In addition, when lateral deflections associated with cyclic loads at or near the mudline are relatively large (e.g., exceeding $y_c$ as defined in API RP 2A for soft clay), consideration should be given to reducing or neglecting the soil-pile adhesion (skin friction) through this zone.

The design of driven anchor piles should consider typical installation tolerances, which may affect the calculated soil resistance and the pile structure. Pile verticality affects the angle of the mooring line at the padeye, which changes the components of horizontal and vertical mooring line load that the pile must resist. Underdrive will affect the axial pile capacity and may result in higher bending stresses in the pile. Padeye orientation (azimuth) may affect the local stresses in the padeye and connecting shackle. Horizontal positioning may affect the mooring scope and/or angle at the vessel fairlead, and should be considered when balancing mooring line pretensions.

## E.7.3 FATIGUE DESIGN

### E.7.3.1 Basic Considerations

Anchor piles should be checked for fatigue caused by in-place mooring line loads. Fatigue damage due to pile driving stresses should also be calculated and combined with in-place fatigue damage. For typical mooring systems, fatigue damage due to pile driving is much higher than that caused by in-place mooring line loads.

### E.7.3.2 In-Place Loading

A global pile response analysis accounting for the pile-soil interaction should be carried out for the mooring line reactions due to the fatigue seastates acting on the system. The local stresses that accumulate fatigue damage in the pile should be obtained by calculating a SCF (Stress Concentration Factor), relative to the nominal stresses generated by the global analysis, at the fatigue critical locations. These locations are typically at the padeye, at the girth welds between the padeye and the pile, and between subsequent pile cans.

The evaluation of SCFs for girth welds needs to account for the local thickness misalignment at the weld. Equations for SCFs are given in [E.15, E.16]. Note that the calculated SCF needs to be corrected by the ratio of the nominal thickness used in the pile response analysis to the lesser of the pile wall thicknesses joining at the weld. The SCF is to be applied to the nominal pile stress range obtained at the weld location due to in-place loads, from which damage is to be calculated.

### E.7.3.3 Installation Loading

Dynamic loads due to hammer impact during pile installation will induce fatigue damage on both padeye and pile girth welds. The evaluation of the cyclic loads involves the dynamic response of the pile-soil system due to the hammer impact. This requires a wave equation analysis per blow for a given hammer type and efficiency, pile penetration, and soil resistance. Various such analyses are to be conducted for judiciously selected pile penetrations. For each analysis, traces of stress versus time at the critical locations along the pile are to be developed, as well as the number of blows associated with the assumed penetration.

For either welds or padeye, fatigue load calculations should be carried out at various pile locations using local stress range, derived from the wave equation analysis at the selected pile penetrations. The location of the girth weld should be determined by the pile makeup schedule. The local response should include the corresponding SCF effect. The number of cycles of the stress history per blow is obtained using a variable amplitude counting method, such as the reservoir [E.17] or rainflow methods.

### E.7.3.4 Fatigue Resistance

Applicable SN curves depend on manufacturing processes and defect acceptance criteria. Typically, pile sections are welded by a two-sided SAW process and are left in the as-welded condition. For this case, the D curve, as defined in [E.18], may be used. Use of a higher SN curve for this application, without additional treatment of the weld, should be demonstrated by relevant data. Use of weld treatment methods, such as grinding, may support the upgrading of the SN curve, provided that (1) the grinding process is properly implemented, (2) weld inspection methods and defect acceptance criteria are implemented, and (3) pertinent fatigue data are generated to qualify the weld to a performance level higher than that implied by the D curve.

### E.7.3.5 Total Fatigue Damage and Factor of Safety

Once the fatigue loading and resistance are determined, fatigue damage due to in-place and installation loads can be evaluated using procedures similar to those described in Section 6 and Section E.5.5. The total fatigue damage should satisfy the following equation for the critical structural elements

$$D = F(D_1) + F(D_2) < 1 \qquad \text{(E.8)}$$

where

$F$ = Factor of safety, equal to 3.0,

$D_1$ = Calculated fatigue damage for Phase 1, i.e., installation (pile driving) phase and transportation phase, if significant.

$D_2$ = Calculated fatigue damage for Phase 2, i.e., in-service phase, during the service life (e.g., 20 years).

Further discussions on fatigue damage design for driven piles can be found in [E.18, E.19].

### E.7.4 TEST LOADING OF DRIVEN PILE ANCHORS

The driven pile installation records should demonstrate that the pile self weight penetration, pile orientation, driving records and final penetration are within the ranges established during pile design and pile driving analysis. Under these circumstances, test loading of the anchor to full intact storm load should not be required. However, the mooring and anchor design should define a minimum acceptable level of test loading. This test loading should ensure that the mooring line's inverse catenary is sufficiently formed to prevent unacceptable mooring line slacking during storm conditions due to additional inverse catenary cut-in. Another function of the test loading is to detect severe damage to the mooring components during installation.

## E.8 References


**E.1.** Doyle, E. H. (1998), "The Integration of Deepwater Geohazard Evaluations and Geotechnical Studies," 30th Annual Offshore Technology Conference, OTC 8590, Houston, May.

**E.2.** Jeanjean P., Andersen K., and Kalsnes, B. (1998), "Soil Parameters for Design of Suction Caissons for Gulf of Mexico Deepwater Clays," Proc. Offshore Technology Conference, Houston, OTC 8830,Houston, May 1998.

**E.3.** Andersen, K. H. and Jostad, H. P. (1999), "Foundation Design of Skirted Foundations and Anchors in Clay," 31st Annual Offshore Technology Conference, OTC 10824, Houston, May.

**E.4.** Murff, J. D. and Hamilton, J. M. (1993), "P-Ultimate For Undrained Analysis Of Laterally Loaded Piles," *Journal of Geotechnical Engineering*, ASCE, Vol. 119, No. 1, pp. 91-107.

**E.5.** J-B. Stevens and J-M. Audibert, "Re-examination of P-y Curve Formulations," OTC #3402, May 1979.

**E.6.** NGI (1999): "Skirted Foundation in Clay. State Of The Art. Set-Up Effects Outside Skirt Wall," Joint Industry Project. NGI Report 524071-2; 11 May 1999.

**E.7.** Andersen, K. H. and Jostad, H. P. (2002), "Shear Strength Along Outside Wall of Suction Anchors in Clay After Installation," Proceedings, 12th ISOPE Conference, Houston, ISOPE 10824.

**E.8.** Randolph, M. and A. House (2002), "Analysis of Suction Caisson Capacity in Clay," Proc. Offshore Technology Conference, Houston, OTC14236.

**E.9.** Dendani, H. And J-L. Colliat (2002), Girassol: "Design Analysis and Installation of the Suction Anchors," Proc. Offshore Technology Conference, Houston, OTC14209.

**E.10.** Naval Civil Engineering Laboratory, *Handbook for Marine Geotechnical Engineering*, Karl Rocker; March 1985.

**E.11.** Erbrich, C. And P. Hefer (2002), "Installation of the Laminaria Suction Piles—A Case History," Proc. Offshore Technology Conference, Houston, OTC14240.

**E.12.** Clukey, E.C., "Suction Caisson Design Issues, Geological, Geotechnical, and Geophysical Properties of Deepwater Sediments," Proc. OTRC 2001 Int'l Conf. Honoring Prof. W. A. Dunlap, Conducted by the Offshore Technology Research Center, edited by C. Aubeny and J--L. Briaud, Houston, May.

**E.13.** Gilbert, R. B. and Murff, J. D. (2001), "Workshop Report—Design Methodologies and Criteria for Suction Caissons for Deepwater Mooring Applications," Prepared by Offshore Technology Research Center, College Station, June.

**E.14.** Norwegian Geotechnical Institute, "Skirted Foundations and Anchors in Clay. State-of-the-Art Report," Joint Industry Project. NGI Report 524071-1. 15 December 1997. Confidential.

**E.15.** Buitrago, Jaime, and Zettlemoyer, Nicholas, "Fatigue Design of Critical Girth Welds for Deepwater Applications," Proceedings, Offshore Mechanics and Arctic Engineering, Paper 98-2004, Lisbon, July 1998.

**E.16.** Connelly, L. M. and Zettlemoyer, N., "Stress Concentrations at Girth Welds of Tubulars With Axial Wall Misalignment," Proceedings, 5th International Symposium of Tubular Structures, Nottingham, U.K., August 1993.

**E.17.** British Standards Institute, *Code of Practice for Fatigue Design and Assessment of Steel Structures*, BS 7608, 1993.

**E.18.** Hunt, R. J-, Chan, Jack H-C, and Doyle, Earl H., "Driving Fatigue Estimation for Ursa TLP 96" OD Piles," Proceedings, 9th International Offshore and Polar Engineering Conference, Brest, France, May 1999.

**E.19.** Buitrago, J. and Wong, P. C., "Fatigue Design of Driven Piles for Deepwater Applications," Proceedings, Vol. II, 13th International Offshore and Polar Engineering Conference, Honolulu, Hawaii, May 25-30, 2003, pp. 726-733.

**E.20.** Bogard, D. "Effective Stress and Axial Pile Capacity: Lessons Learned from Empire, OTC paper # 13059, May 2001.

**E.21.** Bogard, D. and Matlock, H.: "Application of Model Pile Tests to Axial Pile Design," OTC paper # 6376, May 1990.

**E.22.** Doyle, E. H., (1998), "The Integration of Deepwater Geohazard Evaluations and Geotechnical Studies," 30th Annual Offshore Technology Conference, OTC 8590, Houston, May.

**E.23.** Jeanjean, P., Campbell, K., and Kalsnes, B.: "Use of Integrated Study to Characterize the Marlin Deepwater Field.," Proc. Society of Underwater Technology Conference, September 1998, London.

**E.24.** Dutt, R. N., Rainey, W. S., Hamilton, T. K., Pelletier, J. H., and Doyle, E. H., (1997), "Recent Advances in Deepwater Gulf of Mexico Geotechnical Investigations," 29th Annual Offshore Technology Conference, OTC 8303, Houston, May.

**E.25.** Young, A. G., C. D. Honganen, A. J. Silva, and W. R. Bryant, 2000, "Comparison of Geotechnical Properties from Large-diameter Long Cores and Borings in Deep Water Gulf of Mexico," 32nd Annual Offshore Technology Conference, Houston, Texas (OTC 12089).

**E.26.** Borel, D., Puech, A., Dendani, H, and de Ruijter, M., (2002), "High Quality Sampling for Deepwater Engineering: the STACOR Experience," Ultra Deep Engineering and Technology Conference (UDET 2002), Brest, France.

**E.27.** Doyle, E. H. and Kaluza, M. J-, (2001), "A Review on the Use of 3-D Exploration-Level Data for Geohazard Studies," Proc. OTRC 2001 Int'l Conf. Honoring Prof. W. A. Dunlap, Conducted by the Offshore Technology Research Center, edited by C. Aubeny and J--L. Briaud, Houston, May.

**E.28.** Andersen K. H. and Lauritzen, R., (1988), "Bearing Capacity for Foundations with Cyclic Loads," *ASCE Jour Geotechnical Engr.*, v114, no. GT5, pp 540-555.

**E.29.** Andersen, K. H., Kleven, A. and Heien, D, (1988), "Cyclic Soil Data for Design of Gravity Structures," ASCE, *Jour Geotechnical Engr.*, v 114,GT5, pp 517-539.

**E.30.** Dutt, R. N., Doyle, E. H. and Ladd, R. S., (1992), "Cyclic Behavior of a Deepwater Normally Consolidated Clay," *ASCE Civil Engineering In The Oceans V*, Texas A&M University, November 2-4.

**E.31.** Skempton, A. W., (1951): "The Bearing Capacity Of Clays," Proceedings, Building Research Congress, London, pp. 180-189

**E.32.** Lunne, T., Roberston, P.K., Powell, J. (1997): *Cone Penetration Testing in Geotechnical Practice*, ISBN 0419 23750 X, Spon Press. 312 pages.

**E.33.** Newlin, J-A.: "Suction Anchor Piles for the Na Kika FDS Mooring System. Part 1: Site Characterization and Design.," Proceedings, International Symposium "Deepwater Mooring Systems: Concept Design, Analysis, and Materials, Oct. 2-3, 2003, Houston, TX.

**E.34.** Newlin, J-A.: "Suction Anchor Piles for the Na Kika FDS Mooring system. Part 2: Installation Performance," Proceedings, International Symposium "Deepwater Mooring systems: Concept Design, Analysis, and Materials, Oct. 2-3, 2003, Houston, TX.

**E.35.** DNV (2002): *Recommended Practice DNV-RP-E302—Design and Installation of Plate Anchors in Clay*, Issued December 2002.

**E.36.** Andersen, K., Murff, J. D., and Randolph, M. (2004): "Deepwater Anchor Design Practice—Phase II Report to API/Deepstar—Volume II-4 Suction Caisson Anchors—Findings and Recommendations," Report submitted to API/ Deepstar.

**E.37.** Andersen, K., Andresen, L., Jostad, H.P., and Clukey, E., (2004): "Effect of Skirt Tip Geometry on Set-up Outside Suction Anchors In Soft Clays," Paper OMAE2004-51564, Proceedings, 24th OMAE Conference, Vancouver.

**E.38.** Chen, W. F., (1975). *Limit Analysis and Soil Plasticity*, Elsevier Publishing Co., Amsterdam, The Netherlands.

**E.39.** Randolph, M. F. and Houlsby, G. T. (1984), *The Limiting Pressure on a Circular Pile Loaded Laterally in Cohesive Soil*, Geotechnique, London, England, 34(4), 613-623.

**E.40.** Ruinen, R. (2004): "Penetration Analysis of Drag Embedment Anchors in Soft Clays," Presented at ISOPE Conference 2004.

**E.41.** Randolph, M. F. and House, A. R. (2002), "Analysis of Suction Caisson Capacity in Clay," Paper No. OTC14236, Proceedings of the Offshore Technology Conference, Houston, Texas, May 6-9.

**E.42.** McClelland, B. and Focht, J. A. (1958), "Soil Modulus for Laterally Loaded Piles," *Transactions ASCE*, Vol. 123: 1049.

**E.43.** Matlock, H. (1963), “Applications of Numerical Methods to Some Structural Problems in Offshore Operations,” *Journal of Petroleum Technology*, 1040-1046.

**E.44.** Matlock, H. (1970), Correlations for Design of Laterally Loaded Piles in Soft Clay,” Proc. Offshore Technology Conference: OTC Paper No. 1204, Houston.

**E.45.** Hamilton J-M. and J-D. Murff (1995), “Ultimate Lateral Capacity of Piles in Clay,” Proc. Offshore Technology Conference, Houston, OTC 7667, Houston, May 1995.

**E.46.** Grande, L. (1976) “Samvirke mellom pel og jord.” Lic. Techn. Thesis, NTH, Institute for Soil Mechanics and Foundation Engineering. Trondheim, Norway.

**E.47.** Colliat, J-L., Boisard, P., Andersen, K.H. and Schroeder, K. (1995), “Caisson Foundations As Alternative Anchors For Permanent Mooring Of A Process Barge Offshore Congo,” Proceedings of the 27th Annual Offshore Technology Conference, Houston, Texas, OTC 7797.

**E.48.** Randolph MF, Hefer, PA, Geise JM, and Watson PG. (1998b), “Improved Seabed Strength profiling Using T-Bar Penetrometer,” Proc. Conf. Offshore Site Investigation and Foundation, Behaviour ‘98, Society for Underwater Technology.

**E.49.** Whittle, A.J. & Sutabutr, T., (1999), “Prediction of Pile Set-up in Clay,” Transportation Research Record, 1663, 33-41.

**E.50.** Sparrevik, P.: “Suction Anchor—A versatile Foundation Concept Finding Its Place in the Offshore Market,” Proceedings of the 17th International Conference on Offshore Mechanics and Arctic Engineering, OMAE 98-3096.

**E.51.** Randolph, M., O’Neill, M.P., and Stewart, D.P: (1998): “Performance of Suction Anchors in Fine-Grained Calcareous Soils,” Paper #8831, Proceedings, Offshore Technology Conference.

**E.52.** Andersen, K., Murff, J. D., and Randolph, M. (2004): “Deepwater Anchor Design Practice—Phase II Report to API/Deepstar—Volume II—Suction Caisson Anchors,” Report submitted to API/Deepstar.

**E.53.** Andersen, K., Murff, J. D., and Randolph, M. (2004): “Deepwater Anchor Design Practice—Phase II Report to API/Deepstar—Volume II-2 Suction Caisson Anchors—3D Finite Element Analyses and Comparison with Simplified Capacity Prediction Methods,” Report submitted to API/Deepstar.

**E.54.** Andersen, K., Murff, J. D., and Randolph, M. (2004): “Deepwater Anchor Design Practice—Phase II Report to API/Deepstar—Volume II-3 Suction Caisson Anchors—Back Calculation of Prototype Installation Behavior,” Report submitted to API/Deepstar.

**E.55.** Clukey, E, Aubeny, C., and Murff, D. (2003): “Comparison Of Analytical And Centrifuge Model Tests For Suction Caissons Subjected To Combined Loads.,” Proceedings, 22nd International Conference on Offshore Mechanics and Arctic Engineering, Cancun, Mexico.

**E.56.** Aubeny, C., Han, S., and Murff, D. (2003): “Refined Model for Inclined Load Capacity of Suction Caissons,” Proceedings, 22nd International Conference on Offshore Mechanics and Arctic Engineering, Cancun, Mexico.

**E.57.** Clukey, E., Templeton, J-, and Randolph, M. (2004): “Suction Caisson Response Under Sustained Loop Current Loads,” Proceedings, Offshore Technology Conference, Paper #16843, Houston, TX.

**E.58.** Andersen, K., Murff, J. D., and Randolph, M. (2004): “Deepwater Anchor Design Practice—Phase II Report to API/Deepstar—Volume III—Vertically loaded Drag Anchors,” Report submitted to API/Deepstar,

**E.59.** Andersen, K., Murff, J. D., and Randolph, M. (2004): “Deepwater Anchor Design Practice—Phase II Report to API/Deepstar—Volume III-4—Vertically Loaded Drag Anchors—Findings and Recommendations,” Report submitted to API/Deepstar.

**E.60.** DNV (2002): “Recommended Practice DNV-RP-E302—Design and Installation of Plate Anchors in Clay,” Issued December 2002,

# APPENDIX F—DETERMINATION OF AVAILABLE THRUST

## F.1 Introduction

This appendix provides guidelines for the determination of the thrust generated by various types of propulsion devices. Also addressed is the influence of the installation and arrangement of the propulsion devices, which often leads to a reduction of the available thrust (net force acting upon the vessel).

The guidelines apply to typical propulsion devices and installation scenarios for DP vessels supporting offshore operations. These include the following:

a. Open and nozzled propellers installed in the stern of a ship-shaped vessel (conventional main propulsion arrangement).

b. Azimuthing or fixed-direction, nozzled thrusters installed under the bottom of a hull.

c. Tunnel thrusters installed in a transverse tunnel in a hull.

Two methods of thrupst evaluation are provided:

a. Tables and figures for quick and rough estimates that can be used for the design of thruster assisted mooring and preliminary design of a DP system.

b. References for more rigorous determination of available thrust. They can be used for the final design of a DP system.

The estimated available thrust as determined by this appendix should be further reduced under certain conditions as specified in 5.9.

## F.2 Performance Criteria

The performance of a conventional propeller, designed to power a vessel at a certain speed, is normally expressed by the efficiency of the propeller. During stationkeeping, however, the propeller operates at zero inflow velocity (or at very low speeds), and the application of an efficiency expression is not feasible. A popular expression for the performance of a propeller in stationkeeping application is the specific thrust: propeller thrust per horsepower.

Every propeller delivers maximum thrust at zero inflow velocity. Even in the case of a constant power operation (which is feasible, for example, with controllable pitch propellers, or fixed-pitch propellers driven by certain prime movers), the propeller thrust decreases with increasing inflow velocity. Inflow velocity is caused by either current speed, movement of the vessel, or the jet from another propulsion device. For the analysis of the stationkeeping propeller, the maximum thrust at zero inflow (or bollard pull condition) will be considered the benchmark performance.

To determine the available thrust (or net force acting upon the vessel), the propeller thrust at bollard pull has to be calculated first. This thrust has to be corrected by applying thrust deduction factors. These factors depend on the following:

a. Propeller/thruster installation geometry and arrangement.

b. Inflow velocity into the propeller.

c. Propeller sense of rotation (ahead or reverse operation).

## F.3 Propeller Thrust at Bollard Pull

The following paragraphs (F.3.1 and F.3.2) provide guidelines for the calculation of the thrust generated by open and nozzled propellers at bollard pull and certain inflow velocities, if required. Any deduction caused by the factors mentioned in Section 3 have to be applied later. The calculations of these deductions are discussed in Section 5. The calculation of the thrust produced by tunnel thrusters and its associated deductions are described in Section 6.

### F.3.1 OPEN PROPELLERS

Figure F.1 can be used for quick determination of the propeller thrust at zero speed for an open propeller. Required input data are propeller diameter and the power applied. The diagram clearly indicates that, for a given power, the thrust increases with increasing propeller diameter. It also indicates that for a given propeller the specific thrust increases with decreasing load. See C.6 for a list of references [C.1, C.2, C.3, C.4, and C.5] that provide detailed information and data for the design and performance calculation of open propellers.

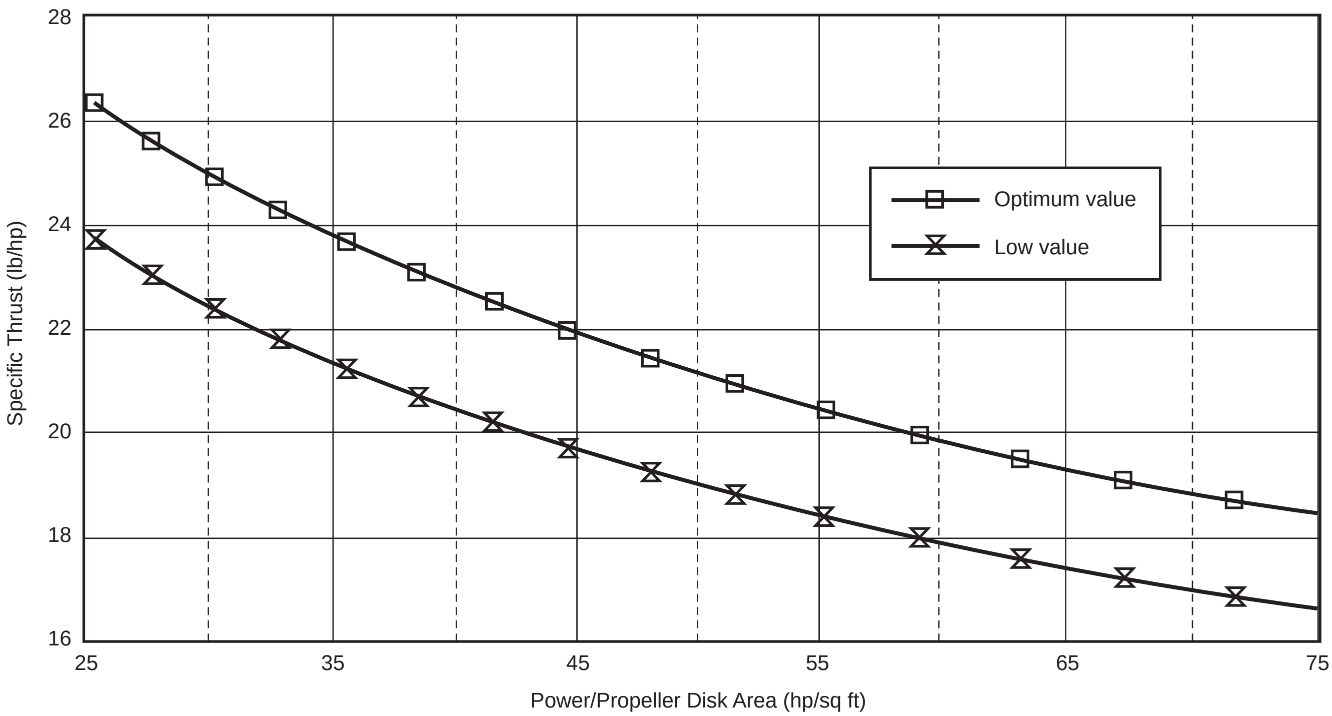
### F.3.2 NOZZLED PROPELLERS

Figure F.2 allows a quick determination of the propeller thrust at zero speed for a nozzled propeller. The same basic considerations apply as discussed above for open propellers. The diagram also indicates the considerable increase of thrust for a nozzled propeller in comparison with an open propeller of same diameter and power load. See C.6 for references [C1, C7, and C8] that provide detailed information and data for the design and performance calculation of nozzled propellers.

## F.4 Calculation of Thrust Deductions

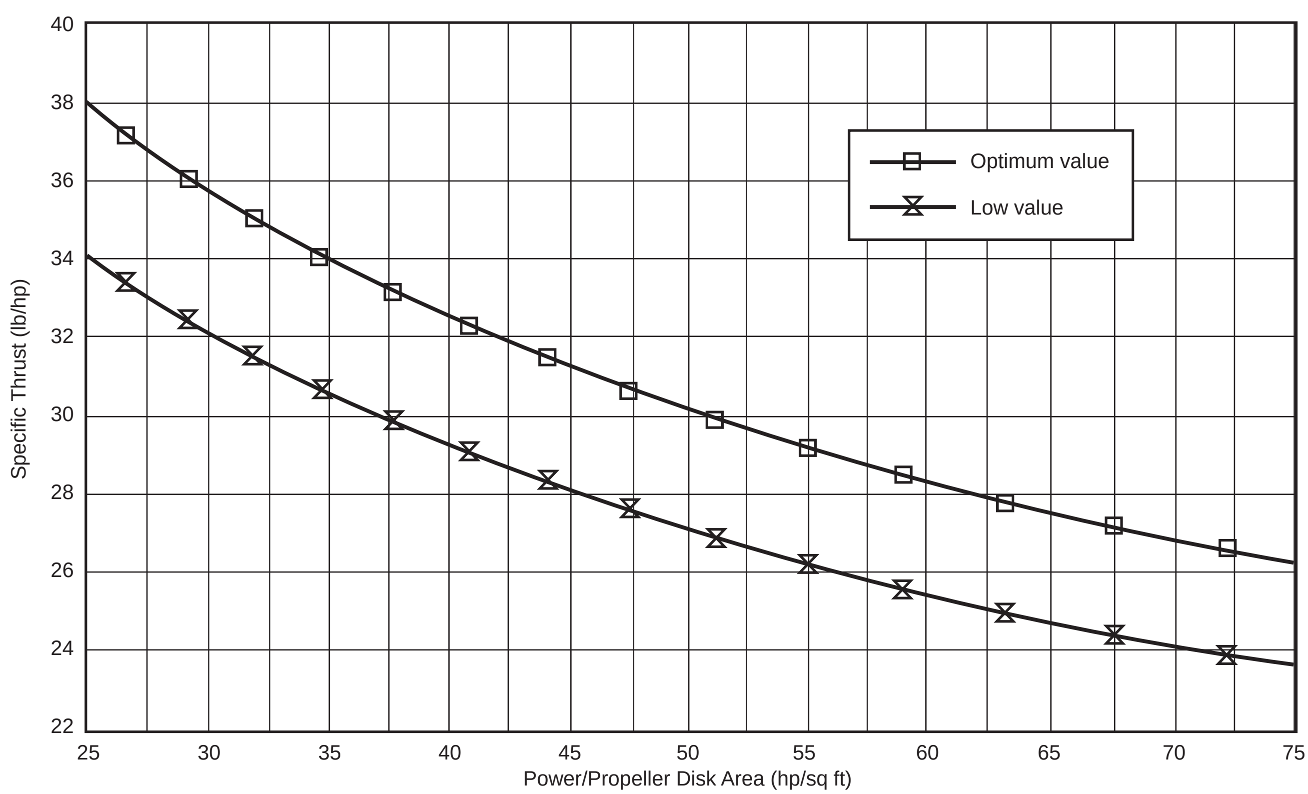
### F.4.1 PROPELLERS INSTALLED AT THE STERN OF A SHIP-SHAPED VESSEL

The suction of the propeller creates a low pressure field at the aft body of the vessel, resulting in a reduction of the available propeller thrust. At zero inflow velocity, and with the propeller turning into ahead direction, this reduction amounts to approximately 5 percent of the propeller thrust. During astern operation, the reduction is about 15 to 20 percent. Detailed data regarding the propeller/hull interaction for conventional vessels are included in F.6, [F.1, F.2, F.4, and F.5].




Note: Power delivered to the propeller.

Figure F.1—Propeller Thrust, Open Propellers




Note: Power delivered to the propeller.

Figure F.2—Propeller Thrust, Propeller with Nozzles

## F.4.2 RIGHT ANGLE GEAR THRUSTER PROPELLERS

The presence of the gear housing and support struts in the flow to the propeller causes a reduction of thrust. For a thruster of average design, this deduction is about 10 percent. In cases where the diameter ratio gear housing to propeller exceeds 0.45, a deduction of 15 percent may be applied.

## F.4.3 THRUST DEDUCTION DUE TO INFLOW VELOCITY

For a propeller applied for stationkeeping, propeller operation in certain inflow velocities is caused by currents as well as by the wake created by thrusters operated in the vicinity. Table F.1 indicates approximate deductions of thrust as a function of the inflow velocity. An accurate prediction for the performance of ducted or open propellers at certain inflow velocities is feasible by a detailed analysis [F.1, F.2, F.3, F.4, F.5, F.7, and F.8]. See F.6 [F.23 and F.24] for references including information regarding the thrust losses caused by the mutual interference of thrusters.

Table F.1— Correction Factor for Inflow Velocity

| | Inflow Velocity in Knots | | | |
|---|---|---|---|---|
| Type Propeller | 1 | 2 | 3 | 4 |
| Open Propeller | .95 | .90 | .85 | .80 |
| Nozzled Propeller | .94 | .88 | .82 | .76 |

## F.4.4 THRUST DEDUCTION DUE TO OBLIQUE INFLOW CROSS-COUPLING EFFECTS

The operation of a propeller in an inflow other than parallel to the propeller axis alters the performance characteristic. Deductions due to inflow velocity may be reduced. However, the creation of cross-coupling forces may cause deduction from the overall balance of forces. The direction of these forces are orthogonal to the propeller axis. These effects are the least researched subjects in propulsion for dynamic positioning. See F.6 [F.17, F.18, F.19, and F.24] for sources of information and qualitative data.

Table F.2—Thrust Losses in Reverse Condition

| Nozzle Type | Loss in Percent |
|---|---|
| Symmetric nozzle | 5–10% |
| Non-symmetric nozzle, elliptic blades | 10–25% |
| Non-symmetric nozzle, cambered blades | 25–50% |

## F.4.5 THRUST IN REVERSE OPERATION

Some of the thrust producing devices applied for dynamic positioning need to reverse the operation of the propeller to produce thrust in reverse direction. Azimuthing thrusters typically produce thrust in one direction only. They control direction of thrust by controlling the azimuth angle.

Some thrusters, such as tunnel thrusters or fixed direction nozzled thrusters are designed as bi-directional devices and are capable of generating approximately equal amounts of thrust in both directions. Propellers optimized for operation in one direction (the majority of marine propellers) are subject to severe deductions while operating in reverse mode. Table F.2 indicates values for thrust losses of nozzled propellers [C24].

## F.4.6 THRUST DEDUCTION DUE TO PROPELLER/ HULL INTERACTION

### F.4.6.1 Coanda Effect

The high-velocity wake from a propulsion device installed under the bottom of a vessel may cause areas of low pressure at the hull that result in considerable deductions from the available thrust forces. The magnitude of these deductions depend on the distance of the propeller from the hull, the location of the propeller relative to the centerline of the hull, from the radius of the bilge, and from the draft of the vessel. A correction factor from 5 percent to 15 percent should be applied to account for this hull interaction. Sources of information and data regarding the thrust losses due to propeller/ hull interference are included [F.17 and F.24].

### F.4.6.2 Twin-Hull Interaction

This effect occurs at twin-hull semi-submersibles equipped with rotatable, under the hull mounted propulsion devices. At certain azimuthing angles, the propeller jet from the thruster is directed towards the neighbor hull, causing a resistance opposite the direction of the thrust. This effect can be amplified by the above mentioned Coanda effect. Little information regarding these effects is available. The magnitude of the thrust losses depends on the thruster installation geometry and the configuration of the semi-submersible hulls. Countermeasures (which apply also to the Coanda effect) include horizontally tilting the propeller axis downwards or fitting guide vanes to the exit of the nozzle. Both methods deflect the propeller jet away from the neighbor hull. An indication has been found of an average thrust loss of 10 to 15 percent due to the above discussed phenomena, with peak losses of over 50 percent at some positions and in particularly unfavorable conditions [F.22]. A discussion of the above interaction effect, including model test results, may be found in *Jet Deflection Vanes for Improved Performance of Rotatable Thrusters* [F.22].

## F.5 Performance of Tunnel Thrusters

Though there are some similarities, tunnel thrusters differ in many ways from the other propulsion devices. They are analytically treated as axial flow pumps. As with marine propellers, the thrust increases with decreased power load. A large propeller diameter yields to a high thrust at a given power. The tunnel thruster is subjected to thrust deductions by factors typical for axial flow pumps, restrictions in the flow to and from the impeller, as well as tunnel entrance and exit losses are the major contributors to the reduction in the net thrust output.

### F.5.1 SIDE FORCE OF TUNNEL THRUSTERS

Figure F.3 can be used for quick determination of the side force of a tunnel thruster. The figure assumes an optimum installation geometry. The tunnel length is about twice the propeller diameter. The hull is perpendicular at the tunnel exits. The exits are conically shaped. No protective bars restrict the tunnel ends. The impeller/hull interaction losses are included.

### F.5.2 THRUST DEDUCTIONS FOR TUNNEL THRUSTERS

In addition to the thrust losses due to the installation geometry typically associated with tunnel thrusters, further thrust losses may occur during certain operational conditions. The performance prediction of a tunnel thruster is based on a nominal design submergence of the tunnel. If this submergence is decreased due to a reduction in the draft, or due to motions of the vessel, the thruster impeller will ventilate (sucking air) and/or cavitate. Both cause a reduction in impeller thrust.

The analytical determination of the losses due to the motions of the vessel is complex. First, a relative motion analysis has to be performed for the environmental conditions in which the vessel is expected to operate. With these data—periodic variations of the submergence at the tunnel location—the thrust losses during the operation of the impeller at reduced submergence can be calculated [F.24 and F.25].

## F.6 References

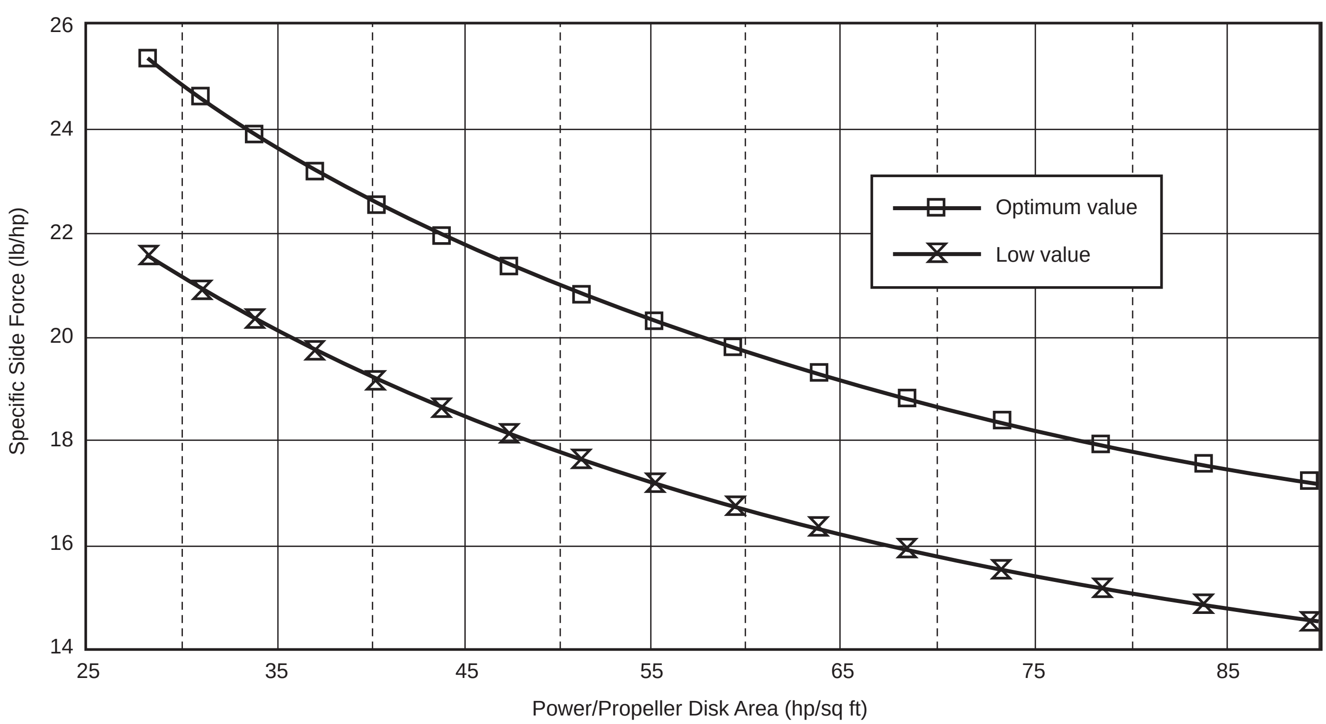
**F.1** Lewis, E. V. (ed), *Principles of Naval Architecture*, Vol. II, SNAME, 1988.

**F.2** an Manen, J. D., *Fundamentals of Ship Resistance and Propulsion, Part B: Propulsion*, NSMB Publication No. 132a.

**F.3** O'Brian, T. P., *The Design of Screw Propellers*, Hutchinson & Co., Ltd., 1962.

**F.4** Saunders, *Hydrodynamics in Ship Design*, SNAME, 1957.

**F.5** Harvald, S. A., *Resistance and Propulsion of Ships*, Wiley, 1983.




Note: Power delivered to the propeller.

Figure F.3—Side Force, Tunnel Thrusters

**F.6** Morgan, M., *Dynamic Positioning of Offshore Vessels*, The Petroleum Publishing Company, 1978.

**F.7** Oosterveld, M. W., *Wake Adapted Ducted Propellers*, NSNB Publication No. 345.

**F.8** Van Manen, J. D. and Oosterveld, M. W., *Analysis of Ducted Propeller Design*, SNAME Transactions, 1966.

**F.9** Oosterveld, M. W. and van Oortmerssen, G., *Thruster Systems for Improving the Maneuverability and Position-Keeping Capabilities of Floating Objects*, OTC No. 1625, 1972.

**F.10** Schneiders, C. C. and Pronk, C., *Performance of Thrusters*, OTC No. 2230, 1975.

**F.11** Brix, J-, *Maneuvering Devices for Offshore Services and their Hydrodynamic Properties*, INTEROCEAN No. 244, 1976.

**F.12** Brix, J-, *Maneuvering Technical Manual*, Schiff & Hafen, June 1988.

**F.13** Sjouke, J. and Lagers, G., *Development of Dynamic Positioning for IHC Drill Ship*, OTC No. 1498, 1971.

**F.14** Wise, D. A. and English, J. W., *Tank and Windtunnel Tests for a Drill Ship with Dynamic Positioning Control*, OTC No. 2345, 1975.

**F.15** English, J. W. and Wise, D. A., *Hydrodynamic Aspects of Dynamic Positioning*, NDL Report Ship 197, 1976.

**F.16** Norrby, R. and Ridley, D. E., *Notes on thrusters for Ship Maneuvering and Dynamic Positioning*, SNAME Transactions, 1980.

**F.17** Deter, D. R., *SEDCO 445: Propulsion System for Dynamic Positioning*, INTEROCEAN No. 243, 1978.

**F.18** Shatto, H. L. and Van Calcar, H., *Improving Dynamic Positioning Performance in the Deepwater, High-Current, Rough Water Environment*, OTC No. 4749, 1984.

**F.19** Symons, P. J. R., et al, *The Design of the SEABED Operations Vessel*, RINA Transactions, 1981.

**F.20** Nienhus, U., *Simulations of Low Frequency Motions of Dynamically Positioned Offshore Structures*, RINA Transactions, 1986.

**F.21** Haslum, K., *Thruster Operation Strategies and Fuel Economy*, Norske Sivllingelorers Forening, 1988.

**F.22** Bjoerheden, O., *Jet Deflection Vanes for Improved Performance of Rotatable Thrusters*, KaMeWa, 1986.

**F.23** Lehn, E., *Thruster Interaction Effects*, NSFI Report R 102.80, 1980.

**F.24** Lehn, E., *Practical Methods for Estimation of Thrust Losses*, Marintek Report No. 513003.00.06, 1992.

**F.25** Olsen, H., *Reduction of Bowthruster Efficiency Due to Ship Motions in Waves*, Det Norske Veritas, 1978.

**F.26** N. A., *Effect of Waves on Thruster Performance*, NSMB Report, 1970.

**F.27** Stuurman, A. M., *Fundamental Aspects of the Effect of Propeller Cavitation on Radiated Noise*, NSMB Symposium, 1975.

**F.28** *Mooring Tankers in Deep Water*, Marin, Report No. 32, 5-1988.

**F.29** Mueller-Graf, B., *Systematische Untersuchungen von Verstellpropellem*, VWS Report No. 568/71.

**F.30** *Guide for the Certification of Thrusters*, American Bureau of Shipping, 1984.

**F.31** *Safety and Reliability of Dynamic Positioning Systems*, Det Norske Veritas, Symposium, 1975.

**F.32** Eck, B., *Technische Stroemungslehre*, Springer, 1978.

# APPENDIX G—MOORING STRENGTH RELIABILITY

## G.1 Basic Considerations

This Appendix discusses mooring system reliability and two approaches for mooring strength design: Deterministic and LRFD (Load and Resistance Factor Design). Mooring system reliability can be affected by many factors such as overloading, component defects/damage, fatigue, corrosion, maintenance, human error, etc. The discussion here addresses the factor of overloading. Therefore the term “reliability” or “probability of failure” are related to failures due to overloading only.

## G.2 Deterministic Approach (Total Safety Factor Format)

The strength design procedure specified in this document is based on a deterministic approach, where the mooring system responses such as line tensions and anchor loads are evaluated for a design environment defined by a return period. The mooring system responses are then checked against the mooring component strength to ensure a factor of safety against mooring breakage. The return periods and factors of safety, which were developed mainly by industry experience and consensus, are generally applicable to all geographical locations, water depths, vessel types (semi-submersible, Ship-shaped, or spar), mooring types (catenary, taut leg, spread, or single point), and mooring components (chain, wire rope, or synthetic fiber rope). The only exception is that lower return periods are used for mobile moorings to reflect less severe consequence of mooring failure for mobile unit operations.

This approach has the advantage of providing a simple design procedure that can yield mooring designs with reasonable effort. It has been used by the offshore industry for more than 30 years, and therefore it has a broad base industry support. However, there are also concerns on the shortcomings of this approach:

1. Industry reliability studies indicate that mooring designs developed by this approach may have significant difference in probability of failure.
2. This approach was calibrated by past experience from shallow water operations with more conventional vessels and mooring systems. There are questions whether this approach will provide sufficient reliability for deepwater operations with new vessel concepts such as spar and new mooring concepts such as taut leg and fiber rope mooring.

## G.3 Load and Resistance Factor Design (Partial Safety Factor Format)

Recently substantial effort has been devoted to the development of a design procedure based on a reliability approach, which is often referred to as LRFD (Load and Resistance Factor Design) or partial safety factor format. Instead of using a total safety factor, this approach employs a number of load factors for the load components and a number of material factors for the line component strength. Since the load and material factors are developed from reliability analysis for a large number of cases representing different geographical locations, water depths, vessel types, mooring types, and mooring components, this approach may yield mooring designs with more consistent reliability.

Up to year 2000, all official mooring design codes use a total safety factor to protect against uncertainties in the design parameters such as environment, mooring line strength, system response, and method of analysis, etc. The International Standards Organization (ISO), however, recommends that the Structural Standards published by the ISO should be level-1 limit state codes with a partial safety factor format. This recommendation is not followed in the draft ISO Stationkeeping standard because of lack of fundamental work in this area. It is expected however, that ISO will eventually adopt a partial safety factor mooring code as the ISO mooring standard.

## G.4 DeepStar CTR 4404 Study “Reliability Based Mooring Design Codes”

Because of the ISO recommendation and the introduction of partial safety factor code in classification rules, the offshore industry is pressed to consider adopting a partial safety factor format for mooring design. In 1997 DeepStar initiated CTR 4404 to investigation the feasibility of developing a reliability based mooring design code. This work is necessary to ensure that the future ISO mooring code will not be biased by a single study.

The scope of this study is summarized as follows:

1. Develop study matrix. There are 25 cases in the study matrix, including the following parameters:
   - Environment: GOM, North Sea, West Africa
   - Water depth: 70m—3,000m
   - Vessel: semi, ship, spar
   - Mooring: catenary, taut leg
   - Mooring line: steel, polyester
2. Develop mooring designs for the study matrix according to API RP 2SK, considering both intact and damaged conditions.
3. Determine random variables and modeling uncertainty.

4. Calculate notional probability of failure. Monte Carlo reliability methods were used for evaluating notional probabilities of failure for intact and damaged conditions.
5. Develop partial factors of safety based on the following principles.
   - Maintain average probability of failure
   - Minimize the spread in probabilities of failure

The conclusions of this study are:

1. The spread in the probabilities of failure for the different mooring systems cannot be reduced by a single set of partial safety factors applied to the mean, low, and wave frequency components of the total line tension. From this point of view, there is no apparent benefit from using a single set of partial safety factors post-applied to mean, low, and wave frequency components of line tensions.
2. Using low safety factors for mean tension appears inappropriate for the following reasons.
   - A low safety factor can be assigned to a load component if the uncertainty for that component is low. The mean line tension consists of 4 components: pretension, mean wind force, mean wave drift force, and current force, among which only the pretension can be considered to have low uncertainty. All the other 3 components have significant uncertainty from the environmental criteria and load calculation.
   - Mean tension often dominates the moorings of many floating systems, especially the deepwater systems and those subjected to high currents. In this case a low safety factor for mean tension is equivalent to a low total safety factor and low reliability for the system.
   - Mean tension has a long duration and therefore can be more harmful than the peak dynamic tension which often lasts only seconds.
3. The most important factor affecting the mooring system reliability is the environment. It has been shown that the different levels of uncertainty inherent in the environmental models for the three geographic areas included in this study lead to different levels of probability of failure for mooring systems designed to a single set of safety factors. However this need not be the case. Theoretically we can design to similar level of probability of failure, by accounting for environmental uncertainty. However, the feasibility of implementing environmental partial safety factors in a stationkeeping code requires further study.
4. Because of differences in the uncertainty associated with extreme environmental conditions, mooring systems in the Gulf of Mexico have a higher probability of failure due to overloading than mooring systems in the North Sea, when the mooring systems are designed to the same single set of safety factors. Similarly, mooring systems in West Africa have a lower probability of failure than mooring systems in the North Sea.
5. The calculation of probability of failure is affected by uncertainty in many parameters, such as environmental model, reliability analysis method, software for mooring and reliability analysis, and mooring design assumptions. Therefore calculated probabilities of failure should be considered notional and should be used comparatively. Setting an absolute target for probability of failure can be a dangerous practice.
6. Taut leg and catenary moorings have similar probabilities of failure. The cases for comparison are limited in the DeepStar study. However, this has been confirmed by other industry studies.
7. Steel and polyester moorings have similar probabilities of failure due to overloading. Again, the cases for comparison are limited in the DeepStar study. However, this has also been confirmed by other industry studies.

## G.5 Conclusions

The deterministic approach recommended in the present revision of API RP2SK has the advantage of providing a simple design procedure that can yield mooring designs with reasonable effort. It has been used by the offshore industry for more than 30 years therefore it has a broad base industry support. However, there are also concerns on the shortcomings with this approach:

1. Industry reliability studies indicate that mooring designs developed by this approach may have significant difference in probability of failure.
2. This approach was calibrated by past experience from shallow water operations with more conventional vessels and mooring systems. There are questions whether this approach will provide sufficient reliability for deepwater operations with new vessel concepts such as spar and new mooring concepts such as taut leg and fiber rope mooring.

DeepStar and other industry reliability studies have addressed the second concern. There is no evidence that floating systems operating in deepwater, using spar, taut leg mooring configuration or fiber rope have lower reliability against overloading than the more conventional systems. These studies, however, confirm the first concern. The DeepStar study further pointed out that the most important factor affecting the mooring system reliability is the environment. The different levels of uncertainty inherent in the environmental models for the three geographic areas included in the DeepStar study lead to different levels of probability of failure for mooring systems designed to a single set of safety factors. Theoretically we can design to similar level of probability of failure by accounting for environmental uncertainty. However, the feasibility of implementing environmental partial safety factors in a stationkeeping code requires further study. Other studies have emphasized uncertainty in the low-frequency response of the platform as a significant factor. Uncertainties related to vortex induced motions of platforms in high currents have not yet been taken into account in reliability-based calibration of mooring design codes.

The DeepStar study indicates that the spread in the probabilities of failure for the different mooring systems cannot be reduced by a single set of partial safety factors applied to the components of the total line tension. For mooring systems dominated by mean tension, using a low safety factor for mean tension may actually result in mooring designs of low reliability. However, other reliability studies indicate that multiple partial safety factors do provide a more flexible design format, with a potential for producing designs with more uniform reliability. This potential can be realized in mooring line design.

The total safety factor approach is retained in the present revision of the recommended practice. However, designers should bear in mind various factors that may affect mooring system reliability. The industry will, no doubt, continue to improve the design procedures for mooring systems. Reliability-based calibration is seen as a rational procedure to incorporate various improvements into revised designed equations.

# APPENDIX H—MOORING DESIGN FOR VORTEX INDUCED MOTIONS (VIM)

## H.1 Basic Considerations

### H.1.1 PURPOSE OF DOCUMENT

Many floaters consisting of cylindrical structures such as spars, TLPs, and semi-submersibles can be susceptible to vortex induced motion (VIM) when exposed to currents. VIM is most prominent for spar platforms where most of industry experience has been acquired. It has been noted recently that under controlled experimental conditions, multi-column floaters such as TLPs and semi-submersibles may experience limited VIM. For these hull types, however, there is no substantiated in-service evidence of VIM. Nevertheless, designers of multi-column floaters should be aware of the potential impact of VIM, particularly if novel concepts are considered.

There are a number of special issues for spar VIM, and guidance is needed by the industry to address these issues. This Appendix presents some high level design guidelines for mooring systems under VIM conditions. More detailed technical information, such as industry experience in model testing and comparison with full scale data, strength and fatigue analysis procedures currently used by the industry, methods to improve mooring design for VIM, and potential future technology development, can be found in the Commentary for Appendix H.

It should be noted that technology dealing with VIM is advancing constantly. Designers are encouraged to pay attention to the latest research in this area and consider the use of more advanced technology when it becomes available.

### H.1.2 VIM FUNDAMENTALS

Cylindrical structures exposed to a current create alternating eddies, or vortices, at a regular period. Figure H.1 shows how these eddies appear in the downstream wake of a cylinder.

The frequency of vortex shedding ($f_s$) is related to the non-dimensional Strouhal number, S:

$$f_s = SV_c/D \qquad \text{(H.1)}$$

where

$V_c$ = Current velocity,

$D$ = Cylinder Diameter.

The eddies create alternating lift and drag forces on the cylinder. When a natural period of a structure falls close to the Strouhal period, oscillations of the structure can occur. This phenomenon is traditionally known as Vortex Induced Vibrations (VIV), and is well known for risers and tendons.

VIV is not restricted to long cylinders. Spar platforms, in particular, experience vortex induced oscillations when their surge/sway or roll/pitch periods are close to the Strouhal period. Figure H.2 shows an example of the motion trajectory of a spar subjected to a current of slightly over 2 kts. The period of the motion in the transverse direction in this case is about 180 seconds, which is close to the natural sway period of the spar. There is also a smaller motion in the in-line direction which is at one-half the transverse period. Because of the

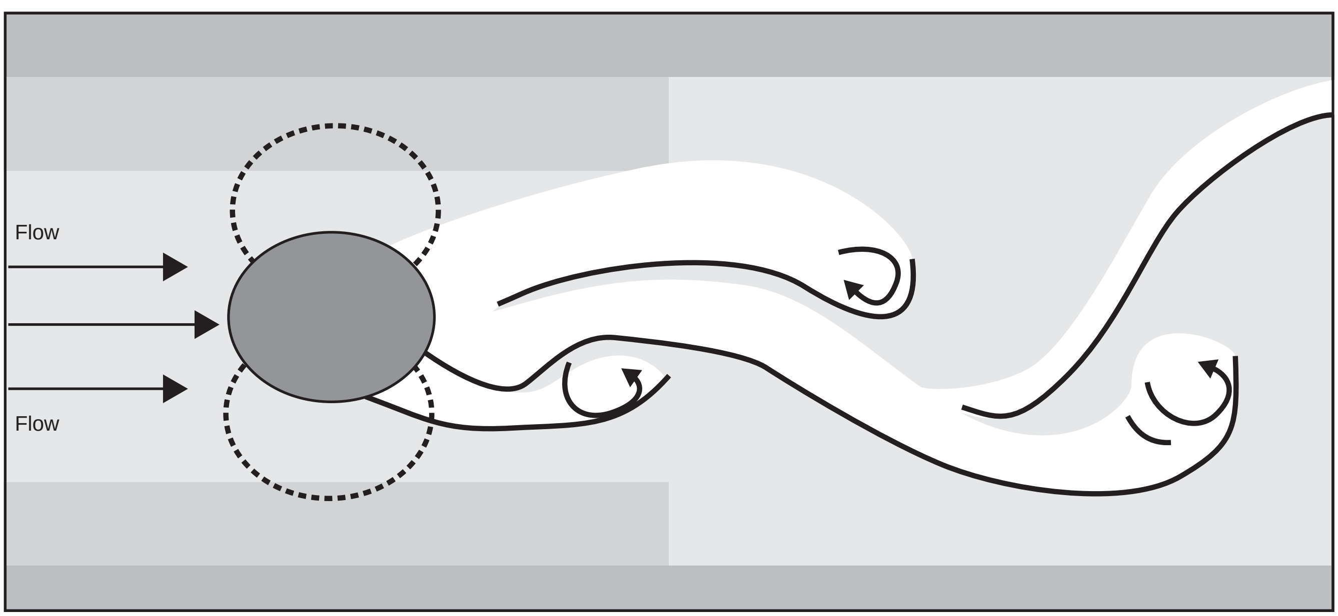



Figure H.1—Eddies in the Downstream Wake of a Cylinder

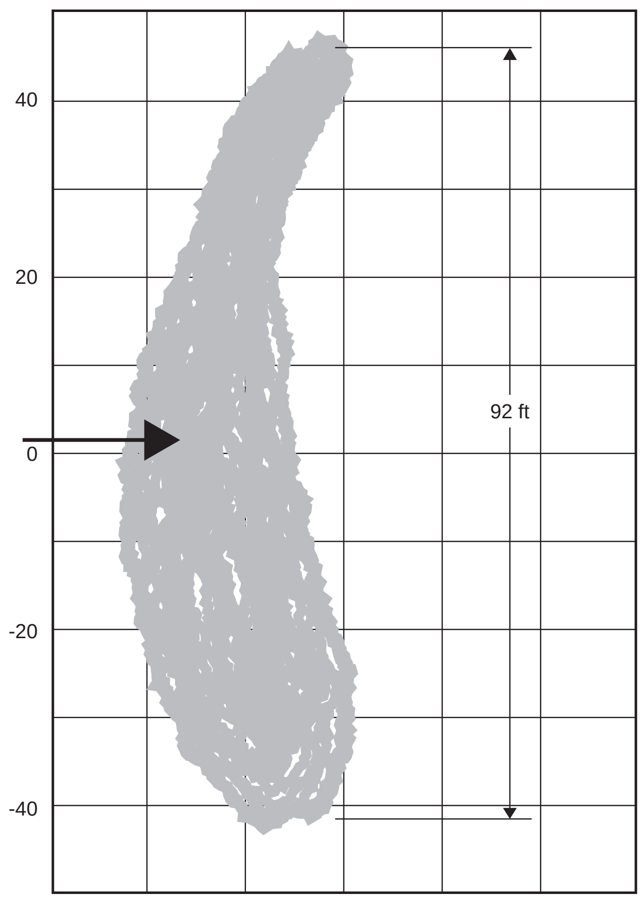



Figure H.2—Motion Trajectory of a Spar Experiencing VIM

long natural periods for floaters, the term "Vortex Induced Motions" (VIM) is typically applied to this phenomenon in lieu of VIV, although the phenomenon is the same.

Semi-submersibles, TLPs and other types of floaters may also be susceptible to VIM, as vortex shedding can occur from the columns or pontoons.

The occurrence of "lock-in" is related to the non-dimensional Reduced Velocity $V_r$:

$$V_r = V_c \, T/D \tag{H.2}$$

where

$T$ = Characteristic period,

$V_c$ = Current velocity.

The definition of can vary (see the Commentary Section CH.2.1). If is the natural period in still water (no current), lock-in can typically occur for values of $4<V_r<10$ for transverse VIM. The precise range of lock-in depends on parameters such as the structural shape, vortex mitigation devices, appurtenances, current profile, mass ratio, and damping.

### H.1.3 EFFECTS OF VIM ON MOORING DESIGN

VIM has two primary effects on the mooring design:

1. The average in-line drag coefficient in lock-in is increased over what it is without VIM, and
2. The motions impose displacements on the mooring line fairleads which cause additional oscillating mooring line tensions.

These effects have to be taken into account for strength and fatigue design of the mooring system. The occurrence of the Loop Current and associated eddies in the Gulf of Mexico make consideration of VIM particularly important. For example, unlike other extreme events, e.g., hurricanes, the Loop Current and associated eddies may affect a particular site for an extended period of time and can contribute to low cycle fatigue of mooring components.

### H.1.4 CURRENT INDUSTRY PRACTICE

Unlike other resonant responses, the amplitude of VIM is bounded. Transverse motion amplitude (*A*) is usually given as a non-dimensional ratio of amplitude to diameter, *A/D*. The largest single amplitude transverse motion observed on bluff bodies is on the order of *A/D* = 1. Helical strakes have commonly been used on spars (and risers) to reduce this amplitude. Strakes can ideally be 95% effective in eliminating VIM, however their effectiveness on spars depends on various factors, e.g., the exact layout and size of the strakes, appurtenances, and current profiles. There is currently no analytical method for determining the motion amplitude, although research is ongoing. The industry currently relies on model testing for this purpose. There is still a lack of sufficient full scale data from which to generalize on the scalability of model testing, particularly for the newer generation of spars: the truss spar and the cell spar. It is usual practice, if current loadings and VIM are determined to be a design driver, to perform well planned model testing to determine motion amplitudes and drag coefficients for purposes of mooring design.

## H.2 Environmental Considerations

### H.2.1 CURRENT CONDITIONS

Current velocity, profile, direction and duration affect vessel VIM, consequently these should be specified in the design criteria for mooring system strength and fatigue. As discussed in Section 3.5, the most common categories of currents are: (1) tidal currents (associated with astronomical tides), (2) circulation currents (e.g., the Gulf Stream, the Gulf of Mexico Loop Current and associated eddies, Brazil current), (3) storm-generated currents, and (4) internal wave generated soliton currents. Although spar VIM was detected in the GOM under eddy currents and hurricane-generated inertial current, other types of current may also induce VIM.

### H.2.2 ENVIRONMENT FOR STRENGTH ANALYSIS

Mooring analysis for strength under the VIM condition is normally conducted for an extreme current with associated wind and waves. The metocean criteria should specify current velocity, profile, and direction as well as the intensity and direction (colinear or non-colinear) of wind and wave conditions associated with extreme currents. The guidelines in this document address mainly mooring design and analysis for this condition. However, recent experience suggests that consideration should also be given to extreme wind and waves with associated current. When the Neptune spar experienced VIM under hurricane generated inertial current in the Gulf of Mexico, VIM was recorded only after the inertial current had deepened and the wind and waves had decreased. For this condition, the application of some of this guideline may need careful evaluation. Further investigation and experience is needed to arrive at a conclusion on this issue.

### H.2.3 ENVIRONMENT FOR FATIGUE ANALYSIS

For long-term fatigue analysis, current events can be represented by a number of discrete current bins, with each current bin consisting of a reference direction, a reference current velocity and profile, associated wave and wind conditions, and probability of occurrence. Industry studies indicate that for some mooring systems, considerable fatigue damage may be caused by a single extreme VIM fatigue event, which should also be addressed. Note that the current for the worst-case VIM fatigue event may not coincide with the 100-year return period current event, but could occur under lower return period current events. For fatigue analysis of single VIM events, the current criteria should specify the current velocity, profile, direction, and duration (build-up and decay) for current events spanning a range of return periods.

## H.3 Vessel Response

### H.3.1 VIM RESPONSE MODES

The exciting force induced by vortex shedding on the hull of bluff body floaters, such as spars, may cause response in any of the 6 rigid body modes of response. The response of primary concern for most floaters is the transverse (sway) response and in-line (surge) response, which are typically included in a mooring analysis. However, other response modes should be checked to ensure that forces due to vortex shedding do not excite them or do not significantly affect the mooring system response. For example, for some floaters large pitch, roll, or yaw responses or large mean transverse displacements could affect the mooring system. Special hydrodynamic effects such as "galloping", which is due to large variations in lift versus vessel heading, are beyond the scope of this document.

### H.3.2 VIM RESPONSE PREDICTION

Model testing has been the primary tool for VIM prediction because of difficulties in obtaining full-scale response data in a timely fashion to support projects, and the lack of a validated numerical or analytical approach. Industry studies indicate, however, that model tests are only able to accurately model certain effects while compromising others. From this point of view, confidence in model test results and VIM design criteria should be established through comparison with field measurement data. The reliance on model testing, the limitations of model testing, and limited validation with full-scale data should be recognized as a potential sources of uncertainty in the design process. A more detailed discussion on model testing can be found in the Commentary Section CH.4.

## H.4 Strength Design

### H.4.1 VIM DESIGN CRITERIA

The first step in strength design is to establish suitable VIM design criteria. VIM-related design parameters for mooring strength design include:

- In-line and transverse VIM response amplitude (*A/D*) as a function of reduced velocity ($V_r$).
- Drag coefficient as a function of VIM response amplitude.
- Definition of ranges for lock-in and lock-out.
- VIM response trajectory or envelope.

These criteria may be based on a combination of project specific model test data and previous VIM design experience, among other aspects. Depending on the approach taken, there will be varying levels of uncertainty in the VIM criteria specified for a particular project. The recommended practice is to develop criteria for a base case (the best estimate) and some sensitivity cases. Tension safety factors for intact and damaged conditions should be met for the base case. Sensitivity cases can be used to check the robustness of the mooring system, i.e. the risk of mooring failure in the event estimates of certain influential parameters such as mooring stiffness, current velocity, drag coefficient, lock-in definition, or VIM amplitude are inaccurate. One of the important roles of the sensitivity check is to determine if, with some changes in system parameters, the system would enter a VIM lock-in regime that would not be apparent for the base case design criteria alone.

An example of $V_r$ versus A/D design criteria is provided in Figure H.3 showing the lock-in transition, locked-in region, and the locked-out region. This type of curve is typically used to define the VIM response amplitude. It is important to note that, for most spars and other moored offshore vessels experiencing VIM, the shape of this curve will likely vary with current direction, i.e., the amplitude of response varies with current direction for the same reduced velocity. Particular attention should be applied to defining the VIM response versus current heading when setting the design criteria.

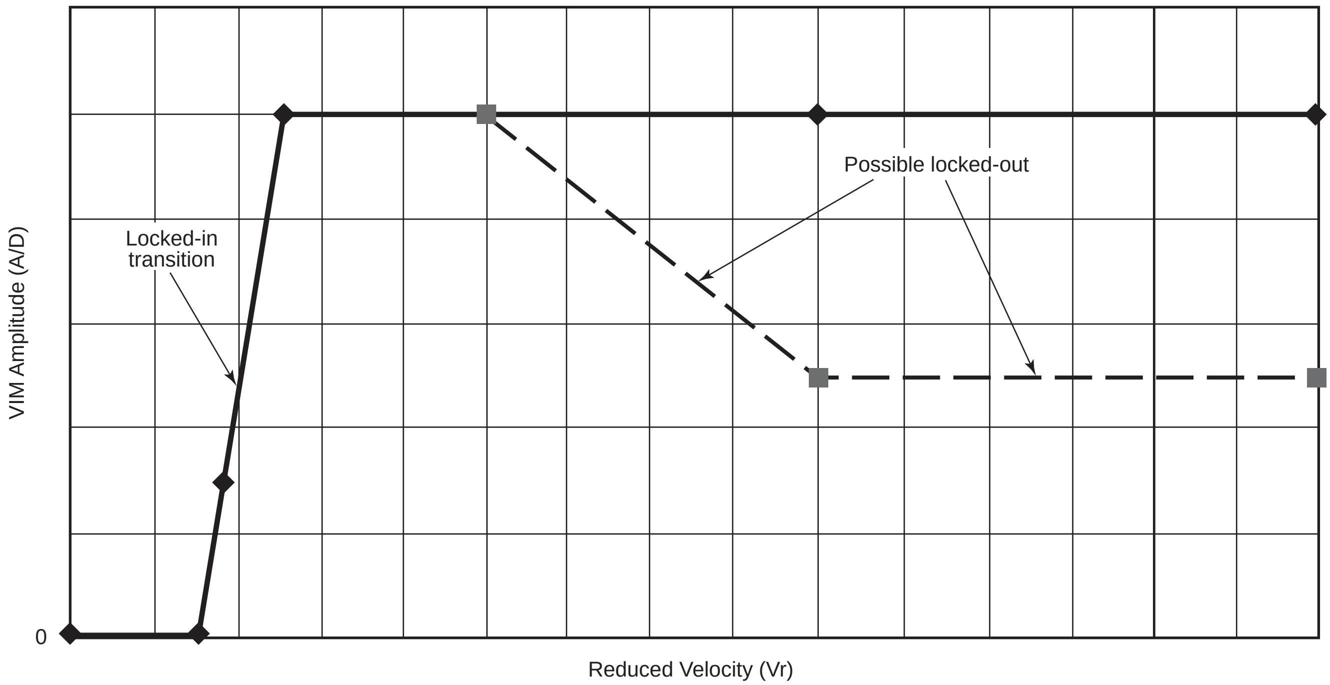



Figure H.3—Example VIM Amplitude versus Reduced Velocity

### H.4.2 STRENGTH ANALYSIS METHOD

Most mooring analysis software was not designed to handle spar VIM analysis, therefore a simplified analysis procedure (Commentary Section CH.5) is typically used. More accurate approaches can be considered when advanced mooring analysis software that can model VIM responses in addition to other typical loads and motions becomes available.

## H.5 Fatigue Design

### H.5.1 BASIC CONSIDERATIONS

Mooring tensions due to VIM are cyclic by nature and contribute to the fatigue damage of the mooring system. The following factors should be considered when assessing fatigue due to VIM:

- The VIM period in the offset position, corresponding to the specific current bin under consideration, should be used when calculating the number of tension cycles. This period can vary with current direction and magnitude and can be different than the still water natural period.
- In addition to a long-term fatigue damage evaluation, a fatigue analysis for the 100-year VIM fatigue event (or other single worst-case event as noted in Section H.5.3) is also recommended.
- Mooring systems experiencing a high mean load and large tension variation may stress the chain to beyond the elastic region, where fatigue test data are not available. To ensure sufficient fatigue life, the mooring system should be designed to avoid this situation.
- Fatigue damage of chain at the fairlead requires special attention since additional bending stress is imposed on the chain in this region, and chain typically has the lowest fatigue life of all the components in the mooring system.
- Sensitivity cases, similar to those used in the strength analysis, may be considered to account for uncertainty in the VIM prediction.

### H.5.2 LONG TERM FATIGUE DAMAGE

For long-term fatigue analysis under the VIM condition, current events can be represented by a number of discrete current bins, with each current bin consisting of a reference direction, a reference current velocity and profile, associated wave and wind conditions, and probability of occurrence. Fatigue damage for each current bin is evaluated, and the sum of fatigue damage due to VIM from all current bins is added to the fatigue damage due to wind and waves to yield total fatigue damage (see Commentary Section CH.6 for more detailed procedure).

### H.5.3 FATIGUE ANALYSIS FOR SINGLE EXTREME VIM EVENT

Industry studies indicate that considerable fatigue damage can be caused by a single extreme VIM event. Therefore in addition to the long-term fatigue damage evaluation, a fatigue analysis for the 100-year VIM fatigue event is also recommended. Since VIM response is largely dependent on reduced velocity, the current for worst-case VIM may not coincide with the 100-year return period loop or hurricane current. The highest VIM amplitudes for fatigue consideration could occur under a lower return period current. The current direction should be the worst direction identified in the strength analysis. Instead of using a constant extreme design current for the whole event, current variation based on field measurements for strong loop currents can be considered. Note the duration of this event may be different from that obtained from the long term current distribution.

Fatigue analysis is typically performed for the intact condition only. However, a fatigue analysis for the damaged condition should be considered for the single extreme VIM event when progressive line failure due to fatigue is a concern.

### H.5.4 FACTOR OF SAFETY

As stated in Section 7.5, the factor of safety for fatigue design is 3.0, which should be applied to long term (due to both wind and waves and VIM) and single event fatigue under the intact condition.

### H.5.5 CHAIN FATIGUE AND WEAR

Fatigue damage of chain at the fairlead is typically higher than that away from the fairlead. For mooring systems where chain fatigue is critical, it is important to shift periodically the links at the fairlead so additional fatigue damage due to bending can be more evenly distributed. If this operational procedure will be carried out in the field, fatigue damage for the links around the fairlead can be evaluated based on percent of time when the links are located at the fairlead. However, the links at fairlead should have sufficient fatigue strength to survive at least a single extreme VIM event (for example the 100-year VIM event).

Mooring systems subjected to VIM may also experience increased wear in the links at the fairlead, which is caused by high contact pressure and large movement between links. This issue should be carefully evaluated, and the measure of periodically shifting the links at the fairlead should be considered to alleviate the wear problem. Wear measurement using go-no-go gage as outlined in API RP 2I can also be considered for detecting excessive chain link wear.

# APPENDIX CH—COMMENTARY ON APPENDIX H—MOORING DESIGN FOR VORTEX INDUCED MOTION (VIM)

## CH.1 Basic Considerations

### CH.1.1 PURPOSE OF COMMENTARY

Many floaters consisting of cylindrical structures such as spars, TLPs, and semi-submersibles can be susceptible to vortex induced motion (VIM) when exposed to currents. VIM is most prominent for spar platforms where most of industry experience has been acquired. Appendix H provides some high level design guidelines for spar mooring systems under VIM conditions. This Commentary, which is not considered to be part of the recommended practice, provides additional background information describing the developing state of the art to supplement Appendix H.

There are a number of special issues for spar VIM, as discussed in the section below. The industry has devoted extensive effort to address these issues, leading to significant improvements in the design procedure. Although improved design procedures have been used in many spar design projects, there are areas where industry consensus has not been completely reached. There are also areas that require further improvement in the future. Designers are encouraged to exercise judgment and caution in applying the information in this Commentary to their designs and to study the references listed in Section CH.9. Results of new studies should be considered when they become available.

### CH.1.2 SPECIAL ISSUES FOR SPAR VIM

1. There is no mature analytical tool for the prediction of spar VIM. Currently VIM design criteria are typically obtained from model testing. Model testing practices need to be validated with field measurement data, which are quite limited.
2. Spar VIM is affected by current velocity, direction, profile, hull geometry and appurtenances.
3. The duration for peak current and resulting VIM can be much longer than peak storm duration.
4. Model tests can only model certain parameters while approximating others. Hence care needs to be exercised in the interpretation and use of model test data.
5. Where VIM results in large tension cycles at high mean load, fatigue life can be short for mooring components with low fatigue resistance such as chain.
6. The calibration of the factors of safety for mooring design does not include the spar VIM condition and the uncertainties associated with spar VIM. Consequently sensitivity checks may be warranted.

Because of the above issues, it is important to address VIM conservatively in the spar mooring design stage. This can be achieved through the following measures:

1. Establish design criteria that recognize the uncertainties in VIM behavior, for example checking sensitivity cases in addition to the base case and checking field measurement data as well as model test data.
2. Conduct fatigue analysis for the 100-year VIM response condition in addition to long term fatigue analysis.
3. Select mooring hardware and system design characteristics (strake configuration, mooring stiffness, etc.) that can better tolerate or mitigate VIM.

## CH.2 VIM Design Parameters

### CH.2.1 REDUCED VELOCITY $V_R$

For the 6-degree of freedom motions of multi-member bodies the definition of reduced velocity is complicated. In general, the definition of $V_r$ involves the eigen-periods of the system under mean load (which depend on the nonlinear mooring system and hydrostatic stiffness, and the full 6-by-6 mass and added mass matrices), the characteristic dimension of the body (which may vary with the eigen-mode under consideration), and the characteristic velocity of the flow incident on the member.

The following discussion applies to translational VIM of a classic spar transverse to the current direction. In this case, transverse VIM occurs when the vortex shedding period (Strouhal period) is close to the natural period of the moored system transverse to the current direction. For a classic spar the relation between VIM response and the natural or observed period of the transverse motion is often given in terms of the reduced velocities $V_{rn}$ or $V_{robs}$,

$$V_{rn} = V_c T_n / D \text{ and } V_{robs} = V_c T_{obs} / D \qquad \text{(CH.1)}$$

where

$V_c$ = the characteristic current velocity, typically the highest velocity in the current profile,

$T_n$ = the still water natural period of the moored vessel transverse to the current direction under mean load,

$T_{obs}$ = the observed period of VIM,

$D$ = spar diameter.

Note that $V_{robs}$ is only defined over the range of current velocities that induce VIM, whereas $V_{rn}$ is defined for all current velocities. Model test data indicate that VIM is a function of $V_{rn}$ or $V_{robs}$ and VIM is negligible when $V_{rn}$ is below a threshold value. For transverse VIM of classic spars a threshold value of approximately 4 has been observed in the model tests. Field data for classic spars support the finding from model tests, but no field data are available to date to validate this for truss spars. The range of $V_{rn}$ or $V_{robs}$ where significant VIM can be induced is often referred to as the "lock-in" range.

$V_{rn}$ is a function of $T_n$, which is a function of mooring stiffness and vessel mass. Mooring stiffness at various offsets can be significantly different, especially for grouped mooring patterns. Vessel mass includes added mass, which is typically determined by analytical tools or model testing. If available, field measurement data should be used to calibrate the added mass values. The transverse stiffness used for calculating $V_{rn}$ is typically evaluated at the mean offset under current and associated wind and waves. Since the mean offset is dependent on the drag force, which is dependent on the VIM amplitude, the process of selecting the appropriate offset for VIM calculation is iterative. It should be noted that the observed period from model tests or field measurements $T_{obs}$ can be different from the calculated still water natural period, $T_n$, which is used in most analyses as it is readily available. Calibration of calculated values with available model test or field measurement data may be desirable when such data are available.

Since $V_{rn}$ is a function of current velocity and natural period of the moored vessel, VIM can be generated under relatively mild currents (for example 1 to 2 knots) if the natural period of the vessel is long. This may be the case with deepwater floating systems that have low mooring stiffness.

### CH.2.2 TRANSVERSE (CROSS FLOW) VIM

Transverse VIM occurs when the vortex shedding period is close to the transverse natural period of the moored vessel, and the spar typically oscillates in the direction perpendicular to the current in a periodic pattern. Transverse motion is normally expressed as the ratio of single amplitude transverse VIM to spar diameter (*A/D*). However, transverse VIM sometimes has an asymmetric pattern. In this case transverse *A/D* should be specified for two opposite directions. Transverse VIM is a function of a large number of parameters such as reduced velocity, spar type (classic or truss), strake configuration (shape, height and coverage), current characteristics (profile, speed, and direction), and hull appurtenances (anode, chain, fairlead, and pipe), etc.

### CH.2.3 INLINE VIM

Inline VIM is typically in the direction of the current, and it may affect the transverse VIM. Inline *A/D* is also a function of the parameters as discussed above for transverse VIM. The magnitude of inline *A/D* is typically much less than the transverse *A/D*. Field measurement data for a classic spar with an equally spaced spread mooring system indicate inline *A/D* of 10% to 15% of the transverse *A/D* [CH.20]. However, the magnitude can be higher if the natural period for the inline motion is about half of the natural period for the transverse motion (resonance condition). Also, unsymmetrical mooring system stiffness could result in a VIM trajectory for which the major axis of the VIM is not transverse to the current direction.

### CH.2.4 DRAG COEFFICIENT

Model tests are typically used to determine drag coefficient $C_d$ to be used in design. A "base drag" $C_{d0}$ is assumed for the case where $A/D = 0.0$ (no VIM) and amplification factors are applied to account for VIM effects. The drag augmentation is a function of *A/D* and $V_r$ and can be expressed as [CH.3, CH.24, CH.29]:

$$C_d = C_{d0}[1+k(A/D, V_r)] \tag{CH.2}$$

The mean drag force on the cylinder is given by

$$F_d = C_d \frac{1}{2} \rho V_c^2 A_p \tag{CH.3}$$

where

$C_d$ = mean drag coefficient (absolute current velocity) in the presence of VIM,

$\rho$ = density of the fluid,

$V_c$ = free stream current velocity,

$A_p$ = projected area.

In the lock-in range the drag coefficient increases almost linearly with *A/D*. For a classic spar where the spar diameter is well defined, the drag coefficient under lock-in condition can be expressed by the following equation:

$$C_d = C_{d0} + f\,(A/D) \tag{CH.4}$$

Where $C_d$ is the spar drag coefficient with VIM, and is spar drag coefficient without VIM. The coefficient $f$ is hull specific, and is normally determined by model testing. It also depends on the definition of *A/D* and $C_d$ (extreme or mean *A/D*, and absolute or relative velocity $C_d$). Some of the reference publications demonstrate the variability to drag observed in model tests. Such variability may warrant sensitivity checks on drag predictions as part of the mooring design.

For truss spars which consist of a large number of components of various diameters, *D* is defined as the diameter of the hard tank. Attention should be given to the possibility of increased drag on the small truss members due to hull VIM.

## CH.3 Effects of Water Depth and Current Turbulence

Generally VIM magnitude is not a function of water depth, but VIM and mooring line tension can be affected by change of stiffness in different water depths. Mooring stiffness typically increases with decreasing water depth. The higher mooring stiffness in shallower water may reduce or even suppress VIM under certain conditions, due to a resulting value of $V_r$ less than the lock-in threshold. However, if higher stiffness fails to reduce or suppress VIM, the mooring line can experience a significant increase in line tension. Industry experience indicates that VIM can cause significant line tension increase for typical steel taut leg moorings in water depths of 2,000 to 3,000 ft, where VIM amplitudes may be a significant fraction of the total offsets. The VIM influence on line tension is much smaller in deeper water, say 5,000 ft water depth, because mooring stiffness generally decreases, while VIM amplitude remains similar in magnitude regardless of water depth. Although VIM of the same magnitude is less damaging to deepwater moorings, the effects of VIM on these moorings still need to be considered. Large seafloor slope may result in significantly different anchor depths for different mooring lines, causing directional change of mooring stiffness. This in turn may induce directional VIM response.

While the correlation between the limited available field measurements of spar VIM and model test results does not indicate that turbulence in ocean currents influences spar VIM response, there is evidence from model testing that high levels of turbulence in the model basin can affect VIM response. The structure and intensity of turbulence in ocean currents and the potential impact of current turbulence on VIM remains an uncertainty for further observation and investigation.

## CH.4 Model Testing

### CH.4.1 BASIC CONSIDERATIONS

Model tests are routinely conducted to investigate spar VIM and VIM mitigation methods. Model testing has been the primary tool for spar VIM prediction because of the difficulties in obtaining full-scale measurement data and the lack of a sufficiently mature numerical approach. A sound VIM model testing practice should pay attention to the following issues:

- Geometric scaling
- Dynamic scaling
- Hydrodynamic scaling
- Modeling of appurtenances
- Mooring stiffness characteristics
- Degrees of freedom
- Current direction and profile
- Directional resolution
- Test rig damping
- Blockage (wall) effect
- Length of response record

Industry has devoted significant effort to improve model testing methodology to obtain better predictions for spar VIM responses. Improved model testing conducted recently yielded VIM predictions that compare reasonably well with field measurements [CH.20]. However, all the model tests conducted to date can only accurately model certain parameters while approximating others. Different model testing methodologies and practices can result in different test results. From this point of view, confidence in model test results and VIM design criteria should be established through adherence to sound engineering principles and comparison with field measurements where available. The reliance on model testing, the limitations of model testing and limited validation with full-scale data should be recognized as potential sources of uncertainty in the design process.

### CH.4.2 MODEL TEST PARAMETERS

#### CH.4.2.1 Flow Similitude

The basis for hydrodynamic model testing is that geometric and dynamic similitude between prototype and model fluid flow is preserved. Reynolds number scaling and Froude number scaling are the two relevant scaling parameters for hydrodynamic model testing of offshore structures [CH.22, Chapter 9].

The Reynolds number is defined as

$$R_e = \frac{V_c D}{V} \qquad \text{(CH.5)}$$

where

$R_e$ = Reynolds number,

$V_c$ = characteristic velocity (e.g., flow velocity),

$D$ = characteristic length (e.g., hull diameter),

$\nu$ = kinematic viscosity of the fluid, $\nu = 1.21 \times 10^{-5}$ $ft^2/sec$ for seawater.

The Froude Number is defined as

$$F_n = \frac{V_c}{\sqrt{gD}} \qquad \text{(CH.6)}$$

where

$F_n$ = Froude number,

$g$ = gravity acceleration, 32.2 $ft/sec^2$.

Satisfying the Reynolds and Froude scaling simultaneously for the model and prototype flows, however, is practically impossible. For a model dimension $D$ that is substantially smaller than prototype, either the gravity $g$ needs to be significantly increased, or viscosity $\nu$ of the testing fluid needs to be significantly decreased. Neither of these changes is practical in a model basin.

For separated flow dynamics that cause VIM, Reynolds number scaling is the governing scaling law. Reynolds scaling is particularly difficult to achieve for an offshore floating structure. For spar hull diameters of 70–140 ft and current velocities of 2–5 knots, the Reynolds number for the full scale structures (prototype) are in a range of 20,000,000 to 100,000,000. To match such Reynolds numbers in the model basin requires that the model experience the same hydrodynamic force as that of the prototype, which is obviously impractical. There are currently two basic testing approaches, supercritical and subcritical Reynolds number, used in the industry:

1. Supercritical Reynolds number model testing. It is important to test at supercritical Reynolds numbers in order to attain a flow regime which is similar to the flow experienced in full scale [CH.20, CH.21, CH.33]. Supercritical model tests conducted at Reynolds numbers of between 600,000 and 2,000,000 for Genesis and Hoover classic spars have shown good agreement with full scale (15,000,000 < $R_e$ < 40,000,000) responses measured in the field. However, supercritical Reynolds number model testing places significant demand on the capacity of the model basin and to date supercritical model tests have only modeled 1-DOF and with a uniform current profile.
2. Subcritical Reynolds number model testing. For a helically straked cylinder, in the near field flow separation is controlled by the sharp edges of the strakes and not by boundary layer effects [CH.4]. In addition, it is possible to include the effects of 6-DOF spar motions and current profile in the model test. Subcritical model tests conducted at Reynolds numbers of between 50,000 and 400,000 for the Genesis spar were conservative when compared to full scale (30,000,000 < $R_e$ < 40,000,000) measurements [CH.17, CH.20].

As stated above, all model tests conducted to date can only accurately model certain parameters while approximating others. Both approaches have advantages and disadvantages, and both show acceptable agreement with full scale data for classic spars [CH.17, CH.20, CH.21, CH.33].

#### CH.4.2.2 Dynamic Similitude

In addition to hydrodynamic scaling, dynamic similitude requires that the rigid body dynamics for the full scale and model scale systems are similar. Dynamic scaling is associated with the vessel's rigid body modes, mass ratio, and reduced velocity.

Modeling all of the rigid-body modes (e.g. surge, sway, heave, roll, pitch, and yaw) may not be important as long as those that are candidates for lock-in are included. For example, a spar might lock-in to sway at lower velocities and roll at higher velocities [CH.11]. The two degrees of freedom might actually couple (lock-in simultaneously). In this case it is important that the sway and roll modes and periods be properly scaled. On the other hand, if the transverse sway is the dominant VIM response, then tests with a single-degree of freedom rigid body mode have shown reasonable agreement between model test and full scale data [CH.21].

The concept of reduced velocity $V_r$ has been introduced in Section H.1. It is an important dimensionless parameter for VIM:

$$V_r = \frac{V_c T}{D} \tag{CH.7}$$

The definition of characteristic period $T$ can vary (see Section CH.2). If $T$ is defined as the still water system natural period, VIM 'lock-in' for a classic spar typically occurs for values of $4 \le V_r \le 10$. The reduced velocity for model flow must correspond to the reduced velocity for the prototype flow in order to achieve proper fluid-structure VIM similarity. That is, in addition to selections of proper scaling for $V_c$ and $D$, scaling for period $T$ should also be appropriate.

Mass ratio has a large effect on the range of lock-in, and possibly the amplitude [CH.10, CH.23, CH.30]. Mass ratio for a free floating body is by definition equal to 1.0 (displacement = weight). This mass ratio should be maintained for model tests as well.

#### CH.4.2.3 Geometric Similitude

Geometric similitude is achieved when the geometry of the model and prototype bodies is similar. The geometry of the hull and strakes (if appropriate) should be accurately scaled. This includes any construction openings in the strakes, brackets (which might affect the flow along the strakes), chains, anodes, external pipes and other appurtenances that may affect the flows around the body. Some members, e.g., the truss members of a truss spar, may result in viscous damping effects that are Reynolds number dependent. Care should be exercised to size these members to result in a representative amount of damping in model tests.

##### CH.4.2.3.1 Model Scale

The model has to be geometrically similar to the prototype, meaning that the shape of the model is the same as that of the prototype while the characteristic length of the model is smaller. Due to hydrodynamic force consideration, the industry typically uses smaller ($^1/_{100}$ ratio) model scale for high, supercritical, Reynolds number model testing and relatively larger ($^1/_{50}$ ratio) model scale for low, subcritical, Reynolds number model testing.

### CH.4.2.3.2 Appurtenances

It is important to model all details of the spar hull accurately. This includes all appurtenances such as fairlead, pipes, chains, anodes, risers and flowlines. Details of strakes including cutouts or holes in strakes should also be modeled correctly. Accurate modeling of appurtenances is particularly important in developing VIM directional sensitivity and testing effectiveness of VIM suppression devices such as strakes.

### CH.4.2.3.3 Model Degrees of Freedom

Models of single degree and multiple degrees of freedom have been used. For the single degree of freedom model, which is mainly used in high (supercritical) Reynolds number testing, only transverse VIM is allowed. For the multiple degrees of freedom model, which is mainly used in low (subcritical) Reynolds number tests, the vessel is free to respond in all six degrees of freedom. The relative importance of multiple degrees of freedom model is determined by the level of coupling between motions of different degrees of freedom.

### CH.4.2.3.4 Mooring Stiffness Characteristics

There are two approaches to modeling the stiffness distribution of the prototype mooring system. One approach is to use the reduced velocity ($V_r$) as a design parameter. In this case, the VIM response in the model is related to the design *A/D* via the reduced velocity. In the model tests, the spar response is measured at different reduced velocities. In the design phase, the transverse period of the spar (hence the $V_r$) is calculated at different offsets. At each offset, the *A/D* is based on the $V_r$ at that location. In this approach, a linear symmetric mooring system can be used for the model test set-up.

Alternately, the actual spread mooring of the spar is modeled. In this case the current speed is the design parameter rather than the reduced velocity. A spar has typically three or four groups of mooring lines. Each mooring line or group of prototype mooring lines is modeled by an equivalent model mooring line. The horizontal force-displacement characteristic of each mooring line or group is modeled by a bi- or tri-linear spring system so as to mimic the nonlinear force-displacement characteristic of each mooring line or group. This allows for modeling of the complete nonlinearity and asymmetry of the stiffness. For some mooring systems such as grouped mooring system, the asymmetry may contribute to highly directional VIM response.

### CH.4.2.3.5 Current Direction and Profile

VIM response for spar hull can be sensitive to small changes in current directions. Fine heading resolutions (e.g., at 10 to 15-degree increment) may be required to capture the maximum VIM response.

Tow tests simulate a slab current uniform with depth. In reality design currents have a profile and current speeds generally decrease with depth. Efforts have been made to simulate shear current profiles in tow, flume and basin tests [CH.27]. Any attempt to generate shear current profiles in model scale generates excessive turbulence. Careful consideration needs to be given while interpreting VIM responses in the presence of turbulent flow. Turbulence in laboratory generated shear flow can be mitigated by using varying density/viscosity stratified liquid layers in the model tests.

### CH.4.2.3.6 Free Surface Effect

The free surface acts as a barrier through which flow cannot pass, and as a means to dissipate energy through wave radiation. Free surface effects can be important when the Froude number is greater than 0.15. For surface-piercing towing test of a spar hull model, the towing speed is limited by wave resistance (Froude number). High speed towing might result in Froude number that far exceeds the field Froude number and exaggerates the free surface effects. One way to avoid the excessive wave resistance for high Reynolds number model testing is to tow a completely submerged, horizontally mounted mirror image of double body with a divider plate in the center. The divider plate is used to prevent flow communication across the divider plate.

### CH.4.2.3.7 Damping

Damping can affect VIM response, therefore the damping (hydrodynamic and mechanical) generated in the model basin should be consistent with the damping expected in the field. Since mechanical damping may be generated by the testing equipment and is absent in the field, care needs to be taken to understand the effect of damping on the VIM response and to mitigate such effects [CH.33]. Hydrodynamic damping, due to mooring lines and wave effects, in the model test should be given careful consideration when estimating the amplitude of full scale VIM.

### CH.4.2.4 Length of Response Record

Sufficiently long response time histories are required to provide meaningful statistics such as standard deviation, significant, and maximum values. The length of time history required depends on the periodicity of the VIM response [CH.21]. Where the VIM motion is well developed and sustained (e.g., fully locked-in), relatively few cycles are required to establish the maximum VIM amplitude. If the VIM response is modulated (e.g., in the lock-in and lock-out transition regions), long records are required to establish statistical values. While these regions don't produce as large of a VIM response, they could be important for computing mooring line fatigue. Hence sufficient time record lengths should be obtained. Note that the start up transient response should be excluded from the record for statistical analysis.

### CH.4.3 CURRENT INDUSTRY PRACTICES

As mentioned above, there are two practices prevalent in the industry today for spar VIM model tests, mainly centered on testing at either supercritical or subcritical Reynolds numbers. The former tests are performed using a horizontal, submerged cylinder in a high speed towing tank [CH.20, CH.21, CH.33], the latter being performed on a floating, surface-piercing vertical cylinder with external spring lines simulating the mooring system [CH.2, CH.11, CH.16, CH.17, CH.19, CH.27, CH.28]. The former approach has so far been limited to classic spars.

Model tests have not been performed for all spars. VIM response itself is self-limiting, and if a bounding analysis indicates that the mooring system will not be governed by high current or VIM responses then VIM tests have not been performed [CH.19].

#### CH.4.3.1 High, Supercritical, Reynolds Number Model Testing

In this approach, model tests are conducted for model Reynolds numbers in the supercritical range beyond $R_e$ = 600,000. The basis for testing the hull model at the supercritical $R_e$ regime is the assumption that, once beyond the transition range, model and prototype flow similitude is preserved. Model tests at supercritical Reynolds number for classic spar VIM show relatively good agreement with field measurements [CH.21, CH.33].

Note that high Reynolds number model testing places significant demand on the capacity of the model basin and is available only at a few test facilities worldwide. An example of the high, supercritical model testing of a classic spar can be found in [CH.21, CH.33]. The described rig has been used to tow the spar hull model at up to $R_e$ =2,000,000.

#### CH.4.3.2 Low Reynolds Number Model Testing

In this approach, the model is either towed at low speed or in-place tested in a flume or wave basin with current generating capability. Froude scaling is not explicitly required. However, the Froude number is typically chosen so that it is less than that of the full scale. The model test Reynolds number is typically in the subcritical range. Model tests at low Reynolds numbers for a classic spar has shown conservative results compared with field measurements [CH.20]. The conservatism may be due to the difference between the current profiles in the model test (uniform) and in the field (may have been non-uniform).

A benefit of this approach is that motions in all 6 degrees-of-freedom can be modeled. This allows for responses in the roll and pitch degrees of freedom to be identified and incorporated in the design. It also allows for the hydrodynamic coupling effects between the different degrees of freedom. The ability to use larger models also facilitates more detailed modeling of the hull details and appurtenances. The vertical moored set-up also gives the ability to model the spatial variation (nonlinearity and asymmetry) of the prototype mooring system. One additional benefit is that such approach has much less requirement for model basin capacity and can be carried out in model basins without high-speed tow capability. Examples of the low, subcritical Reynolds number model testing of spars can be found in [CH.11, CH.17, CH.20, CH.27, CH.28].

### CH.4.4 FIELD MEASUREMENT DATA COMPARED WITH MODEL TEST DATA

Field measurements of VIM response have been recorded for 3 classic spars—Genesis, Hoover, and Neptune [CH.20, CH.21, CH.33]. Note that in the field the current profiles vary in speed and heading with depth, as opposed to the slab current described in the tow tests earlier. Hence, adjustments to the model test values may be required to account for such variations. Although motion measurement systems have been installed on a number of truss spars, no significant VIM responses have been recorded [CH.9].

Of particular interest is the Genesis spar, for which field measurement data, supercritical and subcritical Reynolds number test data are available [CH.17, CH.20, CH.33].

## CH.5 Strength Analysis Procedure

Mooring analysis for high current/VIM conditions may require special computer software that is capable of modeling VIM in time domain. A simplified analysis procedure can be used if the waves associated with the current are mild, resulting in low mooring line dynamics. Once the VIM design criteria are established, the following simplified analysis procedure can be used. An example of strength analysis can be found in [CH.16].

1. Select a current direction

2. Determine the mean vessel offset under the design current with associated wind and waves based on an estimated $C_d$. Spar set-down should be considered to yield realistic results.

3. Calculate in-line and transverse VIM and $C_d$ based on the design criteria established according to Section

H.4. If this $C_d$ value is significantly different from the estimated $C_d$ in Step 2, iteration may be required.

4. Determine the envelope of possible maximum vessel offsets including the effects of current/wind/wave vessel offsets (Step 2), and in-line and transverse VIM (Step 3).

5. Determine line tensions and anchor loads corresponding to the envelope of possible maximum vessel offsets calculated in Step 4 by static mooring analysis.

6. Evaluate additional line tensions and anchor loads due to line dynamics, which is superimposed to the quasi-static values obtained in Step 5.

7. Repeat Steps 1–6 to obtain line tensions and anchor loads for other current directions.

8. Identify the worst direction for design check.

## CH.6 Analysis Procedure for Long Term Fatigue Damage

The recommended procedure for long term fatigue damage evaluation is provided below. An example of fatigue analysis can be found in [CH.16].

1. The long-term current events can be represented by a number of discrete current bins. Each current bin consists of a reference direction and a reference current velocity with associated wave and wind conditions. The probability of occurrence of each current bin must be specified. The number of reference directions depends on the directionality of the current at the site, and the specified directions should include those for which significant VIM is predicted. The required number of reference current velocities normally falls in a range of 10 to 50. Fatigue damage prediction can be fairly sensitive to this number for certain mooring systems, and therefore it is best determined by a sensitivity study.

2. Select a current bin and calculate the duration ti for the current bin in a year based on the probability of occurrence for the current velocity and direction.

3. Determine the natural period $T_n$ of the moored spar under the current bin without VIM based on an estimated $C_d$.

4. Specify extreme in-line and transverse *A/D* values for the current bin based on available model test or field measurement data. The mean *A/D* for fatigue analysis can be evaluated by multiplying the extreme *A/D* by a coefficient *g*, which should be determined by available model test or field measurement data.

5. Determine in-line and transverse VIM amplitude coefficient $C_v$, which is a function of reduced velocity, and is equal to 1.0 at peak VIM under lock-in condition.

6. Calculate the reduced velocity for the current bin and further modify the mean in-line and transverse *A/D* (Step 4) by $C_v$.

7. Determine drag coefficient $C_d$ for the current bin based on the modified mean transverse *A/D* (Step 6). If this $C_d$ value is significantly different from the estimated $C_d$ in Step 3, iteration may be required.

8. Perform VIM mooring analysis based on the modified mean in-line and transverse *A/D* (Step 6), and $C_d$ (Step 7), using the procedure for strength design. Determine average tension ranges $R_i$, and corresponding average response period $T_i$ from the time trace of line tensions for a few VIM cycles. Note the average response period $T_i$ may vary due to the relative orientation of the mooring line and current.

9. Determine number of cycles to failure $N_i$ corresponding to $R_i$ for the mooring component of interest using an appropriate T-N equation. Chain usually has the shortest fatigue life, and chain fatigue life at fairlead is further reduced because of additional stress concentration from bending. Stress concentration factor accounting for bending at fairlead should be determined by testing or finite element analysis. A factor $f_c$, which is defined as ratio of chain stress concentration factor at fairlead to that away from fairlead, can be used for calculating fatigue life of chain links at fairlead. The factor $f_c$ can vary significantly depending on the number of fairlead pocket and fit between chain and fairlead. This factor can be as low as 1.2 for a 7-pocket tight fit fairlead, but it can be higher for a loose fit fairlead. The value of $N_i$ is reduced by a factor of $f_c^M$ at the fairlead, where *M* is the slope of the T-N equation.

10. Calculate annual fatigue damage for the *i*th current bin:

$$D_i = ( t_i / T_i ) / N_i \qquad \text{(CH.8)}$$

11. Repeat Steps 2 to 10 for other current bins

12. Determine cumulative fatigue damage for all current bins, which is combined with the fatigue damage from wind and waves to obtain total fatigue damage $D_i$ (Refer to Section 6.3.2 for methods to combine fatigue damage). The predicted fatigue life is $1 / D_i$ (years), which should be greater than the service life times a factor of safety.

# CH.7 Methods to Improve Mooring Design for VIM

## CH.7.1 POLYESTER ROPE FOR MIDDLE SECTION

Spiral strand wire ropes are commonly placed at the middle section of mooring lines for spars. The use of polyester ropes in this section may sometimes reduce the line tensions due to VIM because the lower rope stiffness makes the polyester mooring more compliant for large vessel movements. Also the use of polyester rope may reduce $V_r$, which in turn may prevent lock-in. Also tension variation due to dynamic loads on the floating vessel can be lower for polyester mooring. This will result in lower fatigue damage to all mooring components including chain, which has the lowest fatigue resistance. A sensitivity study investigating the effects of using polyester ropes instead of spiral strands can be found in [CH.16].

## CH.7.2 SPIRAL STRAND FOR TOP AND BOTTOM SECTIONS

Chains are commonly placed at the top (vessel) and bottom (anchor) sections of mooring lines for spars. The use of spiral strand in these sections may significantly reduce fatigue damage due to VIM because spiral strand has much higher fatigue resistance than chain. This option requires significant hardware modification, which includes replacing chain jack and chain fairlead with linear winch and bending shoe. The industry has good experience with mooring systems using spiral strand, linear winch, and bending shoe.

## CH.7.3 IMPROVED CHAIN FAIRLEAD

The chain section in contact with the fairlead is more susceptible to fatigue failure because of the presence of bending load in addition to tension. Chain fairleads with 7 pockets are commonly used for spar moorings. The use of chain fairleads with 9 pockets can reduce chain bending, thus reducing chain fatigue damage in this section. Also chain fairlead design resulting in a tight fit between the chain and the fairlead pocket can yield a much lower stress concentration factor and longer fatigue life. Alternatively bending shoes that yield low stress concentration in chain can be used.

## CH.7.4 STRAKE DESIGN

VIM can be reduced by improved strake design. Options include improving strake shape, increasing strake height, and eliminating discontinuities and holes in strakes. To evaluate the effectiveness of these options, a rigorous model test program is required.

## CH.7.5 HULL APPURTENANCES

Hull appurtenances such as anodes, chain, fairleads, and pipes may affect spar VIM response. Measures to eliminate or reduce the adverse impact of these appurtenances may reduce VIM. A rigorous model test program is required to evaluate the effectiveness of these measures.

## CH.7.6 TIGHTENED MOORING

VIM is not observed in the model basin when $V_r$ is below a threshold value. This condition can be achieved in some cases by tightening the mooring system, for example using higher initial tensions, or tightening the mooring system in advance of high current events, thus reducing the natural period of the moored vessel, and eliminating VIM for current speeds below the maximum design value. The adoption of this measure should be based on rigorous model testing and analysis, and on addressing sensitivity to higher current and lower threshold $V_r$. An operational procedure to ensure a tight mooring during high current events should also be developed [CH.18].

## CH.7.7 SOFTENED MOORING

Softened mooring lines may significantly reduce the line tensions due to VIM because the lower mooring stiffness makes the mooring system more compliant for large vessel movements. A mooring system can be softened by different methods:

1. Use catenary mooring instead of taut leg mooring.
2. Increase the length of the bottom chain segment in a taut leg mooring.
3. Slacken all mooring lines of a taut leg mooring.
4. Slacken the leeward mooring lines of a taut leg mooring.

For methods 1 to 3, the risers must be able to tolerate the larger vessel offset resulting from lower mooring stiffness. Although Method 3 will reduce line tension for the same VIM, it may cause earlier initiation of VIM due to higher reduced velocity at lower current velocities. Method 4 may require a sophisticated mooring operational procedure because of the possibility of changing current velocity and direction. Implementation of this method requires careful investigation and planning.

# CH.8 Future Technology Development

## CH.8.1 MODEL TESTING AND FIELD MEASUREMENT

To minimize uncertainties in VIM prediction and provide data for calibrating analytical tools, advancement in model testing technology and acquisition of field data are essential. Currently the focus is on measuring the motion and line tension responses. However, more attention should be given to the measurement of the exciting forces in the future, since this may lead to development of reliable analytical tools for spar VIM prediction.

### CH.8.2 PEAK VALUE STATISTICS

Currently mooring strength and fatigue design are typically based on criteria established for extreme VIM values. This is different from the traditional approach that is based on standard deviation and peak value statistics, which is in turn a function of the duration of the extreme environmental event. The traditional approach is not used here because the peak value statistics have not been well established for transverse and inline VIM, and the duration for the extreme environmental event, for example the 100-year current, is difficult to estimate for many locations.

Preliminary investigation of some full scale and model test data for VIM of classic spars in the lock-in range (where the motion is well developed and sustained) indicates the maximum to standard deviation ratio for inline VIM is about 85% to 90% of that determined by the Rayleigh distribution. For transverse VIM in the lock-in range, the ratio of maximum to standard deviation of VIM amplitude can vary from 1.6 to over 2.0 for duration of a few hours to a few days, respectively. Note these values are derived from classic spars and are given for illustration only, and therefore should not be used for a specific project without further investigation. Further data is required to develop recommendations on peak value statistics and extreme current duration so the traditional approach can be adopted.

### CH.8.3 ANALYTICAL TOOL FOR VIM PREDICTION

Currently there is no reliable analytical tool for the prediction of spar VIM. Computational Fluid Dynamics (CFD) is being studied as a means of performing VIM assessment, but three dimensional codes necessary for analysis of spars with strakes are still under development. Attempts have been made to conduct time domain simulations for single or multiple degrees of freedom spar models with forcing functions determined from model testing, field measurement, or CFD. Although this approach is promising, significant effort is still required to advance it to a mature stage, especially in the area of forcing function. Since coupling of surge/pitch and sway/roll can be significant, a 4 degree-of-freedom (surge, sway, pitch, and roll) forcing function based on strip theory may provide satisfactory results. Once reliable analytical tools become available, standard deviation and peak response statistics can readily be generated, and traditional mooring analysis methods can be used.

### CH.8.4 MOORING ANALYSIS TOOL FOR SPAR VIM

Most mooring analysis software was not designed to handle spar VIM analysis therefore a simplified analysis procedure is often used. To improve accuracy, the industry needs advanced mooring analysis software that can model current, inline and transverse VIM in addition to other typical loads and motions.

## CH.9 References

**CH.1** Allen, D. W., and Henning, D. L., “Surface Roughness Effects on Vortex-Induced Vibration of Cylindrical Structures at Critical and Supercritical Reynolds Numbers,” OTC 13302, 2001.

**CH.2** Bangs, A.S., Miettinen, J. A., Mikkola, T. P. J-, Silvola, I, and Beattie, S. M., “Design of the Truss Spars for the Nansen/Boomvang Field Development,” OTC 14090, 2002.

**CH.3** Blevins, R. D., “Flow-Induced Vibration,” 2nd Ed, Kreiger Publishing, Malabar, FL., 2001.

**CH.4** Cowdrey, C. F. and Lawes, J. A. “Drag Measurements at High Reynolds Numbers of a Circular Cylinder Fitted with Three Helical Strakes,” NPL/Aero/384, National Physical Laboratory (UK), July, 1959.

**CH.5** Deep Oil Technology, Inc., “Vortex Induced Motions of Large Floating Structures,” Joint Industry Project Report, R-900501, 1990.

**CH.6** DeepStar Joint Industry Program, “Conceptual Spar Facility for Ultra-Deepwater GOM, Phase 1C, Hull VIV Issues,” Report DeepStar CTR 6405, 2003.

**CH.7** DEEPSTAR VI PROJECT, “DeepStar Ocean Turbulence Measurement Program,” Deepstar Report 6801, 2004.

**CH.8** Ding, Z. J-, Balasubramian, S., Lokken, R. T., and Yung, T-W., (2004) “Lift and Damping Characteristics of Bare and Straked Cylinders at Riser Scale Reynolds Numbers,” OTC 16341, 2004.

**CH.9** Edwards, R., Shilling, R. Theti, R. and Karayaka, M., “BP Horn Mountain Spar—Results of Comprehensive Monitoring of Platform and Riser Responses,” Deep Offshore Technology Conference and Exhibition, Marseille, 2003.

**CH.10** Feng, C. C., “The Measurement of Vortex-Induced Effects on Flow Past Stationary and Oscillating Circular and D-Section Cylinders,” M. A. Sc. Thesis, University of British Columbia, 1968.

**CH.11** Finn, L.D., Maher, J-V., and Gupta, H., “The Cell Spar and Vortex Induced Vibrations,” OTC 15244, 2003.

**CH.12** Gartshore, I. S., Khanna J. and Laccinole, S., “The effectiveness of vortex spoilers on a circular cylinder in smooth and turbulent flow,” Proc. 5th Intl. Conf. on Wind Engineering, Fort Collins, Colorado, 1978.

**CH.13** Hafen, B. E., Meggitt, D. J-, and Liu, F. C., "Strumming Suppression—An Annotated Bibliography," Technical Note N-1456, Naval Civil Engineering Lab, Oct., 1976.

**CH.14** Halkyard, J-, "Evolution of the Spar Floating Drilling, Production and Storage System," SNAME (Texas) Annual Meeting, Offshore Technology: Evolution and Innovation, Houston, Texas, 2001.

**CH.15** Holmes, S., Applied Research Associates, Personal Communication, Nov. 7, 2003. See link: http://www.arasvo.com/platform.htm.

**CH.16** Huang, K., Chen, X. and Kwan, C.T., "The Impact of Vortex-Induced Motions on Mooring System Design for Spar-based Installations," OTC 15245, 2003.

**CH.17** Irani, M. B. and Finn, L. D., "Model Testing for Vortex Induced Motions of Spar Platforms," OMAE2004-51315, 2004.

**CH.18** Kokkinis, T., Sandström, R. E., Jones, H. T., Thompson, H. M., and Greiner, W. L. "Development of a Stepped Line Tensioning Solution for Mitigating VIM Effects in Loop/Eddy Currents for the Genesis Spar," OMAE2004-51546, 2004.

**CH.19** Magee, Allan, Anil Sablok, Joe Gebara, "Mooring Design for Directional Spar Hull VIV," Proceedings, OTC 15243, 2003.

**CH.20** OTRC/MMS Workshop, "Spar Vortex-Induced Motions," Proceedings of Workshop, Navasota, Texas, October 22-24, 2003.

**CH.21** Sandström, R. E., Yung, T-W., Slocum, S. T. and Ding, Z. J-, "Advances in Prediction of VIV for Spar Hulls," Deep Offshore Technology Conference, Marseille, France, 19-21 Nov 2003.

**CH.22** Sarpkaya, T. and Isaacson, M., "Mechanics of Wave Forces on Offshore Structures," Van Nostrand Reinhold Company, New York, 1981.

**CH.23** Sarpkaya, T. "A Critical Review of the Intrinsic Nature of Vortex-Induced Vibration," *Journal of Fluids and Structures*, 19 389–447, 2004.

**CH.24** Skop, R.A., Griffin, O.M. and Ramberg, S.E. "Strumming Predictions for the SAECON II Experimental Mooring," Offshore Technology Conference, OTC 2491, 1977.

**CH.25** Smith, D. W., Kokkinis, T., Thompson, H. M. and Greiner, W. L., "Hindcasting VIM-Induced Mooring Fatigue for the Genesis Spar," OMAE2004-51547, 2004.

**CH.26** Soreide, M., "Experimental Investigation of In-Line and Cross-flow VIV," Proc. 13th Intl. Offshore and Polar Engineering Conf., Honolulu, 2003.

**CH.27** van Dijk R. T., Voogt A., Fourchy P. and Saadat M., "The Effect of Mooring System and Sheared Currents on Vortex Induced Motions of Truss Spars," Proceedings 22nd International Conference on Offshore Mechanics and Arctic Engineering, OMAE '03, Cancun, Mexico, 2003.

**CH.28** van Dijk R. T., Magee Allan, Perryman Steve, Gebara Joe, "Model Test Experience on Vortex Induced Vibrations of Truss Spars," OTC 15242, 2003.

**CH.29** Vandiver, J-K. "Drag Coefficients for Long-Flexible Cylinders," Offshore Technology Conference, OTC 4490, 1983.

**CH.30** Williamson, C.H.K. and Govardham R., "Vortex Induced Vibrations," *Annual Review of Fluid Mechanics*, 36:16.1-16.42, 2004.

**CH.31** Woodgate, L. and Matbrey, J. F. M. "Further Experiments on the Use of Helical Strakes for Avoiding Wind Excited Oscillations of Structures of Circular or Nearly Circular Section," Natl. Phys. Lab. (UK) Aero. Report 381, 1959.

**CH.32** Wu, Z.-J-, "Current Induced Vibrations of a Flexible Cylinder," Ph.D. Thesis, Division of Structural Engineering, The Norwegian Institute of Technology, The University of Trondheim, Norway, 1989.

**CH.33** Yung, T. W., Sandström, R. E., Slocum, S. T., Ding, J. Z. and Lokken, R. T., "Advancement of Spar VIV Prediction," OTC 16343, 2004.

**CH.34** Zdravkovich, M. M., "Review and Classification of Various Aerodynamic and Hydrodynamic Means for Suppressing Vortex Shedding," *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 7, 145-189, 1981.

# APPENDIX I—GLOBAL ANALYSIS GUIDELINES FOR DEEPWATER FLOATING SYSTEMS

## I.1 Basic Considerations

The design and analysis of mooring and riser systems for floating drilling and production systems require global analysis, which evaluates responses of the floating vessel under specific environmental conditions. Traditionally global analysis involves the use of both numerical analysis and physical model tests. Numerical analysis models are used to predict the responses for the floating vessel, and model tests are used to calibrate the numerical models and validate their predictions. The robustness of calibrated and validated numerical models to capture the physics and complex interactions at both model scale and prototype scale is critical to the overall design process. This procedure has provided the basis for many successful floating operations in the past.

As the industry progresses into deep (to 6,000 ft) and ultra deep (to 10,000 ft) water depths, current technology in predicting global responses of floating vessels faces new challenges. First, slender bodies such as risers and mooring lines have much higher impact on the floating vessel in deep and ultra deepwater. Also the number of slender bodies tends to increase as large deepwater fields are developed. The traditional approach of de-coupling the floating vessel and the slender bodies may not be sufficient for deep and ultra deepwater operations. To address this issue, sophisticated computer programs have been developed recently to perform coupled analysis where the vessel and the slender bodies are integrated in a single model to fully capture their interaction. Industry studies indicate, however, some of the computer programs may not be efficient tools for design because of the large computational effort required for the complicated models. Also they may not yield consistent results because different approaches, such as time domain and frequency domain, are used. Some designers still prefer to use the more efficient de-coupled or semi-coupled approaches for conducting a large number of global analyses required for floating system design. The use of various approaches increases the discrepancy in global response predictions.

The second challenge is model testing for deep and ultra deepwater operations. In relatively shallow water, a complete scaled model can be tested in a model basin at reasonable model scales (1:50 – 1:100). Test results can be directly scaled using Froude Number scaling to predict the prototype performance. This approach has been successfully used in many model basins to study floating systems in depths to 3,000 ft or more. Testing floating systems in deep and ultra deep water depths, however, is more difficult. Due to the depth and scope of mooring and riser systems, a complete scaled model of the deep and ultra deep water floating system is simply too large to test in present model basins. The model's mooring and riser systems are often truncated, and a numerical model is validated and then used to interpret and extend the model test results to predict the prototype floating system responses at the full design water depth. This approach is called truncation or hybrid verification method. Alternatively ultra-small scale testing (1:150 – 1:170) can be used. The industry has limited experience with the truncation and ultra-small scale testing methods. This causes additional uncertainty in global response predictions.

Another method that has been used by the industry is to model test components of the full system to gain understanding and confidence in the behavior and properties of the components. For example, in model testing CALM buoys, large scale forced oscillation tests of the buoy alone may be used to develop the hydrodynamic properties of the buoy, with the risers and mooring lines being tested separately, then numerical tools are used to model the entire system. Similarly, for truss spars the hard tank and truss members may be model tested separately, to get the best possible scaling, and combined in the model test setup and in the numerical modeling. However, this approach may not fully capture the coupling between the components or may introduce new uncertainties for the whole system at the expense of greater knowledge of the individual components.

The uncertainties in numerical analysis and model testing may result in either overly conservative design or unrecognized risks, and therefore may have safety and/or cost implications for a project. To address this issue, several studies have been sponsored by DeepStar to better understand the uncertainties in the predicted responses of deepwater systems and to provide a basis for reducing these uncertainties. A summary of the DeepStar studies is presented in Section I.2. The global analysis guidelines provided in this Appendix were derived from the DeepStar studies and industry experience, and have gone through a lengthy process of consensus building. The objective of this Appendix is to provide general design and analysis principles. It is not the purpose of this Appendix to dictate a specific approach for global analysis. Various approaches have been used in the past and this trend is expected to continue. Instead this Appendix points out the important parameters and advantages and limitations of various approaches for global analysis so the designers can make intelligent decisions for their analysis and model testing. This document also points out directions for future technology development.

## I.2 Summary of DeepStar Studies

A series of studies on global analysis were conducted in 3 phases of the DeepStar JIP from 1999 to 2003. In DeepStar Phase IV [Ref I.1] three theme structures—FPSO, Spar, and a TLP—were selected for the studies. Realistic designs were developed for each structure for water depths ranging from 3,000 to 10,000 ft, and design environments for Gulf of Mexico hurricanes and loop currents were used. Global performance analyses of these structures were performed by a number of organizations, and the predicted design responses were compared to characterize the differences and uncertainties among the different analyses. Since the differences and uncertainties were found to be significant, model tests to measure design responses for these three structures were then conducted to provide the basis for further comparing and characterizing the uncertainty between predicted and measured responses.

In DeepStar Phase V [Ref I.2–I.12] a study was undertaken to compare the predicted responses of three theme structures with model test in order to assess the accuracy of and differences between various numerical models. In this study, analysts from a number of organizations conducted detailed analyses of data from the floating vessel model tests to validate their analytical tools. Following the validation process, predicted results from the validated analysis models were compared to model tests for each floating structure to assess the overall accuracy and scatter between the predicted response and measured values from the model tests. Included in DeepStar V is also a first attempt to develop guidelines for global performance analysis and verification of deepwater structures.

In DeepStar Phase VI a review was first carried out on the guidelines for global performance analysis and verification to identify areas for further development [Ref I.13]. Then a study was conducted to further develop global analysis guidelines that can be used to guide global response analysis and model testing and can be directly incorporated into API RP 2SK. In addition, methods for model testing floating structures with truncated mooring lines and risers were further investigated.

These studies represent a major effort by the industry to advance the global analysis technology for deep and ultradeep water operations. The most important piece of work is the model testing for 3 theme structures, which provides a sound basis for bench marking different analysis models. Important findings from the DeepStar and other industry studies have been incorporated in the following sections.

## I.3 Coupling of Floating Structures

Floating drilling and production systems typically consist of single or multiple floating vessels such as spar, TLP, FPSO, or semi-submersible, which are connected with slender bodies such as mooring lines, risers, fluid transfer lines, and umbilicals. Coupling of the floating vessels and slender bodies can be divided into 2 categories:

### I.3.1 SINGLE VESSEL COUPLING

In this case a single floating vessel is connected with mooring lines (or tendons in TLP) and risers, which affect the vessel responses in the following terms:

1. **Mooring and riser system stiffness**—Mooring stiffness provides majority of the restoring force to keep the vessel on station and therefore must be properly accounted for. The importance of riser stiffness depends on number and type of riser. For drilling vessel equipped with a single top tensioned riser, the riser stiffness is often neglected. On the other hand, for floating production vessels equipped with a large number of risers, riser mean load and stiffness can make significant contribution to the total restoring force and therefore should be properly accounted for. In particular, mean riser loads may result in significant asymmetry in the stiffness of combined riser and mooring system. In the case of CALM buoys, mooring and riser system forces may be large compared to first order wave forces, requiring wave frequency responses to be calculated in the presence of the riser and mooring system.
2. **Mooring and riser system damping**—Mooring and riser system damping affects mainly the low frequency motions of the vessel. The importance of this parameter depends on the number, type, and size of the mooring lines and risers, water depth, type of vessel, and the metocean environment. For a semi-submersible drilling vessel equipped with a single riser and 8 mooring lines operating in shallow water depths, mooring and riser damping is often neglected. As the water depth, number of mooring lines and risers increase, mooring and riser damping will become more and more important, especially for vessels such as FPSO where low frequency motions dominate the design. In general, low frequency damping from mooring lines and risers is larger for catenary systems than for taut systems, and increases as the magnitude of the vessel's wave frequency motions increase. Neglecting mooring and riser damping in global analysis is always conservative and may be justified where the impact of the damping is small. However, the cost impact of this practice should be carefully evaluated to avoid significantly over-sizing the mooring and riser system.
3. **Current load on mooring and riser**—Current load on mooring and riser imposes additional loading on the floating vessel. The importance of this parameter depends on the number, type, and size of the mooring lines and risers, the water depth, and the relative mag-

nitude of the current loads compared to wind and wave loads. Since neglecting this loading is always non-conservative, it is good practice to include it in global analysis, especially in deep water or high current area.

4. **Wave load on mooring and riser—**Wave load on mooring and riser has not been found to play an important role on global response of large floating vessels and therefore is often neglected. However for CALM buoys, offloading and mooring lines may have a significant impact on the wave frequency motions of the buoy.
5. **Inertia of mooring and riser**—Inertia of mooring and riser provides additional contribution to the inertia of the floating vessel. This parameter may not be very important for large floating vessels, but can be important for small floating structures such as buoys.

The industry has used a number of approaches to analyze a floating vessel connected with mooring and riser system:

1. **De-coupled analysis—**In this approach the floating vessel and the mooring or riser system are analyzed separately and therefore called de-coupled analysis. The floating vessel is analyzed first, and the slender body parameters such as stiffness, damping, wave and current load, and inertia are either estimated and input to the floating vessel analysis or simply neglected. Mooring stiffness is most important and always required by the floating vessel analysis. This has not been a problem since mooring stiffness can easily be estimated. Inclusion of other parameters often depends on the judgment of the analyst and availability of appropriate parameter values. The responses of the vessel from the analysis are then input to the mooring or riser analysis to obtain the responses of the slender bodies.

   This approach was routinely used in the early stage of the offshore industry because coupled analysis tools were not available. The accuracy of the analysis is uncertain, depending largely on the skill and experience of the analyst.

2. **Coupled analysis**—In this approach the floating vessel, the mooring and riser system are integrated into a single model and the vessel and slender bodies are analyzed dynamically. This approach can fully account for the interaction between the vessel and the slender bodies and therefore allows capture of all the slender body effects such as stiffness, damping, wave and current load, and inertia. Some software may select to ignore minor effects such as wave load on the slender bodies. This approach is most accurate, but the analysis can be very time consuming unless efficient frequency domain solutions are used.
3. **Semi-coupled analysis**—In this approach the floating vessel, the mooring and riser system are integrated into a single model. The vessel is analyzed dynamically, but the slender bodies are analyzed quasi-statically. This approach can accurately account for the current load and nonlinear stiffness of the mooring and riser system but cannot capture the damping and inertia of the slender bodies, which are either estimated separately or simply neglected. By neglecting slender body dynamics, the analysis can be much faster, but the accuracy will depend largely on the slender body damping estimation.

### I.3.2 MULTIPLE VESSEL COUPLING

In this case two or more floating vessels are connected with fluid transfer lines (FTL) and therefore the response of one vessel affects the other vessels. An example of multiple vessel coupling is an FPSO connected to a spar or TLP dry tree unit on one side and an offloading buoy on the other side. Another example is a moored drilling tender vessel operating alongside a floating dry tree unit (spar or TLP), with both vessels inter-connected by mooring hawsers. There are two types of multiple vessel coupling:

1. **Environmental load coupling—**This coupling arises because each vessel influences the incident wave kinematics field in its vicinity due to diffraction and radiation effects. Also there are shielding effects for wind and current loads. This coupling can be negligible if the vessels are far apart from each other.
2. **Structural coupling**—The response of one vessel can be affected by another vessel through the FTLs, and the stiffness, damping, inertia, wave and current loads of the FTLs can influence both vessels connected to its ends.

Similar to single vessel coupling, the industry has used a number of approaches to analyze multiple floating vessels connected with FTLs.

1. **De-coupled analysis**—In this approach the floating vessels are analyzed separately. The environmental load coupling is neglected, and only the mean load of the FTLs is accounted for in the structural coupling. The accuracy of this approach depends on the distance between the vessels, the size of the FTLs and the vessels, the relative stiffness of the FTLs and the vessel's mooring systems, and the severity of the environment. Industry experience indicates that if the FTL stiffness is much smaller than the stiffness of the floating vessels, this approach can provide good results.
2. **Coupled analysis**—In this approach the floating vessels and the FTLs are integrated into a single model and the vessels and FTLs are analyzed dynamically.

This approach can fully account for the interaction between the vessels and the FTLs and therefore can capture all FTL effects such as stiffness, damping, wave and current load, and inertia. This approach is most accurate, but the analysis can be very time consuming unless efficient frequency domain solutions are used. If the floating vessels are close to each other, the coupling effects of waves, wind, and current should also be accounted for in a fully coupled analysis.

3. **Semi-coupled analysis**—In this approach the floating vessels and the FTLs are integrated into a single model. The vessels are analyzed dynamically, but the FTLs, mooring lines, and risers are analyzed quasi-statically. This approach cannot capture the damping and inertia of the FTLs, mooring lines, and risers, which are either estimated separately or simply neglected. By neglecting slender body dynamics, the analysis can be much faster, but the accuracy will depend on distance between the vessels, size of slender bodies and the vessels, the severity of the environment, and the estimate of slender body low frequency damping used.

## I.4 Time Domain and Frequency Domain Dynamic Analysis

Two methods, frequency domain and time domain analysis, are commonly used for predicting dynamic responses of a floating vessel. In the time domain method, all nonlinear effects can be modeled. The term time domain implies recalculation of each mass term, damping term, stiffness term, and load at each time step. Hence the computation can become complex and time consuming. The frequency domain method, on the other hand, depends on the linear principle of superposition. Hence, all nonlinearities must be eliminated, either by direct linearization or by an iterative linearization. It should be noted that linearization in frequency domain analysis is required only for dynamic response about the mean position. Nonlinear properties such as nonlinear mooring stiffness to determine the mean position and amplitudes of low frequency motions can be handled approximately by frequency domain analysis.

### I.4.1 ENVIRONMENTAL LOADS ON FLOATING VESSELS

Dynamic environmental loads such as wind, current, and wave loads are normally analyzed in frequency domain using a wind/current excitation and wave radiation/diffraction solver. The average wetted area of the vessel is used in the analysis. The results of the analysis gives first order excitation forces, hydrostatics, potential damping, added mass, first order RAOs, second order drift force coefficients, and wind/current load spectrum. These results are directly applicable to frequency domain coupled analysis. For time domain coupled analysis, however, the results from the frequency domain hydrodynamic analysis is normally transformed to time domain by convolution. This practice results in generally consistent environmental loads for both frequency domain and time domain analysis. Industry studies indicate that this approach gives satisfactory solutions for typical floating systems [Ref I.14].

It should be noted that low frequency vessel motions are caused, in part, by nonlinear second order drift forces. If the natural period of the moored floater is high, say more than 25 seconds, a linear frequency domain solution can be achieved by Newman's approximation, which eliminates the off-diagonal terms in the QTF (Quadratic Transfer Function) matrix. Newman's approximation generally gives satisfactory results for low frequency motions in the horizontal plane where the natural periods are much larger than the wave periods. For low frequency motions in the vertical plane, for example the pitch motion of a spar, Newman's approximation may underestimate the second order drift forces. If response of this type is important for the design, time domain solution of a full QTF matrix may be required.

Second-order wave forces in a random sea oscillating at the sum-frequencies may excite resonant response in heave, roll, and pitch of a TLP, which is often referred to as springing response. Also deepwater TLPs may experience very large resonant high frequency transient ringing response. Time domain analysis is typically performed to evaluate these high frequency responses for TLP. Cylindrical hull forms such as spar may be subjected to highly nonlinear vortex induced excitation, which is also typically analyzed in time domain.

### I.4.2 DYNAMICS OF SLENDER BODIES

There are 4 types of nonlinearity for slender bodies such as risers and mooring lines, which can be directly modeled by time domain analysis but must be linearized by frequency domain analysis:

1. **Line stretching**—The load versus elongation relationship of slender bodies must be linearized for the dynamic response. The stiffness cannot be a function of dynamic line tension but can vary along the line. This is usually not a difficult requirement even in the case of synthetic material and in most cases, a suitable linearization can be achieved.
2. **Geometry change**—In the frequency domain method it is assumed that the dynamic displacements are small perturbations about a static position. The static shape is fixed and all geometric quantities are computed based on this position. The mass, added mass, stiffness, etc.

are computed only once. Changes in catenary shape due to the dynamic motion contribution are generally not severe, especially for deepwater systems. Hence, a linearization about the position under mean load is generally acceptable.

3. **Fluid loads**—The nonlinear term in the Morrison equation must be linearized. The quadratic relationship in the relative velocity must be replaced by an equivalent linear relationship. The linearization should take into account the frequency content of the line motion spectrum. The development of effective stochastic linearization or energy dissipation method allows accurate linear approximation for fluid loads on slender bodies.
4. **Bottom effects**—The frictional behavior between the grounded line and the seafloor cannot be represented exactly in the frequency domain. Only the average or equivalent behavior of the line can be postulated and included. This simplification should be adjusted to the design objective. Different models may be required for the fatigue and the extreme tension evaluations.

## I.5 Current Industry Practice

As discussed in Sections I.3 and I.4, various analysis techniques such as de-coupled, coupled, semi-coupled, time domain, and frequency domain analysis have been developed to evaluate global responses of floating systems. A survey of current industry design practice indicates that designers are using various approaches, depending on vessel type (FPSO, spar, TLP, semi-submersible, etc.), loading type (extreme, fatigue, etc.), design stage (preliminary, final, etc.), and operation type (drilling, production, offloading, etc.). Following are the approaches identified by the survey, which is focused on single vessel coupling:

### I.5.1 DE-COUPLED FREQUENCY DOMAIN ANALYSIS

In this approach, dynamic responses of the floating vessel alone are calculated by a frequency domain wind/current excitation and wave radiation/diffraction solver. Typically stiffness of the slender bodies is accounted for but the damping is not, resulting in conservative motion predictions. The responses of the floating vessel are then input to mooring or riser analysis software to obtain mooring or riser responses. This approached has been used for various types of floating vessels.

### I.5.2 SEMI-COUPLED TIME DOMAIN ANALYSIS

In this approach the floating vessel, the mooring and riser system are integrated into a single model. The vessel is analyzed dynamically using a time domain solver, but the slender bodies are analyzed quasi-statically (using tension/offset table) to allow fast analysis. The damping of the slender bodies is typically neglected, resulting in conservative motion predictions. The time history of the floating vessel response is then input to a dynamic mooring or riser analysis software to obtained mooring or riser responses. To further simplify the analysis, dynamic amplification factors are often derived from the dynamic mooring analysis, which are then used to modify the quasi-static tensions from the semi-coupled time domain analysis. This approach has been extensively used for spar floating production systems.

### I.5.3 SEMI-COUPLED TIME DOMAIN PLUS COUPLED TIME DOMAIN ANALYSIS

In this approach a large number of design cases are analyzed by the semi-coupled time domain analysis with damping of slender bodies estimated from model test or energy dissipation method. A few critical load cases are identified and analyzed by coupled time domain analysis. This approach has been used for FPSOs.

### I.5.4 COUPLED FREQUENCY DOMAIN ANALYSIS

In this approach coupled frequency domain dynamic analysis software that captures all slender body effects is used in all design phases. This is the most efficient approach but requires the software to be extensively verified by model testing and/or coupled time domain analysis. This approach has been used for FPSOs.

### I.5.5 COUPLED FREQUENCY DOMAIN PLUS COUPLED TIME DOMAIN ANALYSIS

In this approach a large number of design cases are analyzed by coupled frequency domain analysis. A few critical load cases are identified and analyzed by coupled time domain analysis. This approach has been used for FPSOs, spars, and semi-submersibles.

### I.5.6 DE-COUPLED FREQUENCY DOMAIN PLUS COUPLED TIME DOMAIN ANALYSIS

In this approach, responses of the floating vessel alone are calculated by a frequency domain radiation/diffraction solver such as WAMIT. The responses of the floating vessel are then input to finite element models for slender body responses. Coupled time domain analysis or model test is then used to derive calibration factors for the de-coupled frequency domain analysis and to analyze critical cases and fatigue loading. This approached has been used for TLPs.

## I.6 Assessment of Current Industry Practice

### I.6.1 CRITERIA FOR EVALUATING GLOBAL ANALYSIS SOFTWARE

In order to assess current industry practice, the following criteria are established for evaluating global analysis software:

1. **Accuracy**—The software should yield accurate prediction for responses of the floating vessel and slender bodies. This is essential to achieve a good balance between reliability and cost of the floating system. Many designers tend to bias on the conservative side when faced with uncertainty in global response prediction. For example some software cannot predict damping of slender bodies, and the damping is simply neglected by some designers. This is obviously conservative, but the cost impact of this practice can be significant, especially for deepwater systems with a large number of mooring lines and risers. The software is required to yield accurate results not only for maximum responses, but also for high and low frequency standard deviation responses that are important for fatigue analysis. For fiber rope moorings, the minimum responses can also be important.

2. **Efficiency**—To properly design a deepwater floating system, a large number of load cases are often analyzed. This is necessary to address a large number of seastates and environmental directions required in the strength and fatigue design of mooring, riser, and structural components of the vessel. Optimization of mooring and riser systems may further increase the number of load cases. The number of load cases typically ranges from a few hundred to over a thousand using the deterministic design approach. Using response based design approach; however, the number of load cases may exceed 10,000. Therefore it is essential to have efficient global analysis software to ensure a proper design can be completed on schedule and within budget.

### I.6.2 DE-COUPLED FREQUENCY DOMAIN ANALYSIS

This approach cannot accurately account for the interaction between the floating vessel and the slender bodies and therefore is not considered very accurate without calibration with model test or coupled time domain analysis. Also the analysis is conducted in 2 steps: first the floating vessel and second the mooring or riser system. The efficiency of this approach depends on whether there is an efficient link between the two steps. This approach can be used for temporary operations such drilling operations using MODUs where riser and mooring lines are few. It is not an accurate and efficient approach for major deepwater floating production operations, if calibration with model test or coupled time domain analysis is not carried out.

### I.6.3 SEMI-COUPLED TIME DOMAIN ANALYSIS

This approach is more accurate than the de-coupled frequency domain analysis in that the non-linear stiffness and current load of the slender bodies can be better accounted for. However, this approach cannot calculate the damping of the slender bodies, which can be important for most deepwater floating production systems. By using quasi-static solution for slender bodies, the efficiency is much better than the coupled time domain analysis. However, this cannot be considered an efficient analysis tool since dealing with random time history still requires substantial computational effort.

### I.6.4 COUPLED TIME DOMAIN ANALYSIS

This approach is considered the most accurate for its capability to capture all system nonlinearities, but least efficient. It should be emphasized that the accuracy of this approach is not given because mishandling of certain time domain analysis parameters can lead to significant errors in response predictions. Industry studies indicate that to achieve good accuracy, the time step, length of simulation, number of replicates, number of frequencies to represent the loading spectrum, and method to derive the maximum response must be carefully chosen. This often results in long simulation times of hours to hundreds of hours for each load case even with modern high speed PCs. The engineering time to investigate a number of time domain simulations can also be significant. Because of its low efficiency, this approach is not suitable for routine design of floating systems. However, it is a valuable analytical tool for systems of high nonlinearity or large movement, for checking critical cases in the design and verifying the more efficient frequency domain approach.

### I.6.5 COUPLED FREQUENCY DOMAIN ANALYSIS

This approach is most efficient and can be accurate for typical deepwater floating systems. The frequency domain method uses linear principle of superposition. Most slender body nonlinearities such as line stretching, geometry change, and bottom effect are very minor for deepwater systems. The only true nonlinear term is fluid load, which can be effectively linearized by well established technology. From theoretical point of view, coupled frequency domain analysis should yield accurate response predictions for typical deepwater systems if slender body fluid load is properly linearized.

The above view has been verified by DeepStar and other industry studies, which compared frequency domain solutions with model testing and time domain solutions. The comparison was conducted for different types of floating structure operating in GOM and West Africa environments. Frequency domain solutions show good agreement compared with DeepStar model test results and time domain simulations for all 3 DeepStar theme structures—spar, turret moored FPSO, and TLP. Other industry studies show good agreement between frequency and time domain solutions for a semi-submersible and a spar floating production system operating in GOM and a spread moored FPSO operating in West Africa [Ref I.14–I.17]. It should be noted that turret moored FPSO with large low frequency yaw may require special treatment in frequency domain solution, as discussed in Section I.6.8.3.

Coupled frequency domain analysis is very efficient. With a high speed PC, the run time for a load case ranges from seconds to minutes, depending on software efficiency, which is several orders of magnitude faster than coupled time domain analysis. It is very suitable for analyzing a large number of load cases required by the deepwater floating systems. However, it may not be able to handle systems of high nonlinearity or large movement, and it requires significant verification or calibration by model testing and/or time domain analysis.

## I.6.6 COMBINATION OF APPROACHES

As discussed in Section I.5, designers often use a combination of 2 to 3 approaches to achieve the accuracy and efficiency required for deepwater system design. For example semi-coupled time domain analysis or coupled frequency domain analysis is used for large number of load cases and coupled time domain analysis is used for a few critical load cases. This practice has served the industry well in the past and is expected to continue in the future when the industry moves into even deeper water. However, from the accuracy and efficiency point of view, coupled analysis is more superior to de-coupled and semi-coupled analysis, and therefore should be more emphasized for future applications.

## I.6.7 TIME DOMAIN ANALYSIS GUIDELINES

### I.6.7.1 Time Step

Choice of time step is crucial for the stability and accuracy of time domain solution, and is often dependent on the periods of the responses, degree of nonlinearity, and analysis formulation. Time step should be determined by a sensitivity check. Recent studies for deepwater floating systems typically used time steps ranging from 0.1 sec. to 0.5 sec.

### I.6.7.2 Length of Simulation

Traditionally 3-hour duration is often used for model testing and time domain simulation. This duration is generally sufficient for the standard deviation of wave frequency responses because it represents about 1,000 cycles of response with a period of 10 sec. Low frequency responses for deepwater systems, however, typically have periods of several minutes. A 3-hour simulation may contain less than 50 cycles, which is insufficient to provide a good statistical confidence for standard deviation. The requirement for simulation length may even be higher for extreme responses. For example to obtain statistically meaningful wave frequency extreme responses, several 3-hour simulations may be needed. Required length of simulation depends on a number of factors, such as periods of wave and low frequency responses, contribution of wave and low frequency responses to total response, degree of nonlinearity, and system damping. It should be determined by sensitivity check. Recent studies indicate that five to ten 3-hour simulations (15 to 30 hours) or equivalent with different seed numbers may provide standard deviation and extreme responses of good confidence for typical deepwater floating systems [Ref I.14–I.17].

### I.6.7.3 Frequency Discretisation

The number and range of discrete frequencies representing floater transfer functions should be carefully chosen to cover the peaks in the transfer functions and area of significant wave excitation. Also it should be clarified how the actual computer program handles possible excitation outside the frequency range of the floater transfer function since this can be a source for erroneous prediction. Small frequency spacing may be required to avoid repeating time history within the simulation length. This problem can be alleviated by using variable frequency spacing, but the repetition period of the time history is more difficult to assess. The number, range, and spacing of frequency should be determined by sensitivity check. Industry studies have shown that a few hundred equally spaced frequencies yield satisfactory results for typical deepwater floating systems. To determine proper frequency spacing, the following equations can be considered [Ref I.15]:

- To capture the resonant response:

$$\Delta\omega \le min(\zeta\omega_n) \tag{I.1}$$

- To avoid repetition of time history:

$$\Delta\omega \le \frac{2\pi}{T_{simulation}} \tag{I.2}$$

where

$\omega$ = natural frequency of mode *n*,

$\zeta$ = is the model damping as ratio to critical damping.

### I.6.7.4 Initial Transient Response

The length of time domain simulation must allow for transient response during the initial part of the simulation. The time for transient response is normally a function of the period and damping of the response. Industry studies indicate that typical deepwater floating systems may require approximately a couple thousand seconds for transient response. To determine a proper time for initial transient response, the following equation can be considered [Ref I.15]:

$$T_{transient} \geq max\left(\frac{T_n}{2\zeta}\right) \tag{I.3}$$

where

$T_n$ = natural period of mode $n$,

$\zeta$ = model damping as ratio to critical damping.

### I.6.7.5 Extreme Response

The determination of extreme value from stochastic time history is always a challenge, which can be addressed by the following 3 methods:

1. **Probability Density Function (PDF)**—In this approach a PDF for the extreme response is constructed, and the Most Probable Maximum (MPM) is the response where the PDF is a maximum. This is not a practical approach since construction of a smooth PDF may require a large number; say a few hundred, of maximum responses from different realizations [Ref I.15].
2. **Average of Maximum Responses—**In this classical approach the average of the maximum responses from a number of realizations of different random seeds is taken as design maximum response. The average of maximum responses from five to ten 3-hour simulations is typically used for deepwater floating systems.
3. **Fitted Probability Distribution Model**—In this approach a peak probability distribution model, such as Rayleigh, Normal, Gumbel, Weibull, and Exponential, is selected and the parameters in the selected model are determined using available response time histories. Then the expected extreme response can be computed from the fitted model. This approach may require fewer realizations then Method 2. However, in practical applications the fitted parametric model often fails to describe the "true" upper tail behavior, resulting in biased extreme response prediction. Some analysts use special techniques such as fitting the upper tail or taking average of predictions from several realizations to improve accuracy. Nevertheless this is an approach that requires substantial skill and experience.

Method 1 is most rigorous but is not a practical approach for design. Method 2 and 3 are commonly used and considered acceptable approaches, but Method 3 requires more skill and experience.

### I.6.7.6 Application and Limitation of Time Domain Analysis

As can be seen from above discussion, time domain analysis places a heavy burden in terms of computer and engineering time, skill, and experience on a designer. Mishandling of certain analysis parameters can lead to erroneous predictions. Its inefficiency makes it unsuitable to handle a large number of load cases, which unfortunately are often required for modern deepwater systems. The advantage of time domain analysis is its capability to model all system nonlinearity. This advantage may be limited for typical deepwater floating systems since most nonlinearities are either minor or can be effectively linearized. Nevertheless, time domain analysis can be valuable for the following applications:

1. **Critical Load Cases**—Time domain analysis is often performed for a few critical load cases to ensure a reliable design after a large number of load cases have been analyzed by the more efficient frequency domain analysis.
2. **Verification and Calibration Factor for Frequency Domain Analysis**—Time domain analysis can be used to verify frequency domain analysis software or develop calibration factors to modify frequency domain analysis results.
3. **Large Vessel Movement—**Examples of this condition include turret moored FPSO subjected to large low frequency yaw and floating vessel equipped with DICAS (Differentiated Compliance Anchor System)
4. **Transient Condition—**Examples of this condition include broken line transient condition and floater subjected to West Africa squall.
5. **Highly Nonlinear Hydrodynamic Loading—**Structural components subjected to highly nonlinear hydrodynamic loading may require time domain analysis. Examples of this condition include green water effect, ringing and springing of TLP tendons, wave loading on a buoy, and cylindrical components subjected to vortex induced excitation.
6. **Highly Nonlinear Bottom Effect**—An example of this condition is a long length of mooring line or riser moving up and down on the seafloor under a severe storm.
7. **Dynamic Positioning**—Floating vessels equipped with DP system or DP assisted mooring may require time domain stationkeeping analysis.
8. **Highly Nonlinear Mooring Material—**Examples of this condition include certain fiber ropes such as nylon and rubber mooring fender.
9. **Impact or Contact Loading—**Examples of this condition is collision of floating vessels and loads on rubber fender or riser guide.

### I.6.8 FREQUENCY DOMAIN ANALYSIS GUIDELINES

#### CI.6.8.1 Analysis Parameters

The frequency discretisation requirement is similar to that for time domain analysis as discussed in Section I.6.7.3. One exception is that the number of frequencies can be much less because repeating time history is no longer an issue. If a finite element solution is used, the user needs to pay attention to the number of elements representing the floating vessel and slender bodies, but this is the same for time domain analysis.

The primary responses from frequency domain analysis are the standard deviation responses for wave and low frequency components. The extreme responses are derived from the standard deviation responses and a suitable peak probability distribution model. For floating systems where frequency domain analysis yields good approximations, most of the wave and low frequency responses can be represented by a narrow band Gaussian process with Rayleigh distributed peaks, and Equations 5.5 to 5.8 in Section 5.5 in the main document should apply. This does not rule out the use of other peak probability distribution models if they can be shown to better represent the responses of a floating system. Time domain simulation or model testing are often used to identify the most suitable model to determine the extreme responses from the standard deviation responses.

#### I.6.8.2 Verification and Calibration of Frequency Domain Analysis Software

Since frequency domain solution is a linear approximation to nonlinear problems, frequency domain analysis software should be thoroughly verified or calibrated before use for design. Model test data should be used for this purpose if good model test data are available. Time domain solutions can also be used to bench mark frequency domain solutions.

It is common industry practice to check a few critical load cases by model testing and/or time domain analysis after the floating system is designed by the more efficient frequency domain analysis. This is a prudent practice to ensure a reliable floating system.

#### I.6.8.3 Application and Limitation of Frequency Domain Analysis

DeepStar and industry studies have shown that frequency domain coupled analysis yields accurate response predictions for typical deepwater floating systems including spar, TLP, FPSO, and semi-submersible floating systems. In fact frequency domain analysis has been used for the design of many major floating operations around the world. There are cases, however, that are difficult for frequency domain analysis to handle, and time domain analysis may be more appropriate. These cases include large vessel movement, transient condition, dynamic positioning, highly nonlinear hydrodynamic loading, highly nonlinear mooring material, and impact or contact loading, etc., as discussed in Section I.6.7.6. It should be noted that some of these cases can still be solved in frequency domain, but special treatments may be required to yield acceptable or conservative approximations.

An example case that requires special treatment is turret moored FPSO, which may experience large low frequency yaw due to turret located at a significant distance from the bow. The large low frequency yaw causes significant change in vessel heading, wind, wave, and current loading, resulting in non-stationary vessel and slender body responses. The traditional frequency domain approach of fixing the vessel at the stable equilibrium heading will not yield accurate response predictions, and the Rayleigh distribution may not be a good fit for the response peaks. Several special treatments have been used by the industry to provide better frequency domain solutions. Section 5.6.2 of the main document describes a conservative special procedure, which uses a design heading defined as the stable equilibrium heading plus or minus significant low frequency yaw. In another special procedure, frequency domain analyses are performed at different headings that bound the expected range of low frequency yaw. It should be emphasized that special treatments of this nature require significant verification and calibration by model testing and/or time domain solutions.

## I.7 DeepStar Studies for Global Analysis Guidelines

### I.7.1 STUDY SCOPE

DeepStar Phase VI has conducted a series of studies for the development of global analysis guidelines. The studies include:

1. State of technology review for global analysis of floating vessels
2. Comparison of time and frequency domain approach for coupled analysis
3. Sensitivity study on mooring line fatigue
4. Parametric study on effects of slender body and other important parameters to global responses of floating vessels. The following parameters have been investigated:
    - Vessel type: spar, TLP, FPSO, semi-submersible.
    - Water depth: 3,000, 6,000, and 10,000 ft.
    - Environment: GOM hurricane and loop current (Spar VIM in loop current is not considered).
    - Number of slender bodies: small, median, and large.
    - Mooring line material: steel, polyester.
    - Drag coefficient of slender bodies: low, median, and high.
    - Wave period: base value plus and minus 10%.

- Significant wave height: base value plus and minus 10%.
- Wind spectrum: NPD and API.

The spar is a classic spar of 122 ft diameter and 650 ft draft. It is moored by a symmetric 14-point chain/wire rope/chain mooring, and the risers are vertical self standing risers. The TLP is a 4-column, ring pontoon semi-submersible type with a displacement of 59,000 short tons. It is moored by 12 tendons, and the risers are vertical top tensioned risers. The FPSO is a 200,000 DWT, turret moored vessel. It is moored by a symmetric 12-point chain/wire rope/chain mooring, and the risers are catenary type. The semi-submersible is a 4-column, ring pontoon floater with a displacement of 64,000 short tons. It is moored by a symmetric 16-point chain/wire rope/chain mooring, and the risers are catenary type. All the moorings were designed for an offset limit of 5% and tension limit of 60% under intact condition. The change in the number of slender bodies was achieved by changing the number of risers.

Major conclusions from these studies are provided below. It should be emphasized that the response values from the parametric study were generated for the 4 specific vessels by a frequency domain coupled analysis program to indicate some trends. Any use of these values outside this context may not be appropriate.

### I.7.2 MAJOR CONCLUSIONS ON GENERAL DESIGN PRACTICE

- Modern design process requires accurate and efficient analysis tools for global analysis. The industry has used various approaches, often a combination of different tools, to meet this requirement. This practice is expected to continue as the industry moves into deeper and deeper water.
- Differences in global response predictions from different approaches are inevitable. The uncertainties due to these differences have been addressed by conservative analysis procedures and factor of safety in the strength design.
- Differences in standard deviation responses are much more significant than extreme responses, which combine mean and maximum dynamic responses. The differences in standard deviation responses are magnified exponentially in fatigue analysis. Typically model tests are conducted for extreme seastates, which may not provide sufficient data for verifying the lower fatigue seastates. This adds more uncertainty to fatigue life prediction, in addition to the uncertainties known to the industry due to T-N curve, factor of safety, and method to combine wave frequency and low frequency damages.
- The uncertainties in global analysis tend to decrease with increasing water depth, as the dynamic response in terms of total response becomes smaller in deeper water. This indicates that extending the technology and experience in global analysis for shallow and deep water operations to ultra deep water operations is generally acceptable. (Refer to Section I.7.4.)

### I.7.3 MAJOR CONCLUSIONS ON TIME AND FREQUENCY DOMAIN COUPLED ANALYSIS FOR SINGLE VESSEL

- Coupling between floating vessel and slender bodies is important for deepwater operations. Slender body parameters affecting global responses include stiffness, damping, current load, wave load, and inertial load. Some parameters can be neglected under various conditions.
- Analytical approaches addressing the coupling effects can be classified in 3 categories: de-coupled, coupled, and semi-coupled analysis, which can be conducted in time or frequency domain. Coupled analysis is more superior to the others and therefore should be emphasized in the future.
- Time domain coupled analysis can model all non-linearities directly but is time consuming. The accuracy often depends on handling of a number of parameters such as time step, length and number of realizations. It is suitable for final design check, calibration of frequency domain approach, and some highly non-linear problems.
- Frequency domain coupled analysis is efficient but requires linearization of non-linear slender body properties, which include line stretching, geometry change, fluid load, and bottom effect. Industry experience indicates that properly formulated and calibrated frequency domain solutions can be accurate for most applications, especially for deepwater operations. It can meet both accuracy and efficiency requirements. The industry trend is increasing use of frequency domain coupled analysis.
- Most software developments are focused on time domain coupled analysis software. The industry needs also reliable, user friendly, and affordable frequency domain coupled analysis software.
- All coupled analysis software, time domain or frequency domain, needs verification and calibration. Because full scale measurement data are not available, the DeepStar model test data for spar, TLP, and FPSO and similar industry test data provide the best basis for bench marking. Software developers should fully use

this resource. Field measurement program to obtain full scale global response data should be considered in the future.

### I.7.4 EFFECT OF WATER DEPTH ON DYNAMIC OFFSET AND TENSION

Most of the offshore industry experience was accumulated from relatively shallow water (below 3,000 ft) operations. Only in the last few years floating production operations have moved to the deepwater region (up to 6,000 ft), but there is no permanent units installed in the ultra deepwater region (up to 10,000 ft). There is a concern that we may face higher uncertainty in global response analysis when we move to deeper and deeper water where the experience base is small and model testing is difficult. To address this concern, the parametric study examined the effect of water depth to global response. The study found that the dynamic offset and tension (max minus mean) in terms of total offset and tension decreases with increasing water depth for all floaters studied (Figure I.1, I.2). Since most uncertainties in global analysis are related to dynamic response predictions, this trend implies the design uncertainties tends to decrease as water depth increases. The dynamic mooring line tension in terms of the total tension for FPSO appears to be less sensitive to change in water depth than other structural types (Figure I.2).

### I.7.5 EFFECT OF WATER DEPTH ON LOW FREQUENCY NATURAL PERIOD

Natural period of a floating system is an important parameter since it affects the number of low frequency cycles in a time domain simulation or model test record. In the past the industry typically used 3-hour duration, which yields approximately 1,000 wave frequency cycles and 100 low frequency cycles for shallow water systems. The 3-hour duration may result in significant under estimation of standard deviation and maximum low frequency response when we move into deeper and deeper water because the number of low frequency cycles will be much less due to longer natural periods of deepwater systems. As show in Figures I.3 and I.4, translational natural periods increase with increasing water depth for all types of floaters studied, and some floaters may have natural periods exceeding 400 sec. in 10,000 ft water depth. The low frequency yaw natural period is not an important parameter except for FPSO where the low frequency sway-yaw natural period can be over 1000 second (Figure I.5). The traditional approach of using 3-hour duration for time domain simulation or model testing will only provide 11 to 27 low frequency cycles for natural periods of 400 to 1000 sec., which will not be adequate to capture the standard deviation and maximum responses. For the specific floaters studied, FPSO has the longest surge natural period whereas the semi-submersible the shortest.

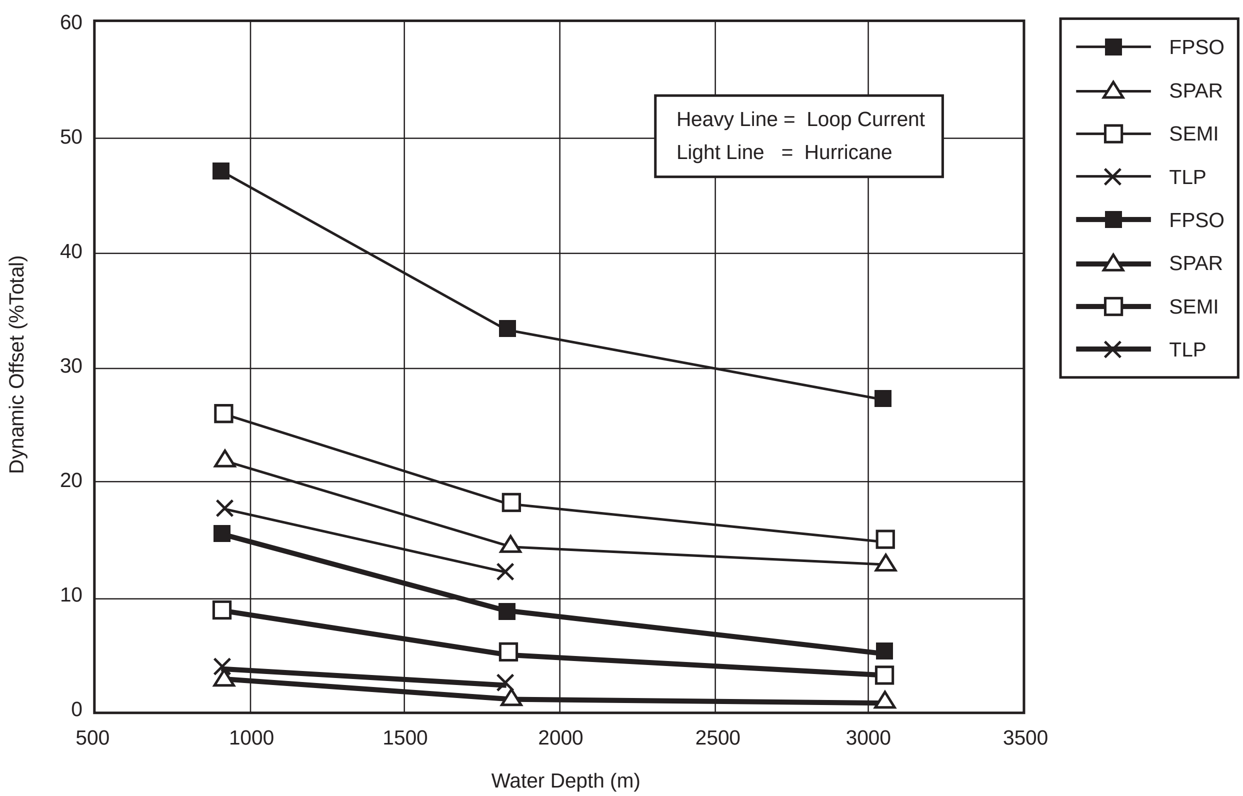



Figure I.1—Dynamic Offset in Terms of Total Offset

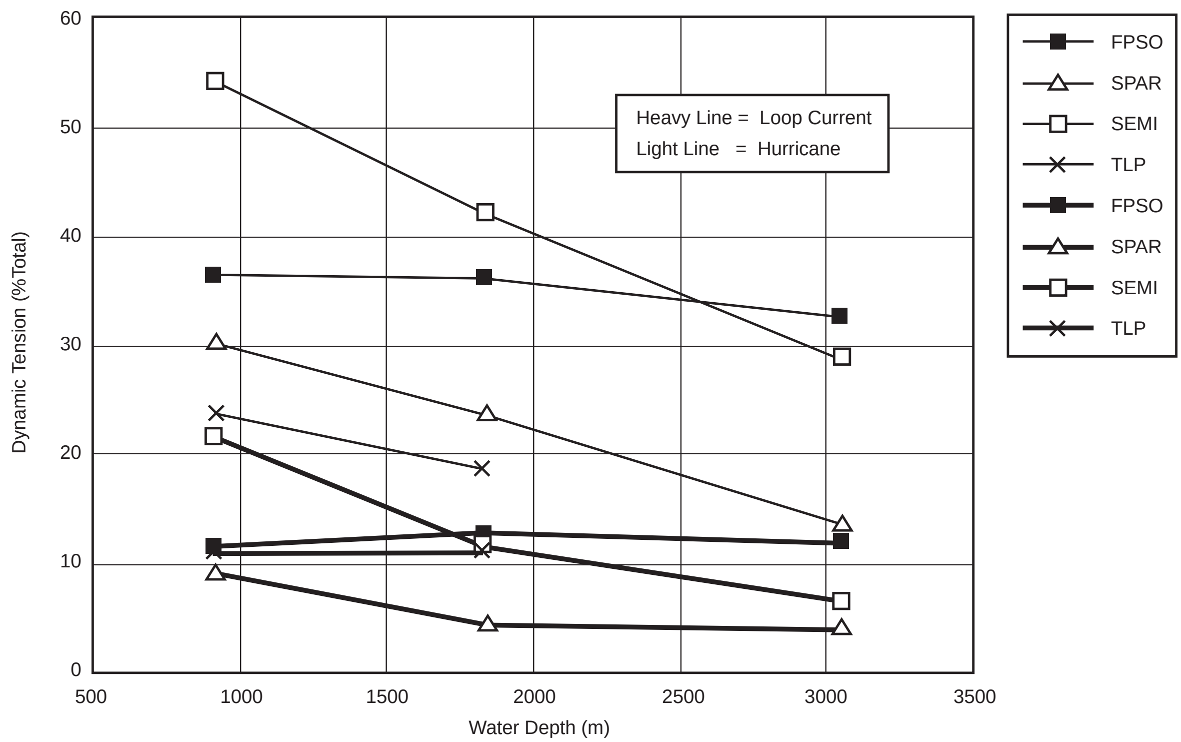



Figure I.2—Dynamic Tension for Most Loaded Mooring Line or Tendon

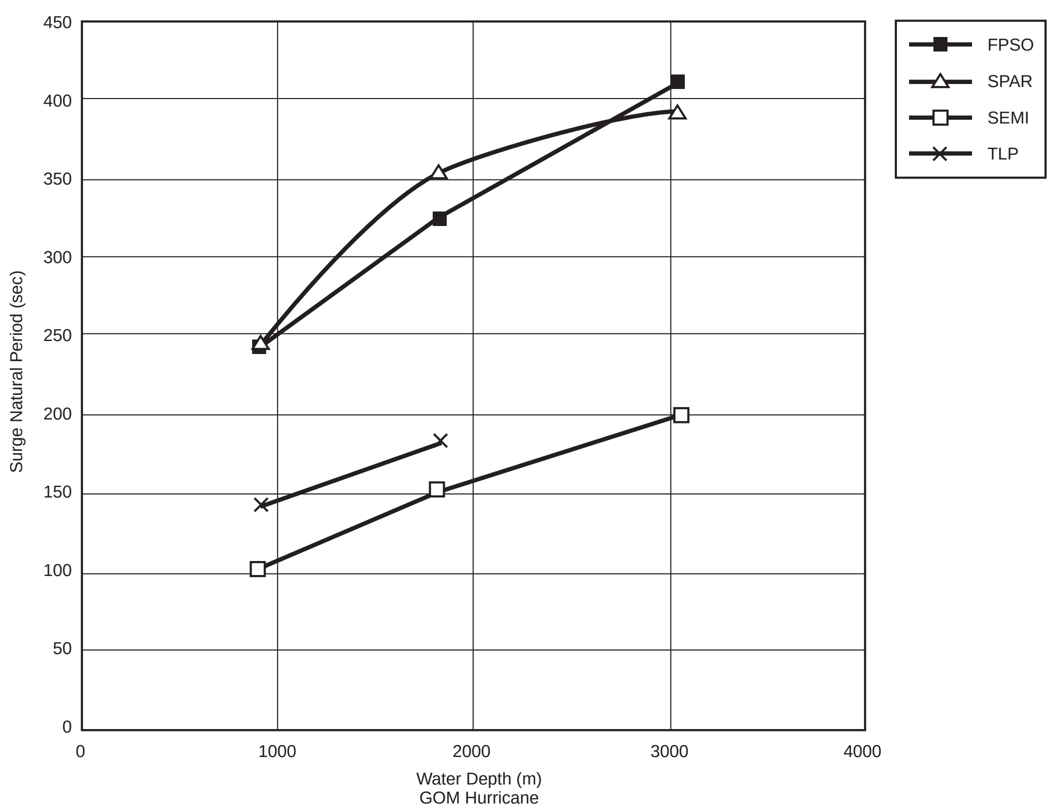



Figure I.3—Surge Natural Period Increases with Water Depth—Hurricane

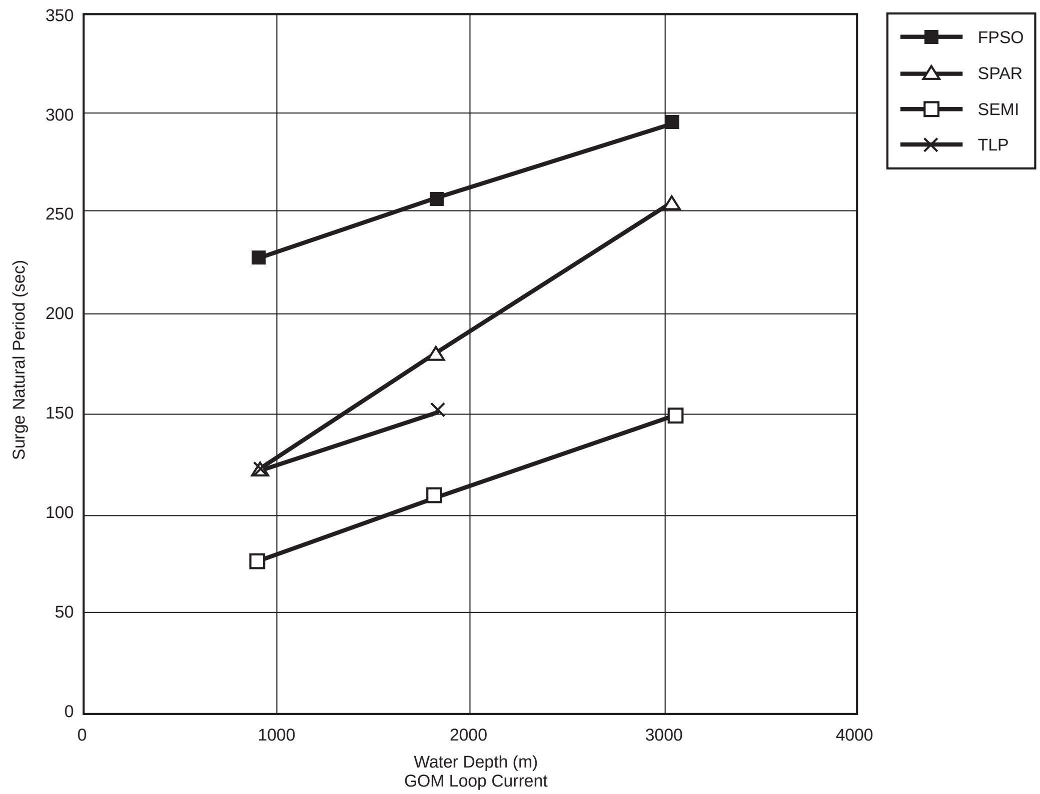



Figure I.4—Surge Natural Period Increases with Water Depth—Loop Current

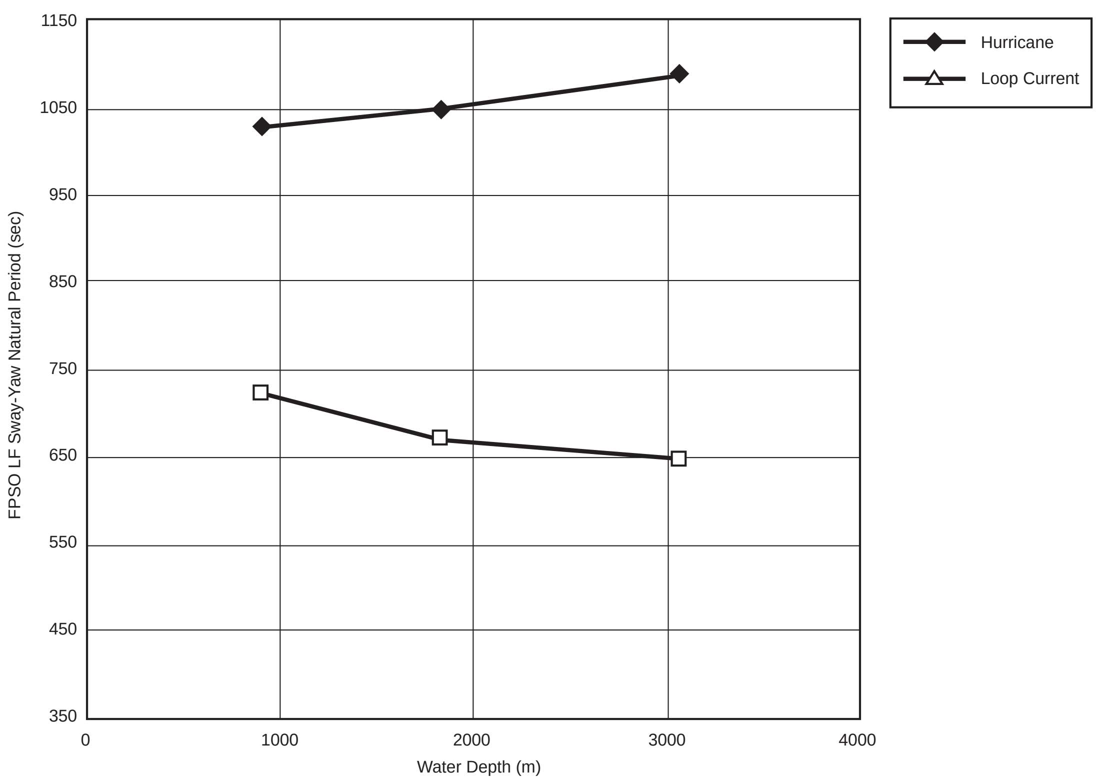



Figure I.5—FPSO Low Frequency Sway-Yaw Mode Natural Period

### I.7.6 EFFECT OF WATER DEPTH AND FLOATER TYPE ON DAMPING

Evaluation of low frequency damping is a difficult task for global response analysis and commercial software often depends heavily on analyst's experience and judgment to provide a damping value. The parametric study included an investigation on how different sources contribute to the low frequency damping and how damping changes with water depth and type of offshore structures. The results for the hurricane condition, where damping is important because of its high dynamic responses, are presented in Figure I.6 and I.7. The results indicate the following trends:

- For all four types of floaters, total damping increases with increasing water depth, primarily due to the increase in current drag on slender bodies.
- semi-submersible is the most heavily damped, with its damping level close to or above the critical damping. Spar is the least damped, with total damping in the range of 20% to 40% of critical.
- Slender bodies such as mooring lines and catenary risers provide significant damping as a result of line motions. An exception is TLP where the vertical tendons and risers contribute little damping. This is also true for spar's vertical risers.
- Hull viscous damping due to current drag is an important source of damping except for FPSO where the current drag is low.

### I.7.7 EFFECT OF NUMBER OF SLENDER BODIES ON DAMPING AND CURRENT LOAD

As shown in Figure I.8, damping increases with increasing number of risers for FPSO, SEMI and TLP. However, damping for SPAR is not affected by the number of risers because its vertical self standing risers contribute little damping. Most spar floating production units were designed using the semi-coupled time domain approach, which neglects mooring and riser damping. To evaluate the accuracy of this approach, analysis was performed for the cases with and without line

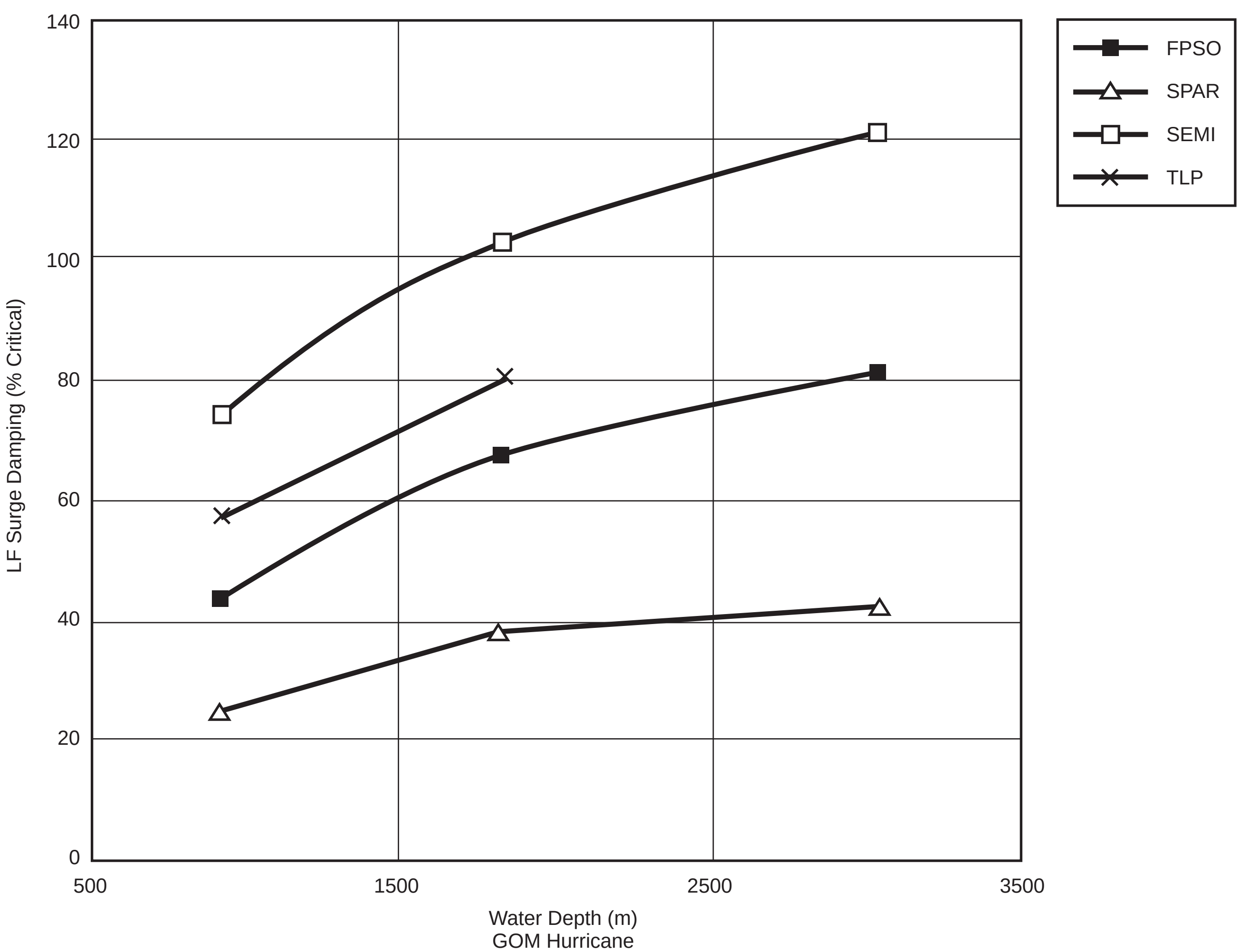



Figure I.6—Low Frequency Surge Damping—Hurricane

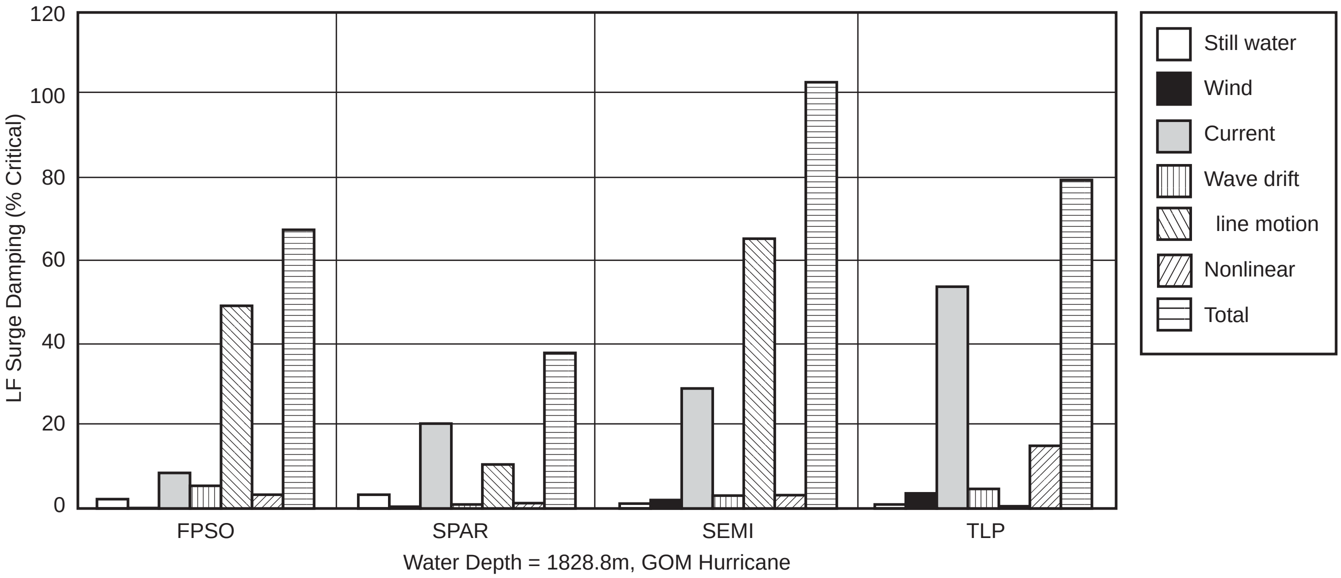



Figure I.7—Low Frequency Surge Damping Contributions in Hurricane

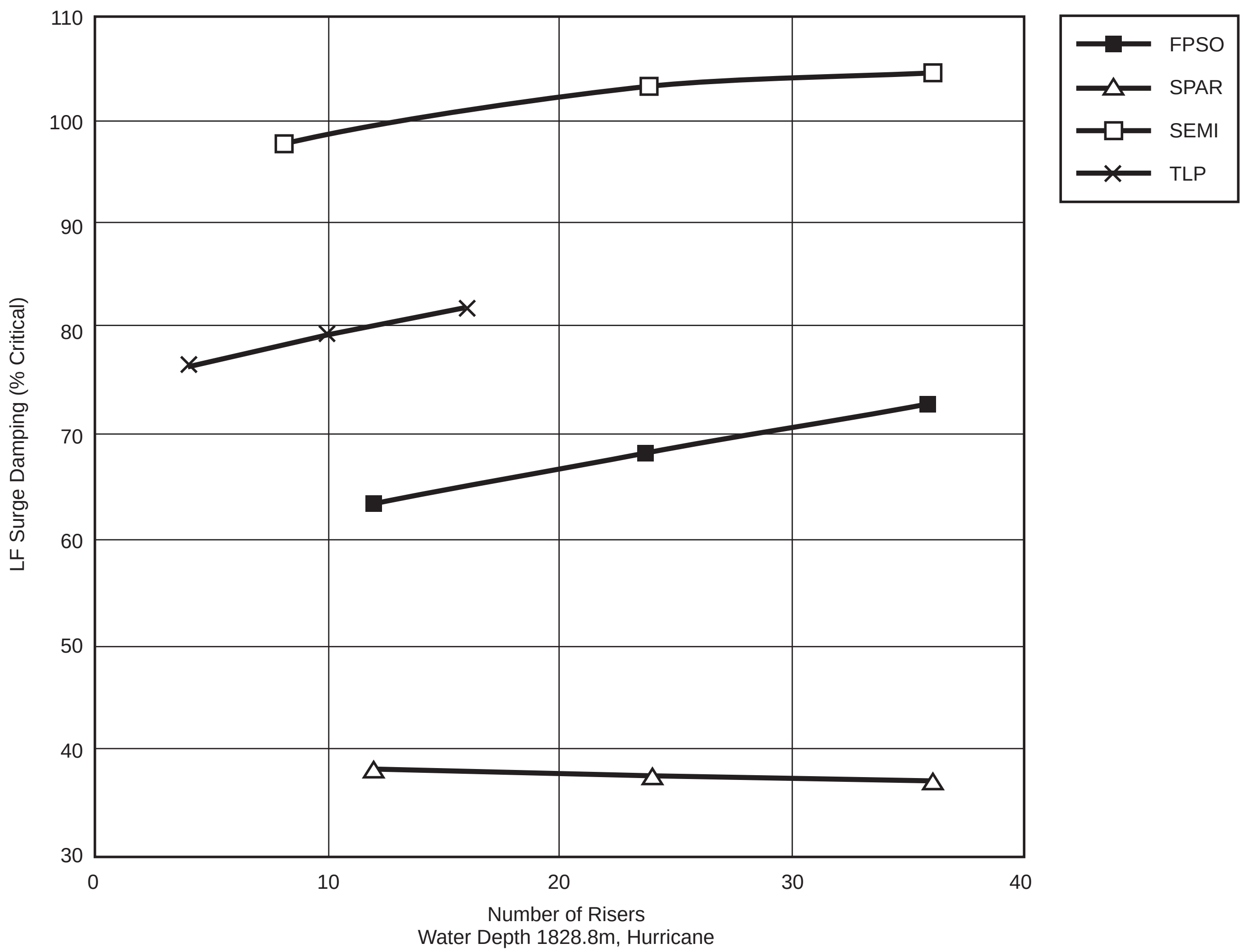



Figure I.8—Effect of Risers on Low Frequency Surge Damping

damping. As shown in Figure I.9, the differences in dynamic offset are small for all water depths and for both hurricane and loop current conditions. Similar trend was observed for dynamic line tensions. This indicates that current spar analysis approach should yield good approximations for spar design, assuming vertical self standing risers are used. This may not be true if catenary risers are used. However, neglecting line damping is always conservative.

The parametric study also investigated the impact of number of risers on slender body current load. As expected, current load increases with increasing number of risers, as shown in Figure I.10. The impact is highest for FPSO and the lowest for spar.

### I.7.8 EFFECT OF SLENDER BODY DRAG COEFFICIENT

As expected, the current drag coefficient of the slender bodies has a noticeable effect on global response under the loop current condition (Figures I.11 and I.12). Increasing current drag coefficient of slender bodies will generally increase vessel offset and line tension due to the increase of current load from the slender bodies. Change of drag coefficient has no significant impact on offset and line tension under the hurricane environment.

### I.7.9 EFFECT OF FLOATER TYPE ON ENVIRONMENTAL LOADS

Different types of structures have different responses to wind, wave, and current excitations. The dynamic responses are illustrated in I.13 and I.14, where the low frequency wind, high and low frequency wave components are presented for the SD (standard deviation) dynamic vessel offset. For the hurricane environment, low frequency wind is dominating for all floating vessels except for FPSO, which is dominated by low frequency wave. For the loop current environment, all three components have significant contribution except for FPSO, which is again dominated by low frequency wave components.

Figure I.15 shows that the dynamic mooring line tension of the most loaded line is heavily dominated by wave frequency component in the Hurricane condition for all floaters. In the loop current condition, the dynamic line tension is almost equally contributed by both low frequency and wave frequency motions of all floaters except for TLP. Since its heave natural period is near the wave period, TLP is highly impacted by wave frequency motions in both environments.

Contributions to the mean environmental load are also investigated. In the Hurricane environment (Figure I.16), wind dominates all floating structures except for FPSO, which is dominated by mean wave drift force. It is apparent that FPSO is dominated by waves in both mean and dynamic responses. In the Loop Current condition (Figure I.17), current load (hull and slender bodies) demonstrates its superior dominance over the total mean environmental load across all types of floaters. The turret moored FPSO has the highest line drag load from current due to the large number of mooring line and catenary risers. SPAR has the highest current load on the hull.

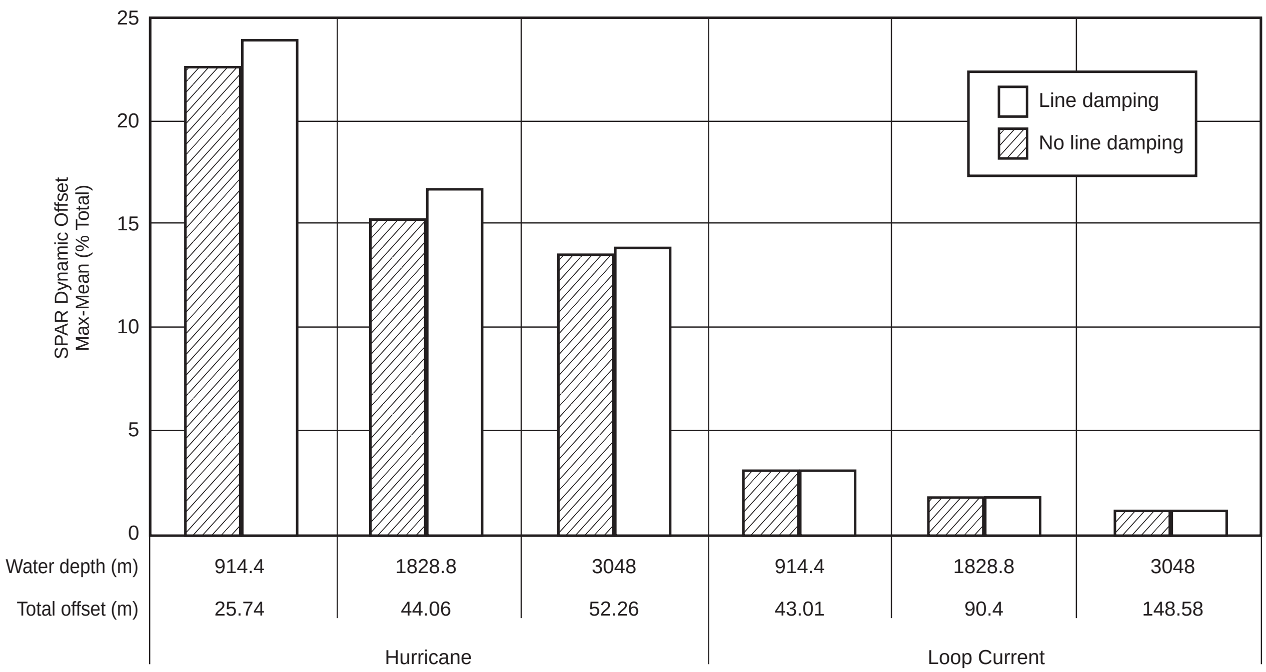



Figure I.9—Effect of Neglecting Line Damping for SPAR

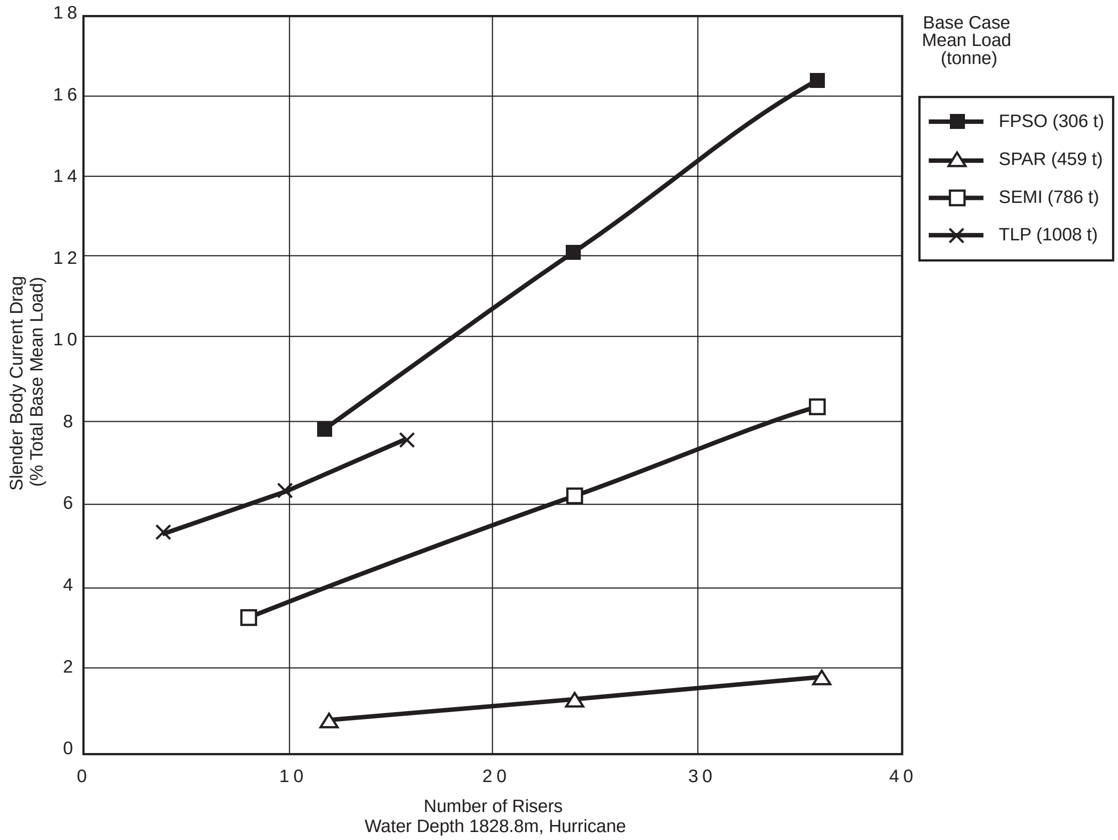



Figure I.10—Effect of Risers on Current Load from Slender Bodies

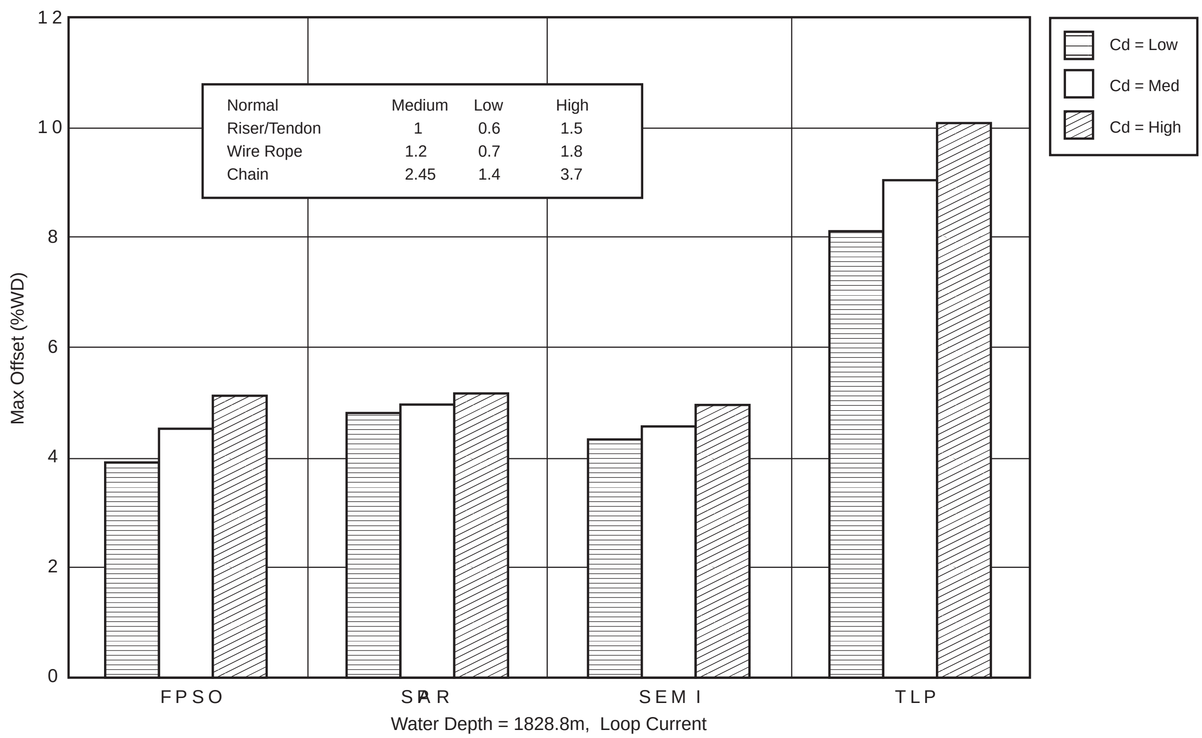



Figure I.11—Effect of Current Drag Coefficient on Vessel Offset—Loop Current

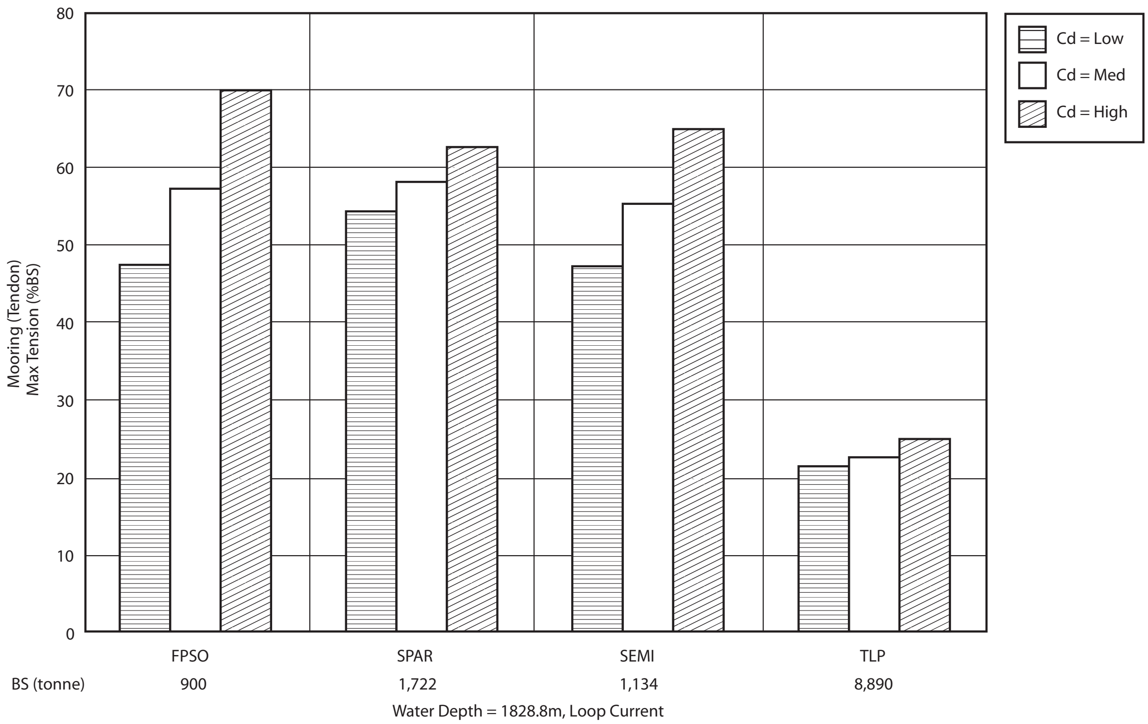
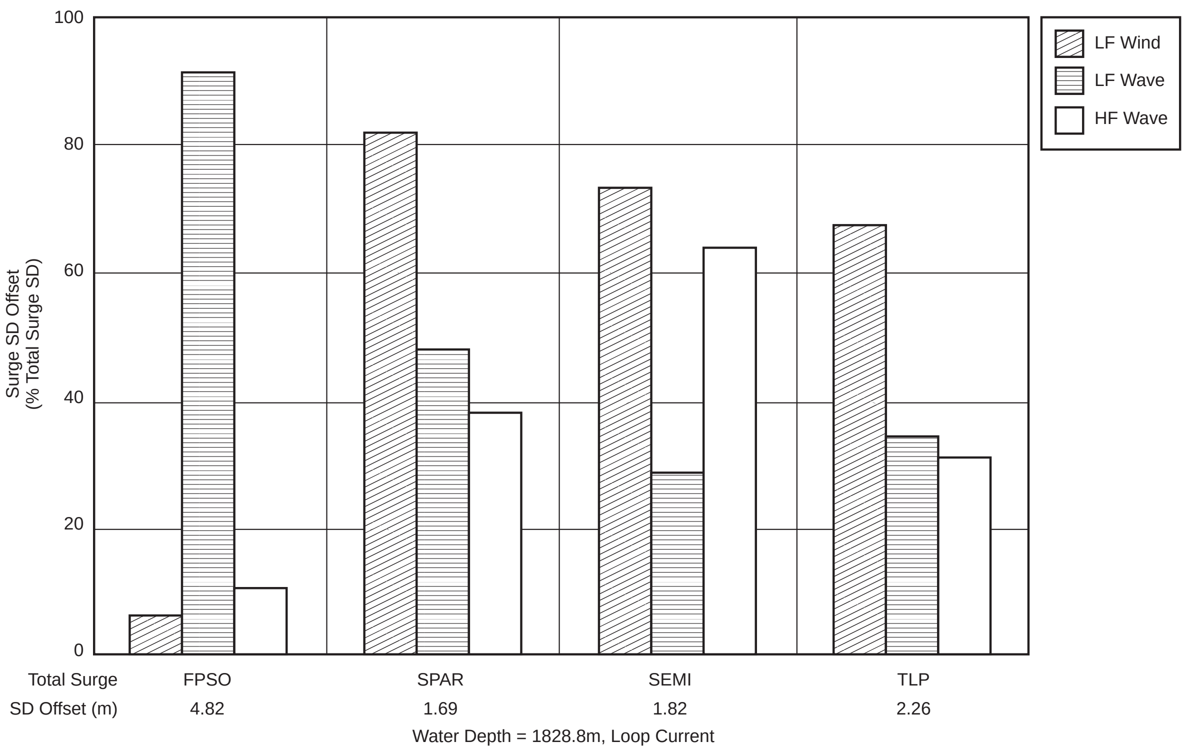
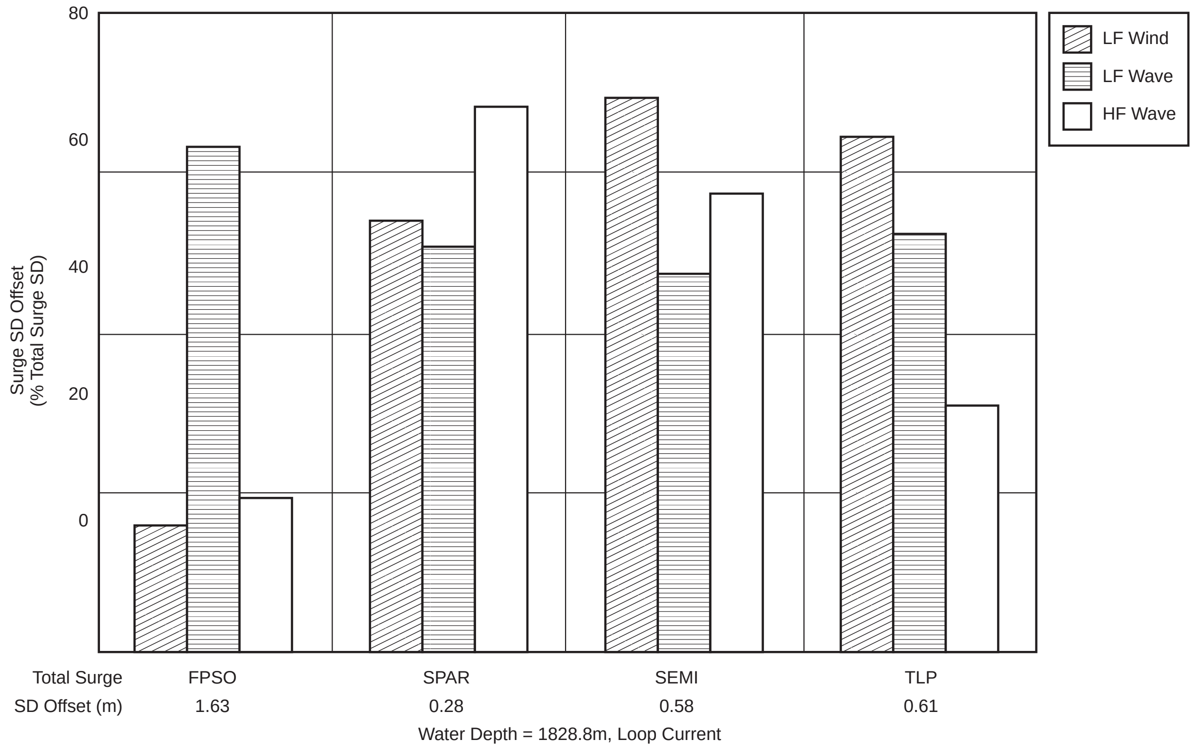



Figure I.12—Effect of Current Drag Coefficient on Line Tension—Loop Current



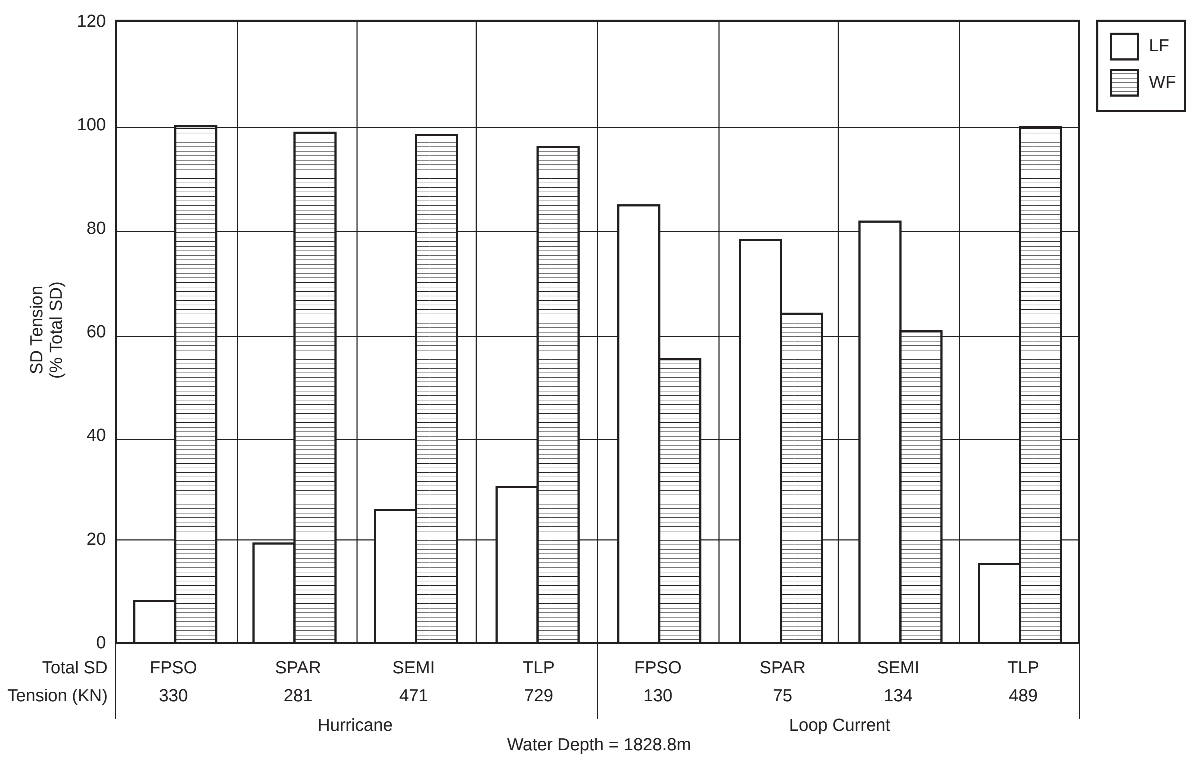

*The total percentage may not equal to 100 due to phase difference.

Figure I.13—Dynamic Offset Contributions in OM Hurricane



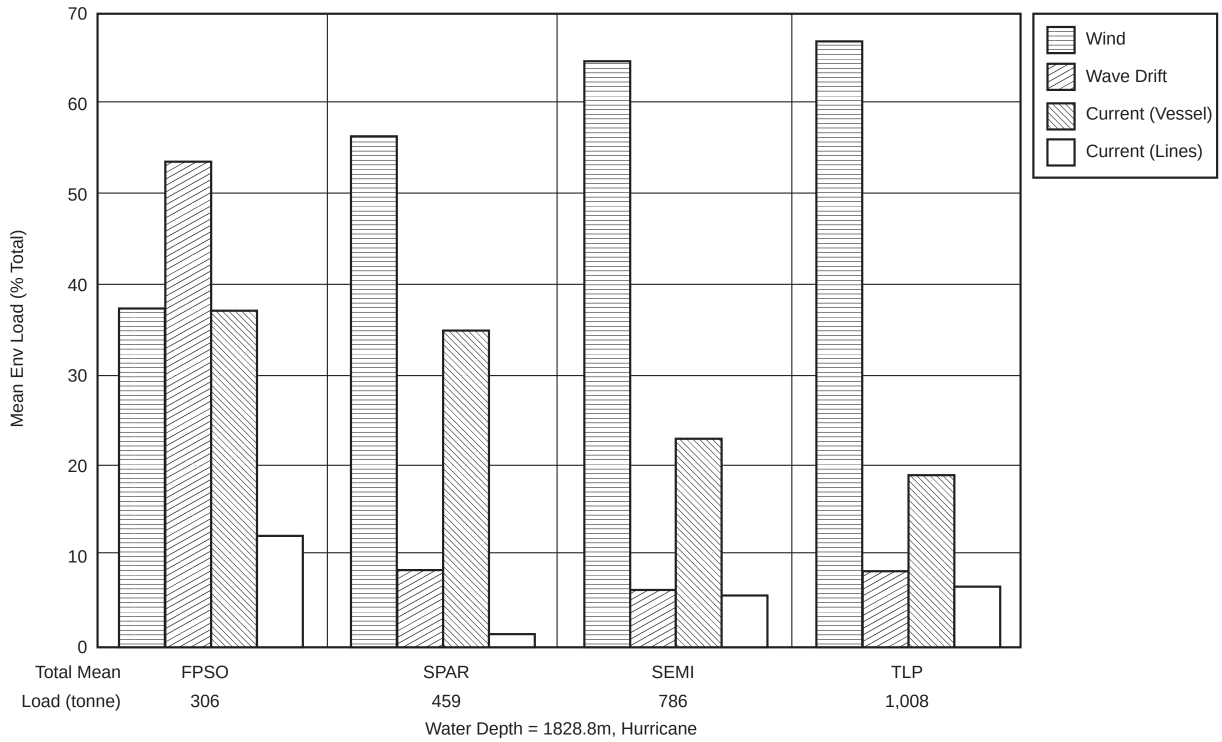

*The total percentage may not equal to 100 due to phase difference.

Figure I.14—Dynamic Offset Contributions in GOM Loop Current



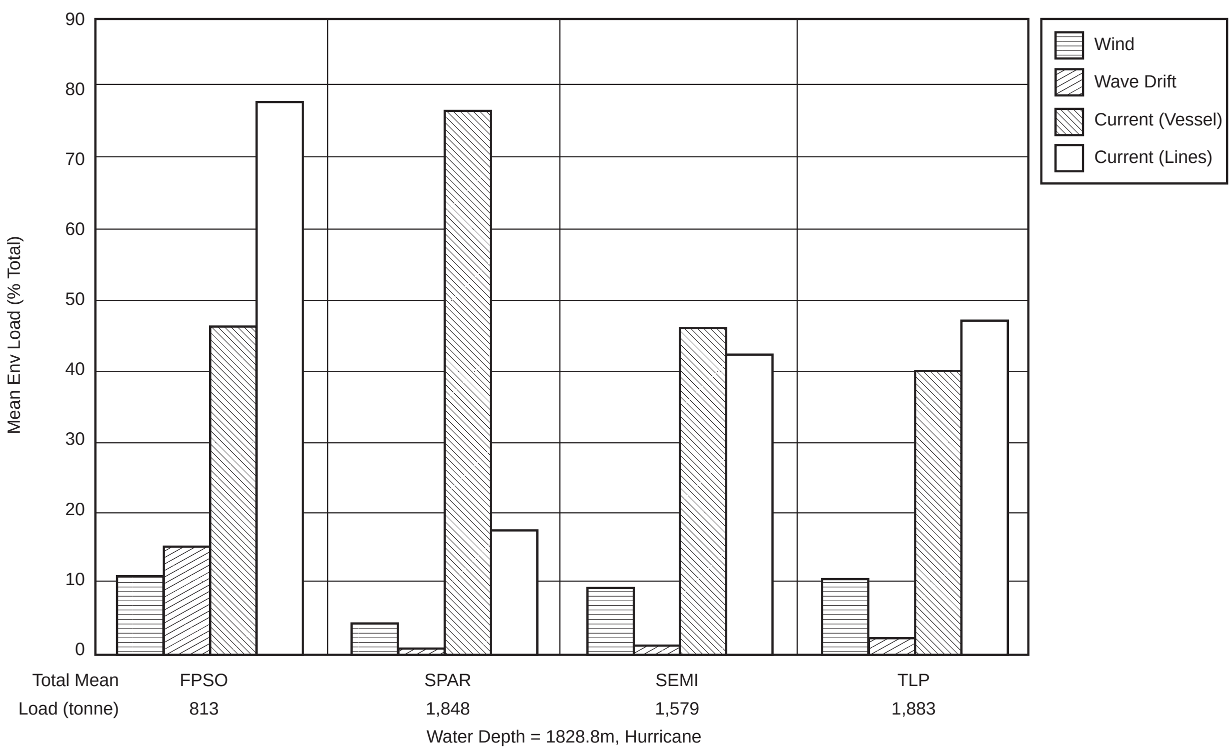

*The total percentage may not equal to 100 due to phase difference.

Figure I.15—Dynamic Tension Contributions for the Most Loaded Line




*The total percentage may not equal to 100 for FPSO due to its non-colinear environmental headings.

Figure I.16—Mean Environmental Loads in GOM Hurricane




*The total percentage may not equal to 100 for FPSO due to its non-colinear environmental headings.

Figure I.17—Mean Environmental Loads in GOM Loop Current

### I.7.10 SENSITIVITY OF WIND SPECTRUM

As discussed in Appendix B, currently the API and NPD spectrum are commonly used by the offshore industry. The API spectrum, which was published in earlier editions of API RP 2A, has much smaller empirical data base than the NPD spectrum. The uncertainty of the API spectrum is addressed through specifying a range instead of a single value for the dimensionless peak frequency. This results in a spectrum defined by upper and lower bound values. In the latest edition of API RP 2A, the API spectrum was replaced by the NPD spectrum, which was also specified by the draft ISO standard. As illustrated in Figure I.18, the NPD curve was fitted to the data with lower periods and extrapolated to higher periods, resulting in potential over estimation of wind energy as the period becomes longer. Since low frequency translational natural periods tend to increase with increasing water depth, a question was raised whether NPD spectrum would be too conservative for ultra deepwater operations.

To answer this question, three spectra (i.e., API lower bound, API upper bound and NPD wind spectra) were investigated for the wind dominating hurricane environment for 4 structures and 3 water depths. The SD (standard deviation) vessel offsets due to wind only are plotted in Figure I.19, and the total SD vessel offsets due to wind/wave are plotted in Figure I.20. These plots show the following trends:

- Response to the NPD spectrum is always higher than the response to the API upper bound spectrum. The difference tends to increase with increasing water depth and natural period.
- The trends are similar for the wind only and the wind/wave cases except for the FPSO, which shows almost no difference between the NPD, API upper and lower bound spectrum when the total offsets due to wind/wave are plotted. This is due to the fact that FPSO is dominated by wave dynamics, and wind dynamics is insignificant.
- Since the API spectrum provides only bounds, the response prediction often depends on the judgment of the analyst. Many analysts select a middle value between the upper and the lower bound, this may significantly under estimate the response.
- Since the natural periods are in a range of 100–400 seconds for all 4 structures in 3 water depths, the over estimation by the NPD spectrum appears not too serious, especially when the maximum response (mean plus maximum dynamic) is considered.

### I.7.11 EFFECT OF WAVE HEIGHT AND WAVE PERIOD

As shown in Figure I.21, increase in wave height always results in increase of dynamic response for all floaters.

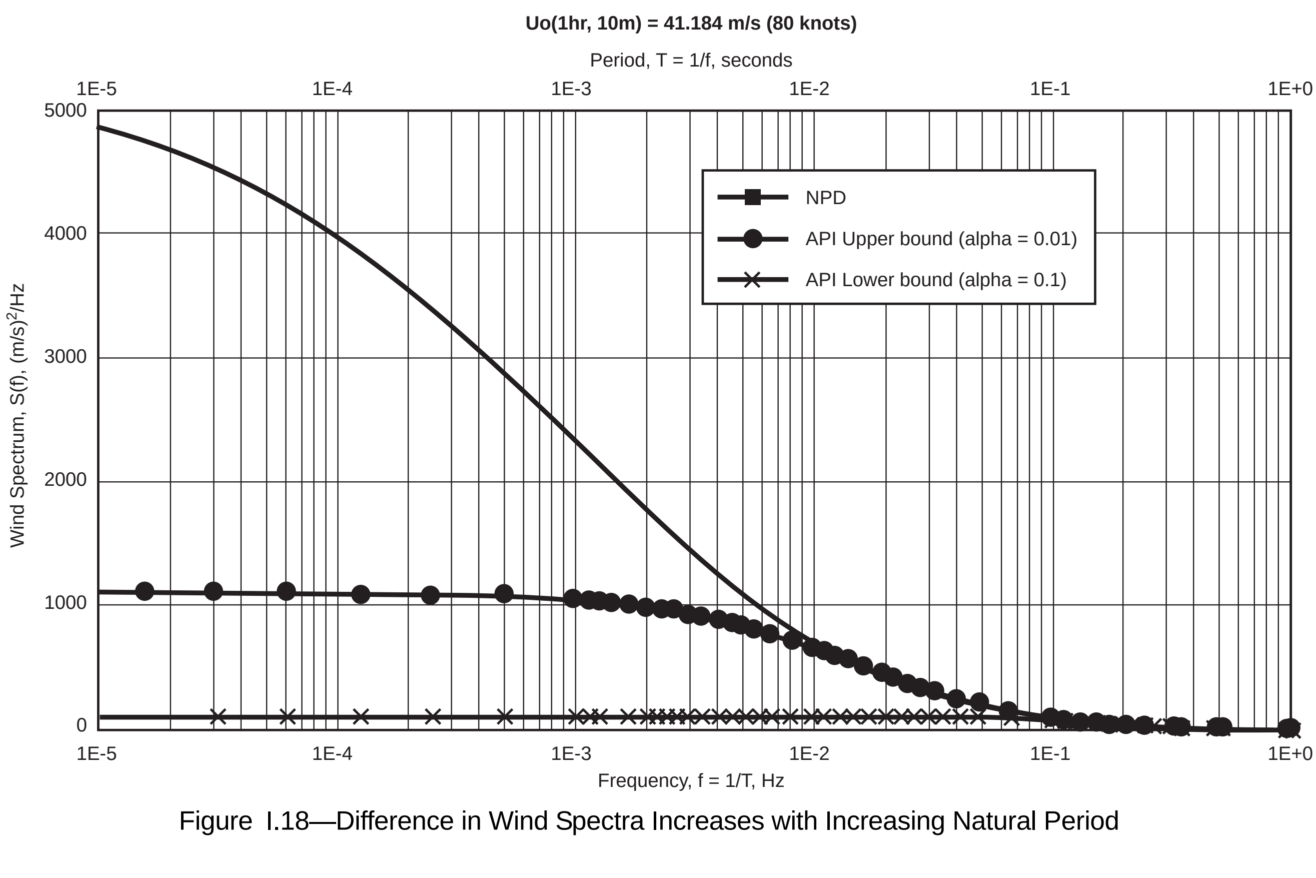



Figure I.18—Difference in Wind Spectra Increases with Increasing Natural Period

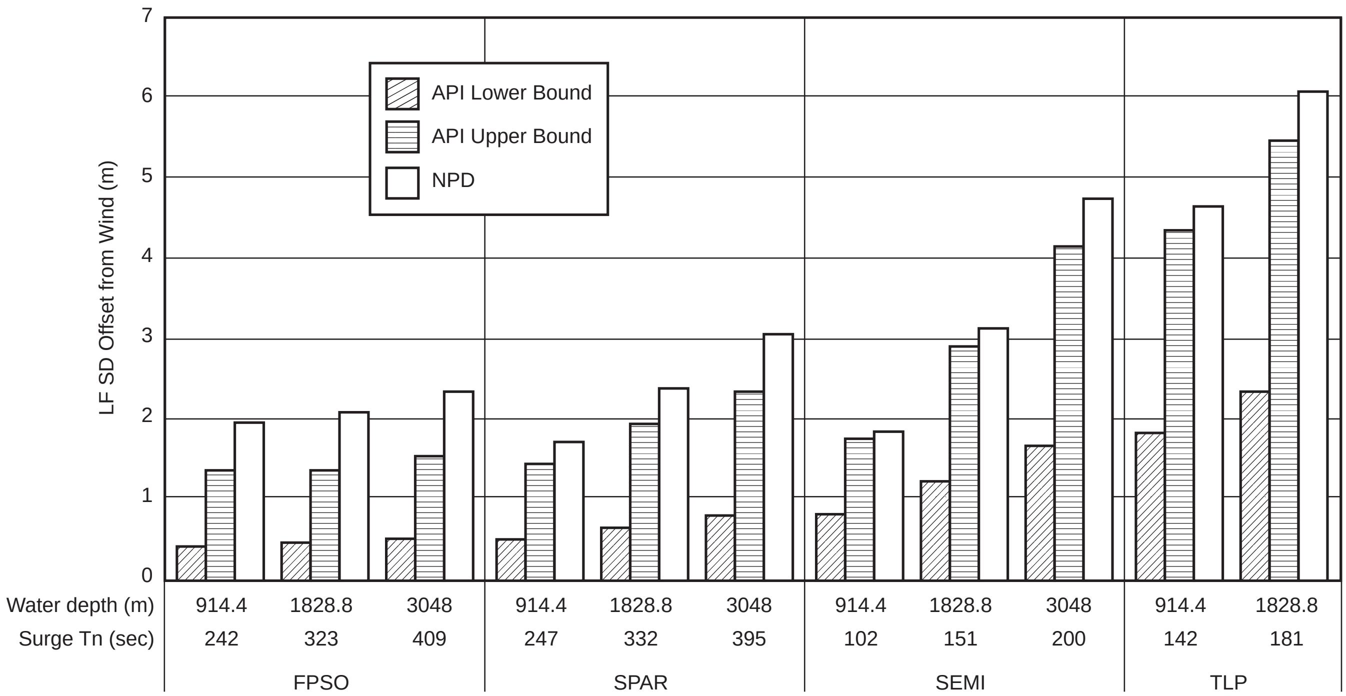



Figure I.19—Sensitivity of Wind Spectrum on SD Offset from Wind—Hurricane Condition

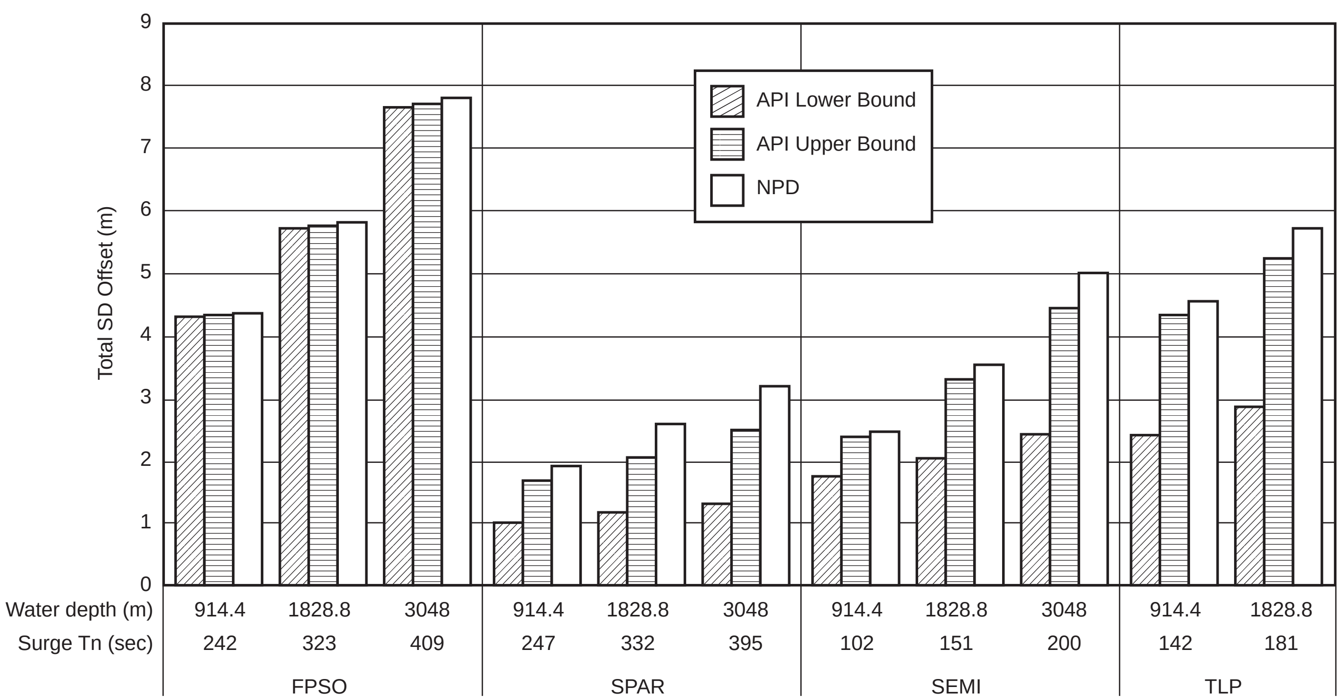



Figure I.20—Sensitivity of Wind Spectrum on Total SD Offset—Hurricane Condition

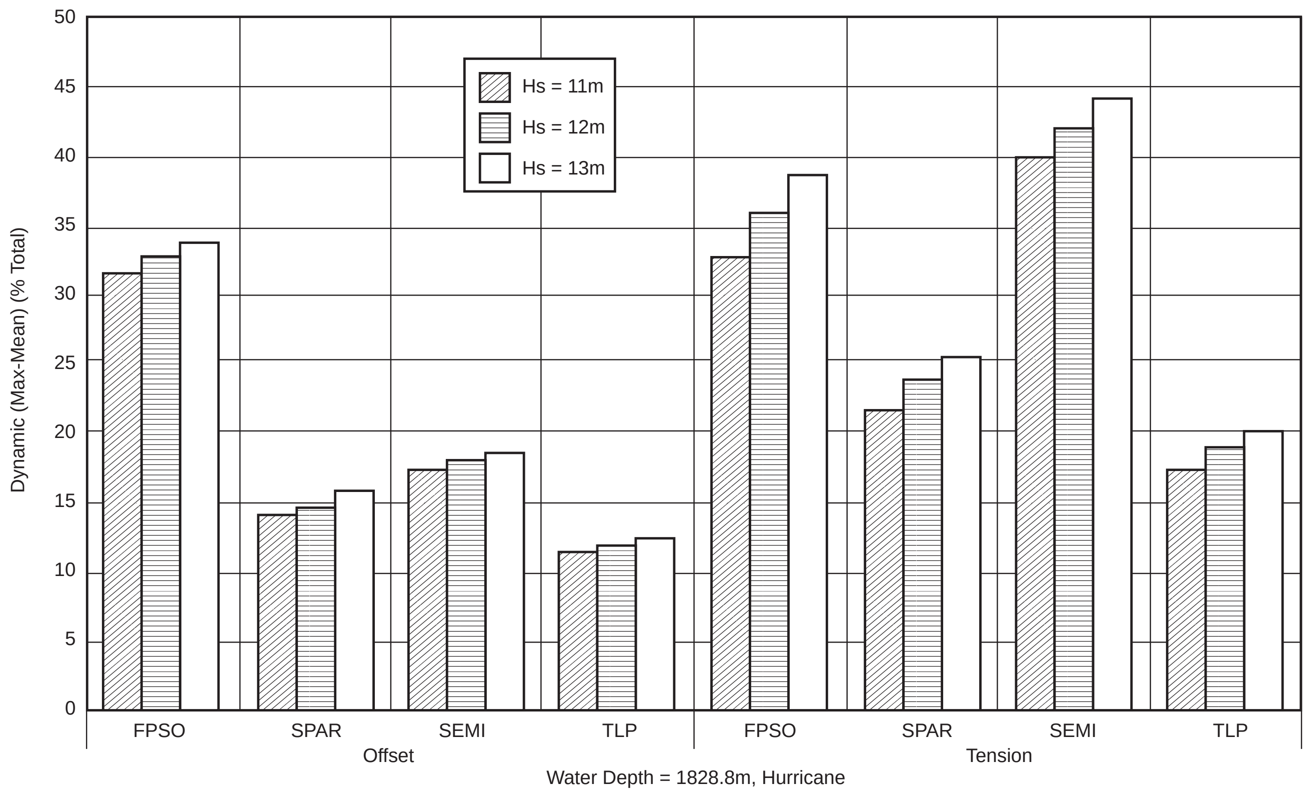



Figure I.21—Sensitivity of Wave Height for Hurricane Condition

Increase in wave peak period, however, does not show the same consistent trend. For example response of FPSO decreases with increasing wave peak period, as shown in Figure I.22.

### I.7.12 POLYESTER MOORING

The parametric study investigated also the impact of using polyester mooring instead of steel mooring. Study results indicate that in addition to generally lower maximum tension and offset, polyester mooring yields much lower wave frequency tension than steel mooring (Figure I.23). The much lower wave frequency tension may reduce fatigue damage by an order of magnitude. This can be a good choice for West Africa or GOM spar operations where fatigue due to swell or VIM can be a dominating design factor.

Another observation is polyester lines provide much lower damping than steel mooring lines because of their lack of catenary change under wave frequency motions (Figure I.24).

## I.8 References


**I.1** DeepStar IV Project, DeepStar CTR 4401B, *DeepStar Theme Structure Specifications*, Sept. 8, 1999.

**I.2** DeepStar V Project, CTR 5401-1, "Coupled Analysis Theme Structures Model Tests," MARIN, April 2003.

**I.3** DeepStar V Project, CTR 5401-2, "Guideline for Global Performance and Verification of DeepWater Structures," DNV, March 2003.

**I.4** DeepStar V Project, CTR 5401-3, "Global Performance of DeepWater Structures–Post Model Test Study," OTRC, April 2003.

**I.5** DeepStar V Project, CTR 5401-4, "Global Performance of DeepWater Structures–FPSO Analysis," MARIN, April 2003.

**I.6** DeepStar V Project, CTR 5401-5, "Global Performance of DeepWater Structures–FPSO Analysis–Marintek," April 2003.

**I.7** DeepStar V Project, CTR 5401-6, "Global Performance of DeepWater Structures–FPSO Analysis–FMC SOFEC," April 2003.

**I.8** DeepStar V Project, CTR 5401-7, "Global Performance of DeepWater Structures–FPSO Analysis–SBM IMODCO," April 2003.

**I.9** DeepStar V Project, CTR 5401-8, "Global Performance of DeepWater Structures–Spar Analysis–Technip Coflexip," April 2003.

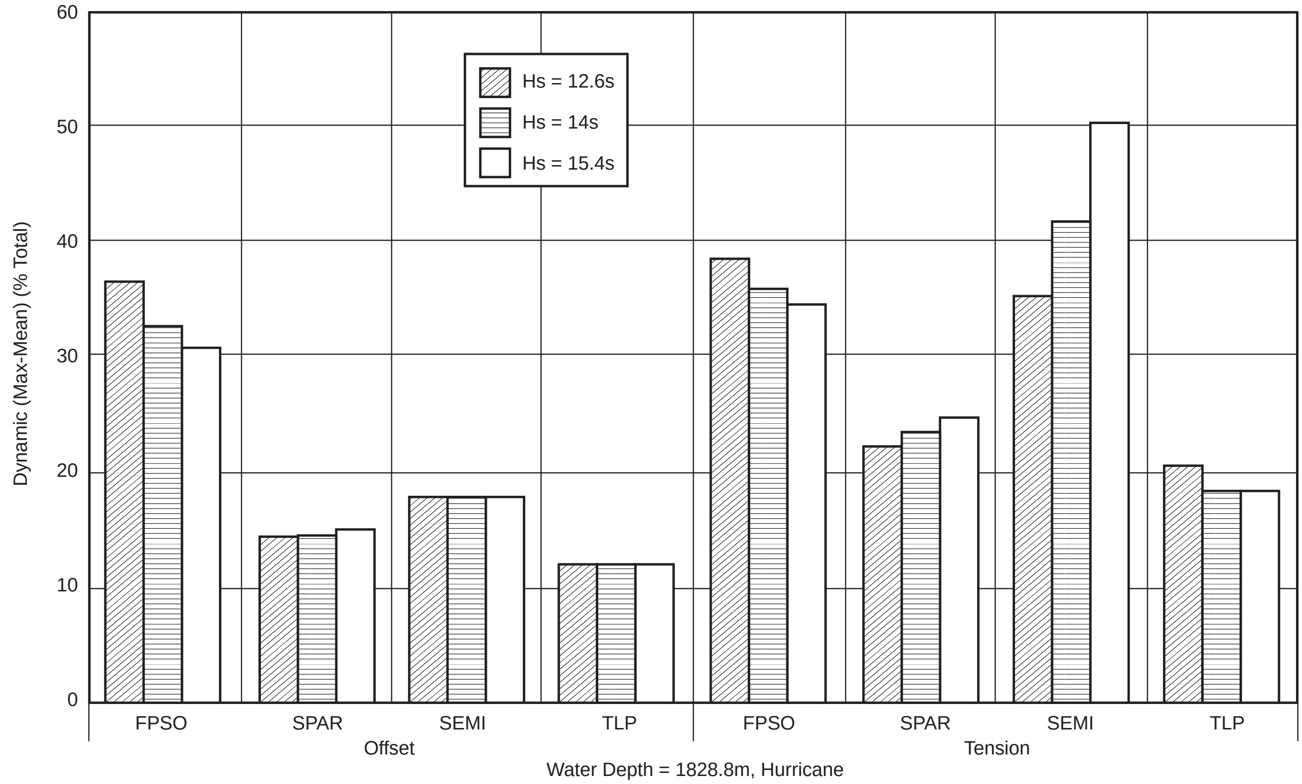



Figure I.22—Sensitivity of Wave Peak Period for Hurricane Condition

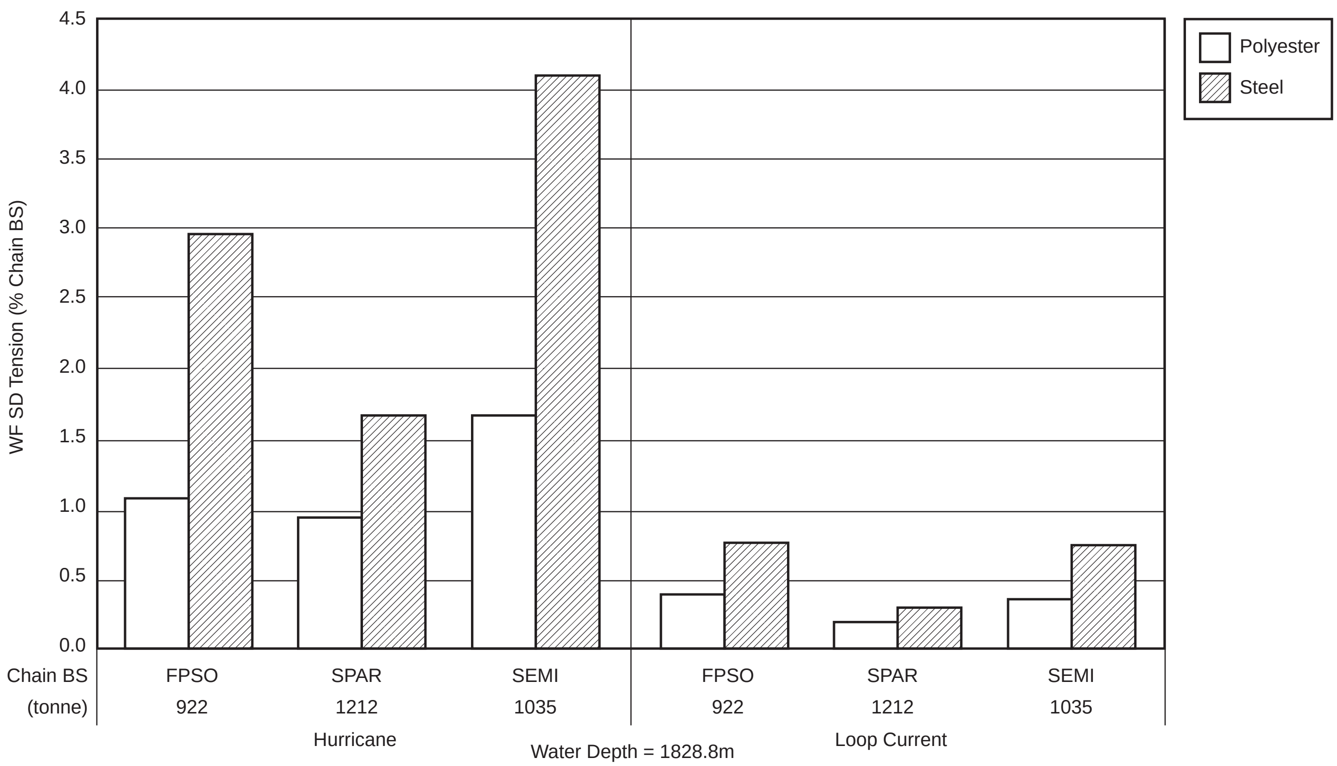



Figure I.23—Comparison of Wave Frequency Tensions for Steel and Polyester Mooring

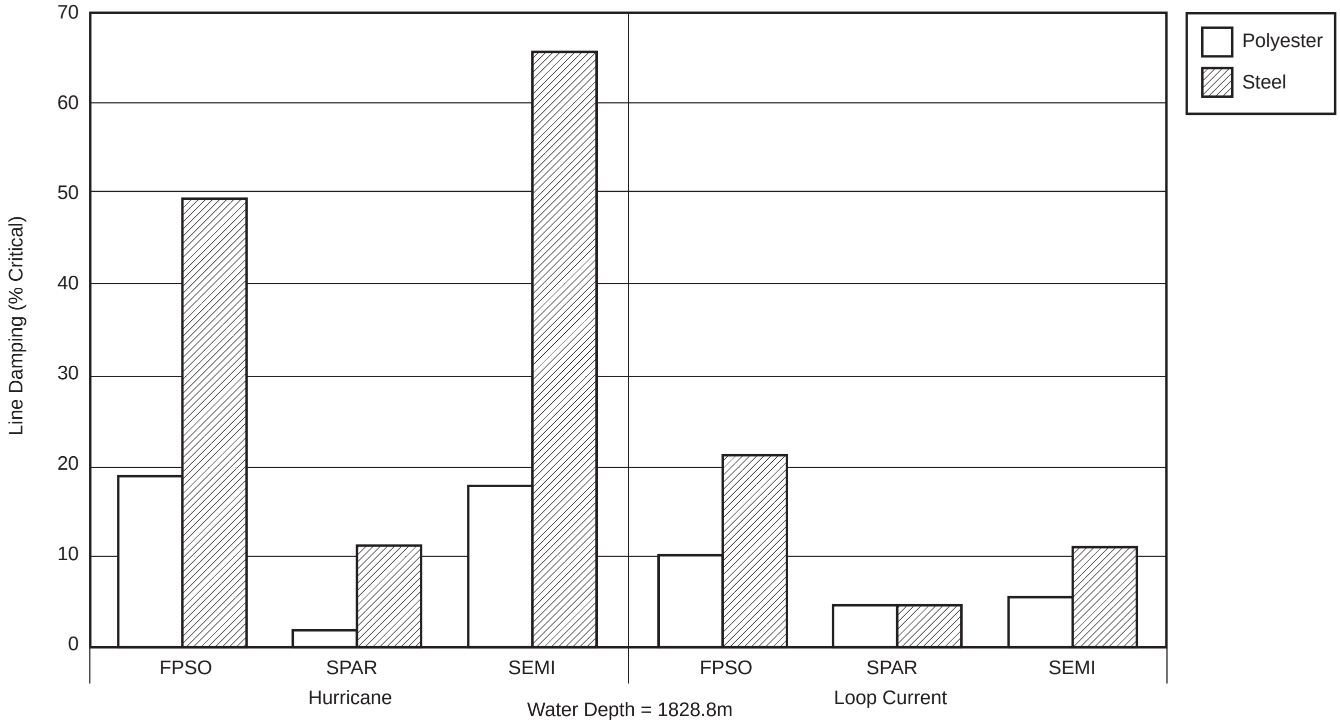



Figure I.24—Comparison of Line Damping for Steel and Polyester Mooring

**I.10** DeepStar V Project, CTR 5401-9, "Global Performance of DeepWater Structures–Spar Analysis–OTRC," April 2003.

**I.11** DeepStar V Project, CTR 5401-10, "Global Performance of DeepWater Structures–TLP Analysis–ABB," April 2003. DeepStar V Project, CTR 5401-11, "Global Performance of DeepWater Structures–TLP Analysis–Marintek," April 2003.

**I.12** DeepStar VI Project, CTR 6401A, "Guideline Analysis and Verification–Guideline Development," Noble Denton, July 2003.

**I.13** Garret, D.L., "Coupled Analysis of Floating Production Systems," Stress Engineering, 2003 International Symposium, Deepwater Mooring Systems: Concepts, Design, Analysis and Materials, Oct. 2-3, 2003, Houston, Texas.

**I.14** Garret, D.L., et al, "Global Performance of Floating Production Systems, Stress Engineering," OTC 14230, Offshore Technology Conference, 2002, Houston, Texas.

**I.15** Garret, et al, "Mooring-And-Riser-Induced Damping in Fatigue Seastates, Stress Engineering," OMAE2002-28550, Proceedings of OMAE'02, 21st International Conference on Offshore Mechanics and Arctic Engineering, June 2002, Oslo, Norway.

**I.16** Garret, et al, "Integrated Design of Risers and Moorings," Stress Engineering, 2003 International Symposium, Deepwater Mooring Systems: Concepts, Design, Analysis and Materials, Oct. 2-3, 2003, Houston, Texas.

## APPENDIX J—MOORING STRENGTH AND FATIGUE ANALYSIS EXAMPLES

## J.1 Strength Analysis Example

The following problem illustrates the procedures for the analysis of a mooring system using three analysis methods:

a. Quasi-static analysis.

b. Time domain dynamic analysis.

c. Frequency domain dynamic analysis.

The example is intended to illustrate the principal steps in the analysis and to give guidance particularly on the dynamic analysis procedure. The method described here is not unique and it may be necessary to modify some of the steps in the procedure to accommodate specific software.

### J.1.1 MOORING SYSTEM DESCRIPTION

The system to be analyzed is:

a. A semi-submersible with a 10 point, 36° symmetric pattern, as shown in Figure J.1.

b. $3^{1}/_{2}$ in. K4 chain 5.100 ft outboard, break test load = 1838 kips.

c. Initial tension = 280 kips.

d. Mooring line description.

1. Diameter = $3^{1}/_{2}$ in
2. Elastic stretch (AE) = 123.4 lbs/ft
3. Weight in air = 107.2 lbs/ft
4. Friction coefficient = 1.0 (with mudline)
5. Break test load = 1,838 kips
6. Line mass = 3.84 slugs/ft
7. Tangential added mass = 0.25 slugs/ft
8. Normal added mass = 0.51 slugs/ft
9. Drag coefficient = 1.2
10. Drag diameter = 7 in. (2 5 nominal diameter)

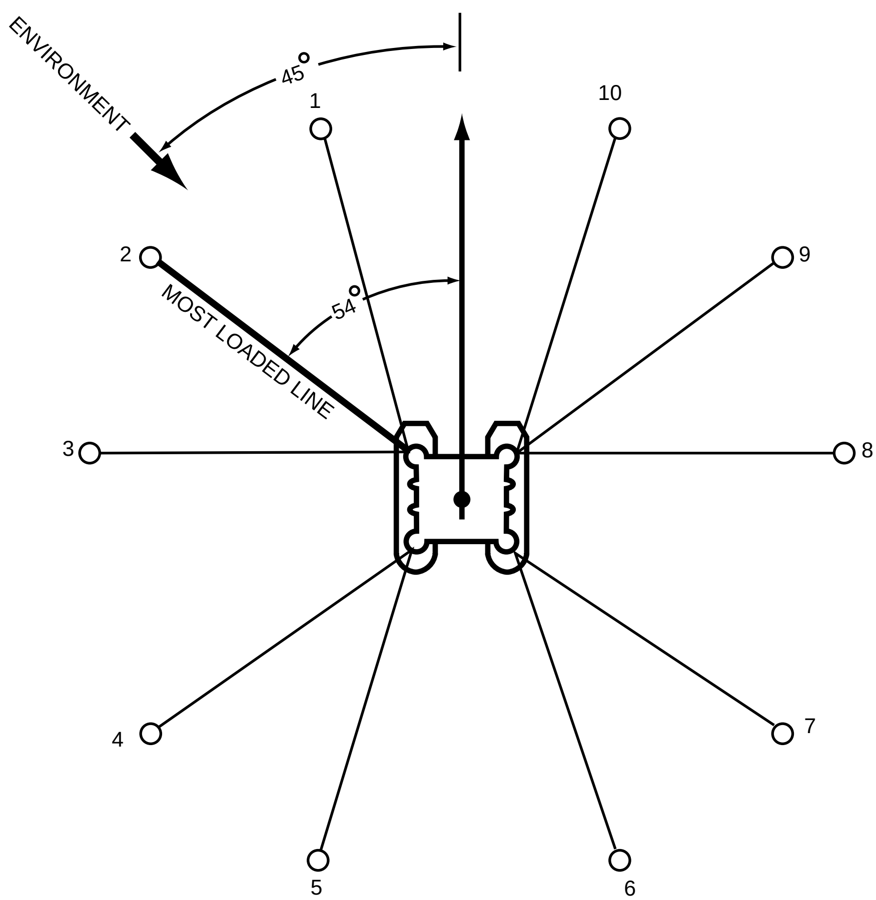



Figure J.1—Mooring Configuration

The mooring properties for chain include provision for a tangential added mass. For wire or synthetic ropes, no tangential added mass is required. The nominal drag diameter is increased by a factor of 2 for chain. For wire rope or synthetic rope this is not required

### J.1.2 THE ENVIRONMENT

The design example environment is as follows:

a. Water depth = 1,233 ft

b. Significant wave height = 55.8 ft

c. Peak spectral period = 17.49 sec

d. JONSWAP spectrum with a peakedness factor 3.3

e. Wind velocity (1 min.) = 100 kt

f. Surface current velocity = 3.1 kt

g. Quartering direction (See Figure J.1)

h. Design storm duration = 3 hours

i. Wind, wave, current colinear

Wind loading on the vessel can be accounted for in two different procedures:

a. Wind load is considered to be applied statically to the vessel. In this procedure the one minute average wind speed is applied on the vessel as a static load.

b. The dynamic effects of wind are considered by combining a steady wind force with a fluctuating wind component. The one-hour average wind speed is applied statically to the vessel. A wind gust velocity spectrum is defined.

The approach described in item a has been used in the present example.

### J.1.3 MEAN LOAD COMPUTATION

The mean loads can be derived from model test data or computed. The computed mean loads on the vessel are as follows:

| | |
|---|---|
| Wind (1 minute average) | 680 kips |
| Current | 378 kips |
| Steady wave drift | 70 kips |
| Total mean load | 1,128 kips |

### J.1.4 VESSEL/FAIRLEAD MOTIONS

Both wave frequency and low-frequency motions are required in all analyses. In general the response amplitude operators, and phases, in the three linear directions (heave, surge, sway) and the three angular directions (roll, pitch, yaw) must be derived. The derivation of these data requires hydrodynamic computer programs or model test data. Any suitable reference point, usually the vessel's center of gravity, can be used to define the motions.

The low-frequency motions can be computed from hydrodynamic computer programs, model test data, or design curves. The computed root mean square (rms) low-frequency motion for this example is 0.97 ft.

The vessel motions at the reference point must be transformed to the end or fairlead of the line to be analyzed. The procedure required varies with the type of analysis.

#### J.1.4.1 Quasi-Static Analysis

In a quasi-static analysis, only surge in the quartering direction is considered. Heave is ignored. The vessel motion RAOs and phases can be transformed into the quartering direction and convoluted over the sea-state spectrum to produce rms line end motion. The computed wave frequency rms motion in this example was 8.6 ft. To establish the dominant frequency response, compare the maximum wave frequency motion with the maximum low-frequency motion as follows:

maximum wave frequency motion:

$$3.72 \times 8.6 = 32.0 \text{ ft}$$

maximum low-frequency motion:

$$3.03 \times 0.97 = 2.9 \text{ ft}$$

where the factors 3.72, 3.03 represent a 1 in 1,000 and 1 in 100 wave maximum respectively, typical for 3-hour storm conditions. Wave frequency dominates. Hence the design condition is:

$$\text{maximum wave frequency motion + significant low-frequency motion} = 32.0 + 2 \times 0.97 = 33.94 \text{ ft}$$

Note the factors 3.72 and 3.03 are approximate values typically used for a 3-hour storm. More precise values can be obtained by Equations 5.5 to 5.8.

#### J.1.4.2 Frequency Domain Analysis

To compute line end motions in the frequency domain, the vessel RAOs and phases in the six degrees of freedom must be translated to the fairlead location. In general, only the motions in the plane of the line are of interest. The line end motions in the horizontal and vertical directions in the plane of the line, and the phases, between them are computed for each frequency. It is extremely important to retain all phase information to this point as the dynamic behavior of the line is heavily influenced by the tangential motion or stretching of the line. The line end motions used in this example are given in complex form in Table J.1 for each frequency. The standard RAO value is the square root of the sum of the squares of the real and imaginary points, in the case of each motion.

### J.1.4.3 Time Domain Analysis

Time domain analysis requires a further step beyond frequency domain analysis. A time history of the motions is required. The following procedure was used in this example:

a. A three hour time series, representing the wave elevation was generated from the sea-state spectrum. The procedure is illustrated in Figure J.2.

$$h(t) = \sum_{jni}^{n} A_j \cos(\omega_j t + \phi_j)$$

where

$Aj$ = $\sqrt{2.5(\omega_j)\Delta\omega}$

$\phi j$ = Random Phase $[0,2\pi]$,

$S(\omega_j)$ = Spectral Density at Frequency $\omega_J$.

At least 50 frequencies are required to develop a realistic time series. Care should be taken in generating the wave elevation, that the time series does not repeat itself prematurely. This is normally achieved by using varied frequency spacing. The fairlead motion RAOs of Table J.1 are used to transform the wave elevation time series into a horizontal and vertical motion at the line end, in the plane of the line. Because phasing has been properly maintained in Table J.1, the resulting horizontal and vertical motions are correctly phased.

The analysis of a full three hour time series is generally not practical in standard design practice. A simplified procedure is illustrated in Figure J.3. The maximum tangential motion is computed. A 120-second segment is selected centered on the maximum tangential motion and the time domain analysis is performed for the 120-second segment.

Table J.1—Fairlead Motion Complex RAO

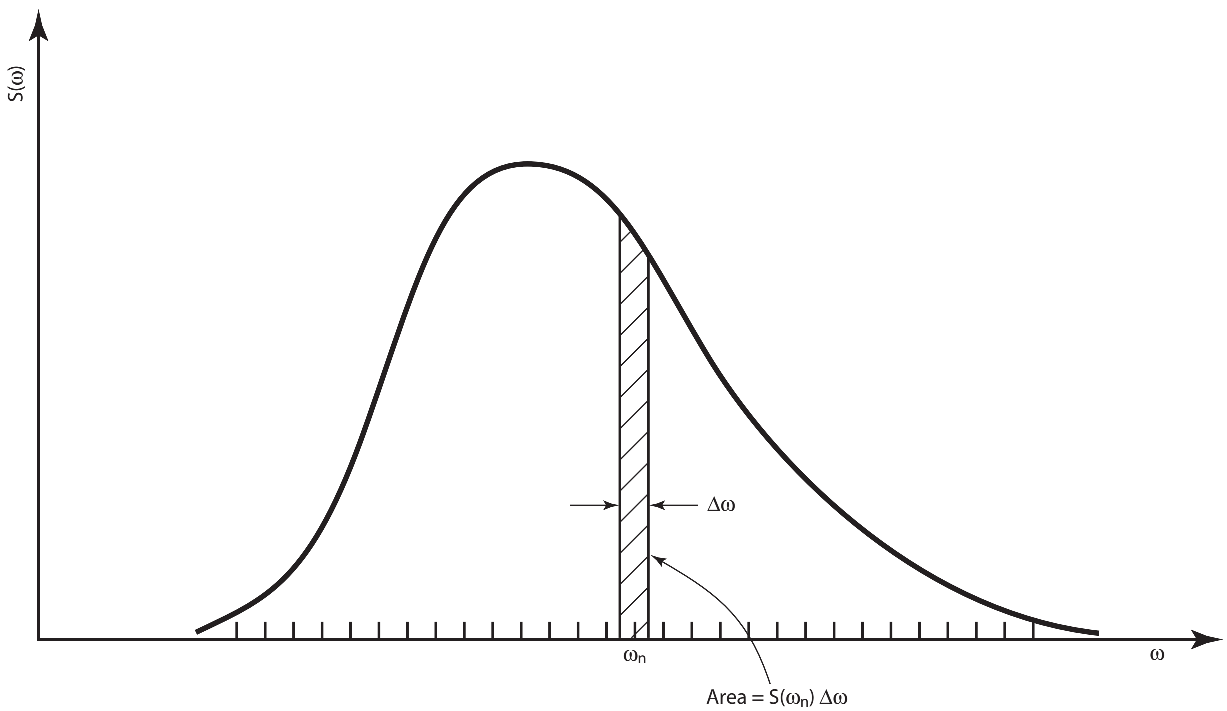
Line No. = 1
Line Heading = 54.00 Degrees
Wave Heading From +X = – 45.00 Degrees
Line End Motions with X in Line from Fairlead to Anchor and Z Vertical

| | | ***Location (X = 110.00 Y = 104.00 Z = –35.00)*** | | | |
|---|---|---|---|---|---|
| Wave Frequency (Rad/Sec) | Low Frequency (Sec) | X Real (AMP/AMP) | X Imag (AMP/AMP) | Z Real (AMP/AMP) | Z Imag (AMP/AMP) |
| 0.20 | 31.42 | –0.0036 | 0.9928 | 1.0976 | 0.0518 |
| 0.26 | 24.32 | –0.0053 | 0.9085 | 0.7092 | –0.2909 |
| 0.32 | 19.84 | –0.0061 | 0.8496 | 0.5965 | 0.2052 |
| 0.37 | 16.76 | –0.0092 | 0.7840 | 0.5820 | 0.3278 |
| 0.43 | 14.50 | –0.0120 | 0.7032 | 0.5158 | 0.3992 |
| 0.49 | 12.78 | –0.0141 | 0.6080 | 0.4273 | 0.4386 |
| 0.55 | 11.42 | –0.0152 | 0.5014 | 0.3289 | 0.4419 |
| 0.61 | 10.33 | –0.0153 | 0.3899 | 0.2315 | 0.4110 |
| 0.67 | 9.42 | –0.0140 | 0.2815 | 0.1452 | 0.3489 |
| 0.72 | 8.67 | –0.1115 | 0.1849 | 0.0773 | 0.2685 |
| 0.78 | 8.02 | –0.0083 | 0.0997 | 0.0249 | 0.1788 |
| 0.84 | 7.47 | –0.0048 | 0.0542 | 0.0090 | 0.1050 |
| 0.90 | 6.98 | –0.0014 | 0.0121 | –0.0047 | 0.0340 |
| 0.96 | 6.56 | –0.0004 | 0.0143 | 0.0025 | 0.0145 |
| 1.02 | 6.18 | 0.0006 | 0.0165 | 0.0096 | –0.0051 |

Table J.1—Fairlead Motion Complex RAO (Continued)

Line No. = 1
Line Heading = 54.00 Degrees
Wave Heading From +X = – 45.00 Degrees
Line End Motions with X in Line from Fairlead to Anchor and Z Vertical

| | | ***Location (X = 110.00 Y = 104.00 Z = –35.00)*** | | | |
|---|---|---|---|---|---|
| Wave Frequency (Rad/Sec) | Low Frequency (Sec) | X Real (AMP/AMP) | X Imag (AMP/AMP) | Z Real (AMP/AMP) | Z Imag (AMP/AMP) |
| 1.07 | 5.84 | 0.0000 | 0.0179 | 0.0114 | –0.0120 |
| 1.13 | 5.54 | –0.0023 | 0.0186 | 0.0073 | –0.0051 |
| 1.19 | 5.27 | –0.0047 | 0.0193 | 0.0032 | 0.0018 |
| 1.25 | 5.03 | –0.0071 | 0.0200 | –0.0009 | 0.0087 |
| 1.31 | 4.80 | –0.0073 | 0.0173 | –0.0013 | 0.0079 |
| 1.37 | 4.60 | –0.0073 | 0.0142 | –0.0013 | 0.0061 |
| 1.42 | 4.41 | –0.0074 | 0.0111 | –0.0012 | 0.0044 |
| 1.48 | 4.24 | –0.0074 | 0.0079 | –0.0011 | 0.0026 |
| 1.54 | 4.08 | –0.0074 | 0.0048 | –0.0011 | 0.0008 |
| 1.60 | 3.93 | –0.0075 | 0.0033 | –0.0011 | –0.0001 |




$$h(t) = \sum_{j=1}^{\omega} \sqrt{2\,S(\omega_j)\,\Delta\omega}\,\mathrm{Cos}\,(\omega_j t + \phi_j)$$

Figure J.2—Spectral Decomposition

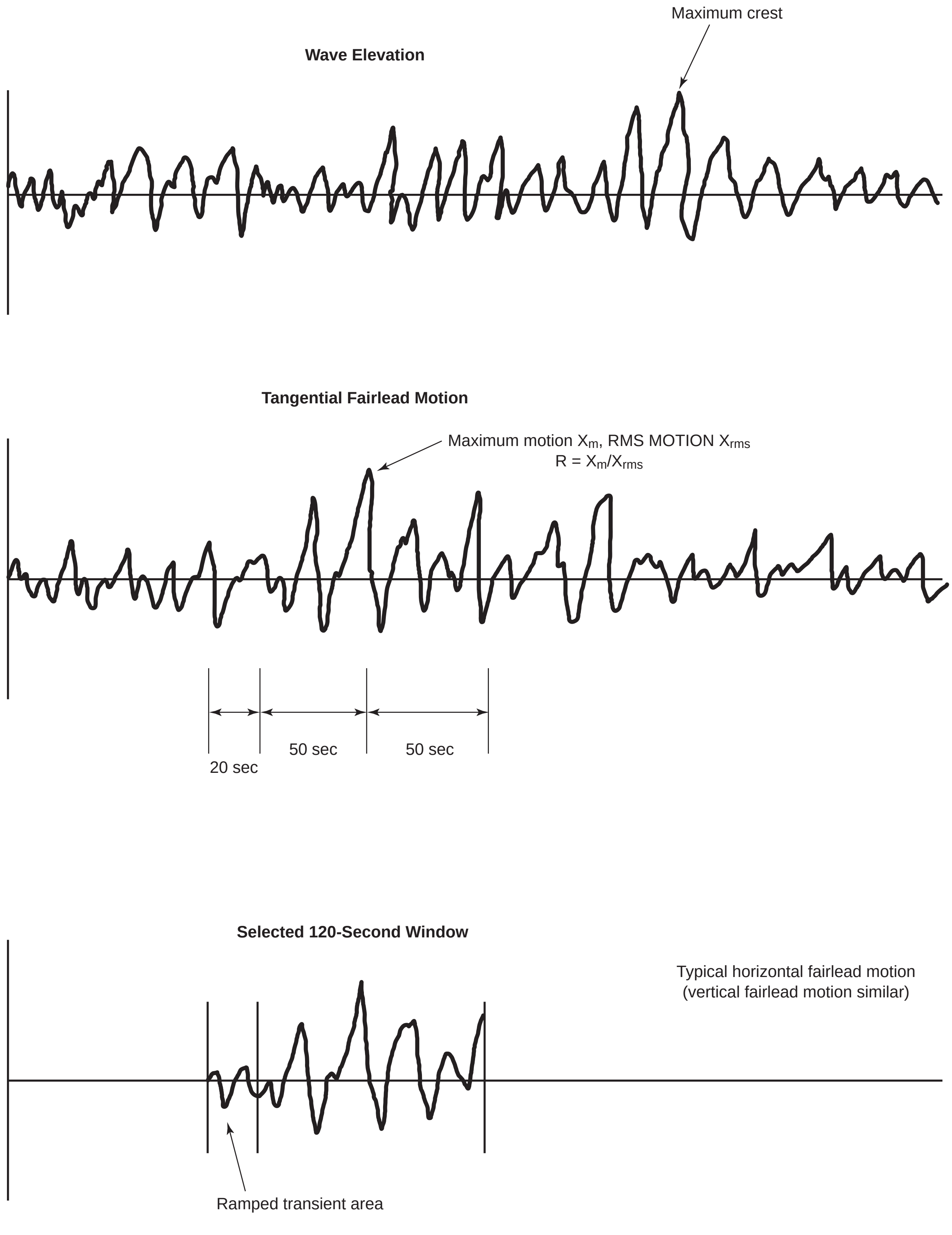



Figure J.3—Selection of Input Motion

The tangential motion is used because tangential motion greatly influences the line stretch. The use of a limited time segment based on the maximum wave elevation is not in general recommended. Alternative methods, which ensure a statistically realistic response level can be used. The 120 seconds of data is adequate and will give results of comparable magnitude to those developed in a full three hour simulation for the computer programs used in this analysis. However, the length of the data should be determined after a sensitivity study for a particular computer program has been conducted.

### J.1.5 MOORING ANALYSIS RESULTS

#### J.1.5.1 Quasi-Static Analysis

The mooring system was analyzed using a computer program based on catenary equations modified for elastic stretch and bottom friction. The computed line tensions were for the most loaded line No. 2 (54 degree direction).

Mean Tension = 643 kips (35 percent BTL).

Mean + Low-Frequency = 650 kips (35 percent BTL).

Maximum Tension = 779 kips (42 percent BTL).

Other derived parameters are:

Maximum Anchor Load = 353 kips.

Maximum Suspended Line Length = 3,986 ft

#### J.1.5.2 Frequency Domain Analysis

The most loaded line No. 2 was analyzed. Initially, a quasi-static analysis is required under mean plus low-frequency tension. From a review of the data it was concluded that wave frequency tensions would dominate. Hence, the line is initially analyzed under mean plus significant low-frequency conditions:

Mean Tension = 643 kips.

Mean + Low Frequency = 650 kips (35 percent BTL).

The line was analyzed under a 650 kip tension with the specified end motions. The output tension spectrum is given in Table J.2. The computed rms tension was 111.5 kips. The maximum wave frequency tension was 3.72 x 111.5 = 415 kips. The total tension is:

Maximum Design Tension = 1,065 kips (58 percent BTL).

The other derived parameters are:

Maximum Anchor Load = 870 kips.

Maximum Suspended Line Length = 4,284 ft

Table J.2—Tension Spectrum

| Frequency Rad/Sec | Tension Spectrum |
|---|---|
| 0.200 | 0.1067002E-09 |
| 0.258 | 0.3302278 |
| 0.317 | 80.27989 |
| 0.375 | 586.9768 |
| 0.433 | 2172.754 |
| 0.492 | 1969.006 |
| 0.550 | 510.8987 |
| 0.608 | 187.6015 |
| 0.667 | 60.40662 |
| 0.725 | 21.31249 |
| 0.783 | 13.28411 |
| 0.842 | 5.786316 |
| 0.900 | 1.439414 |
| 0.958 | 0.1947693E-01 |
| 1.017 | 2.250558 |
| 1.075 | 4.270370 |
| 1.133 | 2.902684 |
| 1.192 | 1.725605 |
| 1.250 | 0.8388341 |
| 1.308 | 0.5401919 |
| 1.367 | 0.3427716 |
| 1.425 | 0.2073264 |
| 1.483 | 0.1237301 |
| 1.542 | 0.7565913E-01 |
| 1.600 | 0.5461835E-01 |

The line tensions are acceptable (below 60 percent BTL) and suspended line length ensures adequate grounded length. Anchor loads are substantially higher then those produced by quasi-static analysis.

#### J.1.5.3 Time Domain Analysis

Only the most loaded line, No. 2, was analyzed. The procedure is similar to the frequency domain in that a quasi-static analysis was first carried out under mean plus significant low-frequency tension. The applied end motions were then combined with the resulting tensions.

Mean Tension = 643 kips (35 percent BTL).

Mean + Low-Frequency = 650 kips (35 percent BTL).

Maximum Tension = 1,101 kips (60 percent BTL).

Other derived parameters are:

Maximum Anchor Load = 886 kips.

Maximum Suspended Line Length = 3,903 ft.

The peak quantities are compared to allowables as before. Tensions are acceptable (60 percent limit); suspended line length ensures adequate grounded length; and a suitable anchor must be designed.

## J.2 Fatigue Analysis Example

The following example illustrates the computation of lifetime fatigue damage on a mooring line and the estimation of allowable fatigue life. A frequency domain method of dynamic analysis is used.

### J.2.1 MOORING SYSTEM DESCRIPTION

**J.2.1.1** A semi-submersible system with a 12 line, 25 degrees/45 degrees/65 degrees mooring is considered. The system is shown in Figure J.4.

**J.2.1.2** Mooring parameters used in the analysis for wire rope:

| | |
|---|---|
| Elastic stiffness (EA) | 94,355 kips |
| Air weight | 22.7 lbs/ft |
| Submerged weight | 19.3 lbs/ft |
| Mass | 0.705 slugs/ft |
| Normal added mass | 0.133 slugs/ft |
| Drag diameter | 3.5 in. |
| Drag coefficient | 1.2 |
| T-N curve | $NR^{4.09} = 731$ (For example only) |
| Reference breaking strength | 1,110 kips |

**J.2.1.3** Mooring parameters used in the analysis for chain:

| | |
|---|---|
| Elastic stiffness (EA) | 147,074 kips |
| Air weight | 123 lbs/ft |
| Submerged weight | 107 lbs/ft |
| Mass | 3.73 slugs/ft |
| Tangential added mass | 0.25 slugs/ft |
| Normal added mass | 0.50 slugs/ft |
| Drag diameter | 7 in. |
| Drag coefficient | 1.2 |
| T-N curve | $NR^{3.36} = 370$ (For example only. See Table 3 for recommended equation.) |
| Reference breaking strength | 1,383 kips |

Fatigue analysis are provided here for the wire rope at the fairlead and for the chain at the chain/wire rope intersection.

### J.2.2 ENVIRONMENTAL CONDITIONS

A fatigue analysis consists of multiple individual line tension analyses as described in 7.5. The environment is defined as a set of the following:

a. Directions.

b. Wind speeds.

c. Current speeds.

d. Wave heights, spectral shapes, spectral peak periods or equivalent.

e. Probability of occurrence of the above.

The mean forces associated with wind, wave and current are first computed. The low-frequency motions associated with each environmental condition can be obtained from computer programs, model tests, or design curves.

In the usual case, eight directions at 45 degree intervals are sufficient to define the environment. In this example we will consider one such direction, 225 degrees, in detail. The environmental conditions to be analyzed are given in Table J.3. The sea-states, cumulative probability of occurrence, mean loads and rms low-frequency motions are given. The following additional parameters are used:

a. Water depth 1,476 ft.

b. Pierson Moskowitz Spectral form.

c. Environment is in the analyzed direction 16 percent of the time.

In this example, wind, wave, and current are assumed colinear. All low-frequency motions are assumed to be from wave effects and are based on a reference stiffness of 18 kips/ft. Wind effects are included in the mean load. For a mooring system with a stiffness $k$ kips/ft at the mean position, the rms low-frequency motions are computed as:

$$rms(k) = rms(18) \times \sqrt{18/k}$$

where

$rms(k)$ = the motion at stiffness $k$,

$rms(18)$ = the motion at the reference stiffness 18 kips/ft.

If low-frequency wind effects were included, a square root relationship between the actual and reference stiffness values cannot be used. It is necessary to define the rms motions for the actual mooring stiffness.

### J.2.3 FATIGUE ANALYSIS

The fatigue damage is computed as follows. The annual fatigue damage is initially computed. Low-frequency effects are minor but are included. A year is assumed to have 3.15576 x 10 seconds. By Narrow Band Theory, the fatigue damage in a given sea-state is:

$$D = N_w (\sqrt{2}\ R_{wrms})^M \bullet \Gamma(1 + M/2)/K$$
$$+ N_l (\sqrt{2}\ R_{lrms})^M \bullet \Gamma(1 + M/2)/K \qquad (11.1)$$

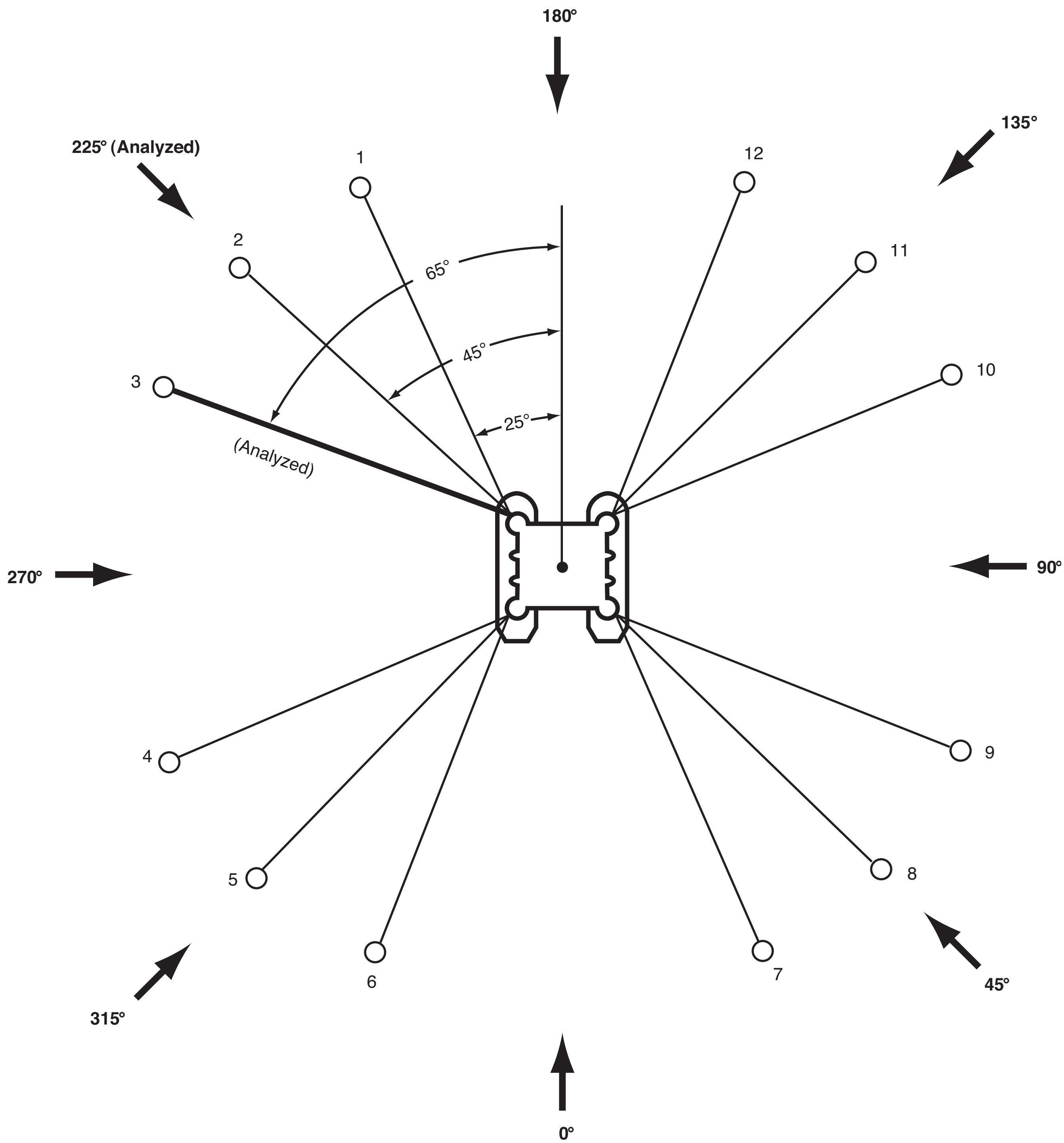



Figure J.4—Mooring System and Environmental Directions

where

$D$ = annual fatigue damage,

$K$ = the intercept parameter of the $T/N$ curve (731 for wire rope and 370 for chain, these values are for example only),

$M$ = the slope of the $T/N$ curve (4.09 in this example for wire rope and 3.36 for chain, these values are for example only),

$R_{wrms}$ and $R_{lrms}$ = the ratios of wave frequency and low-frequency *rms* tension range (twice the single amplitude value) to reference breaking strength,

$N_w$ and $N_1$ = the numbers of wave frequency and low-frequency tension cycles per year.

$N_w$ can be computed as:

$$N_w = v \times 3.15576 \times 10^7 \times P_d \times P_s \tag{11.2}$$

where

$v$ = zero up-crossing frequency of the tension spectrum (hz),

$P_d$ = probability of the direction (16 percent for this example),

$P_s$ = probability of the sea-state given the direction. (Table J.3).

Low-frequency fatigue damage and wave frequency fatigue damage are computed independently. The low-frequency zero crossing period is estimated to be the natural period of the vessel/mooring system as a function of applied mean load. The appropriate number of low-frequency cycles per year can be computed as:

$$N_l = (3.15576 \times 10^7 \times P_d \times P_s)/T_n$$

where

$T_n$ = vessel/mooring system natural period and the remaining quantities are as previously defined.

If procedures are used that incorporate a general low-frequency (from wind or wave or both) tension spectrum, the actual zero up-crossing period can be used, rather than the natural period. In the usual case, low-frequency effects are minor and the natural period is adequate. The wave frequency zero up-crossing period or frequency should be computed directly from the tension spectrum generated in a line dynamic analysis, as follows:

$$v = \sqrt{M_2 / M_o} \text{ hz,}$$

$$M_2 = \int \infty f^2 \quad S(f) d_f,$$

$$M_o = \int_o^\infty \quad S(f),$$

$f$ = Frequency (hz),

$S(f)$ = Tension spectrum ($\text{kips}^2$/hz).

### J.2.4 FATIGUE DAMAGE

A detailed computation of the fatigue damage per year associated with a 225 degree direction (wave heading towards), for line 3 (65 degree spreading angle) is given in Table J.4 for wire rope and Table J.5 for chain. The total annual accumulated fatigue damage is:

$$D = 0.689 \times 10^{-2} \text{ (chain).}$$

$$= 0.155 \times 10^{-2} \text{ (wire rope).}$$

In this example, low-frequency motions have essentially no effect on the fatigue life, being about three orders of magnitude less severe than wave frequency effects. This, however, may not be true for the cases where low-frequency motions are dominant. The fatigue damages for chain and wire rope associated with all directions are given in Table J.6. The total annual fatigue damage on the line, counting all directions is:

$$D = 0.218 \times 10^{-1} \text{ (chain).}$$

$$D = 0.418 \times 10^{-2} \text{ (wire rope).}$$

The useful fatigue life, computed as:

Life = $1/(3D)$ years.

= 15 years (chain).

= 80 years (wire rope).

The safety factor applied is 3, based on 6.8.

Table J.3—Environmental Condition, Mean Loads and Low-Frequency Motions for the Analyzed Direction

| Sig. Wave (ft) | Peak Period (sec) | Probability (percent) | Mean Loads (kips) | Low Freq. rms[a] (ft) |
|---|---|---|---|---|
| 3.60 | 8.4 | 16.96 | 28.7 | 0.40 |
| 8.30 | 9.2 | 36.29 | 61.4 | 0.66 |
| 13.01 | 10.4 | 26.07 | 131.4 | 1.07 |
| 17.71 | 11.6 | 13.05 | 212.2 | 1.22 |
| 22.42 | 12.7 | 5.31 | 309.3 | 1.35 |
| 27.12 | 13.6 | 1.64 | 418.8 | 1.43 |
| 31.83 | 14.4 | 0.52 | 552.1 | 1.50 |
| 36.53 | 15.3 | 0.13 | 714.9 | 1.60 |
| 41.24 | 16.1 | 0.02 | 891.7 | 1.68 |
| 45.94 | 17.7 | 0.01 | 1239.1 | 1.75 |

[a] Low frequency motions based on a reference stiffness of 18 kips/fftt

Table J.4—Wire Rope Damage

Annual Accumulated Fatigue Damage by Sea-State for Direction 6, 225.0
(All Periods and #'s of Cycles Refer to Tensions)
All Data is Per Year

| Mean Ten. | Sig. Height | RMS Tension (Wave) | RMS Tension (Low) | # of Cycles (Wave) | # of Cycles (Low) | Zero Cross. Period (Wave) | Zero Cross. Period (Low) | Wave Frequency Damage | Low Frequency Damage |
|---|---|---|---|---|---|---|---|---|---|
| 153.0 | 3.60 | 2.2 | 0.4 | 0.121E+06 | 0.780E+04 | 7.10 | 109.76 | 0.196E–06 | 0.172E–10 |
| 162.0 | 8.30 | 4.4 | 1.4 | 0.243E+06 | 0.168E+05 | 7.56 | 109.14 | 0.748E–05 | 0.526E–08 |
| 175.7 | 13.01 | 7.2 | 2.6 | 0.163E+06 | 0.122E+05 | 8.09 | 108.22 | 0.037E–04 | 0.438E–07 |
| 192.0 | 17.71 | 10.8 | 3.3 | 0.723E+05 | 0.616E+04 | 9.11 | 106.94 | 0.863E–04 | 0.599E–07 |
| 215.5 | 22.42 | 16.3 | 4.2 | 0.263E+05 | 0.258E+04 | 10.19 | 103.97 | 0.169E–03 | 0.619E–07 |
| 242.4 | 27.12 | 24.0 | 4.9 | 0.734E+04 | 0.816E+03 | 11.23 | 100.89 | 0.226E–03 | 0.380E–07 |
| 278.2 | 31.83 | 34.7 | 5.5 | 0.212E+04 | 0.265E+03 | 12.25 | 97.79 | 0.296E–03 | 0.200E–07 |
| 324.8 | 36.53 | 49.3 | 6.5 | 0.442E+03 | 0.629E+02 | 13.34 | 93.75 | 0.258E–03 | 0.900E–08 |
| 376.9 | 41.24 | 67.7 | 7.3 | 0.799E+02 | 0.130E+02 | 14.60 | 89.52 | 0.170E–03 | 0.300E–08 |
| 480.2 | 45.94 | 96.1 | 8.4 | 0.325E+02 | 0.633E+01 | 16.08 | 82.44 | 0.290E–03 | 0.300E–08 |
| | | | | | | | Total | 0.154E–02 | 0.243E–06 |

Total damage measure for this direction:

| | |
|---|---|
| Direction (6) | 225.0 |
| Probability percent for direction | 0.160E+02 |
| First order damage | 0.154E–02 |
| Second order damage | 0.243E–06 |

Total number of tension cycles per year for this direction:

| | |
|---|---|
| Wave frequency tension cycles | 0.634E+06 |
| Low frequency tension cycles | 0.466E+05 |
| Average wave tension period | 0.795E+01 (zero crossing) |
| Average low tension period | 0.108E+03 |

## Table J.5—Chain Damage

Annual Accumulated Fatigue Damage by Sea-state for Direction 6, 225.0
(All Periods and #'s of Cycles Refer to Tensions)
All Data is Per Year

| Mean Ten. | Sig. Height | RMS Tension (Wave) | RMS Tension (Low) | # of Cycles (Wave) | # of Cycles (Low) | Zero Cross. Period (Wave) | Zero Cross. Period (Low) | Wave Frequency Damage | Low Frequency Damage |
|---|---|---|---|---|---|---|---|---|---|
| 153.0 | 3.60 | 2.0 | 0.4 | 0.121E+06 | 0.780E+04 | 7.08 | 109.76 | 0.507E–05 | 0.182E–08 |
| 162.0 | 8.30 | 4.3 | 1.3 | 0.243E+06 | 0.168E+05 | 7.55 | 109.14 | 0.119E–03 | 0.229E–06 |
| 175.7 | 13.01 | 7.1 | 2.7 | 0.162E+06 | 0.122E+05 | 8.11 | 108.22 | 0.437E–03 | 0.123E–05 |
| 192.0 | 17.71 | 10.8 | 3.4 | 0.721E+05 | 0.616E+04 | 9.14 | 106.94 | 0.789E–02 | 0.143E–05 |
| 215.5 | 22.42 | 16.4 | 4.2 | 0.262E+05 | 0.258E+04 | 10.22 | 103.97 | 0.120E–02 | 0.122E–05 |
| 242.4 | 27.12 | 24.2 | 4.9 | 0.733E+04 | 0.816E+03 | 11.24 | 100.89 | 0.123E–02 | 0.670E–06 |
| 278.2 | 31.83 | 35.0 | 5.6 | 0.211E+04 | 0.265E+03 | 12.26 | 97.79 | 0.123E–02 | 0.330E–06 |
| 324.8 | 36.53 | 49.8 | 8.5 | 0.442E+03 | 0.629E+02 | 13.33 | 93.75 | 0.850E–03 | 0.130E–06 |
| 376.9 | 41.24 | 68.4 | 7.4 | 0.801E+02 | 0.130E+02 | 14.57 | 89.52 | 0.440E–03 | 0.400E–07 |
| 480.2 | 45.94 | 97.1 | 8.5 | 0.326E+02 | 0.633E+01 | 16.04 | 82.44 | 0.590E–03 | 0.300E–07 |
| | | | | | | | Total | 0.689E–02 | 0.531E–05 |

Total damage measure for this direction:

| | |
|---|---|
| Direction (6) | 225.0 |
| Probability percent for direction | 0.160+02 |
| First order damage | 0.689E–02 |
| Second order damage | 0.531E–05 |

Total number of tension cycles per year for this direction:

| | |
|---|---|
| Wave frequency tension cycles | 0.634E+06 |
| Low frequency tension cycles | 0.466E+05 |
| Average wave tension period | 0.795E+01 (zero crossing) |
| Average low tension period | 0.108E+03 |

## Table J.6—Annual Fatigue Damage as a Function of the Environment

Line 3 (65 degrees)

| Direction | Probability of Direction (percent) | Annual Fatigue Damage: Chain | Annual Fatigue Damage: Wire Rope |
|---|---|---|---|
| 0 | 6.0 | 0.426 5 $10^{-3}$ | 0.412 5 $10^{-4}$ |
| 45 | 8.0 | 0.535 5 $10^{-3}$ | 0.436 5 10–$^{4}$ |
| 90 | 13.0 | 0.125 5 $10^{-2}$ | 0.109 5 $10^{-3}$ |
| 135 | 12.5 | 0.375 5 $10^{-3}$ | 0.337 5 $10^{-4}$ |
| 180 | 14.0 | 0.133 5 $10^{-2}$ | 0.151 5 $10^{-3}$ |
| 225 | 16.0 | 0.689 5 $10^{-2}$ | 0.155 5 $10^{-2}$ |
| 270 | 18.0 | 0.100 5 $10^{-1}$ | 0.213 5 $10^{-2}$ |
| 315 | 12.5 | 0.983 5 $10^{-3}$ | 0.131 5 $10^{-3}$ |
| Total | 100.0 | 0.218 5 $10^{-1}$ | 0.418 5 $10^{-2}$ |

# APPENDIX K—GULF OF MEXICO MODU MOORING PRACTICE FOR HURRICANE SEASON

## K.1 Scope

This appendix provides guidance for design and operation of MODU mooring systems in the Gulf of Mexico during the hurricane season. The guidance was developed through a cooperative arrangement with the American Petroleum Instituteís Subcommittee on Offshore Structures RP 2SK Work Group and the Joint Industry Project entitled "US Gulf of Mexico (GOM) Mooring Strength Reliabilty" (MODU JIP). The information presented herein is premised on the existence of a MODU evacuation plan, the intent of which is to assure timely and safe evacuation of all MODU personnel in anticipation of hurricane conditions.

This guidance is supplemental to the following documents:

- API RP 2SK, *Design and Analysis of Stationkeeping Systems for Floating Structures*, 3rd Edition (2005);
- API RP 2I, *In-service Inspection of Mooring Hardware for Floating Structures*, 3rd Edition (2008);
- API RP 2SM, *Recommended Practice for Design, Manufacture, Installation, and Maintenance of Synthetic Fiber Ropes for Offshore Mooring*, 1st Edition (2001), and the 2007 Addendum.

This guidance replaces API RP 95F, *Gulf of Mexico MODU Mooring Practices for the 2007 Hurricane Season—Interim Recommendations*, 2nd Edition, April 2007.

## K.2 Basic Considerations

### K.2.1 BACKGROUND

In 2004 and 2005, Hurricanes Ivan, Katrina, and Rita moved through the Gulf of Mexico with extreme winds and waves, causing a number of MODU mooring failures in their paths. Mooring failures have occurred in previous hurricanes, including Hurricanes Andrew and Lili, but the number of failures was much lower.

Assessment of MODU mooring systems for worldwide operations has frequently been based on API recommended practices. The first API MODU mooring recommended practice (API RP 2P), released in 1987, specified a design environment lower than the five to ten year return period specified in 3.1, principally driven by the MODU mooring capacities available at that time. Building on the results of a joint industry project focused on MODU mooring code calibration (Reference 1), this document incorporates increased MODU mooring design return periods. These criteria are as follows:

- 5-year return period (away from other structures);
- 10-year return period (in the vicinity of other structures).

There have been significant modifications in the underlying calibration parameters and Gulf of Mexico operations since the 1995 mooring code calibration study which may influence the applicability to future activities. Differences include the following.

1. There are more floating and subsea installations and pipelines. This may result in higher risk of property damage or environmental impact, should a MODU break loose or drag its anchors under hurricane conditions.
2. The number of deepwater permanent installations has increased significantly. These are high production rate installations that often share a pipeline to shore. Therefore the cost for an incident can be much higher.
3. There are more deepwater MODU operations that typically use taut leg moorings with pile anchors. These systems may respond to hurricanes differently than catenary moorings with drag anchors. These types of mooring systems in deeper water were not part of the 1995 calibration study.

### K.2.2 MOORING ISSUES

This appendix supplements 3.1, Section 5 and Section 6 for Gulf of Mexico MODU mooring design and operating practice during the hurricane season. Topics addressed herein that will be part of the overall mooring design and MODU operations include:

- site- and well-specific data;
- design criteria for the mooring;
- indicative Gulf of Mexico hurricane extreme metocean conditions;
- mooring analysis;
- site-specific risk assessment and mitigation;
- mooring hardware issues such as anchor system and mooring system upgrade;
- mooring operation issues such as deployment, hurricane preparedness, and inspection.

### K.2.3 SITE- AND WELL-SPECIFIC DATA

When planning a MODU mooring operation, the following site- and well-specific data should be collected:

1. location description;
2. description of planned well operation;
3. site-specific metocean data and source;
4. mooring installation hazards;
5. surface and subsea infrastructure.

### K.2.4 STACKED MODUS

These guidelines also apply to MODUs that are “stacked” and not working. MODUs that are not actively working should be moored in accordance with the provisions of this document to minimize the likelihood of breaking free and inflicting damage. Alternate methods of stacking MODUs, e.g., setting on bottom for MODUs that can accommodate bottom founding, may be acceptable provided appropriate engineering is performed to assure performance comparable to or better than that of moored MODUs.

### K.2.5 EXCEPTIONAL MODU MOORING OPERATIONS

It is recognized that a MODU may be required to perform exceptional operations, for example, to prevent major losses or pollution. Alternately, it may be necessary to relocate a MODU (e.g., to a low consequence location) with a damaged mooring while it awaits repair. In these exceptional cases a risk assessment should be performed to assess the consequences of not performing the MODU mooring operation and the risks associated with mooring system failure. In these special circumstances an environmental return period of less than 10-years may be acceptable for the particular operation under consideration.

### K.2.6 MOORING INSPECTION

Mooring inspection is critical to ensure the integrity of the mooring system and minimize the probability of mooring failure resulting from premature failure of substandard components. Guidance for inspection and reuse of MODU mooring components is contained in API 2I, 3rd Edition, with special reference to Annex B on MODU mooring inspection in areas of tropical cyclone.

## K.3 Mooring Analysis

### K.3.1 MOORING ANALYSIS METHOD

Following API 2SK, quasi-static or dynamic analyses may be utilized for MODU moorings. Either the 1-hour wind speed with wind spectrum or the 1-minute steady wind speed may be used for the wind force calculation. It should be noted that the wind spectrum approach requires good estimates of low-frequency damping.

Wind, wave, and current forces and vessel motions shall be evaluated using the best available, updated MODU information. Many MODUs have gone through significant modifications, involving additional hull structures and deck equipment, that can change the environmental loads on the vessel. Wind, wave, and current force coefficients and models for hydrodynamic analysis should be adjusted to reflect the changes. The adjustment can be based on new model tests, analysis, or combination thereof.

It is not possible to predict precise wind, wave, and current directions under hurricane conditions; therefore, sufficient environmental directions shall be investigated to capture critical cases for line tensions and anchor load and uplift angle. As a minimum, bow, beam, quarter, down-line, and between-line environmental directions should be analyzed. Analysis for the damaged condition should investigate as many conditions as necessary to capture the critical cases, including, as a minimum, damage of the most highly loaded line and adjacent lines. For mooring systems with lines of unequal strength, damage of the most utilized lines and adjacent lines should also be considered.

### K.3.2 IDEALIZED MOORING SYSTEM BEHAVIOR: ROBUSTNESS CHECK OR WEAK POINT ANALYSIS

#### K.3.2.1 General

In addition to the safety factor check, a mooring sensitivity or weak point analysis should be performed. The objective of this analysis is to determine the probable failure mode of the mooring system. It is a useful tool for comparing different mooring systems for a given design criteria. Such an analysis can provide useful information for risk assessment and mitigation strategies. As such, there are no defined acceptance criteria for mooring analysis results discussed in this section.

The mooring sensitivity or weak point analysis should be conducted for both the intact and the damaged conditions. Performing this analysis does not guarantee MODU mooring survival because of other potential failure modes, such as bending over the fairlead, wire fretting, elasto-plastic fatigue damage, etc.

For line components such as chain, wire rope, and fiber rope, the capacity of the component is normally taken as the break strength [minimum break load (MBL), catalog break strength (CBS), minimum break strength (MBS), as appropriate] adjusted for the condition of the component. For example, API 2I allows a mooring component to remain in use until its break strength is reduced to 90% of its catalog break strength. In addition, wire rope bending around the fairlead experiences further strength reduction; for example, a *D/d* (fairlead diameter/wire rope diameter) ratio of 16 may reduce the strength of the wire rope to 90% of CBS. Strength reduction can also be expected for chain.

#### K.3.2.2 Illustrative Example

Following is an example demonstrating how this analysis may be used for risk assessment and mitigation. The mooring is a chain/wire rope combination system with high efficiency drag anchors. Based on mooring analysis results, plots of utilization versus return period are generated for anchor load and line tension under intact (see Figure K.1) and damaged (see Figure K.2) conditions. For line tension, utilization is the ratio of the maximum line tension to break strength. For anchor load, utilization is the ratio of the maximum anchor load to anchor holding capacity. These two figures provide the following information.

1. Utilization limits are 0.6 (intact line tension), 0.8 (damaged line tension), and 1.25 (MODU intact anchor load for drag anchor) based on dynamic analysis (see 7.2 and 7.4). These utilization limits are satisfied for environmental return periods of 12 years for the intact mooring system and 10 years for the damaged system. Therefore, this mooring system meets the line tension utilization requirements for a 10 year return period hurricane.
2. As an example, consider a wire rope with a reduced break strength of about 80% CBS (e.g., 10% strength reduction due to wire condition and 10% strength reduction due to bending over the fairlead). If there is no faulty component in the windward lines, the intact mooring system may survive a 20- to 25-year return period hurricane (see Figure K.1). However, the anchors of the most loaded lines are expected to move and bury deeper, resulting in redistribution of the load between the highly loaded lines and a reduction in the maximum line tension and anchor load. For hurricane conditions that exceed the 25-year return period, a complete stationkeeping failure—breaking of a number of lines and dragging the anchors of the remaining lines a large distance—is possible if further reduction in mooring line and anchor load cannot be achieved by anchor movement.
3. If a faulty component results in a premature failure of a highly loaded mooring line, then a complete stationkeeping failure can be expected to occur in about a 10-year return period hurricane, based on Figure K.2. This highlights the importance of keeping the mooring system in good condition through mooring inspection and maintenance.

## K.4 Site Assessment Background for MODU Mooring

### K.4.1 EXISTING CRITERIA

This document provides the basis for mooring analysis for both site assessment of MODU moorings and the design of mooring systems for permanent installations.

### K.4.2 MODIFICATIONS FOR SITE ASSESSMENT OF GULF OF MEXICO MODU MOORINGS

The 2004 and 2005 Gulf of Mexico hurricanes resulted in a number of total and partial failures of MODU mooring systems, but no failures of permanent mooring systems. As a result of these MODU mooring system failures, a risk based method for site assessment of MODUs operating in the Gulf of Mexico during hurricane season was introduced in API 95F, 1st and 2nd Editions, for use in the 2006 and 2007 hurricane seasons.

The most significant change in this appendix, from the previous 5- or 10-year return period environmental conditions used for MODU site assessment in 3.1, is the use of risk assessment methods to determine the adequacy of the MODUs mooring system for the planned operation and location. Other differences between the MODU site assessment method recommended in this appendix and those in Section 3 include the following.

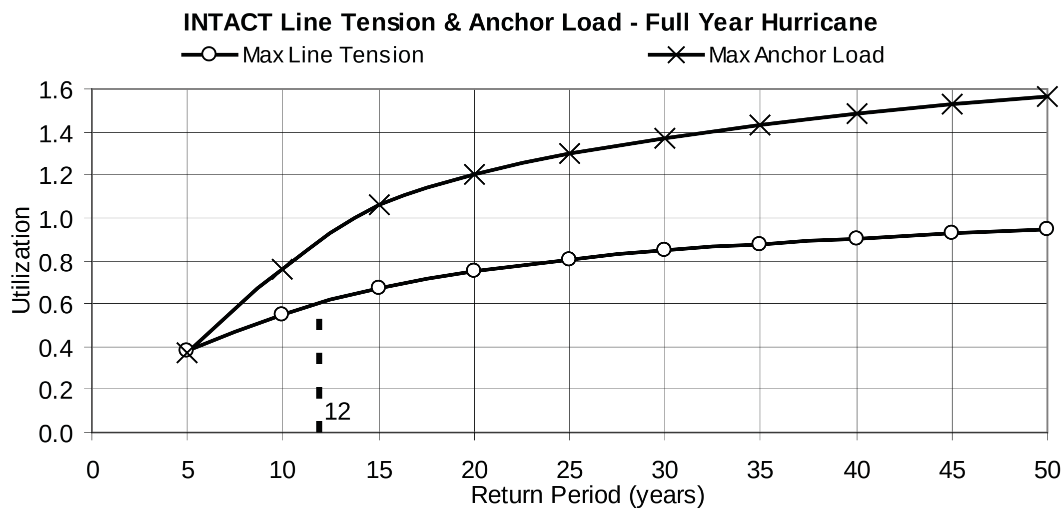



Figure K.1—Utilization versus Return Period for Intact Condition

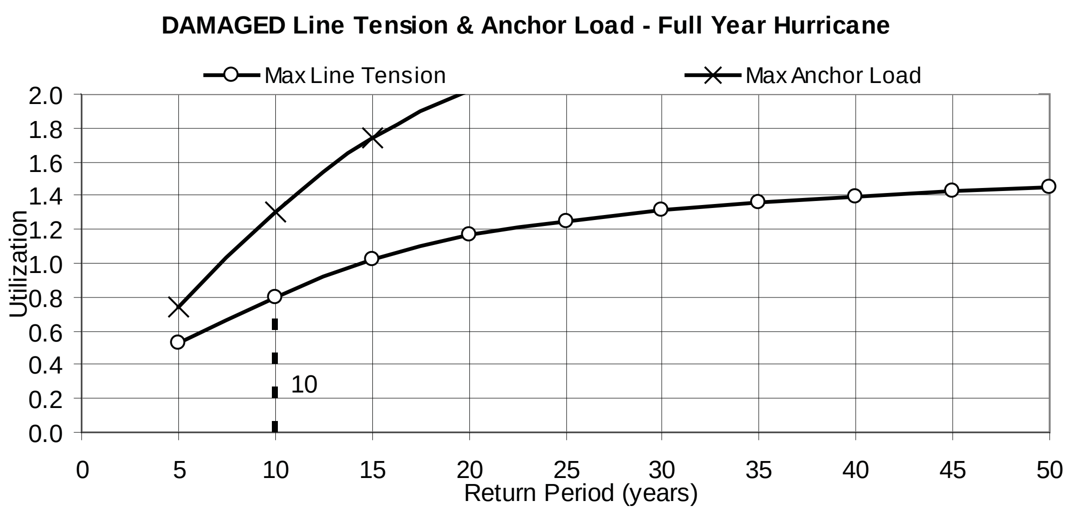



Figure K.2—Utilization versus Return Period for Damaged Condition

1. Recommended design requirements with increased return periods and consequence categories (see K.6).
2. The metocean conditions used for site assessment of MODUs performing typical or atypical operations shall have a return period of not less than 10 years (see K.6).
3. For operations within the peak of the hurricane season (as defined in K.11), the wind, wave, and current conditions used for site assessment of typical or atypical operations shall not be less than those associated with a threshold Category 1 hurricane. During the pre- and post-peak hurricane seasons, the wind, wave and current conditions used for site assessment should not be less than those associated with a threshold Category 1 hurricane unless it can be shown that the overall risk associated with the MODU operations can be significantly reduced with marginally lower metocean criteria.

   Note: In some cases, mitigation methods (the use of alternative mooring line or anchor types) can result in an increase in the probability of system failure (reduction in line or anchor safety factors). In these cases, it can sometimes be shown that despite the increase in probability of mooring system failure, the overall risk of the operation (namely damage to surrounding infrastructure) is substantially reduced.

4. Site and seasonal metocean conditions may be used. Guidance is provided for establishing site and seasonal metocean parameters (see K.11).

5. For typical MODU operations, guidance is provided for performing a financial risk assessment and decision analysis (see K.5, K.6, K.13 and K.14).
6. For atypical MODU operations, an appropriate risk assessment is required to evaluate suitability of the operation (see K.6 and K.14).
7. Mitigation and prevention strategies for reducing the consequences and likelihood of mooring failure should always be considered when designing the mooring system and planning and scheduling the operation (see K.5 through K.10 and K.14).

### K.4.3 SITE-SPECIFIC AND MOORING INFORMATION

The general and local site-specific information to be obtained by the Operator should include the following (existing and anticipated during operation):

1. Location description:
   a. Gulf of Mexico Block designation;
   b. location coordinates;
   c. water depth and seafloor bathymetry;
   d. seabed conditions (soils) and hazards;
   e. site characteristics (e.g., chemosynthetics, archeological, etc.).
2. Description of planned well operation:
   a. well type such as exploratory, development, workover;
   b. time of year for the planned operations;
   c. expected duration;
   d. confidence in duration and potential overrun;
   e. possible causes of delay.
3. Site-specific metocean data and source (see K.11).
4. Mooring installation hazards: restrictions to anchor placement and drag.
5. Surface and subsea infrastructure (see K.6, K.13 and K.14):
   a. distances and directions;
   b. other mooring lines, tendons, etc., within mooring pattern;
   c. mooring lines crossing subsea infrastructure (pipelines, umbilicals, wells, etc.).

The information related to the mooring system that affects the consequences, or mitigates the consequences, of mooring failure includes:

1. type of anchors: drag embedment, plate, pile, etc.;
2. types of mooring components that could damage subsea infrastructure, if dragged;
3. other components used to mitigate the consequences of mooring failure (buoyancy, polyester, etc.).

## K.5 Risk Based Site Assessment for MODU Mooring Operations

### K.5.1 GENERAL

The probability and consequences of a MODU losing station when operating at any location shall be assessed. The intent of the assessment process is to determine the characteristics of the area near the drilling operation and identify options related to mooring component selection, mooring system design, and mitigation opportunities prior to finalizing the mooring design and installing the mooring system. For the planned MODU operation, the mooring system should be associated with an acceptable risk, either by minimizing potential consequences of mooring component or system failure (mitigation) or by reducing the probability of mooring component or system failure (prevention).

### K.5.2 RISK AND CONSEQUENCE TYPES

Risk is defined as:

*Risk = [Probability of an adverse event occurring] × [The consequences associated with that event]*

The risk can be reduced either by reducing the probability of experiencing an incident (prevention) or by reducing the consequences of that incident should it occur (mitigation). A fundamental part of reducing the risk associated with MODU operations is to ensure that all parties, including owners, operators, regulators, etc., have a clear understanding of their "risk exposure."

The different types of consequences that are associated with MODU mooring failures are as follows:

1. health and safety;
2. environmental;
3. financial;
4. corporate reputation and image;
5. industry reputation and image;
6. national interest.

For MODU operations in the hurricane season where the MODU is evacuated and wells and pipelines are shut-in, health, safety, and environmental consequences associated with MODU mooring system failure are relatively low. Assessments of consequence types 4 through 6 will be subject to considerable corporate interpretation, and there will be large variations in risk tolerance. In the case of industry reputation (5) and national interest (6), the consequences depend on the performance of all MODUs operating in the Gulf of Mexico at any one time. The consequences of failure will include public and regulatory perception, which will be influenced by the number of MODUs that fail and the result of those failures on other industry infrastructure in a single hurricane, hurricane season, or few years.

While risk assessments may be performed for all six types of consequences, the one primarily addressed in this appendix is the third, financial. Other types of risk, namely health and safety and environmental, should be evaluated as required for the operation at hand. For example, if there is a significant risk for an environmental release of hydrocarbons from a drifting MODU colliding with a facility that stores hydrocarbons or dragging an anchor over a pipeline resulting in a release, then such possible environmental hazards should be considered in the assessment process. Additional information on the other types may be found in K.14.

### K.5.3 OVERVIEW OF RISK ASSESSMENT

The consequences (to infrastructure) of a MODU mooring failure depend on the density and type of subsea and surface infrastructure that surrounds the location of interest and, to some extent, on the type of MODU mooring system (e.g., the consequences of dragging chain over the seabed will be different from those due to dragged polyester). The risk assessment procedures described in this Appendix address the consequences of damage to surrounding infrastructure. For MODU operations in the hurricane season, where the MODU is evacuated, it is the responsibility of the Drilling Contractor and Operator to manage the risk associated with damage to the MODU and its mooring system, and to the Operator's drilling program.

An introduction to risk assessment methods and acceptance criteria (decision analysis) for performing different levels of risk assessment is provided in K.14. Recommended procedures and guidance are provided in K.6.

The potential consequences to infrastructure from a stationkeeping failure depend on:

- financial consequence values (including both the cost of replacement and lost production);
- distances and directions between individual components of infrastructure and the MODU's location;
- mitigation strategies;
- different likelihoods of adverse consequences given a mooring failure.

The probability of MODU mooring system failure decreases with increases in the design return period [the return period for which the mooring system satisfies all of the requirements of this document (intact and damaged line tension, anchor load, and clearance requirements)]. Generally, the management of risk to surrounding infrastructure requires that the design return period increases or additional mitigation measures be put into place as the consequence of failure increases, but the required return period is independent of the duration of the operation and the season of operation. However, for a given return period, the intensity of the environmental conditions (wind, wave, and current) is dependent on the particular site and the season(s) of operations.

The following example serves to illustrate that the return period is independent of duration.

An operator has two wells that need to be drilled near an existing facility with identical consequences. Each well will take one month to complete, and they will be drilled consecutively. If two independent Applications for Permit to Drill (APDs) are submitted, each for a drilling program of one month operation, then the design return period for each well should be the same as for a single APD for a MODU operating on the same location for a duration of two months. Clearly, different return periods for two one month APDs compared to a single two month APD is not a logical solution: the exposure risk for the facility is the same in both cases, so the design return periods should be the same. In effect, the daily risk to the infrastructure should be consistent, so the duration of an operation should not influence the design return period for MODU operations.

Figure K.3 shows the general methodology for carrying out a risk assessment for MODU operations when considering the potential consequences of mooring failure to the surrounding infrastructure.

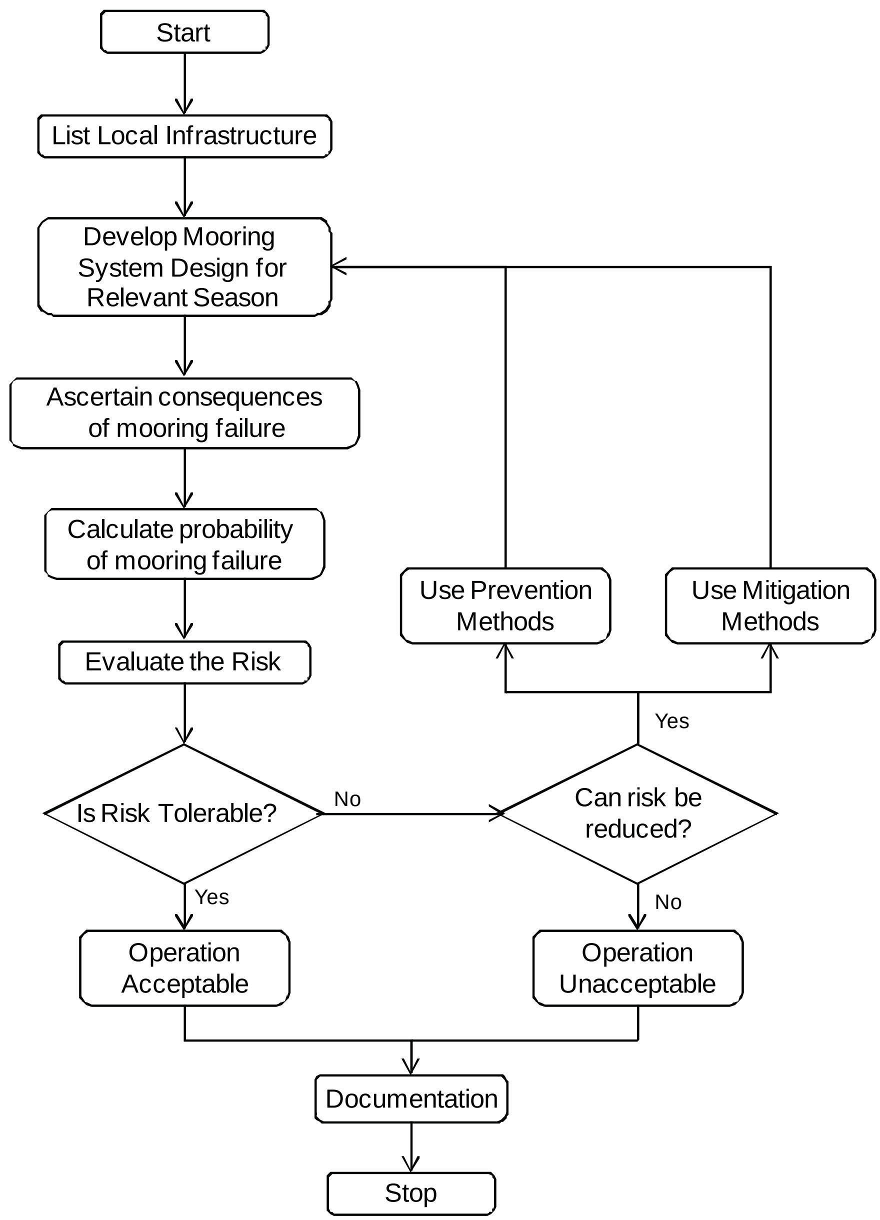



Figure K.3—Financial Risk Assessment—Overall Process

Tolerable risk levels should provide a balance between absolute safety requirements and costs and benefits of proposed risk reduction measures. Additional discussion and guidance on risk acceptance criteria and means to reduce risk are provided in K.14. In particular, changing operating season may be considered a prevention method that can reduce risk to acceptable levels. Documentation is the responsibility of the Operator and should include all necessary information to allow verification of the risk assessment.

## K.6 Assessment Procedures and Criteria

### K.6.1 GENERAL

This Appendix recognizes five consequence categories that depend on the type of MODU operation being performed and site characteristics. Two of the five consequence categories are associated with exceptional (see K.2.5) and atypical (see K.6.4) MODU operations. Three of the five consequence categories are associated with typical MODU operations and are:

C-1 lower consequences in event of stationkeeping failure;

C-2 intermediate consequences in event of stationkeeping failure;

C-3 higher consequences in event of stationkeeping failure.

For typical operations, an initial assessment and basic risk assessment shall be conducted for all locations as discussed below. A supplemental risk assessment should be conducted when necessary. The results of the basic or supplemental always override the results of the initial assessment.

An outline of the recommended risk based assessment process is as follows.

1. *Initial Assessment (required)*. The initial assessment process determines the unmitigated consequence category for the location to be evaluated based on distance and class of nearby infrastructure. In the initial assessment, the consequences categories listed in Figure K.4 are intended as a starting point for the basic risk assessment of the mooring system.
2. *Basic Risk Assessment (required)*. The basic risk assessment is used to establish the acceptability of the risk and the design return period. A basic risk assessment shall consider each of the following elements.
   a. *Infrastructure and Design Return Period Evaluation*. In determining the design return period, the actual infrastructure proximity shall be considered for a given moored MODU operation. Detailed evaluation of nearby infrastructure and potential to damage such infrastructure shall be conducted for all MODU operations. This evaluation includes assessing the MODU mooring system performance and the weak point analysis per K.3.3.
   b. *Operational Planning and Evaluation*. Additional issues that affect mooring system reliability and risk exposure should be evaluated. During peak hurricane season, lower risk locations should be given priority in operational planning. Adequate contingency plans shall be in place with operations near peak hurricane season if the risk levels during peak season are not tolerable.
   c. *Mitigation Evaluation*. As part of the risk evaluation, possible actions should be evaluated that can reduce the potential for mooring failure and consequence of failure. The design criteria referenced in this document are not intended to preclude reasonable and practical actions that can result in improved mooring systems.
3. *Supplemental Risk Assessment (as required)*. For higher risk locations or areas where more detailed assessment is warranted, a supplemental risk assessment should be conducted to determine suitability to drill with a given mooring system at a specific location. When a supplemental risk assessment is used for a typical MODU operation, it shall include considerations of elements from the basic risk assessment.

Sections K.5, K.13 and K.14 provide more detailed information, discussion, and guidance for evaluating site-specific consequences associated with MODU mooring failure and for assessing the risk of MODU operations.

### K.6.2 INITIAL ASSESSMENT PROCEDURE FOR TYPICAL MODU OPERATIONS

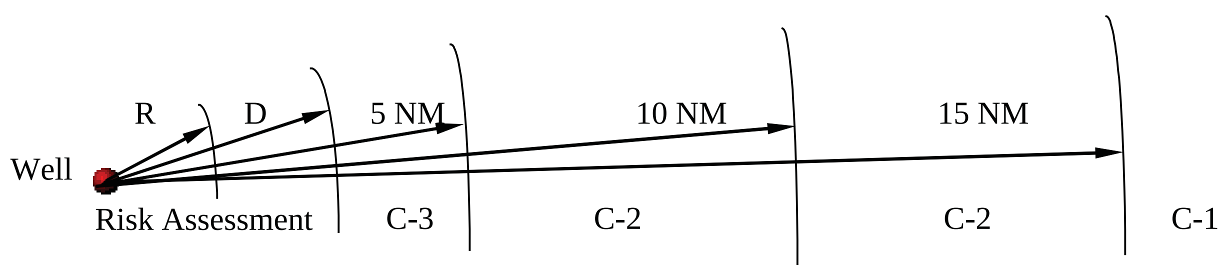
The initial assessment process determines the unmitigated consequence category for the location to be evaluated based on distance and class of nearby infrastructure.

The consequence category from the initial assessment depends on two factors: production rate and distance. A larger facility or pipeline represents a higher consequence at the same distance as a smaller facility or pipeline. Therefore, the design return period for a MODU operating in the proximity of a large facility or pipeline is higher than that of a MODU operating near a smaller facility or pipeline.

The greater initial investment and future tie-in potential for surface facilities dictates that the consequence assessment for surface facilities be based on rated production. Pipelines can be repaired in a shorter time and production potentially rerouted; therefore, the consequence assessment for pipelines may be based on actual throughput rather than rated capacity. If actual pipeline throughput is unavailable, a conservative estimate of pipeline throughput shall be used.

The initial assessment is a process to determine the relative consequence level of MODU operations. The initial assessment process works by determining the consequence category based on distance and capacity of nearby and most important infrastructure and then finding an approximate design return period for that consequence class. However, the initial assessment does not take into account multiple facilities or pipelines which would increase the consequence of a particular MODU mooring operation. Likewise, the initial assessment does not take into account mitigation measures that would reduce the consequence of the operations. The Operator must take these factors into consideration when conducting an initial assessment. The initial assessment may be useful in helping an Operator schedule operations or determine MODU mooring requirements. For example, a location that falls under the C-1 consequence category, the Operator may be able to use, after performing the required basic assessment, a MODU of opportunity at any time of the year. If a location falls under the C-3 consequence category, it is an indication that the Operator may have to take more care in selecting a MODU or in scheduling the operation for a more benign season of the year. Figure K.4 presents recommended unmitigated consequence categories based on distance to surface and subsea infrastructure. The consequence category for a given location may change for other assessment methods, e.g., basic and supplemental.The unmitigated consequence category for the location determined by the initial assessment is the highest consequence category for any facility or pipeline as determined from Figure K.4. The recommended design return period is obtained from Table K.1 based on the consequence category.

Figure K.4—Initial Assessment Unmitigated Consequence Categories for Typical MODU Operations




Note: *d* = distance to well center. *R* = mooring radius = distance between well and most distant anchor; *D* = mooring diameter = 2*R*

Surface Facilities

| Rated Capacity | $d \le R$ | $R < d \le D$ | $D < d \le$ 5NM | $5 < d \le$ 10NM | $10 < d \le$ 15NM | 15NM $< d$ |
|---|---|---|---|---|---|---|
| > 75K BOE/D | Risk Assessment | | C-3 | C-2 | C-2 | C-1 |
| 25 to 75K BOE/D | | | C-2 | C-2 | C-1 | C-1 |
| < 25K BOE/D | | | C-1 | C-1 | C-1 | C-1 |

Active Pipelines

| Actual Throughput | $d \le R$ | $R < d \le D$ | $D < d \le$ 5NM | $5 < d \le$ 10NM | $10 < d \le$ 15NM | 15NM $< d$ |
|---|---|---|---|---|---|---|
| > 75K BOE/D | Risk Assessment | C-3 | C-2 | C-2 | C-2 | C-1 |
| 25 to 75K BOE/D | C-2 | C-2 | C-2 | C-2 | C-1 | C-1 |
| < 25K BOE/D | C-2 | C-1 | C-1 | C-1 | C-1 | C-1 |
| Note: An "Active Pipeline" is defined as any pipeline located within the OCS that currently has throughput. Pipelines that are abandoned, cancelled, out of service, proposed, or relinquished are not active. However an active pipeline could be temporarily shut-in. Actual throughput should be based on all active pipelines in a corridor. When selecting the initial consequence category, due consideration should be given to the number of active pipelines and total throughput. Details of active oil and gas pipelines in the Gulf of Mexico are available at: http://www.gomr.mms.gov/homepg/pubinfo/freeasci/pipeline/freepipe.html and http://www.gomr.mms.gov/homepg/pubinfo/PI%20Catalog.pdf | | | | | | |

Table K.1—Consequence Category Return Period

| Consequence Category | Return Period, Years |
|---|---|
| C-1 | 10 |
| C-2 | see note |
| C-3 | 50 |

Note: In principle, the return period for C-2 and C-3 categories is a range. It is expected that the C-2 category covers most operations in the GOM and a 20-year return period should be suitable for the majority of locations. When the consequence of the MODU operation is close to the boundary between consequence categories, particular care needs to be taken to ensure that a suitable design return period is used.

### K.6.3 BASIC RISK ASSESSMENT PROCEDURE FOR TYPICAL MODU OPERATIONS

The purpose of a basic risk assessment is to facilitate planning and follow-up operations that reduce as much as practically possible the risk exposure from MODU mooring systems, as discussed previously.

A basic risk assessment should, as a minimum, qualitatively assess and fully document the following:

1. potential mooring failure modes (see K.14.2);
2. probability of mooring failure (see K.14.7);
3. nearby infrastructure and the potential for damage with various types of failure and consequence of damage;
4. operations plans and impact on analysis assumptions;
5. mechanical integrity of systems;
6. possible mitigation actions to improve reliability and reduce potential consequence of failure.

A basic risk assessment may be based on a methodology that determines a mitigated consequence score, which allows the stakeholders to assess, on a relative basis, the consequences of MODU mooring failure associated with the proposed operations. The intent of this approach is to be more conservative by comparison to a more detailed risk assessment. However, the basic consequence assessment can be completed with the routinely available information and data that should be available to the Operator and Drilling Contractor.

The basic consequence assessment should be based on:

1. consequence values based on location (infrastructure that could be damaged in the event of a mooring failure);
2. consequence factors based on mooring components and system details.

The Consequence Assessment Methodology, in K.13, is an example of a methodology that can be used to determine the mitigated financial consequences of a given MODU operation as part of a risk assessment. Guidance on acceptance criteria for risk assessment (i.e., acceptable return period) is provided in K.14.

Table K.2 shows a risk matrix based on consequence and probability of occurrence for hazardous events which can be used as part of a qualitative MODU mooring risk assessment. In situations with moderate risk, reasonable and practical actions should be evaluated that reduce the potential of mooring failure and consequences of failure.

Table K.2—Sample Risk Matrix

| Consequences | | Probability | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| | | Less Likely | | | More Likely |
| IV | High | | | | High Risk |
| III | | | Moderate Risk | | |
| I | | Low Risk | | | |
| I | Low | | | | |

### K.6.4 SUPPLEMENTAL RISK ASSESSMENT

For typical operations, a supplemental risk assessment may always be used to perform a more detailed assessment of specific risk elements of interest associated with the proposed operations.

For atypical MODU operations, a detailed risk assessment of all applicable mooring risk elements shall be performed.

The risk assessment process contains a series of steps to formally assess the risk at any given location. Due consideration should be given to the time required to complete this process. The steps can be summarized as:

1. definition of location and well parameters;
2. identification of local and distant infrastructure;
3. undertaking a hazard identification (HAZID) study;
4. determination of probability of mooring failure (mooring system reliability analysis, anchor holding capacity uncertainty, etc.);
5. quantification of the consequences of failure (e.g., through event tree analysis);
6. risk mitigation;
7. documentation.

Further information on risk assessment methods is provided in K.14.

The risk assessment entails a documented and structured identification of options available, impact of these options, and leads to the selection of the lowest consequence mooring system available, and is a valuable tool in designing the mooring system.

### K.6.5 ATYPICAL MODU OPERATIONS

There may be atypical MODU operations associated with exceptionally high consequences that may require very high environmental return periods. MODU operations that may be associated with exceptionally high consequences include but are not limited to:

- MODU offset drilling;
- tender assisted drilling adjacent to a permanent facility;
- MODU operations within a mooring radius of a permanent surface facility.

Such operations are subject to a detailed supplemental risk assessment to determine if the operation is acceptable (see K.6.4).

## K.7 Mooring System Improvement

There are various options to improve the survivability of the mooring system and reduce the consequences of a mooring failure under hurricane conditions, such as the use of higher strength components, additional mooring lines, and steel or fiber rope inserts. These options have design and hardware issues that require special attention, as discussed below.

### K.7.1 HIGHER STRENGTH COMPONENTS AND ADDITIONAL LINES

Replacement of existing chain and wire ropes with higher strength components may be considered.

Additional lines may also be placed on a MODU to increase mooring system strength. The additional lines may be terminated in a number of ways, such as:

- standard fairlead and tensioning equipment with full tensioning capability;
- alternate fairlead and tensioning equipment with limited tensioning capability;
- fixed terminations with no tensioning capability.

The following items may be affected by the additional lines or the lines with stronger components:

- required anchor holding capacity;
- required stall and brake load capacity of winch/windlass;
- global structural strength of the vessel;
- local structural strength of the tensioning equipment frame and foundation;
- local structural strength of the fairlead foundation and support structure;
- vessel variable deck load and loading conditions;
- vessel stability if new downflooding points are introduced by the mooring modifications;
- available space.

### K.7.2 FIBER ROPE INSERT

Fiber rope (polyester or HMPE) sections may be inserted in the existing mooring line to improve mooring performance and mitigate the potential for damage due to lines dropped on or dragged over subsea equipment. The selection and design of such systems shall be based on mooring analysis using a proper fiber rope stiffness model. The rope should be protected by soil blocking devices such as a filter or jacketing since contact with seafloor is possible under hurricane conditions due to large offsets, anchor drag or line failure.

### K.7.3 FIBER ROPE AND STEEL WIRE ROPE INTERACTION—STRENGTH AND FATIGUE CONSIDERATIONS

The torque relationship between fiber and steel mooring components requires special consideration to ensure the mooring system performance is not compromised.

#### K.7.3.1 Strength Considerations

For MODU mooring systems, Reference 2 indicates that neither the strength of a fiber rope nor the strength of a 6- or 8-strand wire would be degraded by any noticeable amount when they are connected in the same line for the duration of a severe hurricane.

#### K.7.3.2 Fatigue Considerations

Typically, no fatigue design analysis is required for a MODU mooring system. However, laboratory testing demonstrates that a 6- or 8-strand wire rope's fatigue performance, when connected with a torque-neutral fiber rope, could be significantly degraded (Reference 3) although the scale effect of such testing is yet to be quantified. Industry experience indicates that there are at least two viable design approaches to address this issue.

1. **Torque-matched approach**
   A steel wire rope's fatigue life is best preserved by connecting to a torque-matched rope. A rope is considered torque-matched if its torsional characteristics over the design load range are essentially the same as that of the connected wire rope. Due to the inherent difference in material properties, a fiber rope typically can only match a wire rope's torsional characteristics at a pre-determined tension level. The difference between the torque of the fiber rope and wire rope increases as the line tension deviates from the match point with changing environmental loading or heading. Other factors to be considered in the torque-matched design include torque characteristics, lay direction, presence of swivels in the mooring lines, swivel lock-up load, and the presence and length of chain segments.
2. **Non-torque-matched approach**
   Available torque-neutral fiber ropes can be used for short term MODU mooring systems if the dynamic torsion of the steel wire could be restrained at the interface between the fiber rope and wire rope (Reference 2). A properly designed submersible buoy could provide such restraint. Available experience shows that wire fatigue damage in such a system is lower than some of the earlier scaled test data suggests (Reference 2).

The fatigue damage to wire rope tends to be concentrated near the interface with the fiber rope. The wire rope can be returned to service after the damaged end is re-terminated during a MODU move. It is also possible to insert a short connecting wire rope (200 ft to 300 ft) between the MODU mooring wire and the fiber rope to minimize the need for re-socketing wire ropes in the field.

## K.8 Anchor System Considerations

### K.8.1 GENERAL

The anchor system plays an important role in hurricane survivability of the mooring system and the consequence of mooring failure. Consideration should be given to alternative anchor types, where necessary, to achieve adequate performance and mitigate consequences of failure. Anchor handling vessel and MODU winch system capabilities should be considered in selecting the best anchor option.

Drag anchors are commonly used for catenary moorings, while fixed anchors such as suction piles or normally loaded plate anchors, including drag or direct embedded plate anchors, are often used for taut or semi-taut moorings. Drag anchors of the heavily loaded lines may move a short distance (tens to hundreds of feet), causing redistribution of the mooring load among the mooring lines. This redistribution of load may help the mooring system survive. However, for locations where pipelines, subsea trees, manifolds or other subsea infrastructure exist, excessive anchor movement can cause damage to these infrastructure elements.

The use of fixed anchors may increase the likelihood of mooring line failure under similar conditions because redistribution of mooring load cannot be achieved.

For all types of anchors, behavior and performance under severe loading must be understood to assess and mitigate the risk of moored MODU operations.

### K.8.2 ANCHOR HOLDING CAPACITY, SAFETY FACTORS AND INSTALLATION REQUIREMENTS

Anchor holding capacity for the types of anchors being used shall be considered in the design of the mooring system. Anchor installation requirements should be included in anchor type selection consideration, especially when the anchor is to be installed near sea floor infrastructure and where an adequate safety zone should be maintained around the infrastructure during anchor handling. Anchor selection and safety factors should consider capacity, availability, and potential to minimize damage to subsea infrastructure should an anchor failure occur in conditions such as but not limited to:

- a marine installation such as a pipeline lies in the dragging path of an anchor or in the potential dragging path, i.e. a location such that mooring system failure could result in an anchor dragging across the installation;
- a mooring line that can cross another mooring line;
- density or importance of seafloor or water column infrastructure that merits a higher safety factor than those stated in Tables 6 and 7.

Unless site-specific soil data are available, appropriate upper and lower bound soil conditions for the general area of operation shall be considered. Any evaluation of anchor holding capacity should take into consideration the uncertainties of the local soil strength profile and other geotechnical properties.

### K.8.3 DRAG ANCHOR

Drag anchors should have a proven performance or be closely similar to an anchor with proven performance. Performance may be proven through scale testing, field tests, etc. Drag anchors should be in an undamaged condition to preserve symmetry and hence holding capacity.

When drag anchors are used for a MODU mooring operation, they should be test loaded to ensure the anchor is right side up and sufficient embedment is achieved.

#### K.8.3.1 Windward Line Loading

Due to equipment limitations for MODU operations, a drag anchor is typically subjected to a test load below the maximum storm load. When the anchor experiences loads higher than the test load and uplift angles are within the anchor's design tolerance, a properly set anchor will typically penetrate deeper, developing higher holding capacity. When the storm load exceeds the anchor holding capacity and uplift angles are within the anchor's design tolerance, the anchor will stop penetrating and move horizontally below the seafloor. In this process the mooring line either breaks due to overloading or remains intact due to mooring load redistribution or storm passage. If the uplift angle exceeds the anchor's design tolerance, then the anchor may lose holding capacity, lose penetration and may drag to the surface.

Note: In the 2004 and 2005 hurricane seasons, anchor drag distances for the windward lines were typically less than one mile. However, on some occasions, windward anchors dragged over 20 miles off location.

#### K.8.3.2 Out-of-plane Loading

When several windward lines fail, resulting in large directional changes of the remaining lines, out of plane loading at the anchor shackle may occur. Although anchor behavior under this loading condition is still a subject of research, there is evidence suggesting that drag anchors will rotate to a new orientation and maintain their holding capacities. Under this loading condition, a pipeline or subsea equipment that was not originally in the dragging path of an anchor may become in the dragging path due to change of line direction. Consequently, the site-specific assessment should account for the possibility of anchors dragging in directions other than towards the center of the mooring pattern.

When windward lines fail, some drag anchors on the leeward lines may be subject to reverse loading. These anchors may be pulled out and dragged some distance. Some drag anchors may re-embed, limiting the drag distance.

Note: Industry experience in the 2004 and 2005 hurricane seasons shows that most of the leeward and side anchors stayed in the vicinity of their original locations.

#### K.8.3.3 Oversized Anchor

When drag anchors are oversized for a MODU operation to protect the surrounding structures, they should be test loaded to ensure the anchor is right side up and sufficient embedment is achieved. If the anchors are conventionally set, the MODU must have sufficient winch capacity to apply the required test load.

### K.8.4 PLATE ANCHOR

The behavior of drag embedded and direct embedded plate anchors for MODU operations must be understood in order to determine suitability of the anchor for the intended operation.

#### K.8.4.1 Fluke Angle Setting

Drag embedment plate anchors may have several options for fluke angle setting: embedment, near-normal, and normal. In the embedment or near-normal fluke angle setting, the plate anchor behaves as a drag embedment anchor. The smaller embedment fluke angle is generally used to obtain initial anchor penetration, changing to the larger near-normal fluke angle allows even deeper penetration. In the near normal setting, the anchor may behave as a plate anchor under design loading conditions, but under overload conditions the anchor can drag, penetrate deeper and reach a new equilibrium depth with a higher holding capacity. In the normal setting, the anchor ultimately behaves as a fixed plate anchor, and overloading will either result in failure of the mooring line or cause the anchor to pull out. Selection of these options should be based on evaluation of the specific MODU operation.

Some direct embedment plate anchors have also demonstrated diving behavior. Diving behavior is a result of an eccentricity between the line of action of the mooring line and the center of soil pressure on the fluke. In an overload condition, plate anchor movement through the soil will cause the fluke to tilt with respect to the mooring line direction developing an effective near-normal shank or fluke angle.

#### K.8.4.2 Triggering the Anchor and Ultimate Holding Capacity

Drag embedment plate anchors typically have two operating modes: an installation mode and a normal or near-normal loading mode. In the installation mode, depending on the consistency of the soil, the load is applied at an angle of 40º to 60º to the fluke. After failing a shear pin or triggering the anchor, the load becomes perpendicular (normal) or nearly perpendicular (near normal) to the fluke.

Design holding capacity should be based on rigorous anchor design and installation analysis for a defined set of upper and lower bound soil conditions and installation loads. Once a normally loaded anchor is triggered the holding capacity will always be greater than the installation load with relatively small anchor movement. The actual holding capacity will depend on anchor and mooring line dimensions, the sensitivity of the soil, and the change in load direction. Guidance on installation analysis can be found in DNV RP E302 (Reference 4).

#### K.8.4.3 Out-of-plane and Reverse Loading

Some drag embedment plate anchors for MODU moorings are designed to be retrieved by loading in the reverse direction to operate a release mechanism, permitting low load recovery by the mooring line. These plate anchors cannot resist reverse loading and therefore may also have limited resistance to out of plane loading. However, where the risk of damaging pipelines and subsea equipment by anchors needs to be minimized, the reverse retrieval device can be disabled temporarily to provide reverse loading capability during the hurricane season, with recovery being achieved by means of a drogue tail or submerged buoy attached to the anchor fluke. These anchors may be set from the MODU or preset.

Plate anchors with a normal loading fluke angle setting that have the retrieval device temporarily disabled or do not incorporate a reverse retrieval device will therefore have capability of resisting out of plane and reverse loading.

Care should be taken when using drag embedment and direct embedment plate anchors designed with near-normal features; they may lose capacity if rotated approximately 90º in the vertical plane after windward mooring line failure and leeward line direction change as the MODU drifts off location over a leeward anchor.

### K.8.5 SUCTION PILE

Suction piles have been observed to fail at the padeye due to a combination of tension and excessive out-of-plane bending. The out-of-plane bending occurs due to large vessel offset after first and subsequent line failures. Consideration should be given in the padeye design to applying the breaking load of the mooring line at any angle.

### K.8.6 SOIL CONDITIONS

Unless site-specific soil data are available, appropriate upper and lower bound soil conditions for the general area of operation shall be considered. However, caution should be exercised at locations where unusual soil conditions beyond the notional bounds may be encountered—e.g., underconsolidated or weak soil, shallow cementation, sand layers and overconsolidated or hard soil. Unusual soil conditions may be identified at specific locations by interpretation of 3-D seismic data, usually analyzed in support of EP submissions for exploration drilling—see 30 *CFR* Part 250, Subpart B, and NTL 2006 G14. Features that may be interpreted from 3-D seismic data that provide evidence for unusual soil conditions include (but are not limited to):

- shallow gas;
- erosion features, such as canyons and furrows;
- shallow mass transport deposits;
- seafloor expulsion features;
- seafloor faults.

Continental shelf areas, where interpretation of 3-D seismic data for shallow geologic features is extremely limited, may warrant dedicated site surveys and soil sampling where data for the general area of operation are sparse.

## K.9 Hurricane Preparedness

### K.9.1 GENERAL

This section addresses specific mooring related issues that are part of a hurricane preparedness plan. The overall hurricane preparedness plan should include suitable provisions for other activities, such as personnel evacuation and suspension of drilling activities.

### K.9.2 PREPAREDNESS OVERVIEW

#### K.9.2.1 Hurricane Preparedness Plan

The hurricane preparedness plan shall be a written plan and should address as appropriate the following mooring specific items:

- ballasting operations;
- repositioning the vessel to a more favorable storm safe position within the already set anchor positions;
- mooring line payout and/or tension adjustments to optimize the mooring's storm survivability;
- engaging storm survival brakes and stoppers or securing and dogging winches;
- optimum mooring pattern and positions to maximize mooring performance;
- provision of sufficient battery power, computer disc storage space, etc., to ensure that critical systems, including MODU trackers, remain operational from the time the crew disembarks until the time the crew re-boards the MODU;
- confirmation that towing bridles or lines, navigation aids, and position tracking devices are installed and functional.

The hurricane preparedness plan should also include a schedule that reflects the time required to complete necessary mooring activities, operations to secure the well and the MODU, evacuate the crew to a safe location and allow for some contingency time.

#### K.9.2.2 MODU Recovery

All units should be prepared to the extent feasible for towing. Each MODU should be equipped with a primary and secondary tow line or bridle.

#### K.9.2.3 Contingency Planning

Contingency plans shall address operations identified as critical to both hurricane survival and resumption of normal activities. The contingency plans shall address the need to have suitable personnel available to respond to the problem at hand. For example, if a mooring winch is inoperable and cannot be repaired, then it is necessary to have a mooring analyst determine suitable payouts and pretensions on the remaining lines in order to maximize survivability.

### K.9.3 LOOP AND EDDY CURRENTS

When a MODU is in a loop or eddy current, the drilling contractor or operator shall determine the mooring line adjustments required to abandon the MODU in a condition that provides its best chance of riding out the storm with due consideration to the anticipated surface current velocity and direction.

The drilling contractor or operator should obtain the following information:

- existing line payouts and tensions;
- stall capacity of the winches;
- latest measurements of the currents, particularly velocity and direction at the sea surface;
- forecasts of the loop/eddy current velocity and direction.

The drilling contractor or operator should determine the optimum line payouts and pretensions that serve to maximize intact mooring line safety factors without exceeding equipment limits or endangering human life. The environmental conditions used for analysis should include the following weather combinations:

- omnidirectional hurricane metocean criteria;
- hurricane-driven surface currents vectorially added to the local loop or eddy current;
- the payouts and pretensions updated as surface current velocities or headings change.

### K.9.4 MODU TRACKERS

Satellite location transponders should be installed and tested on board all moored MODUs operating in the Gulf of Mexico. These transponder systems should be function tested prior to hurricane season to ensure the system is functioning properly. Sufficient care should be given to ensure these systems have adequate battery backup to enable the transponders to function after the MODU has been abandoned for a minimum period of seven days. Sufficient battery life should allow for reasonable assurance that the system will be operational through a given cyclonic storm event and for a period of time after potential passage of the storm, to allow for speedy recovery operations in the event of mooring failure. The tracker system should be fully operational with seven day capacity within 48 hours of reboarding the MODU.

Redundancy in systems should be considered.

### K.9.5 POST-STORM REPORTING

Section K.15 contains a form that may be utilized to capture the MODU particulars and any storm related consequences. Completion of the appropriate sections of this form immediately after installation is recommended.

Every reasonable effort should be made to retain, preserve, and label the failure surface of any failed mooring line component for future examination. The label should include: site name, failure date, MODU name, line number, location along the line, and component serial number if applicable.

## K.10 Mooring Installation

### K.10.1 MOORING INSTALLATION PLAN

The mooring system for a specific site should be deployed according to an installation plan that specifies a number of items related to the mooring design:

- MODU heading;
- mooring line headings, including installation tolerance;
- anchor locations, including installation tolerance;
- line segment lengths and composition;
- pretensions;
- anchor test loads.

The installation plan should also include information on:

- minimum anchor handling vessel (AHV) specification (bollard pull, winch capacity and pull, and any other equipment requirements);
- maximum sea states for safe operations;
- weather window requirements (i.e., duration of installation activities);
- weather forecast requirements;
- contingency and management of changes to the plan.

### K.10.2 AS-INSTALLED MOORING SYSTEM INFORMATION

Once the installation is completed, it is the Operator's responsibility to ensure that the information on the as-installed mooring system is recorded and transmitted, as applicable, in a timely fashion. This information should be provided to all relevant parties, including the drilling contractor, for post installation verification, operating the mooring system, and planning for evacuation. Completion of K.14 may facilitate recording most of this information.

This information can be used for a number of purposes.

- verify that the mooring system is installed within design tolerances;
- verify that any deviations from the design tolerances will not have a negative impact on mooring system performance.

As a minimum, the as-installed information shall include the following:

- Global geometry:
  - MODU heading and global position;
  - individual line headings;
  - initial and final anchor locations.
- Mooring composition:
  - length, general condition, composition and location of all mooring line sections;
  - number, location, general condition, and type of connectors (e.g., shackles, connecting links, subsea connectors, etc.);
  - anchor type, size, general condition, serial number, and fluke angle, as applicable.
- Anchor test load:
  - test load at fairlead;
  - estimated test load at anchor shackle;
  - estimated anchor drag distance.
- Mooring pretension:
  - pretension or line angle at fairlead, and estimation of accuracy.

### K.10.3 POST INSTALLATION VERIFICATION

Based on the information specified in K.10.1 and K.10.2, the operator and drilling contractor shall verify that the as-installed mooring meets the original safety factor requirements. If the as-installed mooring system does not meet the design safety factor requirements, then appropriate plans should be developed and implemented in a timely fashion that will provide acceptable mooring safety factors.

## K.11 Gulf of Mexico Hurricane Criteria

### K.11.1 GENERAL

Guidance on development of metocean extremes are contained in API Bull 2INT-MET, including Addenda A and B. This section provides a summary and parameterization of the deepwater Gulf of Mexico hurricane criteria contained in API 2INT-MET specifically tailored for use in performing site assessments of MODUs. In all cases, the criteria contained in API 2INT-MET take precedence over the summary presented in this section.

### K.11.2 SELECTION OF METOCEAN CRITERIA

The consequences to surrounding infrastructure of mooring system failure determine the required minimum design return period. The flow chart shown in Figure K.5 describes a method for selection of metocean criteria. API 2INT-MET provides default metocean criteria that are intended to be conservative for the Gulf of Mexico and may be used instead of site-specific criteria.

For operations during the peak hurricane season, the wind, wave, and current conditions used for site assessment of typical or atypical operations shall not be less than those associated with a threshold Category 1 hurricane. During the pre- and post-peak seasons, this restriction may be eased subject to the constraints listed in K.4.2. This minimum Category 1 hurricane condition is not required if the metocean conditions associated with the required return period are more severe.

### K.11.3 BACKGROUND TO DEEPWATER GULF OF MEXICO HURRICANE CONDITIONS

Hurricane season in the North Atlantic Basin officially runs from June 1st through November 30th, with the most severe storm activity generally occurring in August, September and October. The storms which occur in these three months effectively control

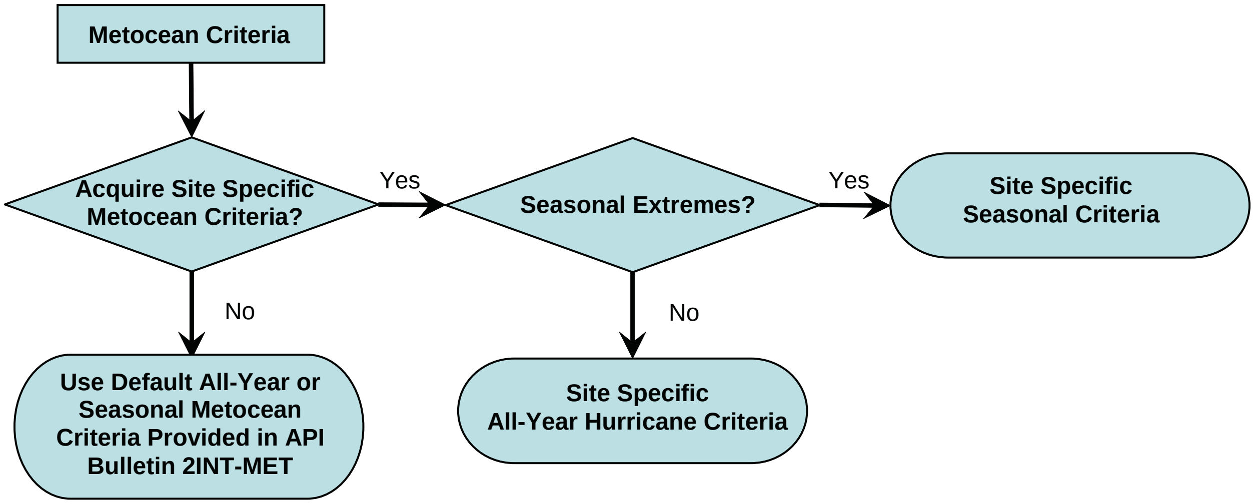



Figure K.5—Metocean Criteria Flowchart

the annual (all-year) hurricane extremes; extremes derived just considering storms which occur in these three months will be essentially identical to extremes derived using the full population of storms irrespective of month. The severe months are preceded by a period of moderate cyclone activity during June and July and then followed by a period of rapidly decreasing cyclone activity from the end of October through November. While rare, tropical storms have formed or entered in the Gulf of Mexico in both May and December, outside the official hurricane season.

The regional conditions presented in API 2INT-MET have been derived assuming an exposure period to hurricane encounters over the full year. Should a facility operate in such a manner as to restrict its exposure to hurricanes in the Gulf of Mexico (or one of the regions in the Gulf of Mexico) to periods less than one year, i.e., a seasonal operation, it would be reasonable to consider the facility subject to hurricane conditions derived from a limited exposure period.

A set of seasonal hurricane conditions for water depths greater than or equal to 300 m (984 ft) are provided in Addendum A to API 2INT-MET, for each of the four Gulf of Mexico regions described in Section 3 of API 2INT-MET. Conditions are provided for a pre-peak season period, covering June 1st to August 1st, and a post-peak season period, covering October 21st through November 30th. Peak hurricane season is considered to cover the period from August 14th through October 7th; during this period, the hurricane conditions from API 2INT-MET, Section 4.5, i.e., the annual full-population conditions, should be used. For the periods between August 1st to August 14th, and between October 7th and October 21st, conditions should be derived by linearly interpolating over two-week ramp periods between the full-population conditions in API 2INT-MET and the pre- and post-peak conditions presented in Addendum A to API 2INT-MET. For the 1-minute mean wind speed at 10 m above mean water level this is illustrated in Figure K.6.

The following conditions apply to the Gulf of Mexico hurricane criteria summarized in this section.

- The conditions presented in this section are for water depths of 300 m (984 ft) or greater in the regions covered by API 2INT-MET. They should not be interpolated or extrapolated to shallow water.
- The seasonal conditions are for the full population of pre- and post-peak tropical cyclones. They do not include winter storms, which should be treated as a separate storm population with its own set of derived extremes. Some of the extremes presented in this section, particularly in the post-peak period, may not represent the highest storm-driven n-year wind or wave conditions which could be encountered in the periods described.
- The pre- and post-peak conditions summarized in this section should be treated as complete load cases, and the wind, waves, and current should be treated as omni-directional. That is, the factors provided in Sections 4.2.2 and 5 of API 2INT-MET should not be used with the seasonal information in this section; however, the seasonal wave conditions should not be higher in any given direction than the appropriate Section 4.5 of API 2INT-MET independent extreme waves adjusted for direction.

For MODU operations planned to take place in the pre-peak (ending before August 1st) or post-peak (starting after October 21st) the following should be considered.

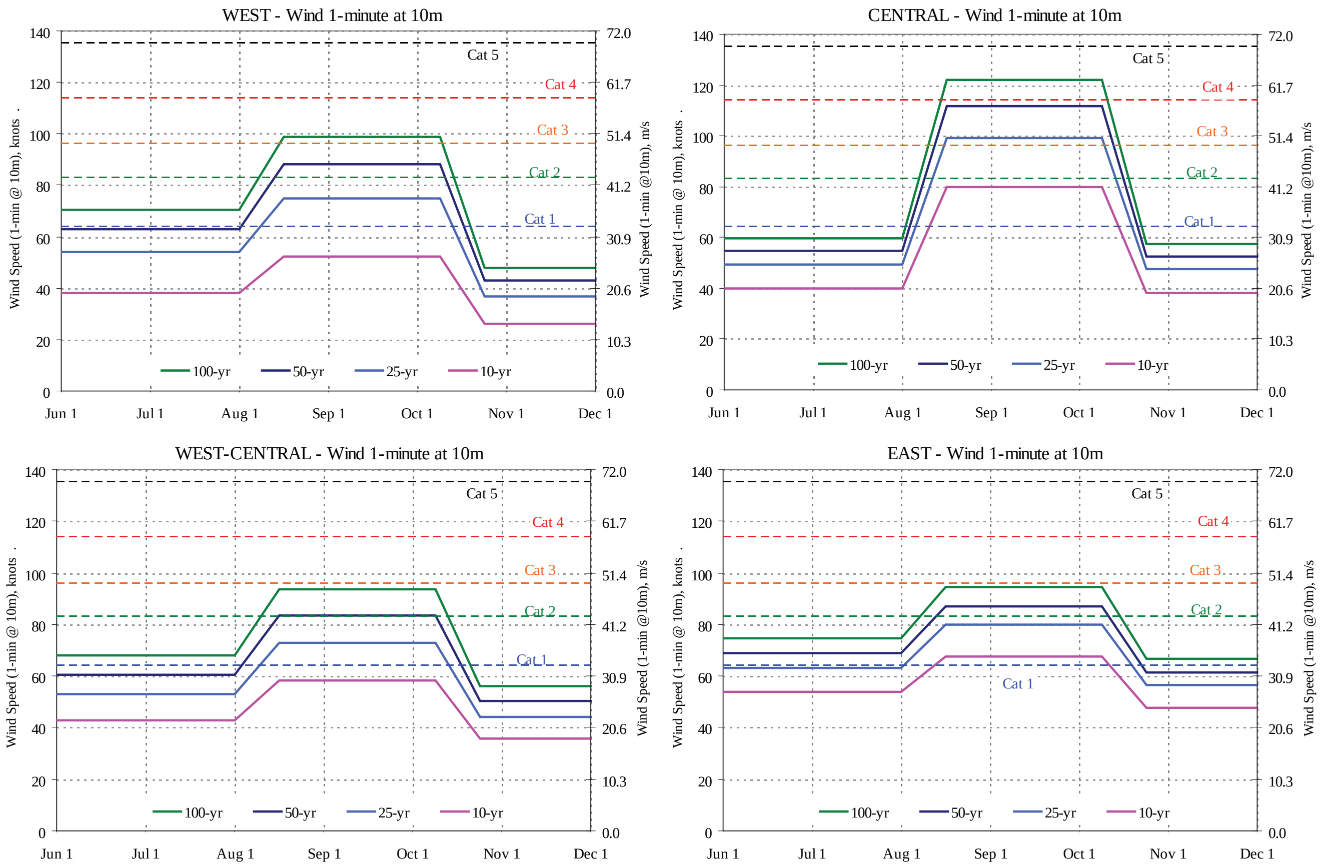



Figure K.6—Deepwater Seasonal Hurricane Wind Speeds (1-minute at 10 m) for Four Regions [API 2INT-MET and 2INT-MET Addendum A]

- Planning for operations in the pre-peak hurricane season should consider the possibility of delayed completion due to late arrival of equipment at the beginning of the operation, delays due to Loop current intrusions, and delays due to tropical storm occurrences. Wind, waves, and current corresponding to the latest likely completion date should be used in planning.
- Planning for operations in the post-peak hurricane season should consider the possibility of an early start due to early availability of equipment. Wind, waves, and current corresponding to the earliest likely start date should be used in planning, or the operator should be prepared to delay the start until it is clear that no hurricane will approach the Gulf in the next few weeks.

Addendum A to API 2INT-MET contains guidelines and recommendations for the derivation of seasonal hurricane conditions in the Gulf of Mexico, and should be followed when a site-specific study is performed.

### K.11.4 SUMMARY OF API BULLETIN 2INT-MET HURRICANE CRITERIA FOR DEEPWATER MODU SITE ASSESSMENT

API 2INT-MET provides all-year hurricane criteria for four regions with transition zones between the regions. The four regions are described in Figure K.7 and defined as:

- West, between longitude 97.5º W and 95º W;
- West-Central, between longitude 94º W and 90.5º W;
- Central, between longitude 89.5º W and 86.5º W;
- East, between longitude 85.5º W and 82.5º W.

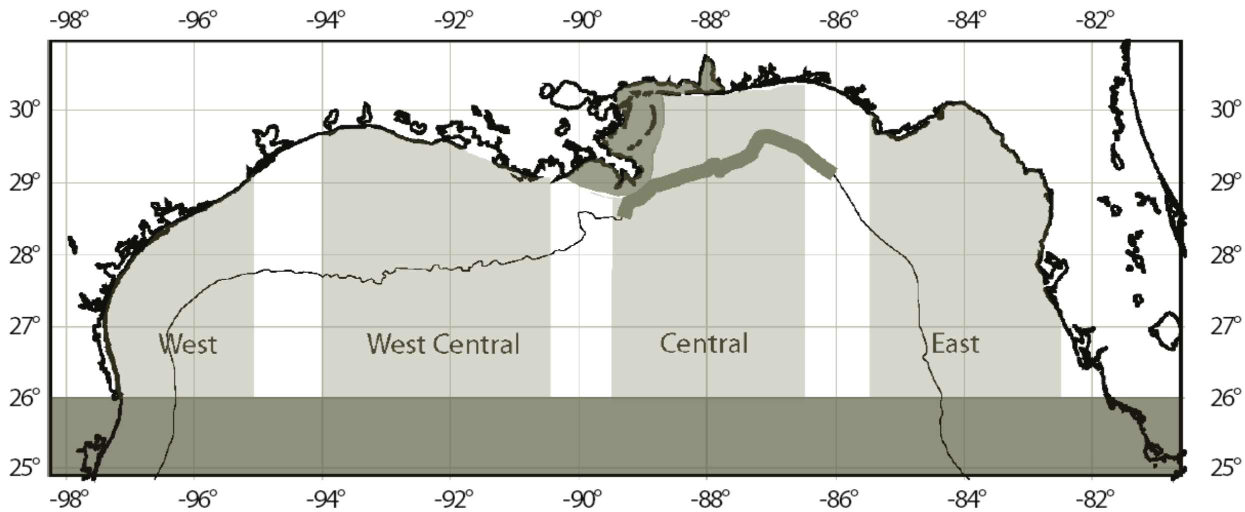



Figure K.7—Four Gulf of Mexico Regions [API 2INT-MET]

Between each region are areas of transition (unshaded), 1º longitude wide. Conditions for these transition areas should be derived by linearly interpolating between the values of the two adjacent regions across the width of the transition, see API 2INT-MET.

In addition Addendum A to API 2INT-MET divides the hurricane season into three parts:

- Pre-Peak hurricane season from June 1st to August 1st;
- Peak hurricane season from August 14th to October 7th;
- Post-Peak hurricane season from October 21st to November 30th.

Two week transition periods separate the pre-peak, peak, and post-peak parts of the hurricane season as shown in Figure K.6.

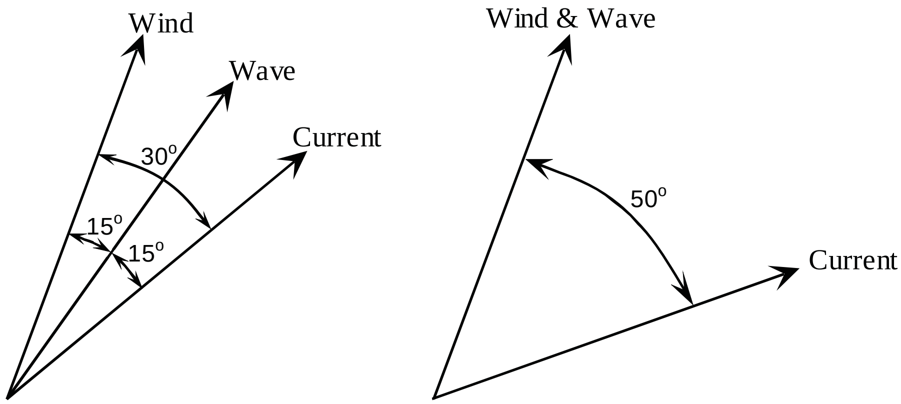
Tables 4.5.1-1A and 1B, 4.5.2-1A and 1B, 4.5.3-1A and 1B, and 4.5.4-1A and 1B of API 2INT-MET contain the independent extremes for the 10 to 10,000 year return period hurricane winds, waves, currents, and surge. For the site assessment of MODU mooring systems in deepwater a sub-set of the parameters provided in API 2INT-MET are required. Table K.3 illustrates the parameters required for the site assessment of MODU mooring systems in deepwater.

Table K.3—Central Region—All-Year Independent Extremes for Deepwater MODU Site Assessment

| Return Period (years) | 10 | 25 | 50 | 100 | 200 |
|---|---|---|---|---|---|
| $V_{1\text{-}hr}$, 1-hr average wind @ 10 m, knots | 64.20 | 78.00 | 86.30 | 93.30 | 99.10 |
| $V_{1\text{-}min}$, 1-min average wind @ 10 m, knots | 79.70 | 99.30 | 111.60 | 122.10 | 131.00 |
| $H_s$, significant wave height, ft | 32.80 | 43.60 | 48.60 | 51.80 | 54.10 |
| $T_p$, peak period, s | 13.00 | 14.40 | 15.00 | 15.40 | 15.70 |
| Surge, ft | 1.05 | 1.71 | 2.17 | 2.62 | 3.05 |
| $V_{cs}$, current, surface speed, knots | 3.21 | 3.89 | 4.32 | 4.67 | 4.96 |
| $V_{cm}$, current, mid-depth speed, knots | 2.41 | 2.92 | 3.25 | 3.50 | 3.71 |
| $D_0$, current, zero speed current depth, ft | 227.40 | 276.20 | 305.80 | 330.70 | 351.40 |

Table 5-1 of API 2INT-MET contains factors, for deepwater, to be used with the independent extremes for developing peak-wind, peak-wave, and peak-current cases for return periods between 10 and 10,000 years. Table 5-1 also provides the relative directions between wind, wave, and current for the three peak cases, and these are summarized in Figure K.8.

Addendum A to API 2INT-MET contains tables summarizing seasonal (pre- and post-peak) deepwater Gulf of Mexico hurricane wind, waves, currents, and surge for the four regions and for return periods between 10 and 10,000 years.




(a) Peak-wind and Peak-wave Cases (b) Peak-current Case

Figure K.8—Directional Relationship for Peak Wind, Wave and Current Cases [API 2INT-MET]

In performing site assessments of MODU mooring systems return periods other than the 10, 25, 50, 100, etc. provided in API 2INT-MET are usually required. To assist in developing metocean parameters of interest for a given return period, the following equation may be used:

$$E_R(\varepsilon, \alpha, \beta) = E_{10}\left\{\varepsilon - \alpha\left[-\ln\left(1-\frac{1}{R}\right)\right]^{\frac{1}{\beta}}\right\} \text{for } 10 \le R \le 200 \tag{K.1}$$

where

$E_R$ = $R$-year return period value of environmental parameter,

$R$ = return period (years),

$E_{10}$ = 10-year return period value of environmental parameter,

$\varepsilon$ = threshold parameter,

$\alpha$ = scale parameter,

$\beta$ = shape parameter.

For the various load cases the parameters, $\varepsilon$, $\alpha$, $\beta$, and $E_{10}$ are given in Table K.4.

In all cases, the NPD spectrum shall be used to describe the frequency content of the wind energy, and the JONSWAP spectrum shall be used to describe the frequency content of the wave energy. The peak enhancement factor, $\gamma$, should be in the range of 2.0 to 2.5 for hurricane seastates.

The relationship between 1-hour and 1-minute wind speeds at 10m above mean water level, based on the NPD spectrum, is:

$$V_{1\text{-min}} = V_{1\text{-hr}}\,(1.10070 + 0.004331V_{1\text{-hr}}), \text{ where } V_{1\text{-hr}} \text{ and } V_{1\text{-min}} \text{ are in m/s}$$

$$V_{1\text{-min}} = V_{1\text{-hr}}\,(1.10070 + 0.002228V_{1\text{-hr}}), \text{ where } V_{1\text{-hr}} \text{ and } V_{1\text{-min}} \text{ are in knots} \tag{K.2}$$

$$V_{1\text{-min}} = V_{1\text{-hr}}\,(1.10070 + 0.001320V_{1\text{-hr}}), \text{ where } V_{1\text{-hr}} \text{ and } V_{1\text{-min}} \text{ are in ft/s}$$

## Table K.4—Parameters for fits to API 2INT-MET Gulf of Mexico Hurricane Criteria

| Season | Load Cases | Parameters | West | | | West-Central | | | Central | | | East | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Wind 1-hr @10m | Wave Hs | Current Surface | Wind 1-hr @10m | Wave Hs | Current Surface | Wind 1-hr @10m | Wave Hs | Current Surface | Wind 1-hr @10m | Wave Hs | Current Surface |
| All-year wind, wave, and current (peak season, Aug. 14 to Oct. 7) | Peak Wind Case | ε | **2.439** | 6.691 | 2.038 | **6.577** | 2.026 | 2.747 | **2.042** | 1.766 | 1.615 | **2.126** | 4.462 | 1.481 |
| | | α | **3.011** | 6.619 | 2.726 | **6.137** | 1.762 | 2.294 | **1.804** | 2.116 | 1.449 | **1.612** | 3.910 | 1.047 |
| | | β | **3.048** | 14.903 | 2.332 | **23.526** | 4.162 | 8.262 | **4.101** | 2.216 | 2.625 | **6.266** | 18.477 | 2.895 |
| | | $E_{10}$ (m/s, m, m/s) | **22.5** | 6.8 | 0.90 | **24.9** | 8.1 | 1.00 | **33.0** | 10.0 | 1.32 | **28.4** | 8.2 | 1.14 |
| | | $E_{10}$ (kt, ft, kt) | **43.7** | 22.3 | 1.76 | **48.4** | 26.6 | 1.94 | **64.2** | 32.8 | 2.57 | **55.2** | 26.9 | 2.21 |
| | | $E_{10}$ (ft/s, ft, ft/s) | **73.8** | 22.3 | 2.97 | **81.7** | 26.6 | 3.28 | **108.3** | 32.8 | 4.33 | **93.2** | 26.9 | 3.73 |
| | Peak Wave Case | ε | 2.675 | **3.949** | 2.038 | 7.116 | **1.856** | 2.747 | 4.419 | **1.780** | 1.615 | 7.269 | **2.421** | 1.481 |
| | | α | 2.771 | **4.231** | 2.726 | 6.585 | **2.087** | 2.294 | 3.830 | **2.873** | 1.449 | 6.554 | **2.126** | 1.047 |
| | | β | 4.473 | **6.230** | 2.332 | 30.493 | **2.525** | 8.262 | 19.845 | **1.726** | 2.625 | 50.590 | **5.589** | 2.895 |
| | | $E_{10}$ (m/s, m, m/s) | 22.5 | **6.8** | 0.90 | 24.9 | **8.1** | 1.00 | 33.0 | **10.0** | 1.32 | 28.4 | **8.2** | 1.14 |
| | | $E_{10}$ (kt, ft, kt) | 43.7 | **22.3** | 1.76 | 48.4 | **26.6** | 1.94 | 64.2 | **32.8** | 2.57 | 55.2 | **26.9** | 2.21 |
| | | $E_{10}$ (ft/s, ft, ft/s) | 73.8 | **22.3** | 2.97 | 81.7 | **26.6** | 3.28 | 108.3 | **32.8** | 4.33 | 93.2 | **26.9** | 3.73 |
| | Peak Current Case | ε | 2.276 | 3.685 | **2.504** | 6.139 | 1.732 | **9.257** | 1.906 | 1.661 | **2.086** | 1.984 | 2.260 | **2.026** |
| | | α | 2.544 | 3.764 | **2.989** | 5.585 | 1.643 | **8.797** | 1.465 | 2.222 | **1.825** | 1.324 | 1.794 | **1.534** |
| | | β | 3.264 | 6.663 | **3.277** | 27.013 | 2.785 | **35.559** | 4.681 | 1.857 | **4.333** | 7.592 | 6.370 | **5.589** |
| | | $E_{10}$ (m/s, m, m/s) | 16.9 | 5.1 | **1.13** | 18.7 | 6.1 | **1.25** | 24.8 | 7.5 | **1.65** | 21.3 | 6.2 | **1.42** |
| | | $E_{10}$ (kt, ft, kt) | 32.8 | 16.7 | **2.20** | 36.3 | 19.9 | **2.43** | 48.1 | 24.6 | **3.21** | 41.4 | 20.2 | **2.76** |
| | | $E_{10}$ (ft/s, ft, ft/s) | 55.4 | 16.7 | **3.71** | 61.3 | 19.9 | **4.10** | 81.2 | 24.6 | **5.41** | 69.9 | 20.2 | **4.66** |
| Pre-peak (Jun. 1 to Aug. 1) | All Cases | ε | 2.440 | 4.437 | 2.594 | 9.279 | 1.846 | 8.545 | 1.993 | 1.744 | 1.962 | 2.183 | 2.352 | 2.085 |
| | | α | 2.981 | 4.613 | 2.971 | 8.820 | 2.047 | 8.080 | 1.775 | 3.127 | 1.793 | 1.660 | 2.084 | 1.589 |
| | | β | 3.094 | 7.649 | 3.615 | 35.559 | 2.549 | 32.820 | 3.877 | 1.567 | 3.615 | 6.652 | 5.201 | 5.900 |
| | | $E_{10}$ (m/s, m, m/s) | 16.7 | 4.4 | 0.84 | 18.7 | 5.1 | 0.94 | 17.5 | 4.5 | 0.87 | 23.0 | 5.9 | 1.15 |
| | | $E_{10}$ (kt, ft, kt) | 32.5 | 14.4 | 1.63 | 36.4 | 16.7 | 1.83 | 34.0 | 14.8 | 1.69 | 44.7 | 19.4 | 2.24 |
| | | $E_{10}$ (ft/s, ft, ft/s) | 54.8 | 14.4 | 2.76 | 61.4 | 16.7 | 3.08 | 57.4 | 14.8 | 2.85 | 75.5 | 19.4 | 3.77 |
| Post-peak (Oct. 21 to Nov. 30) | All Cases | ε | 2.420 | 4.378 | 2.475 | 5.804 | 1.842 | 8.743 | 2.048 | 1.812 | 2.016 | 1.981 | 2.620 | 2.060 |
| | | α | 2.998 | 4.605 | 2.962 | 5.371 | 2.138 | 8.275 | 1.818 | 2.822 | 1.786 | 1.503 | 2.272 | 1.548 |
| | | β | 3.013 | 7.268 | 3.229 | 20.154 | 2.417 | 33.870 | 4.087 | 1.807 | 3.988 | 5.276 | 6.652 | 5.946 |
| | | $E_{10}$ (m/s, m, m/s) | 11.7 | 3.0 | 0.59 | 15.7 | 4.2 | 0.79 | 16.8 | 4.4 | 0.84 | 20.7 | 5.6 | 1.04 |
| | | $E_{10}$ (kt, ft, kt) | 22.7 | 9.8 | 1.15 | 30.5 | 13.8 | 1.54 | 32.7 | 14.4 | 1.63 | 40.2 | 18.4 | 2.02 |
| | | $E_{10}$ (ft/s, ft, ft/s) | 38.4 | 9.8 | 1.94 | 51.5 | 13.8 | 2.59 | 55.1 | 14.4 | 2.76 | 67.9 | 18.4 | 3.41 |

Note: For all-year hurricane conditions the independent extremes are calculated by using the parameters in **bold**.

For return periods other than those specified in API 2INT-MET the peak period, $T_p$ in seconds, may be calculated from the significant wave height, $H_s$, using the following relationship,

$$T_p = A \cdot (H_s)^B \tag{K.3}$$

where

| | | Peak Hurricane Season | | | | Pre- & Post-Peak |
|---|---|---|---|---|---|---|
| | | West | West-Central | Central | East | All Regions |
| $H_S$ in meters, | A = | 5.868 | 5.719 | 4.776 | 4.311 | 5.427 |
| $H_S$ in feet, | A = | 3.763 | 3.676 | 2.872 | 2.466 | 3.416 |
| $H_S$ in meters or feet, | B = | 0.374 | 0.372 | 0.428 | 0.470 | 0.390 |

The mid-depth current speed, $V_{cm}$, may be calculated from the surface current speed, $V_{cs}$, as follows,

$$V_{cm} = 0.750 \cdot V_{cs} \tag{K.4}$$

And the depth at which the current speed is zero, $D_0$, is given by,

$$D_0 = C \cdot V_{cs} \tag{K.5}$$

where

$D_0$ in metres and $V_{cs}$ in m/s, C = 42.00

$D_0$ in ft and $V_{cs}$ in knots, C = 70.88

$D_0$ in ft and $V_{cs}$ in ft/s, C = 42.00

Finally the storm surge can be calculated from the 1-hour mean wind speed at 10 m above mean water level, $V_{1\text{-hr}}$, as follows,

$$\text{Surge} = F \cdot V_{1-hr} + G \tag{K.6}$$

where

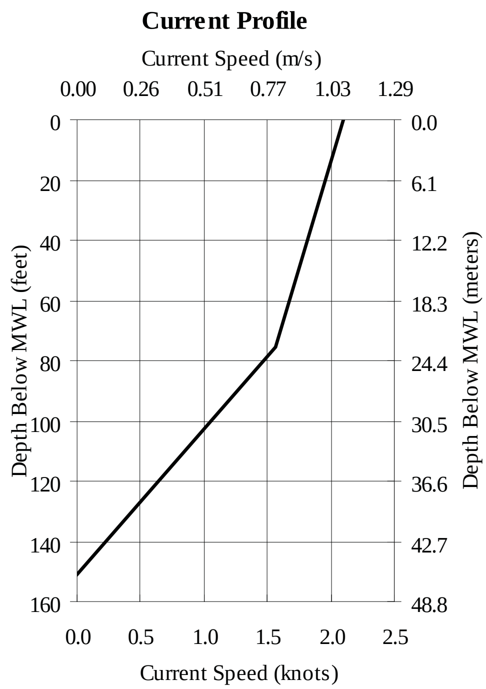
| | F | G |
|---|---|---|
| Surge in metres and $V_{1\text{-hr}}$ in m/s | 0.0283 | – 0.5171 |
| Surge in feet and $V_{1\text{-hr}}$ in knots | 0.0478 | – 1.6965 |
| Surge in feet and $V_{1\text{-hr}}$ in ft/s | 0.0283 | – 1.6965 |

### K.11.5 MINIMUM CATEGORY 1 HURRICANE WIND, WAVE, AND CURRENT CONDITIONS

Site and seasonal specific metocean criteria may be used in developing the wave and current conditions associated with a minimum Category 1 hurricane. That is, the wave and current conditions that occur for the same return period as the 64 knot 1-minute wind speed may be determined based on site and seasonal specific hurricane criteria. Alternatively, the wind, wave, and current conditions specified in Table K.5 may be used.

Table K.5—Minimum Category 1 Hurricane Conditions

| | | | |
|---|---|---|---|
| *Wind* | | | |
| 1-hour mean at 10 m, $V_{1\text{-hr}}$ | 27.0 m/s | 52.6 knots | 88.6 ft/s |
| 1-minute mean at 10 m, $V_{1\text{-minr}}$ | 32.9 m/s | 64.0 knots | 107.9 ft/s |
| *Wave* | | | |
| Significant Wave Height, $H_s$ | 8.0 m | 26.2 ft | 26.2 ft |
| Peak Period, $T_p$ | 12.2 s | 12.2 s | 12.2 s |
| *Current* | | | |
| Surface Speed, $V_{cs}$ | 1.08 m/s | 2.10 knots | 3.5 ft/s |
| Mid-depth Speed, $V_{cm}$ | 0.81 m/s | 1.57 knots | 2.7 ft/s |
| Zero Speed Depth, $D_0$ | 46 m | 151 ft | 151 ft |
| JONSWAP wave spectrum $2.0 < \gamma < 2.5$, and NPD wind spectrum | | | |




If the operator has a set of site-specific metocean criteria for the mooring location, the Operator may elect to derive suitable associated waves and current through the following procedure:

1. determine the return period $R_{64kt}$ for a 1-minute average 64kt wind speed;
2. determine the associated significant wave height ($H_s$) for the return period $R_{64kt}$;
3. determine the peak wave period ($T_p$) using regression analysis of $H_s$ and $T_p$;
4. determine the surface current velocity for the return period $R_{64kt}$;
5. determine the mid depth current velocity and zero current depth using regression analysis.

Use of the above procedure should provide an appropriate set of parameters for a minimum Category 1 hurricane at the location of interest.

## K.12 References


1. Noble Denton, "Calibration of ABS, API, DnV, HSE (Den), and NMD Mooring Design Codes for Floating Drilling and Production Platforms," NDAI Rpt No. 92489, December 1995.
2. Shu, H. and Loeb, D.A., "Extending the Mooring Capability of a Mobile Offshore Drilling Unit," OTC paper 17995, 2006.
3. Chaplin, C.R. Rebel, G. and Ridge, I.M.L., "Tension/Torsion Interactions in Multi-component Mooring Lines," OTC paper 12173, 2000.
4. DNV-RP-E302, "Design and Installation of Plate Anchors in Clay," December 2002.


## K.13 Consequence Assessment Checklist

Notes regarding usage of the Consequence Assessment Checklist.

1. This checklist contains many of the items that need to be addressed when determining the potential consequences of operating a MODU at a specific location. The list may not be complete, and there may be other items that need to be addressed.
2. A user can base decisions as to preferred mitigation options in the design of the mooring system by use of the checklist, or a similar consequence assessment method.
3. If a user desires to compute numerical scores, a factor of unity may be used as a multiplier on the base value and values less than unity developed for terms such as Slightly Better, Better, Much Better, Significantly Better. "Similar" refers to a factor close to unity, i.e., a value close to the base value. Factors greater than unity may be used for terms such as Worse, Much Worse, etc. User-selected factors allow for rapid sensitivity assessments.
4. Some questions have multiplier ranges provided. The factors provided are indicative of values that may be used in a numerical analysis to account for relative importance or consequence. The user is encouraged to select values and factors commensurate with these items and consistent throughout the evaluation.
5. Some questions concern the number of items (e.g., pipelines, wells, umbilicals, etc.) and may generally be taken to act as a direct multiplier. A response of zero should be equivalent to ignoring the question. In other cases the response is simply "yes/no": the base can be taken as unity with the other term taken as zero.

### K.13.1 OVERVIEW TO CHECKLIST

#### K.13.1.1 General Information

The checklist approach is a simple consequence ranking methodology that allows the stakeholders to assess, on a consistent relative basis, the likelihood of adverse consequence that the well operations represents. The risks associated with drilling a specific location can be considered to be a combination of the probability that a unit will suffer a mooring failure and the consequences should such a failure occur. The primary purpose of the approach set out below is to help the stakeholders develop a high level overview of the consequences of a mooring failure, and a measure of the likelihood of realizing those consequences. The stakeholders must then determine an acceptable level of risk for the operation by selecting the minimum environmental return period to be used in the design of mooring system, and other mitigation measures that may be suitable.

Some variations of the questions in the checklist set out below are associated with atypical operations, as defined in K.6.5 (e.g., a permanent facility within one mooring radius of the MODU, etc.). The checklist can be used as a method of initially estimating the consequences of atypical operations, but it is not sufficiently refined to assess all the nuances of such an operation. Atypical operations should always be subject to a detailed supplemental risk assessment in addition to any basic assessment.

Note: *Likelihood* is the conditional probability of an event (consequence) occurring, given that a mooring failure has already occurred (i.e., at a minimum the likelihood of an event occurring is conditional on partial or complete mooring system failure). The *probability* of mooring system failure largely depends on the environmental return period for which the mooring system meets the design criteria specified in Section 7.

It is anticipated that any risk analysis would consider the issues set out below, but this does not represent the only approach, and there may be other factors that need to be taken into account that have not been described. Any risk assessment should assess the overall issue to ensure completeness.

#### K.13.1.2 Definitions

**K.13.1.2.1 Within the mooring pattern** is taken to be anywhere within a smooth closed curve drawn through all the anchors in a projection on the horizontal plane.

**K.13.1.2.2 Within one mooring radius of any anchor** can be taken as the area covered by a series of circles, one based on each anchor, having a radius equal to the greatest distance from the nominal center of the mooring pattern to the furthest anchor.

**K.13.1.2.3 Crossed moorings** are defined as when the smooth curve drawn through the MODU anchors intersect a similar curve for an adjacent permanent facility.

**K.13.1.2.4 Surface facility** is defined as the platform and its mooring pattern where applicable.

### K.13.2 BLOCK AREA

Certain areas of the Gulf of Mexico are more densely populated with both surface and subsea infrastructure. Despite the answers to the questions below about pipelines and permanent facilities close to the proposed location, there is a certain “overhead” consequence for drilling in any given location. A higher consequence designation of some block areas may be chosen to account for the proximity and density of infrastructure that cannot be explicitly calculated within the checklist approach. Due consideration should be given to the infrastructure that is in adjacent block areas, not only that within the block area in question. Size and criticality of the infrastructure should also be taken into account. In some areas there may not be many pipelines, but those that exist are large, service deepwater areas, would be costly to repair, and carry a significant percentage of the Gulf production.

There are metocean variations across the Gulf. The Block Area Value does NOT account for these since they will be accounted for in the site specific metocean criteria used in the design. Use of infrastructure maps, such as the MMS map referenced in the main document (see K.6.2), may be used in evaluating infrastructure importance for a given site and nearby waters.

Areas of high density infrastructure may include Mississippi Canyon, Green Canyon, Eugene Island, Garden Banks, East Breaks, West Cameron, South Timbalier, High Island, Ship Shoal, Viosca Knoll, South Marsh Island, Vermillion, Ewing Bank, and Main Pass.

Areas of low density may include Destin Dome, Desoto Canyon, Henderson, Port Isabel, and Corpus Christi.

If computing numerical values, a Block Area Consequence Value ($Block_v$) in the range of 5 to 20 may be considered appropriate and should be reported as:

$$Block_v = \text{value based on response to Table K.6}$$

### K.13.3 WATER DEPTH

The cost and impact of damage to subsea infrastructure in deepwater is often greater than in the shallower waters. In addition, there can be additional delays in contracting the larger marine vessels required to handle repairs in deepwater due to limited availability. This contracting problem can, in turn, further increase the cost and delay production. The checklist accounts for an increase in the subsea damage consequence of failure in deepwater.

If computing numerical values, the Water Depth Factor ($WD_f$) should be reported as:

$$WD_f = \text{factor based on response to Table K.7}$$

### K.13.4 SEABED INFRASTRUCTURE WITHIN THE MOORING PATTERN

#### K.13.4.1 General

This section, and those that follow, contain specific questions about the local subsea infrastructure. This specific question concerns the subsea infrastructure that actually lies within the mooring pattern of the MODU. “Within the mooring pattern” is taken to be anywhere within a smooth closed curve drawn through all the anchors in a projection on the horizontal plane. Because the infrastructure is within the mooring pattern, mooring failure in any direction could lead to failed mooring components being dragged over, or dropped on the subsea facilities. Since the effective consequence value for a pipeline or umbilical within a mooring pattern is higher than for one outside the mooring pattern, any item that is accounted for in this question does not need to be included in the responses to other questions, unless there is either a significant change in potential consequence of damage, or the size of a pipeline changes (e.g., after picking up production from a subsea well). (Note that the “effective

Table K.6—Block Area

| Block Area | Response | Block Area | Response |
|---|---|---|---|
| Alaminos Canyon | | Lund South | |
| Amery Terrace | | Main Pass | |
| Apalachicola | | Matagorda Island | |
| Atwater Valley | | Miami | |
| Bay Marchand | | Mississippi Canyon | |
| Brazos | | Mobile | |
| Breton Sound | | Mustang Island | |
| Campeche Escarpment | | North Padre Island | |
| Chandeleur | | Pensacola | |
| Charlotte Harbor | | Port Isabel | |
| Corpus Christi | | Pulley Ridge | |
| DeSoto Canyon | | Rankin | |
| Destin Dome | | Sabine Pass (LA) | |
| Dry Tortugas | | Sabine Pass (TX) | |
| East Breaks | | Ship Shoal | |
| East Cameron | | Sigsbee Escarpment | |
| Eugene Island | | South Marsh Island | |
| Ewing Bank | | South Padre Island | |
| Florida Middle Ground | | South Pass | |
| Florida Plain | | South Pelto | |
| Gainesville | | South Timbalier | |
| Galveston | | St. Petersburg | |
| Garden Banks | | Tarpon Springs | |
| Grand Isle | | The Elbow | |
| Green Canyon | | Tortugas Valley | |
| Henderson | | Vermilion | |
| High Island | | Vernon Basin | |
| Howell Hook | | Viosca Knoll | |
| Keathley Canyon | | Walker Ridge | |
| Key West | | West Cameron | |
| Lloyd Ridge | | West Delta | |
| Lund | | | |

Table K.7—Water Depth

| Response | Subsea Factor |
|---|---|
| <1000 ft | Slightly Better |
| 1000 ft – 4000 ft | Base |
| >4000 ft | Slightly Worse |

consequence value" implicitly includes such issues as likelihood of drifting over the relevant subsea infrastructure and the "Likelihood Factor" explicitly given in Table K.8. However, this allowed exclusion does not account for the possibility that the pipeline, umbilical, etc. is damaged in more than one location by the drifting MODU which would increase the direct repair costs, but have limited effect on "lost" production.)

#### K.13.4.2 Likelihood Factor

The Likelihood Factor is a measure of the likelihood that a mooring component will be dragged over the pipeline, umbilical, etc., taking into account where the item is within the mooring pattern. If computing numerical values, a value of 0.5 to 0.7 may be considered as a base for pipelines and umbilicals. There is a lower likelihood that a component will be dragged over a subsea well because of its size. This factor should not be used to account for the likelihood that the subsea item is damaged by the dragged mooring component. That is addressed separately within the checklist.

Table K.8—Subsea infrastructure within Mooring Pattern

| | Number of Items | Likelihood Factor | Possible Value[a] | Total Value |
|---|---|---|---|---|
| **ACTIVE PIPELINES WITHIN MOORING PATTERN** | | | | |
| How many pipelines are there within the mooring pattern, and are: | | | | |
| < 10 in. diameter? | | Base: 0.5 to 0.7 | Better | |
| 10 in. ≤ D < 15 in.? | | Base: 0.5 to 0.7 | Base: 15 to 35 | |
| 15 in. ≤ D < 20 in.? | | Base: 0.5 to 0.7 | Worse | |
| 20-in. ≤ D? | | Base 0.5 to 0.7 | Much Worse | |
| | | | | |
| **UMBILICALS** | | | | |
| How many umbilicals are within the mooring pattern? | | Base: 0.5 to 0.7 | Base: 1 to 15 | |
| | | | | |
| **SUBSEA WELLS** | | | | |
| How many subsea wells or completions are within the mooring pattern? | | Better | Base: 1 to 15 | |
| | | | | |
| **Total Sum of Subsea Consequences within Mooring Pattern** (see note below) | | | | |

[a]If computing a numerical value, then the numbers given in this column may be considered suitable values modified, as appropriate, by the expressions "Better," Worse," etc. Additional discussion can be found below the table.

### K.13.4.3 Pipelines

If computing numerical values, a value of 15 to 35 may be considered as a base for a pipeline in the range of 10 in. to 15 in. within the mooring pattern. Lines that are less than 10 in. diameter will be of less consequence, i.e., "Much Better" than the base value. Those over 15 in., and particularly those over 20 in., can be considered of high consequence, i.e., "Much Worse" than the base value, particularly as they may be carrying hydrocarbons from a number of wells and facilities. Pipeline values chosen here should be consistently used throughout the checklist as they are modified by "Likelihood Factors," or equivalents, to account for changing locations relative to the MODU. In some cases, it is not known what hydrocarbon is being transported in the pipeline, however, this can be taken into account when assigning values, if it is known. Generally, a gas line will carry less barrels of oil equivalent (BOE) than an equal size line carrying oil. However, due consideration needs to be given to the possibility that damage to a gas line could shut down production through an oil line having the same starting point, but possibly different destination. (If the gas cannot be exported from a facility, and cannot be re-injected, then all hydrocarbon production will be shut down due to losing only the gas export facility.) The flow capacity of a pipeline is approximately proportional to the square of its diameter, although not all lines are flowing at their capacity. Actual pipeline flow rates may be taken into account when computing numerical values: it is permissible to use less than the design capacity if actual flow rates are known.

### K.13.4.4 Umbilicals

If computing numerical values, a value of 1 to 15 may be considered as a base for an umbilical, depending on the factors discussed below. Umbilicals are easily damaged, but have a relatively low consequence rating within this approach. While they are important, they generally affect only one well or a small number of wells, and may not have a major impact on the overall production levels within the Gulf of Mexico. However, they can be extremely difficult to replace, and may have long lead times on replacement. Companies may want to increase the significance of umbilicals for their own internal purposes, but from a Gulf of Mexico production perspective, they are less important than pipelines. For umbilicals that service a number of different wells, or a single high production rate well, a value in the upper range may be considered.

### K.13.4.5 Subsea Wells

Subsea wells, over and above the well being worked on, should be included if they are active. If computing numerical values, a value of 1 to 15 may be considered as a base for a subsea well within a mooring pattern. The cost and consequences of damage are slightly larger than those for an umbilical within a mooring pattern, although the likelihood of damaging a subsea well is somewhat lower due to its physically smaller size.

#### K.13.4.6 Computation of Numerical Values

If computing numerical values, the consequence value for Subsea Infrastructure within the mooring pattern, (Subsea-in$_v$) should be reported as:

$$\text{Subsea-in}_v = \Sigma(\text{Number} \times \text{Likelihood Factor} \times \text{Value})$$

See Table K.8 for for guidance on the numerical values for Likelihood Factors and Values for pipelines, umbilicals, and subsea wells.

As discussed in K.13.13, if a more detailed assessment is being undertaken, it may be advantageous to keep separate the calculated values for pipelines and umbilicals as they have different damage probabilities, depending on what mooring component is potentially being dragged over the seabed. Combining the values at this stage will be conservative.

### K.13.5 SEABED FACILITIES WITHIN ONE MOORING RADIUS OF ANY ANCHOR

This question is an extension of the previous one relating to infrastructure within the mooring pattern, except that it addresses subsea infrastructure outside the mooring pattern, but within one mooring radius of any anchor. The subsea infrastructure questions are identical, but it would be expected that somewhat lower values would be used since there is an increased likelihood that no mooring component would be dragged over, or dropped on, the pipelines, etc. if the mooring system was to fail. This change in likelihood of having a mooring component dragged over the subsea item should be included through modification of the "Likelihood Factor" if calculating numerical values. The base values for pipelines, umbilicals, etc. should remain the same since their "value" does not alter.

The definition of "within one mooring radius of any anchor" can be taken as the area covered by a series of circles, one based on each anchor, having a radius equal to the greatest distance from the nominal center of the mooring pattern to the furthest anchor. This will tend to be slightly conservative for some asymmetrical mooring patterns, but should generally be followed unless there is good evidence that it would produce unrealistic results. The intent is to cover the entire area that can be "swept" by the MODU should it swing on any anchor, allowing for some limited anchor slippage for drag embedment anchors.

No account has been taken for directionality to this nearby infrastructure as there is a relatively high likelihood of damage due to dragged components over a relatively wide swath.

As with the pipelines or umbilicals within the mooring pattern, any component that is accounted for in this question does not need to be included in the responses to other questions, unless there is either a significant change in potential consequence of damage, or the size of a pipeline increases (e.g., after picking up production from a subsea well).

If computing numerical values, a value of 15 to 35 may be considered as a base for a 10-in. to 15-in. pipelines, 1 to 15 for umbilicals, and 1 to 15 for subsea wells within a mooring radius of any anchor. See K.13.4 on "subsea infrastructure within the mooring pattern" for further discussion of these items.

If computing numerical values, the consequence value for Subsea Infrastructure within one radius of any anchor (Subsea-out$_v$) should be reported as:

$$\text{Subsea-out}_v = \Sigma(\text{Number} \times \text{Likelihood Factor} \times \text{Value})$$

See Table K.9 for for guidance on the numerical values for Likelihood Factors and Values for pipelines, umbilicals, and subsea wells.

As discussed in K.13.13, if a more detailed assessment is being undertaken, it may be advantageous to keep separate the calculated values for pipelines and umbilicals as they have different damage probabilities, depending on what mooring component is potentially being dragged over the seabed. Combining the values at this stage will be conservative.

### K.13.6 MOORING LINES CROSSING THE MOORINGS OF OTHER TEMPORARY FACILITY

If the MODU mooring lines cross the moorings of another temporary facility, there is clearly a threat that if either of the mooring systems fail, then there could be an adverse effect on the moorings of the other unit. This would be a high consequence event. There is, however, a relatively high likelihood that even in the worst case, the MODU and other facility will not be irreparably damaged. Crossed moorings are defined as when the smooth curve drawn through the MODU anchors intersect a similar curve for an adjacent temporary facility. See also K.13.9.

Table K.9—Subsea Infrastructure within Mooring Radius of Anchor

| | Number of Items | Likelihood Factor | Possible Value* | Total Value |
|---|---|---|---|---|
| **ACTIVE PIPELINES WITHIN ONE MOORING RADIUS** | | | | |
| How many pipelines are there within the mooring radius of any anchor, and are: | | | | |
| < 10 in. diameter? | | Base: 0.4 to 0.6 | Better | |
| 10 in. ≤ D < 15 in.? | | Base: 0.4 to 0.6 | Base: 15 to 35 | |
| 15 in. ≤ D < 20 in.? | | Base: 0.4 to 0.6 | Worse | |
| 20 in. ≤ D? | | Base: 0.4 to 0.6 | Much Worse | |
| | | | | |
| **UMBILICALS** | | | | |
| How many umbilicals are within the mooring radius of any anchor? | | Base: 0.4 to 0.6 | Base: 1 to 15 | |
| | | | | |
| **SUBSEA WELLS** | | | | |
| How many subsea wells or completions are within the mooring radius of any anchor? | | Better | Base: 1 to 15 | |
| | | | | |
| **Total Sum of Subsea Consequences within One Mooring Radius** (see note below) | | | | |
| *If computing a numerical value, then the numbers given in this column may be considered suitable values modified, as appropriate, by the expressions "Better," "Worse," etc. Additional discussion can be found below the table. | | | | |

## K.13.7 SURFACE FACILITIES WITHIN THE MOORING PATTERN

There is a potentially high consequence of a MODU mooring failure to a surface facility within the mooring pattern of the MODU. The primary concern is the loss of the permanent facility. The secondary concern is damage to the facility resulting in lost production. In addition, there is a possibility that the surface facility could suffer a failure and thereby affect the MODU. "Within the mooring pattern" is taken to be anywhere within a smooth closed curve drawn through all the anchors in a projection on the horizontal plane. This question, and the next addressing surface facilities within one mooring radius, has the same drivers, however, the likelihood of interaction decreases with distance between the facilities.

It is important to note that if there are crossed moorings with another temporary facility, as addressed in K.13.6, then it may be advisable to add an additional overall multiplier to both this and the next question to account for the increased likelihood of a MODU mooring failure.

### K.13.7.1 Likelihood Factor

The likelihood factor is to account for the likelihood that the MODU, having had a mooring failure, will interact with the surface facility. It should take into account how close the facility is to the MODU, its size, any special features that make it more or less likely to be damaged, etc. If calculating numerical values, a base value of 0.3 to 0.6 may be used. The likelihood factor is higher for spread moored surface facilities since there is a higher likelihood of adverse interaction leading to loss of station, damage to the facility, and damage to the wells.

### K.13.7.2 Design Production Capacity

The design production capacity will affect the consequences of an interaction between the MODU and a surface facility. High production facilities both cost more as capital investments and are also likely to have higher "lost production" costs. Although not explicitly included in the table above, if computing a numerical value, the design production capacity should be taken into account. The upper end of the range of base values for a full-size TLP has been estimated for a major TLP with high production rates of approximately 150,000 barrel of oil equivalent (BOE) per day. The cost of a facility is not directly proportional to the design production rate, but there is a close relationship.

Referring to Table K.10, if computing numerical values, the consequence value for Crossed Mooring Lines with a temporary facility ($Cross_v$) should be reported as:

$$Cross_v = \text{“Yes/No factor”} \times \text{Value}$$

Table K.10—Mooring Lines Crossed with Temporary Facility

| **MOORING LINES CROSSED WITH OTHER TEMPORARY FACILITY** Directionality does not matter | Factor | Possible Value |
|---|---|---|
| Does the smooth curve drawn through the MODU anchors intersect a similar curve for an adjacent temporary facility? | Yes = 1<br>No = 0 | 20 to 30 |

### K.13.7.3 TLP

A TLP is a relatively robust facility, as long as the tendons remain intact. When a TLP is very close to a MODU, and the MODU suffers a mooring failure, then there is a relatively high likelihood that the MODU mooring lines will interact with the TLP tendons. At best this will damage the tendons so that they only need to be replaced. At worst, the tendons will fail during the storm, thereby leading to TLP mooring failure. Generally, TLPs are not stable when tendons are lost, so the damage may be catastrophic and the TLP can capsize. Another possibility is that a tendon pile is damaged by a mooring component being dragged over it. Depending on the specific TLP, this may be an irrecoverable event that requires facility replacement at a new location, necessitating the re-drilling of all the wells. Most of the larger TLPs have dry trees, so loss of the TLP could lead to very expensive P&A operations, with a very low likelihood of well recovery.

### K.13.7.4 Spar

A spar is relatively robust from the standpoint of stability: it is possible for them to sustain significant damage and still remain upright. The greater threat to a spar is from damage to the mooring system, thereby possibly leading to loss of stationkeeping and damage to risers. Conversely, subsurface damage to a spar hull would be extremely difficult and costly to repair. Most spars have dry trees, so loss of stationkeeping would result in wells only protected by a sub-surface safety valve. It may be possible to re-enter the wells and get them flowing again once the surface facility has been replaced, but this would be an expensive and relatively high risk operation.

### K.13.7.5 Semi-Submersible

A semi-submersible production unit is going to have similar responses to those of a spar, although the cost of recovery from a complete mooring system failure may be lower. It should be possible to either take the unit into a shipyard for repairs, or possibly “dry dock” it on submersible barges although disconnecting and safely laying down the production and export SCRs will be challenging.

### K.13.7.6 Synthetic Moored

Synthetic mooring components can be more easily damaged than conventional wire and chain. There is a possibility that dragging MODU mooring components over the steel mooring lines could cause failure, but it is relatively low likelihood. A more likely outcome is that the moorings are damaged and have to be replaced after the storm, but with limited impact on production. Synthetic lines, however, will very likely fail if subjected to interaction with the MODU steel mooring components, thereby letting the permanent facility drift off location resulting in massive repair costs and loss of production. To account for this increased likelihood of damage, there is a possible multiplier of 1.5 to 2.5 that may be used if numerical values are being computed. Redundancy in the mooring system and grouping of mooring legs should be taken into consideration when selecting this factor.

### K.13.7.7 Compliant Tower

Compliant towers will tend to be lower consequence than most of the floating facilities as they are reasonably likely to survive in a repairable condition, generally have lower capital investment, and riser damage would likely be more easily repaired.

#### K.13.7.8 Jacket

Similar to a compliant tower, except for hub jackets. Hub jackets have been included at a higher level to account for the effect on overall GoM production if they were to be damaged.

#### K.13.7.9 Computation of Numerical Values

If computing numerical values, the consequence values for Surface Facility within the mooring pattern (Surface-in$_v$) should be reported as:

$$\text{Surface-in}_v = \text{Both}_f \times \Sigma(\text{Number} \times \text{Likelihood Factor} \times \text{Synthetic Multiplier} \times \text{Facility Value})$$

where "Both$_f$" is defined below and the "Synthetic Multiplier" is either unity, or adjusted to account for the presence of synthetic moorings.

See Table K.11 for for guidance on the numerical values for Likelihood Factors and Values for TLPs, spars, semi-submersibles, compliant towers and jackets.

Table K.11—Surface Facility within Mooring Pattern

| **SURFACE FACILITIES WITHIN MOORING PATTERN** Directionality does not matter | Number of Items | Likelihood Factor | Synthetic Multiplier | Possible Value[a] | Total Value |
|---|---|---|---|---|---|
| Is there a permanent installation(s) within the mooring pattern? If yes, indicate which of the following types; | | | | | |
| | | | | | |
| Full-size TLP | | Base | | Base: 400 to 700 | |
| Mini TLP | | Base | | Better | |
| Spar | | Worse | | Similar | |
| Synthetic moored? | | | 1.5 to 2.5 | Similar | |
| Semi-submersible | | Worse | | Slightly Better | |
| Synthetic moored? | | | 1.5 to 2.5 | Slightly Better | |
| Compliant Tower | | Base | | Better | |
| Hub Jacket | | Base | | Better | |
| Other Jacket | | Base | | Much Better | |
| | | | | | |
| **Total Sum of Surface Facility within Mooring Pattern Consequences** (see note below) | | | | | |

[a]If computing a numerical value, then the numbers given in this column may be considered suitable values modified, as appropriate, by the expressions "Better," "Worse," etc. Additional discussion can be found below the table.

### K.13.8 SURFACE FACILITIES WITHIN ONE MOORING RADIUS OF ANY ANCHOR

#### K.13.8.1 General

The danger of having a surface facility within one mooring radius of any anchor is that it is possible for the MODU to break most of its mooring lines and swing into the facility. This can be particularly severe if the mooring line length at any anchor is comparable to the distance from that anchor to the permanent facility. Under these circumstances swinging on the anchor will facilitate a direct hull-on-hull collision. Many of the interaction discussions given in K.13.7 apply in this case as well, as do the damage and consequence discussions. If computing numerical values, the Likelihood Factor value range has been reduced to account for the increased distance from the MODU to the facility.

This scenario should also be used to cover the case in which the moorings of the MODU are crossed with those of a spar, permanent semi-submersible or other spread moored facility, even if the platform is not actually within one anchor radius of the MODU (see definition for surface facility). The potential for mooring line interaction is significant, particularly if any of the MODU anchors drag. "Crossed moorings" occur when the smooth curve drawn through the MODU anchors intersect a similar curve for an adjacent permanent facility.

### K.13.8.2 Likelihood Factor

If calculating numerical values, a base value of 0.2 to 0.3 may be used. The likelihood factor is higher for spread moored surface facilities since there is a higher likelihood of adverse interaction leading to loss of station, damage to the facility, and damage to the wells.

### K.13.8.3 Computation of Numerical Values

If computing numerical values, the consequence values for Surface Facility within one radius of any anchor (Surface-out$_v$) should be reported as:

$$\text{Surface-out}_v = \text{Both}_f \times \Sigma(\text{Number} \times \text{Likelihood Factor} \times \text{Synthetic Multiplier} \times \text{Facility Value})$$

where “Both$_f$” is defined below and the “Synthetic Multiplier” is either unity, or adjusted to account for the presence of synthetic moorings.

See Table K.12 for for guidance on the numerical values for Likelihood Factors and Values for TLPs, spars, semi-submersibles, compliant towers and jackets.

Table K.12—Surface Facility within Anchor Radius

| **SURFACE FACILITIES WITHIN ONE ANCHOR RADIUS**<br>Directionality does not matter | Number of Items | Likelihood Factor | Synthetic Multiplier | Possible Value[a] | Total Value |
|---|---|---|---|---|---|
| Is there a permanent installation(s) outside the mooring pattern, but within one anchor radius of any anchor? If yes, indicate which of the following types. | | | | | |
| | | | | | |
| Full-size TLP | | Base | | Base: 400 to 700 | |
| Mini TLP | | Base | | Better | |
| Spar | | Worse | | Similar | |
| Synthetic moored? | | | 1.5 to 2.5 | Similar | |
| Semi-submersible | | Worse | | Slightly Better | |
| Synthetic moored? | | | 1.5 to 2.5 | Slightly Better | |
| Compliant Tower | | Base | | Better | |
| Hub Jacket | | Base | | Better | |
| Other Jacket | | Base | | Much Better | |
| | | | | | |
| **Total Sum of Surface Facility within One Radius Consequences** (see note below) | | | | | |

[a]If computing a numerical value, then the numbers given in this column may be considered suitable values modified, as appropriate, by the expressions “Better,” “Worse,” etc. Additional discussion can be found below the table.

## K.13.9 BOTH CROSSED MOORINGS WITH A TEMPORARY FACILITY AND AN ADJACENT PERMANENT FACILITY

If the MODU has crossed moorings with another temporary facility AND there is an adjacent permanent facility, then there is an increase in risk to the permanent facility due to the increased likelihood of interaction between the MODU and other temporary facility.

This scenario could be considered as “double dipping” since it is possible the other temporary facility will also have been assessed through some form of risk assessment. However, the probability that any one of the two facilities suffers a significant failure is higher than the sum of the two individual probabilities of a failure (there are scenarios in which mooring line interaction could result in failure, a dragged anchor could increase the likelihood of mooring line interaction, a single line damage case may result in interaction, etc.). In addition, the consequences of a failure of one of the temporary facilities, leading to failure of the other, would have a far greater impact on an adjacent permanent facility with a much increased consequence of failure.

It is not known what the design return period of the mooring system of the adjacent temporary moored facility will be, but it may be comparable to that of the MODU in question. If two units are close together and one fails, there is an increased likelihood that the other will fail due to unfavorable interaction. The increased failure probability on the nearby surface infrastructure needs to be taken into account as a result of this scenario. If computing numerical values, consideration should be given to increasing the consequence values for permanent facilities within either mooring pattern or a radius of an anchor if the MODU has crossed moorings with another temporary facility. Such a multiplier may have a value of between 1.3 and 1.8.

If computing numerical values, the factor for both crossed moorings and a surface facility anywhere within one radius of any anchor ($Both_f$) should be reported as:

$$Both_f = \text{Additional multiplier from Table K.13.}$$

Table K.13—Both Crossed Mooring Lines and Adjacent to Permanent Facility

| **MOORING LINES CROSSED WITH OTHER TEMPORARY FACILITY AND ADJACENT PERMANENT FACILITY** | Factor | Additional Multiplier for Surface-in$_v$ and Surface-out$_v$[a] |
|---|---|---|
| Does the MODU have crossed moorings (as defined in K.13.6) AND have an adjacent permanent facility within one mooring radius of any anchor? | Yes = 1<br>No = 0 | 1.3 to 1.8 |

[a]If computing numerical values, this additional multiplier may be appropriate.

## K.13.10 FACILITIES BETWEEN ONE MOORING RADIUS AND 15 NAUTICAL MILES WITHIN EACH OCTANT

Having accounted for close proximity infrastructure in the previous sections, it is now necessary to establish what infrastructure there is within the likely striking distance of a drifting MODU that has broken its moorings. This section addresses infrastructure between one mooring radius of the well center, and 15 nautical miles. The use of a 15 nautical mile limit can be considered somewhat arbitrary, however, it does have some foundation in experience and calculations. (Herein, 15 miles should be read as "15 nautical miles" or 91,200 feet)

Mooring failures in the 2004 and 2005 hurricane seasons resulted in MODUs drifting up to 120 miles, but in only two cases were anchors dragged in excess of 5 miles. While there were a number of MODUs that could have hit surface facilities, there were only two that were in a position to cause significant subsea infrastructure damage at a distance. The importance of this information is that pipelines offer large targets to be hit by a dragged anchor, but surface facilities are relatively small targets. At 15 nautical miles, there is less than 0.15% likelihood of a drifting MODU directly hitting a surface facility, and this conditional probability still needs to be reduced to include the probability that the MODU suffers a complete mooring system failure, the initial event. This event was considered to be suitably low to not necessitate specific additional inclusion. In addition, the item addressing the general block area of operations should inherently account for some increase in risk in certain areas, and reductions in others, thereby implicitly accounting for additional (or lack of) distant infrastructure.

In order to better account for the number and sizes of pipelines and umbilicals, the circle of 15 mile radius, centered on the MODU location, needs to be split into subsections. When undertaking a detailed formal risk assessment, each individual piece of infrastructure within the 15 mile radius would be addressed. Due consideration would be given to the likelihood of drifting towards, say, a pipeline based on its length, distance from the MODU, and possibly its direction from the MODU. Such detail is beyond the capability of a simplified consequence assessment checklist approach. In order to make the problem tractable, the 15 mile circle has been split into 8 equal octants, each representing a 45° arc. A separate reporting table should be completed for each octant.

The octants for which these tables need to be completed are N (337.5° to 022.5°), NE (022.5° to 067.5°), E (067.5° to 112.5°), SE (112.5° to 157.5°), S (157.5° to 202.5°), SW (202.5° to 247.5°), W (247.5° to 292.5°), and NW (292.5° to 337.5°), as shown in Figure K.9.

Any pipeline or umbilical that was addressed as within a mooring pattern, or within one mooring radius of an anchor, does not need to be included in the responses to this series of octant questions, unless there is either a significant change in potential consequence of damage, or the size of a pipeline increases (e.g.; after picking up production from a subsea well).

Pipelines and umbilicals need to be counted in all octants they pass through where they are less than 15 nautical miles from the MODU. This way there is due consideration given to the real likelihood of impacting the pipeline or umbilical.

### K.13.10.1 Pipelines and Umbilicals by Octant

The possible values have been discussed in Table K.14. There is some reduction in value for distance, but the greatest effect of distance is that a distant pipeline will be in fewer octants than a near pipeline (reduced subtended angle). This will inherently lead to a reduced factor in the calculations when the drift direction is considered. The total sum pipeline and umbilical consequence for the octant ($Pipe\text{-}dist_v$) can be calculated as:

$$Pipe\text{-}dist_v = \Sigma(\text{Number} \times \text{Flow Ratio} \times \text{Value})$$

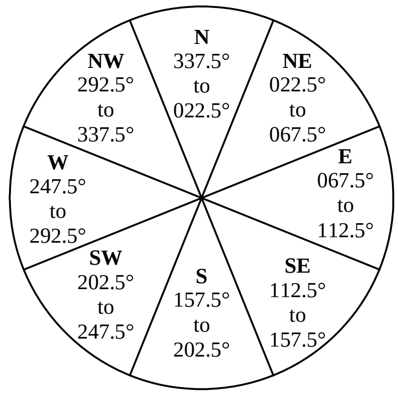



Figure K.9—Octant Definition

Table K.14—Pipelines and Umbilicals in Octant

| Octant | | | | |
|---|---|---|---|---|
| | Number of Items[1] | Flow as Ratio of Capacity[2] | Possible Values[3] | Subsea Total Value |
| **ACTIVE PIPELINES** | | | | |
| How many active pipelines are there in this octant that are less than 91,200 ft (15 nautical miles) from the well center, but outside the mooring pattern, and are: | | | | |
| < 10 in. diameter? | | | Better | |
| 10 in. ≤ D < 15 in.? | | | Base: 15 to 35 | |
| 15 in. ≤ D < 20 in.? | | | Worse | |
| 20 in. ≤ D? | | | Much Worse | |
| | | | | |
| **UMBILICALS** | | | | |
| How many umbilicals are in the octant that are less than 60,800 ft (10 nautical miles)1 from the well center, but outside the mooring pattern? | | | Base: 1 to 15 | |
| **Total Sum of Pipeline and Umbilicals Consequences for Octant** (see note below) | | | | |

[1]It is acceptable to ignore smaller pipelines of less than 12-in. diameter in the range between 10 and 15 miles, and all umbilicals may be ignored in the 10 to 15 mile range.

[2]The BOE flow in a pipeline affects the cost of downtime due to the magnitude of the lost or delayed “production”. It is therefore reasonable to account for the actual BOE flow rate in the pipeline when assessing the potential damage due to dragged mooring components. If computing numerical values, the base values given in the “Possible Values” column are for the pipeline transporting oil at 100% capacity. Generally gas lines will transport a lower BOE than oil lines. This reduction in BOE flow may be accounted for unless loss of such flow would disrupt the flow in another undamaged pipeline (e.g., by requiring production be shut-in).

[3]If computing a numerical value, then the numbers given in this column may be considered suitable values modified, as appropriate, by the expressions “Better,” “Worse,” etc.

### K.13.10.2 Subsea Wells by Octant

If computing a numerical value, Subsea wells, as discussed in Table K.15, may have a value of 1 to 15, however, this needs to be reduced to account for the distance from the MODU, and hence the reduced likelihood of interaction, even if the MODU is dragging a mooring component along the seabed. The simplest way to do this is to calculate the likelihood of drifting within the angle subtended by the wells from the MODU, and use this as a simple multiplier. The angle subtended can be conservatively taken as 5° at one mile. So the likelihood of drifting towards the subsea wells, given that the MODU is drifting within this specific octant, can be calculated as (L-well$_f$):

$$\text{L-well}_f(\text{Drifting towards wells} \mid \text{drifting in this octant}) = [5°/(\text{distance in feet}/6{,}800)]/45°$$

The total subsea well consequence value for the octant (Wells-dist$_v$) can therefore be calculated as:

$$\text{Wells-dist}_v = \Sigma(\text{Number} \times \text{Value} \times \text{L-well}_f)$$

The total subsea consequence value for the octant (Subsea-dist$_v$) can be calculated as:

$$\text{Subsea-dist}_v = \text{Pipe-dist}_v + \text{Wells-dist}_v$$

Table K.15—Subsea Wells in Octant

| Octant | | | |
|---|---|---|---|
| **SUBSEA WELLS (Not P&A)** | Number of Items | Possible Values* | Subsea Total Value |
| Are there any subsea wells or completions in the octant that are less than 60,800 ft (10 nautical miles)[1] from the well center, but outside the mooring pattern? | | Base: 1 to 15 | |
| If "Yes," calculate L-well$_f$ as described below | | | |
| | | | |
| **Total Sum of Subsea Well Consequences for Octant** (see note below) | | | |
| [1]It is acceptable to ignore any individual subsea wells in the range between 10 and 15 miles if their individual production rates are lower than 50,000 BOE per day. However, any wellhead cluster that has a production rate of over 50,000 BOE per day should be included. A wellhead cluster is taken to be any close group of wells that could be effectively shut-in by dragging an anchor through their midst. | | | |

### K.13.10.3 Surface Facilities by Octant

#### K.13.10.3.1 Spread Mooring Factor

As discussed in K.13.10.3.2, the distance of the surface facility from the MODU will affect the likelihood that there is an interaction between the two structures. However, a spread moored facility offers a much larger "target" to a MODU that is dragging mooring components below the keel. If calculating a numerical value, this factor may be considered to lie in the range of 1 to 5. Clearly, if the MODU is not dragging any components below the keel, then there is no increase in likelihood. Conversely, if it is dragging anchors, then the increased likelihood of an interaction is vastly increased due to the spread on the facility mooring system.

#### K.13.10.3.2 Calculation of Sum

If computing a numerical value, Surface Facilities, as discussed in Table K.16, may have a value of 400 to 700 with a potential additional multipliers to account for spread mooring and synthetic moorings. This needs to be reduced, however, to account for the distance of the facility from the MODU, and hence the reduced likelihood of interaction. The simplest way to do this is to calculate the likelihood of drifting within the angle subtended by the facility from the MODU, and use this as a simple multiplier.

Table K.16—Surface Facility in Octant

| **Octant** | | | | |
|---|---|---|---|---|
| **SURFACE FACILITIES UP TO 15 NAUTICAL MILES AWAY** | Number of Items[1] | Synthetic & Spread Multiplier | Possible Value[2] | Surface Total Value |
| Are there any permanent installations in the octant that are less than 91,200 ft (15 nautical miles)[1] from the well center, but outside the mooring pattern? If yes, calculate L-surface$_f$ as described below. | | | | |
| Full-size TLP | | | Base: 400 to 700 | |
| Distance (ft) | | | | |
| Mini TLP | | | Better | |
| Distance (ft) | | | | |
| Spar | | | Similar | |
| Distance (ft) | | | | |
| Spread Mooring Factor | | 1 to 5 | | |
| Synthetic moored? | | 1.5 to 2.5 | | |
| Semi-submersible | | Slightly Better | | |
| Distance (ft) | | | | |
| Spread Mooring Factor | | 1 to 5 | | |
| Synthetic moored? | | 1.5 to 2.5 | | |
| Compliant Tower | | | Better | |
| Distance (ft) | | | | |
| Hub Jacket | | | Similar | |
| Distance (ft) | | | | |
| Other Jacket | | | Better | |
| Distance (ft) | | | | |
| | | | | |
| Other structures, or additional of above | | | | |
| | | | | |
| **Total Sum of Surface Facility Consequences for Octant** (see note below) | | | | |

[1]It is acceptable to ignore any facilities with design production capacity of below 50,000 BOE per day in the range between 10 and 15 miles.

[2]If computing a numerical value, then the numbers given in this column may be considered suitable multipliers modified, as appropriate, by the expressions "Better," "Worse," etc. Discussion of the values and influences, including possible reductions based on the design production capacity, can be found beneath Table K.5.

The angle subtended can be conservatively taken as 7° at one mile. So the likelihood of drifting towards the surface facility, *given that the MODU is drifting within this specific octant*, can be calculated as (L-surface$_f$):

$$\text{L-surface}_f\text{(Drifting towards facility | drifting in this octant)} = [7°/(\text{distance in feet}/6{,}800)]/45°$$

The total distant surface facility consequence value for the octant (Surface-dist$_v$) can therefore be calculated as:

$$\text{Surface-dist}_v = \Sigma(\text{Number} \times \text{Spread Moor} \times \text{Synthetic} \times \text{Value} \times \text{L-surface}_f)$$

The "Spread Moor" and "Synthetic Multiplier" are either unity, or adjusted to account for the presence of spread moorings or synthetic moorings.

### K.13.10.4 Summation over Octants

If computing numerical values, it becomes necessary to sum the results for the various octants, adjusting for the likelihood that the unit drifts within the specific octant. The results for each octant should be summarized in Table K.17, and then the total value, over all octants, can be calculated.

Table K.17—Summation Over Octants

| Octant | Direction Likelihood | | Subsea-dist$_v$ | Surface-dist$_v$ |
|---|---|---|---|---|
| | Base L.[a] | Modified L.[a] | | |
| N | 0.125 | 0.12 | | |
| NE | 0.125 | 0.06 | | |
| E | 0.125 | 0.06 | | |
| SE | 0.125 | 0.10 | | |
| S | 0.125 | 0.12 | | |
| SW | 0.125 | 0.18 | | |
| W | 0.125 | 0.18 | | |
| NW | 0.125 | 0.18 | | |
| **Individual Sum Totals Over all Octants** (see K.13.10.4.5) | | | | |

[a]The "Base Likelihood" is the conditional probability that the MODU will drift towards the relevant direction, given that the moorings have failed, if it is assumed that the drift direction is completely random. The sum over all directions is equal to 1.0, as it must be. "Modified Likelihood" has divided the drift direction conditional probabilities based, in part, on the direction of the worst winds in the series of storms used to create the database of metocean extremes over the four zones within the Gulf of Mexico. The data have been smoothed and also sum to 1.0. It should also be noted that even if the extreme winds are in the relevant direction, the direction at the MODU location will change over time, hence the drift direction will change.

Experience from the 2004 and 2005 hurricane seasons tends to confirm that in general drift is towards the northwest, but that is based on a very limited number of events. An alternative way of viewing this is that the highest winds within a hurricane are normally in the northeast quadrant, blowing towards the northwest. It is likely that a MODU will first be affected by the northern side of a hurricane before the southern side. Hence, the most likely direction of MODU drift can be estimated.

It can be seen that there are a number of arguments for either a relatively even distribution of drift directions, or a weighted distribution based on a combination of experience (albeit limited), logic concerning the likely MODU location with respect to the storm, and the data on metocean extremes.

When deciding which set of values to use, it may be helpful to consider the MODU location, and the adjacent infrastructure. If infrastructure is randomly distributed around the MODU, then it makes little difference which series of numbers is used, however, there may be merit is using both approaches, and reporting results from the more conservative. This exercise would be particularly important if there was a strong directionality to the infrastructure.

If computing numerical values, the sums over all octants can be calculated as:

$$\text{Subsea.all-dist}_v = \Sigma(\text{Subsea-dist}_v \times \text{Direction Likelihood})$$

$$\text{Surface.all-dist}_v = \Sigma(\text{Surface-dist}_v \times \text{Direction Likelihood})$$

### K.13.11 COMBINED LOCATION CONSEQUENCE FACTORS

If computing numerical values, the intermediate consequence factors can be taken from the parameters described in K.13.3 through K.13.10, and included in Table K.18.

Table K.18—Combined Location Consequence Factor

| Parameter | Factors | Surface Value | Subsea Value | Description |
|---|---|---|---|---|
| $\text{Block}_v$ | | | | Block area value (same value for both surface and subsea) |
| $\text{WD}_f$ | | | | Water depth factor (on subsea values only) |
| $\text{Subsea-in}_v$ | | | | Subsea within the mooring pattern |
| $\text{Subsea-out}_v$ | | | | Subsea outside mooring pattern but within one radius |
| $\text{Cross}_v$ | | | | Crossed moorings with a temporary facility |
| $\text{Surface-in}_v$ | | | | Surface facility within the mooring pattern |
| $\text{Surface-out}_v$ | | | | Surface facility outside mooring pattern but within one radius |
| $\text{Subsea.all-dist}_v$ | | | | Subsea facilities within 15 miles |
| $\text{Surface.all-dist}_v$ | | | | Surface facilities within 15 miles |
| | | | | |
| $\text{Surface.total}_v$ | | | | Total site Surface Consequence Factor (see note below) |
| $\text{Subsea.total}_v$ | | | | Total site Subsea Consequence Factor (see note below) |

The total consequence factors are summed from Table K.18, although the subsea factor needs to be modified by the water depth to account for the increased difficulty of repairs. They are each given by:

$$\text{Surface.total}_v = 0.5 \times \text{Block}_v + \text{Cross}_v + \text{Surface-in}_v + \text{Surface-out}_v + \text{Surface.all-dist}_v$$

and

$$\text{Subsea.total}_v = 0.5 \times \text{Block}_v + \text{WD}_f \times (\text{Subsea-in}_v + \text{Subsea-out}_v + \text{Subsea.all-dist}_v)$$

### K.13.12 LIKELIHOOD ADJUSTMENT FACTORS

#### K.13.12.1 General

The previous sections address the consequences of a MODU suffering a mooring failure. There are, however, some factors associated with the mooring system design that influence the likelihood of suffering the consequences and can have a direct influence on the consequences. These are discussed below.

If computing numerical values, in most cases the base case factor will have a value of 1 with alternates reducing that value.

#### K.13.12.2 Anchor Type and Pullout

To mitigate subsea infrastructure damage, the best anchors are those that will not pull out of the seabed and will not drag components across the seafloor in the event of any mooring failure. Drag anchors have a higher modified consequence value than other anchor types. The reason is that there is a relatively high likelihood that the leeward anchors will pull out in the event of windward mooring line failure. Hence, even if the anchor has a higher holding capacity than the strength of the mooring line, if the windward lines fail, the leeward anchors likely will be dragged as the MODU drifts over them and pulls them in the "wrong" direction. Notwithstanding this, drag anchors performed remarkably well during the MODU failures in Katrina and Rita. There were only four units that dragged anchors for over 1 mile.

It is important that if piles, Normally Loaded Plate Anchors, or Near Normally Loaded Plate Anchors are used to mitigate the likelihood of damaging subsurface equipment, then their holding capacity should be greater than the maximum breaking strength of the mooring line in any possible loading direction, and in all reasonably possible soils conditions, even considering all possible mooring line failure combinations. In addition, the installation procedure should be developed to ensure that this holding capacity can be achieved. This is particularly important for the Normally Loaded Plate Anchors which will tend to pull out if overloaded rather than drag at depth, or continue to embed. It is of note that during Hurricane Rita one unit fitted with normally loaded plate anchors suffered a mooring failure and dragged some of the anchors for over 100 miles. The likely reason is that the anchors were installed with insufficient horizontal load. However, even though the plate anchors were dragged for over 100 miles, crossing many pipelines, the extent of damage caused was reported to be limited. The manufacturer asserts that this is an advantage of the anchor not being designed to self-embed, and hence not as likely to grab subsea equipment. While the results of the historical experience are compelling, it may be difficult to justify a low numerical value for a dragged a plate anchor, particularly if used to offset the increased likelihood due to low holding capacity. Hence, unless pull out can be absolutely guarded against through design and installation, normally loaded and near normally loaded plate anchors should be rated at a comparable level to “high holding capacity anchors [less capacity than mooring minimum break load (MBL)]” with some possible reduction for their reduced propensity o grab subsea equipment.

### K.13.12.3 Anchor Pullout

The historical database of MODU mooring failures during 2004 and 2005 hurricane season contains 17 documented cases of MODUs drifting for over one mile, and in only four cases were anchors dragged for a significant percentage of that drift distance. Of those four cases, in only two were anchors dragged for over 5 miles. While the database is not large, it is reasonable to say that the likelihood of dragging an anchor over the subsea infrastructure itemized above is in the range of 12% to 25%. If computing numerical values, then it is reasonable to include an Anchor Drag Factor as follows:

Anchor Drag Factor: in the range 0.12 to 0.25

If computing numerical values, the subsea factor for anchors ($\text{Anchor}_f$) should be reported as:

$$\text{Anchor}_f = \text{Subsea Multiplier from Table K.19} \times \text{Anchor Drag Factor}$$

Table K.19—Anchor Type

| Anchor Type | Subsea Multiplier[a] |
|---|---|
| Suction Pile (designed for significant out-of-plane loading) | Best Option |
| Suction Pile (NOT designed for significant out-of-plane loading) | Significantly Better |
| High Holding Capacity Drag Anchor (HHCDA) (capacity greater than capacity of mooring line) | Much Better |
| High Holding Capacity Drag Anchor (HHCDA) (less capacity than capacity of mooring line) | Better |
| Conventional Drag Anchor | Base: 1.0 |
| Normally Loaded Plate Anchor | Much Better |
| Near Normally Loaded Plate Anchor | Much Better |
| [a]The anchor subsea multiplier should be chosen based on two criteria: (i) what is the likelihood that an anchor will pull out of the seabed before all the mooring lines have parted, and (ii) the potential consequences of dragging such an anchor over the subsea infrastructure. | |

### K.13.12.4 Mooring Component at the Seafloor

The mooring component that is at the seafloor will have a major effect on the amount of subsea infrastructure damage in the event of a mooring line failure at, or close to, the anchor. Due consideration should be given to the potential extent of damage and its significance to operations. A pipeline that is damaged, but can continue in service until it can be scheduled for repair at a time of no flow is a far less serious loss than a pipeline of comparable flow that has to be immediately taken out of service. Conversely, even minor damage to a pipeline’s insulation or cathodic protection may lead to long term problems that can be expensive to repair, particularly if left and forgotten about. As a general rule, damage that can be repaired later is far less critical than damage necessitating immediate action. This needs to be borne in mind when deciding on the damage factor for dragged mooring components.

The intent of the question is to ascertain what types of component can be dragged along the seabed, regardless of where the mooring line failure occurs, not just the component at the anchor. For example, if a mooring line is comprised of rig wire, intermediate chain, and anchor wire, and the rig wire is sufficiently long such that both it and the intermediate chain can be dragged along the seabed, then the least favorable chain would be assumed to be at the seabed for the purpose of determining the subsea factor for the component at the seabed

It is of note that in the 2005 hurricane season approximately 5% of mooring lines failed at the anchor, and an additional nearly 15% failed in intermediate mooring component between the fairlead wire or chain and the anchor, so there is a relatively high likelihood that a MODU suffering a mooring line failure will have components that have broken at both the fairlead and away from the fairlead.

If computing numerical values , then the base factor, to be used for chain, has to be chosen depending on the expected quantity of damage due to a dragged chain by comparison to other components (note, drag anchor factor = 1.0). The suggested method of combining consequences given below is to use the factor for the worst dragged mooring component or anchor as the factor to be applied to all subsea equipment. The base number if one is following this approach, therefore, is the amount of damage that a dragged chain would cause by comparison to a dragged anchor with a value of 1.0. While chain may cause significant damage, it is unlikely to cause as much as a dragged anchor. A suggested range of 0.1 to 0.3 has been included for consideration. Based on this discussion, and that above, the subsea factor for the component at the seabed ($\text{Component}_f$) should be reported as:

$$\text{Component}_f = \text{Multiplier from Table K.20}$$

Table K.20—Mooring Component at Seafloor

| Component at Seafloor (Worst case should be chosen) | Subsea Multiplier |
|---|---|
| Chain at seafloor | Base: 0.1 to 0.3 |
| Wire at Seafloor | Much Better |
| Synthetic at seafloor | Significantly Better |

### K.13.12.5 Subsea Buoyancy Used to Mitigate Local Subsea Damage

In certain mooring arrangements, it may be advantageous to include either additional buoys or lengths of synthetic mooring line in order to give additional protection to subsea components (see Table K.21). It is possible that these buoys were used to aid in mooring line installation or mitigate the risk of dropped mooring lines during installation. The primary area of interest in this particular question concerns mooring equipment falling onto the subsea infrastructure, not being dragged over it. This item is addressed in K.13.12.6.

Table K.21—Mitigation of Close Subsea Infrastructure

| BUOYANCY OR SYNTHETIC USED TO MITIGATE DAMAGE TO CLOSE SUBSEA INFRASTRUCTURE | Factor | Possible Reduction Factor |
|---|---|---|
| Are buoys or length of synthetic mooring line included within the mooring arrangement that will mitigate the risk to subsea infrastructure due to falling mooring components? | Yes = 1<br>No = 0 | See discussion in this section. |

A buoy will be very effective at mitigating damage to local subsea infrastructure under certain limited circumstances, as follows.

- The mooring line fails between the buoy and the fairlead and the buoy prevents anchor chain or wire from falling on subsea components.
- The buoy is positioned such that if the mooring line fails between it and the anchor, it will prevent the upper mooring components from falling onto the seabed. In addition, the length of line between the buoy and the anchor should be insufficient to drop on any subsea infrastructure, or should be synthetic.

The buoy will not be effective if:

- the mooring line fails at the fairlead (in most cases the rig wire plus intermediate component would be long enough to allow the rig wire to fall onto the seabed);
- the buoy has insufficient buoyancy to carry the additional load of the failed components;
- the buoy collapses due to increased hydrostatic head when dragged down by mooring components.

Similar scenarios exist for the use of synthetic components to mitigate damage to subsea infrastructure.

Some additional information from the 2005 hurricane season may help illuminate the issues. Approximately 80% of all the mooring failures were in the rig component at, or close to, the fairlead. This means that there is a very high likelihood that out of, say, four lines that may be extended over a piece of subsea infrastructure, there will be at least one that fails at the fairlead. If the buoy or synthetic line does not mitigate damage to the subsea infrastructure under this circumstance, then it will have almost no effect on the likelihood of damage.

It is clear from the preceding discussion that there will be a limited number of cases in which buoyancy or synthetics can be effectively used to mitigate the risk of mooring components falling onto local subsea infrastructure. However, if it can be conclusively shown that there is real mitigation offered through the use, then advantage may be taken. It is suggested that only a relatively small reduction should be taken if numerical values are being calculated (i.e., use a factor close to unity), and only for subsea infrastructure within the mooring pattern, or within one mooring radius of any anchor.

If computing numerical values, the reduction factor for use of buoyancy or synthetics to mitigate damage to subsea infrastructure (Buoy-in$_f$) should be based on the discussion above, and reported as:

$$\text{Buoy-in}_f = \text{Factor representing mitigation from Table K.21}$$

### K.13.12.6 Subsea Buoyancy Used to Mitigate Remote Subsea Damage

It is possible to envisage scenarios in which subsea buoys could be used to help prevent mooring components being dragged across the seabed in the event of a MODU mooring failure. However, for similar arguments to those given when discussing the use of buoyancy to mitigate damage due to falling components on close in subsea infrastructure, there are too many ways in which the system can fail to work effectively unless carefully designed. As an example, approximately 5% of the 2005 hurricane season line failures occurred at the anchor, so if there is any anchor chain at the anchor, there is a relatively high likelihood that it will be dragged over the seabed due to insufficient buoyancy. In addition, approximately 20% of MODUs that suffered a mooring failure dragged their anchors for over 1 mile, so no amount of buoyancy would have prevented the potential for damage.

For these reasons, it is suggested that this factor not normally be considered unless it can be conclusively shown that there is real mitigation offered through the use of the buoys under all credible scenarios. Buoyancy can only be considered if there is no anchor dragging, so is limited to mitigating the effects of dragged wire, chain, or synthetic and can only be realistically used with anchor piles.

If computing numerical values, the reduction factor for use of buoyancy to mitigate damage to subsea infrastructure outside one mooring radius of any anchor (Buoy-out$_f$) should be based on the principles presented in this section, and reported as:

$$\text{Buoy-out}_f = \text{Factor representing mitigation from Table K.22}$$

Table K.22—Mitigate Damage Due to Dragged Mooring Components

| BUOYANCY USED TO MITIGATE DAMAGE TO SUBSEA INFRASTRUCTURE FROM DRAGGED COMPONENTS | Factor | Possible Reduction Factor |
|---|---|---|
| Are buoys or length of synthetic mooring line included within the mooring arrangement to mitigate the risk to subsea infrastructure? | Yes = 1<br>No = 0 | See discussion in this section. |

## K.13.13 COMBINED CONSEQUENCE VALUES AND ADJUSTMENT OR MITIGATION FACTORS

Having determined the main consequence values and likelihood adjustment factors, it is necessary to consider the effects of combining them. In principle, the various components can be combined in a number of different ways, depending on how and what was developed. However, if computing numerical values, the following discussion may help give some guidance on one possible approach.

The total surface infrastructure consequence value (Surface.total$_v$) is basically as it is, without any modifiers included. It has been assumed that the reliability of the mooring system is the driver for preventing damage to surface infrastructure, and the suitability of that should be determined, in part, based on the results of this consequence assessment. This is, in reality, a simplification that will tend to produce conservative results. It may be possible to more precisely determine the potential interactions between a drifting MODU and the surface facility of interest and therefore refine the numerical values used. An example could be the interaction between a drifting MODU and the mooring lines of a spar. Simple probabilistic analysis can give the likelihood that different MODU mooring components will interact with the spar's moorings, and therefore affect the likelihood of damage. Detailed methodology is beyond the scope of this simplified checklist approach, but the basic information can be used in many ways to refine the analysis results.

The total subsea infrastructure consequence value (Subsea.total$_v$) will also be affected by the mooring system reliability, but will be modified by the anchors used, and components at the seabed. These components have a direct effect on the extent of damage that can be caused and the likelihood of sustaining that damage. It may also be modified by factors designed to prevent mooring components dropping or dragging, if these have been defined. The simplified method of determining total consequences based on the subsea adjustment factor given below will tend to produce a conservative result. It is possible, and reasonable, to assess different mooring component drag factors for the different subsea infrastructure. As an example, an umbilical will likely be severely damaged by a dragged wire rope whereas a pipeline may suffer very minimal damage. As discussed in K.13.12.4, immediate failure tends to be far more serious than damage that can be scheduled for later repair without affecting production. Hence, by subdividing the subsea infrastructure into constituent parts and using different component damage factors for each subsea item, it is possible to determine a refined subsea consequence factor. This will tend to be important when there is a large amount of subsea infrastructure within the 15 mile radius of the well location.

A simple summation methodology is to assume that the worst of the factors for either the mooring component at the seabed or the anchor should be used and then applied to the subsea infrastructure consequence factor. Hence, the subsea adjustment factor (Drag$_f$) is given by:

$$\text{Drag}_f = \text{Greater of Anchor}_f \text{ or Component}_f$$

The total adjusted consequence value (Total-Consequence$_v$) is then given by:

$$\text{Total-Consequence}_v = \text{Surface.total}_v + \text{Drag}_f \times \text{Subsea.total}_v$$

## K.14 Risk Assessments for MODU Operations

### K.14.1 OVERVIEW AND INDUSTRY EXPERIENCE

The purpose of this appendix is to describe the basic elements of a risk assessment and to identify the primary drivers of MODU operational risks as they relate to mooring failure. This appendix does not provide detailed guidance on risk assessment methods, only a general overview related to this specific subject.

The offshore industry has been extremely successful in evacuating platforms in advance of hurricanes without loss of life. Given the number of people that have had to be evacuated, and the limited time available, this has been a remarkable achievement.

Also, in the 2004 and 2005 hurricane seasons there was very little pollution due to the failures that did occur in hurricanes. There were some small spills from damaged pipelines, but these were minor by comparison to the potential losses that could have occurred.

It was in the areas of asset loss and industry reputation that the greatest damage occurred. There was a strong perception that industry had not done a good job because of the number of fixed platforms destroyed, pipelines damaged, jackups lost or damaged, and MODUs that drifted. The overall result was that the majority of the Gulf of Mexico production was shut-in for months. The following discussion is aimed specifically at the issues affecting moored MODUs.

There were approximately 20 semi-submersible MODUs that lost station in the 2004 and 2005 hurricane seasons. Some of these units drifted for well over 100 miles while others remained in the same vicinity, with a reduced number of mooring lines holding. Notwithstanding the number of units that went adrift, the actual damage to Gulf of Mexico infrastructure caused by drifting semi-submersible MODUs was small. The greatest damage was to the Mars export pipelines. This damage could have resulted in the field and tieback production being shut-in for three months, but the actual consequences were more than an order of magnitude less because the facility had wind induced damage that resulted in an eight month shut-in. [Ref. 1, 2, 3 and 4]

Despite the lack of significant damage to Gulf of Mexico infrastructure from these mooring failures, there is an expectation that industry should improve the reliability of MODU moorings. There is also an expectation that the potential consequences of a mooring failure be fully understood, and where necessary, suitable mitigation measures implemented to reduce the potential infrastructure damage should a MODU break adrift. One of the outcomes of the work undertaken to achieve these ends has been the increased use of risk analysis techniques to assess proposed drilling locations.

The question may be raised as to why mooring failure risk assessment techniques are not required for floating permanent facilities by this document while they are now recommended for mobile platforms? A key difference is MODUs are designed as mobile facilities, and are intended to deploy and retrieve their mooring systems repeatedly and in a limited time frame with readily available equipment. Permanent floating facilities utilize specialized installation vessels due to the size of their mooring components and installation takes more time requiring a longer mild weather window. In addition, the consequence of a mooring failure for a MODU can be very different compared to a permanent floating facility. The list of these differences is numerous, but in general, the financial consequence of a permanent mooring line or system failure will be far greater than that of a MODU. Therefore, it is not appropriate to use the same criteria as used for permanent facilities. However, to offset the increased probability of suffering a mooring failure due to a lower design return period, the consequences of such a failure need to be suitably assessed.

## K.14.2 TERMS AND DEFINITIONS

For the purposes of this section, the following terms and definitions apply.

**K.14.2.1 consequence:** The effects of a mooring failure.

**K.14.2.2 hazard:** An event with potential adverse consequences.

**K.14.2.3 initiating event:** Hardware failure, control system failure, human error, extreme weather or geophysical event, which could lead to hazards being realized.

**K.14.2.4 likelihood:** The conditional probability of an event (consequence) occurring. That is, at a minimum the likelihood of an event occurring is conditional on partial or complete mooring system failure.

**K.14.2.5 mooring failure:** The definition of mooring failure will vary depending on the circumstances, but for the purpose of a risk assessment, a mooring failure is a failure of any parts of the mooring system that could lead to adverse consequences. For some mooring systems, for example deployed over pipelines, a mooring failure could be as simple as an anchor slipping a short distance. For a MODU operating far from any infrastructure, a mooring failure may necessitate breaking a number of mooring lines and dragging anchors for a considerable distance: lesser failures, such as loss of some mooring lines and limited anchor slippage, may be considered acceptable if the MODU remains in the vicinity of its drilling location.

**K.14.2.6 probability:** The relative frequency that an event will occur, as expressed by the ratio of the number of occurrences of the event to the total number of possible occurrences.

**K.14.2.7 reliability:** A measure of the probability of an item or system to adequately perform a required function under stated conditions of use and maintenance for a stated period of time.

**K.14.2.8 risk:** The product of the probability of occurrence of a hazardous event and its consequence(s).

**K.14.2.9 tolerable risk:** risk which is accepted in a given context based on the current values of society. Also referred to as acceptable risk.

## K.14.3 RISK ASSESSMENT GENERAL

Risk is defined as:

$$\textit{Risk} = [\textit{Probability of an adverse event occurring}] \times [\textit{The consequences associated with that event}]$$

Risk assessment is the study of the probability of an adverse event occurring, the potential consequences of that event, and the measures taken to reduce the probability and consequences of such an event. A fundamental part of reducing the risk is to ensure that all parties have a clear understanding of their "Risk Exposure." The risk can be reduced either by reducing the probability of experiencing an incident, prevention, or by reducing the consequences of that incident should it occur, mitigation.

Accidental and extreme environmental events may result in a number of adverse consequences, including:

- injury or fatality;
- environmental damage;
- property damage, potentially leading to a disruption of production due to damage to Gulf of Mexico infrastructure;

- damage to corporate image;
- deterioration in public perception and industry reputation.

Risk management can be used to assess and maintain risks within acceptable levels. Risk assessment techniques can be used to evaluate frequencies and potential consequences of accidents. Risk assessment methods can also be used to help evaluate and sort the risks. Further analysis can then be used to help determine and implement mitigation measures, where applicable. Risk management is a process that can be effectively integrated into future operations to provide continuous improvement: past experiences can be reviewed with the aim of improving both system reliability and reducing adverse consequences.

A major advantage of undertaking a risk assessment is that it requires the stakeholders to be involved in the process and give consideration to the issues that make up the potential risk. This does not require massive attendance at, for example, the HAZID workshops, but there is a danger in making any risk assessment process too automated. Risk analysis is an efficient way of helping to comprehend and evaluate issues that cannot be quantified by conventional means or design codes. It is through *the process* that the participants understand the risks which, in turn, help them to decide on the appropriate acceptance criteria and correct mitigation measures.

## K.14.4 MODU-SPECIFIC RISK ASSESSMENT OVERVIEW

### K.14.4.1 General

The probability and consequences of a MODU losing station when operating at any location within the US Gulf of Mexico should be assessed. The intent of the assessment process is to determine the characteristics of the area of operation and identify options related to mooring component selection, mooring system design, scheduling of operations, and mitigation opportunities prior to finalizing the mooring design and installing the mooring system. For the planned MODU operation, the mooring system should be associated with an acceptable risk, either by minimizing potential consequences of mooring component or system failure or by reducing the probability of mooring component or system failure.

The general risk definition is composed of the following elements:

1. Probability of a hurricane producing extreme environmental conditions at site:
   - hurricane occurring;
   - distribution of hurricane intensity;
   - distance of hurricane track from MODU location.
2. Probability of MODU mooring failure:
   - strength of mooring line:
   - holding capacity of anchor;
   - mooring component resistance (anchor or line) is less than demand.
3. Likelihood and consequence:
   - likelihood of MODU drifting toward surface infrastructure;
   - likelihood of MODU dragging or slipping anchors toward subsurface infrastructure;
   - likelihood of damage to infrastructure of differing values (damaged or lost asset or lost production).

The metocean criteria for a specific location reflects the combination in item 1. The probability of suffering a mooring failure reflects the combination in item 2. The likelihood of causing damage, having suffered a mooring failure, is given in item 3. The risk is determined by the product of the probability and suitably summed consequences.

The financial consequences of a MODU mooring failure can be divided into three types:

1. consequences of damage to the surrounding subsea and surface infrastructure;
2. consequences of damage to the MODU and its mooring system;
3. consequences to Operator's drilling program.

The risk assessment described in this appendix addresses the consequences of damage to surrounding infrastructure. For MODU operations in the hurricane season, where the MODU is evacuated, it is the responsibility of the Drilling Contractor and Operator to manage the risk associated with damage to the MODU and its mooring system.

The probability of mooring failure can be assessed through analysis of the mooring design, and the consequences of failure assessed by giving due consideration to the infrastructure local to the proposed drilling location. Figure K.10 shows the general

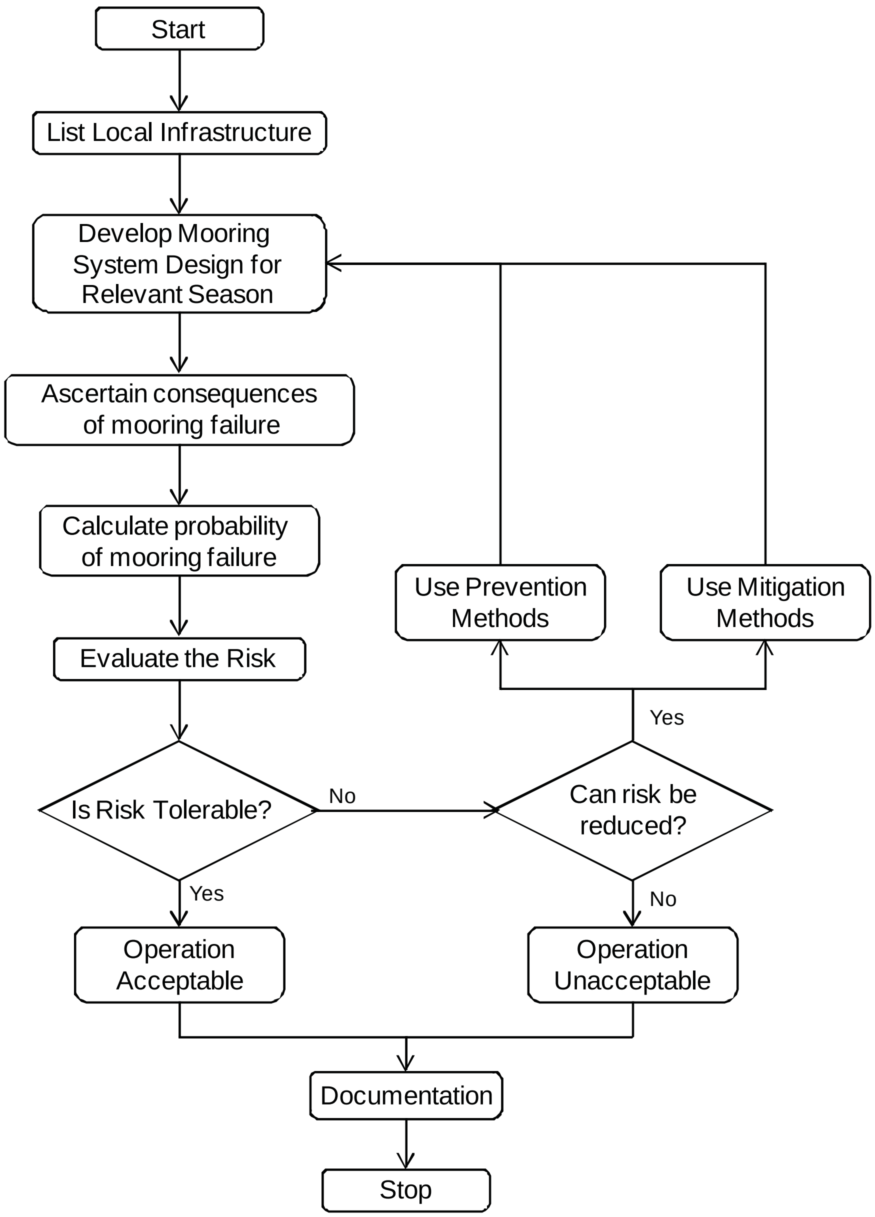



Figure K.10—Mooring System Failure Risk Assessment Overall Process

methodology for carrying out a risk assessment of MODU mooring failure when considering the potential for damage to the surrounding infrastructure.

Each element of this overall process is discussed as follows.

a. **List Local Infrastructure:** This covers all surface and subsurface infrastructure that can suffer consequences from a MODU mooring failure. The list may be divided up according to infrastructure that is within the mooring pattern, within one mooring diameter, within 15 nautical miles and infrastructure beyond 15 nautical miles that may require special consideration due to size and importance.

b. **Develop Mooring System Design for Relevant Season:** This requires performing a mooring analysis of the proposed mooring system to API 2SK using appropriate metocean design criteria. See K.11 for additional information.

c. **Ascertain Consequences of Mooring Failure:** Using an appropriate process, such as hazard identification (HAZID), all the potential consequences of a MODU mooring failure are identified. Further information can be found in K.14.8. Information on logic to perform a consequence assessment is found in K.13.

d. **Calculate Probability of Mooring Failure:** The probability of MODU mooring system failure decreases with increasing design return period for which the mooring system satisfies all of the requirements of API 2SK. Further information can be found in K.14.7.

e. **Evaluate the Risk:** This involves taking information from probability of mooring failure and consequence of mooring failure and assessing the risk of the proposed MODU mooring operation. See K.14.9.

f. **Is Risk Tolerable?** Once the risk is evaluated, the Operator's standards shall be used to determine if the risk is tolerable. Further information can be found in K.14.10.

g. **Can Risk Be Reduced?** Practical mitigation or prevention measures should be evaluated and implemented as appropriate to reduce the risk. See K.14.11.

h. **Operation Acceptable or Unacceptable:** Using all the information from the risk assessment process, the Operator can determine if the location for a given season and MODU mooring system is acceptable or not. If no further mitigation or prevention measures are possible and the risk is deemed high, then the operation should be rejected. Usually changing the season of operation, in particular if it is planned for peak of hurricane season, and rescheduling to off-peak or non-hurricane season will make the operation acceptable. See K.14.10 for additional information.

i. **Documentation:** The risk assessment process must be formally documented. The documentation should include all information required to support the risk assessment results. Further information can be found in K.14.11.

Without participation by personnel experienced in MODU moorings, their failure modes, and infrastructure sensitivity, the risk assessment process will not advance to the stage where it can become meaningful.

## K.14.5 TYPES OF CONSEQUENCES

Most corporate risk assessments include five types of consequence. These five have been supplemented by an additional consequence type, national interest, that is specific to offshore operations in the Gulf of Mexico. The six consequence types are:

1. health and safety;
2. environmental;
3. financial;
4. corporate reputation and image;
5. industry reputation and image;
6. national interest.

National interest has been added since Gulf of Mexico oil and gas production has a direct influence on the cost and availability of hydrocarbons to the public and the requirements for imports.

For MODU operations in the hurricane season where the MODU is evacuated and wells and pipelines are shut-in, health, safety, and environmental consequences associated with MODU mooring system failure are relatively low. Assessments of consequence types 4 through 6 will be subject to considerable corporate interpretation, and there will be large variations in risk tolerance. In the case of industry reputation (5) and national interest (6), the consequences depend on the performance of all MODUs operating in the Gulf of Mexico at any one time. The consequences of failure will include public and regulatory perception, which will be influenced by the number of MODUs that fail and the result of those failures on other industry infrastructure in a single hurricane, hurricane season, or few years.

In addition to the risk analysis of specific MODUs, there are also considerations for the performance of the entire MODU fleet. There is an expectation that a large hurricane in the Gulf of Mexico may cause multiple MODUs to break adrift, however, there are limits on the acceptable number of drifting MODUs. Fleet performance is difficult to address through individual MODU risk assessments: it is more driven by overall minimum design return period requirements and fleet distribution throughout the Gulf of Mexico.

## K.14.6 RISK ANALYSIS

### K.14.6.1 General

Risk assessments can be either qualitative or quantitative. The first step for either of these risk assessments is a Hazard Identification (HAZID), resulting in event trees that can be evaluated either qualitatively or quantitatively.

The normal expectation is that the HAZID would be used to assess the broad risks that may be associated with a location, including issues such as:

- recovery of a drifting rig;
- personnel evacuation;
- sudden hurricane survival capability;
- mooring installation and recovery risks;
- additional effects of mitigations that may be required by the main mooring failure risk assessment (e.g., consequences of necessitating additional or stronger mooring equipment).

Direct consequence mooring risk assessment is the main issue discussed in this appendix. This type of assessment may be used to assess the specific probability and consequences of suffering a mooring failure at a particular location (e.g., probability of mooring failure, and the consequences and likelihood of the MODU or its mooring system interacting with both near and far infrastructure). In this assessment, frequency of occurrence of hazardous events, the likelihood of escalation into further accidents, and the magnitude of potential consequences are evaluated. The methods used could be qualitative or quantitative depending on the system definition and the objectives of the risk assessment. The initiating event frequencies and the likelihood (conditional probabilities) of the chains of events leading to accident scenarios are established using a combination of historical databases, fault tree, event tree, and reliability analysis techniques. Historical data [1, 2, 4] are available for some failure probabilities, but there is limited experience within the Gulf of Mexico. Experience from other areas may not be appropriate due to differences in environmental conditions, water depth, and mooring line components.

Fault tree analysis can be used to estimate the probability of suffering a mooring failure, but in most cases it will be more accurate and easier to use a combination of historical data and structural reliability calculations. Event trees are often used to estimate the likelihood of event escalation and the associated consequences.

Consequence analysis involves analyzing the range of possible outcomes of an accident or initiating event. Consequence analysis can be carried out in either a qualitative or quantitative manner. When undertaking quantitative analysis it is important to have sufficient data available to make realistic estimates.

Qualitative consequence analysis, typical of a HAZID, involves verbal development of an accident scenario and then subjectively evaluating the consequence intensities. It often uses a Risk Matrix to assess the results.

In most mooring failure risk analyses it is anticipated that there will be a combination of qualitative and quantitative risk assessment, both using a risk matrix approach, or similar, to assess acceptability.

### K.14.6.2 Levels of Detail

The detail within the risk analysis process should be designed to complement the level of work required to understand the site-specific situation and demonstrate compliance, or lack thereof. There is little point in performing extremely detailed risk analyses if the proposed location is far from any significant infrastructure. Equally, one can expect to perform a detailed analysis if the intent is to prove acceptability of a location that is in a highly congested area either because of production infrastructure or multiple MODUs.

A basic risk assessment may entail following a process similar to that contained in K.6.3. A detailed/supplemental risk assessment may include a quantification of the risks, including quantification of both the probability of mooring failure and the consequences and likelihood of interacting with surrounding infrastructure.

### K.14.6.3 Data Requirements

The data required for a risk assessment will depend on the level of analysis being undertaken, as discussed below. The following information is required in order to determine the consequences of a mooring failure. In addition, it will be necessary to determine some measure of the probability of suffering a mooring failure.

#### K.14.6.3.1 Data Requirements for All Level of Analysis

As a minimum, the following data will be needed to undertake a MODU mooring failure risk assessment.

- The global mooring system description and mooring analysis report, including anchoring system assessment.
- Details on the mooring line components including jewelry and interconnecting hardware (to help ascertain potential for damaging subsea infrastructure).
- Anchor details including type, weight, capacity, fluke angle (if relevant) and any specific geotechnical limitations.
- List of all surface and subsurface infrastructure within 15 nautical miles of the wellsite, including:
  - infrastructure within the mooring pattern;

- infrastructure within one mooring radius of any anchor;
- infrastructure within 15 nautical miles of the wellsite.

One source of infrastructure information is the Gulf of Mexico infrastructure map maintained by the Minerals Management Service.

The mooring system inspection should be current and in compliance with API RP 2I. While this will not, by itself, assure that the mooring components will not fail at loads significantly below their catalogue values, it should help minimize the probability of such failures.

Infrastructure details should include size and type of any pipeline. If information on flow rate is unavailable conservative estimates should be used.

Information will be required on the physical size of surface facilities. Such information should include both the size of the structure above the waterline and the size of its seabed footprint as well as rated hydrocarbon processing capacity. The likelihood of interacting with surface infrastructure, in the event of a mooring failure, will depend on the size and distance of the surface infrastructure from the wellsite. For a TLP, jacket, etc. the surface and subsurface-footprint size will be similar, although export risers may be considered to have the same effect on subsurface-footprint size as mooring lines (discussed below).

Spread moored infrastructure needs to be considered differently from other structures. With spread moored infrastructure there is a possibility that broken MODU mooring components will be dragged and damage the moorings of the moored permanent facility. The likelihood and consequences of this interaction will depend on:

- size of the facility's mooring pattern (footprint size);
- mooring components used on the facility (chain, wire, polyester, etc.);
- likely mooring components being dragged by the MODU.

As an example, if the facility's moorings include polyester mooring lines and the MODU mooring lines are steel, then it is more likely that facility's moorings will be damaged by dragged MODU components, assuming there is some interaction. Conversely, if the MODU moorings are largely synthetic, and the facility's moorings are steel, the interaction is less likely to cause substantial damage.

#### K.14.6.3.2 Data Requirement for Detailed/Supplemental Risk Assessment

In some cases it may not be necessary to gather additional information for the detailed risk assessment. However, if close operations between the MODU and a permanent surface facility are considered, then it will be necessary to fully understand the potential interactions between the MODU's mooring system and the surface facility's above and under water structure. This type of situation may be addressed within a detailed risk assessment and would likely be classified as an "Atypical Operation" in K.6.4.

As an example, consideration needs to be given to the following list which is not all inclusive.

a. The possibility of damaging a mooring line, TLP tendon or their foundations by dragging a MODU mooring line over it.

b. Mooring line interaction with SCR(s), subsea flowlines, manifolds or trees.

c. Mooring line interaction with the hull of a floating structure or the structural elements of a jacket and any of their critical appurtenances.

d. Potential to damage fuel or crude oil tanks leading to pollution (e.g., if such tanks exist in exposed pontoons on a permanently moored semi-submersible).

e. Damage survivability of the surface facility including:

   - damaged stability;
   - tendon failure;
   - structural reserve;
   - damaged mooring capability;
   - effect on risers.

The intent is to document the realistic scenarios so that their criticality and likelihood can be determined. For example, in most cases any collision between a MODU and a surface facility will occur during the passage of the storm, so survivability of the surface facility should be viewed in light of the likely storm condition at the time of damage. It may be that, for example, a TLP has a single tendon damage survival case that includes a severe storm, so it may then be reasonably assumed that the TLP will survive a "loss of tendon" scenario. Conversely, should the TLP have limited storm survival capability with tendon damage, then it would not be unreasonable to assume that loss of the tendon is equivalent to loss of hull system, based on the high likelihood of the entire tethering system failing.

What is imperative is that sufficient data is gathered so that all possible interaction scenarios can be adequately considered and, where necessary, quantified.

## K.14.7 PROBABILITY OF MOORING SYSTEM FAILURE

### K.14.7.1 General

An integral part of any risk analysis is the determination of the probability of the initiating event. In the case of a mooring risk assessment, the initiating event is mooring failure. The definition of failure will vary depending on the circumstances, but for the purpose of a risk assessment, a failure is any event that could lead to adverse consequences. For some mooring systems deployed over pipelines, that could be as simple as dragging an anchor a short distance (see definitions for more details).

A simple estimate of the probability of MODU mooring system failure can be taken as the inverse of the return period corresponding to the load where failure is expected. In doing this, consideration should be given to actual anchor capacity and mooring line strength. Mooring line strength may be reduced due to component degradation, bending over the fairlead, etc., in comparison to catalog values. (See K.3.2.)

One way to determine the probability of suffering a mooring failure is through the use of reliability analysis, thereby incorporating to some degree the uncertainty in the environmental loads and the component strengths. Due to uncertainty in the definition of basic variables, methods of structural reliability analysis are best suited to the calculations of comparative system probabilities of failure. Caution should be exercised when using structural reliability methods to calculate absolute probabilities of failure.

The sophistication of the reliability analysis can vary depending on the level of detail required, but in all cases the following factors should be considered.

a. The slope and shape of the metocean extreme curves (parameter vs. return period). This will vary depending on location and season within the Gulf of Mexico.

b. The distribution of mooring component strength. The mean value will vary with age, use, maintenance, etc. A mooring system reliability analysis should not be based on the full CBS of the mooring components but should reflect the component age, inspection history results, and fairlead bending considerations.

c. Variability in reliability depending on storm approach direction with respect to MODU heading.

d. Different reliability of mooring systems with different redundancy.

### K.14.7.2 Mooring Component Failure Locations

The location of the failure along the length of the mooring line affects the possible consequence of mooring failure. The effects of having multiple mooring lines needs to be carefully considered when determining the likely mooring line failure locations. As an example it is reasonable to assume that 80% of mooring lines fail close to the fairlead, but on an eight line unit there is approximately an 85% chance that at least one line will fail away from the fairlead. Similarly, only approximately 5% of lines failed at the anchor in the 2005 hurricane season, but that equates to over 33% probability that on an eight line unit there will be at least one mooring line failure at the anchor. These factors become extremely important when determining the likelihood of various components being dragged over the seabed [1].

## K.14.8 CONSEQUENCES OF MOORING FAILURE

The greatest concerns for potential asset damage are described in the following.

a. **Surface Facilities—**Relatively low likelihood for direct vessel to vessel collision unless the MODU is very close, but the cost for a major installation can be well in excess of $1 billion for a major deepwater facility, not accounting for shut-in loss. Contact likelihoods depends on the “target” size, including any mooring and riser spread. While contact with the surface facility does not necessarily equate to total loss, due consideration needs to be given to the environment in which the collision will occur (i.e. during a storm). Similar, consideration should be given to potential damage to the mooring or risers. This kind of event has a higher likelihood than the direct vessel to vessel collision but would not be a complete facility loss but may still result in significant repair costs and lost or deferred production.

b. **Pipeline/Flowlines—**Relatively large “target” if the drifting MODU is dragging an anchor. Cost of damage needs to include the impact on production upstream of the pipeline, and if those facilities feeding the pipeline can produce through any alternative route. Repair costs in deepwater can be high, with long duration. Recent experience is that only approximately one in five MODUs that broke adrift dragged anchors for any significant distance. While dragging an anchor can cause significant damage, it is less clear how much damage can be caused by a dragging a failed wire or chain. The loads imposed by a dragged wire or chain will not be very large, however, damage to the coating or insulation of a pipeline may be sufficient to necessitate costly repairs in the future. There were cases reported in the 2004 and 2005 hurricane seasons where a dragged wire or chain that apparently did not damage steel pipelines that they crossed. However, if a groove gets started, then a wire or chain will tend to cut into the steel. For flowlines, there is lower likelihood of damage than a pipeline as they tend to be shorter, but conversely may be closer to the MODU operations site. Likelihood of interaction depends on the angle subtended. Flowlines being infield or intra-field versus pipelines which are for export to shore, tend to be smaller and thus lower throughput does lesser consequence if damaged.

c. **Wellheads**—Low likelihood of damage by dragged anchor or broken mooring line(s) due to small size, however likely to be damaged by a dragged wire or chain. Normally the consequences would be limited because safety features would prevent flow even if the wellhead were pulled off the well. However, there have been wells where the sub-surface safety valve has been taken out for repair prior to hurricane arrival. Damage to the wellhead in these cases could lead to a significant hydrocarbon spill.

d. **Umbilicals—**Similar likelihood of damage to flowlines, although often less robust, so more prone to damage from dragged mooring components. Normally would not lead to large shut-in production, although there are some umbilicals used for flow assurance for a number of wells. Loss of one of these umbilicals would result in significant shut-in.

There is a high concern for extended shut-in of production. In most cases, the production of a facility can be reasonably estimated, but there are some facilities, often older ones, that are producing relatively little, but significant quantities of hydrocarbon transit across the facility. These hub facilities represent a special risk, as do the main pipelines that feed them.

Determining the cost of shut-in production is not simple. The production is not normally lost, unless the damage is so great that the economics of replacing the facility are unacceptable. However, the delay in production leads to significant loss of cash flow and may impact the ability to finance the development of other fields.

Due consideration should be given to some of the massive pipelines that carry crude from hub facilities and the LOOP. Damage to these could have an impact on oil supplies to the U.S. and therefore should be subject to special assessment.

While drifting of a MODU into a major surface facility may be a low likelihood event, it is one of the cases in which the consequences could significantly outweigh the direct financial damage. The perception of such a collision could be quite detrimental to the industry.

### K.14.9 RISK EVALUATION—THE RISK MATRIX

Although risk is defined as the product of the frequency of occurrence of a hazardous event and its consequences, this definition is sometimes inadequate. In the case of a financial assessment, the result is the "expected loss," in real currency, if the venture is undertaken. A high probability event of low consequence can have the same risk as a low probability event with high consequences. This is a common definition, but for some extremely high consequence events with low probability of occurrence, this definition can lead to misleading conclusions; society will accept frequent low consequence events more readily than less frequent high consequence events.

The usual method of presenting results makes use of a risk matrix, whether the mooring failure risk assessment is undertaken at a simplified or detailed level. It is difficult for people, even those trained in risk analysis, to get a good understanding of what an event with a probability of $10^{-4}$ means when that event is associated with consequence of \$1 billion. In order to put these numbers into perspective it is best to use a risk matrix such as shown in Figure K.11. This is a table of cells, each cell representing a single combination of probability range with consequence range. Figure K.11 is a four-by-four matrix, but many companies use different shapes and sizes. Each cell is assigned either a low, moderate or high risk rating.

In many cases, particularly when considering mooring failures, the detail of the quantification, the magnitude of the numbers, and level of accuracy is such that the best way to interpret and present the results is through the use of a risk matrix. The selection of levels probability and consequence depends on the Operator, and these must be defined at the start of the risk analysis. Different matrices may be used at different stages of the analysis.

The risk matrix can be used to assess various type of consequence as discussed in K.14.5. As discussed previously, financial consequence may not be the critical consequence type for some stakeholders, but it is the most easily quantified. Based on the position in the matrix, a risk classification such as low, moderate, and high may be used to decide the risk potential from an individual hazard. Figure K.11 has not been drawn with specific risk categories, but they should be revised to suit the requirements of the stakeholder. An example of three "Risk Levels" follows.

a. *Low risk (L)*: The bottom left corner of Figure K.11. Severity is low and likelihood is also low. Minimal risk that may be tolerated because it represents low probability of relatively small loss and may be addressed as part of the normal on-going continual improvement processes.

b. *Moderate risk (M)*: The middle area of Figure K.11, from the upper left corner to the lower right corner. There is either a low probability of suffering a large loss (upper left), or a relatively high probability of suffering a small loss (lower right). Generally, the risk may be within tolerable limits. However, the expected loss is sufficiently high to require best attempts to mitigate the risk. In a more general sense, this risk level requires implementation of reasonable and practicable risk reducing measures, or more detailed analyses to better define probability and consequences.

c. *High risk (H)*: The top right corner of Figure K.11. A high probability of suffering a large financial loss that is not tolerable without implementation of effective risk reduction measures. Risk reduction measures may include reducing the probability of mooring failure or reducing the consequences should one occur. One way to reduce the probability of failure may be to change the season of operation. Possible consequence reduction measures are discussed in the following section.

| Consequences | | Probability | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| | | Less Likely | | | More Likely |
| IV | High | | | | High Risk |
| III | | | Moderate Risk | | |
| II | | | | | |
| I | Low | Low Risk | | | |

Figure K.11—Typical Risk Matrix used for MODU Mooring Risk Assessment

## K.14.10 RISK ACCEPTANCE, ALARP AND RISK REDUCTION

### K.14.10.1 Risk Acceptance

Risk acceptance involves deciding whether a risk is tolerable and whether risk reduction measures are needed. Tolerable risk levels should provide a balance between absolute safety requirements and cost and benefits of proposed risk reduction measures. Acceptability is generally determined by comparing mooring failure risk against the acceptable risks established for similar or other offshore systems with acceptable operating experience or with those established by other industries.

Operators and other stakeholders may have different risk acceptance criteria that may be driven by different consequence types. The situation becomes increasingly complex when considering either extremely high financial losses or the other consequence types. It may be possible to develop a relatively simple set of criteria for financial loss, however the other consequences of a MODU mooring failure may be difficult to objectively assess.

Other consequences besides financial, such as corporate image, may be involved at this stage. Considering only financial risk may mislead the overall assessment conclusions. In addition, when dealing with extremely low probability but high consequence events, the normal approach of cost-benefit analysis can break down.

*Example 1: Consider an event with a 0.1 annual probability of occurrence with an associated $100,000 financial consequence produces the same financial risk as an event with a 0.0001 annual probability of occurrence with an associated $100 million financial consequence that is a $10,000 annual risk. However, an event that produced a $100 million loss is likely going to have far greater impact on corporate image than a $100,000 event.*

*Example 2: Consider the case where there is a 0.001 probability of doing $2 billion damage. The financial risk can be calculated as $2 billion × 0.001 or $2 million. By implication, it would not be financially advisable to spend more than $2 million (the total financial risk) to reduce the probability to a lower level. However, because the $2 billion loss may be more than the company could tolerate, it would likely be advantageous to attempt to reduce the probability to a lower level, even at an expense of over $2 million.*

Financial risk may be useful to determine what mitigation or prevention measures are worth investing in to reduce the risk. The ALARP (As Low As Reasonably Practicable) process can also be used to determine if the risk is tolerable or that the Operator has done everything reasonable to reduce the risk.

### K.14.10.2 ALARP

When evaluating the risk of a mooring failure an operator should make an effort to satisfy themselves that the risk is ALARP. The ALARP concept is part of the risk decision analysis. ALARP goes beyond determining the risk level based on the adequacy of existing mitigation measures by asking *"What else can be done to further reduce the risk and what is the argument for not doing it?"* The key to the ALARP concept is that the burden of proof is to demonstrate why you wouldn't do more, as opposed to explaining what has or will be done to reduce the risk.

Philosophically, all risks should be reduced to a level that is ALARP. The ALARP concept hinges on what is reasonably practicable in terms of effort and benefit, recognizing that it is not reasonable to expect every possible risk reduction measure to be implemented. To demonstrate that a risk is ALARP it is necessary to show that there is a gross disproportion between the benefit (risk reduction) gained and the resources of implementing further risk reduction measures. Gross disproportion implies there is a bias towards risk reduction. What constitutes gross disproportion will depend to a great extent on the potential consequences associated with the risk—the greater the consequences the greater should be the bias towards risk reduction. The ALARP concept is consistent with a philosophy of continuing risk reduction while recognizing the principle of diminishing returns.

Consider the ALARP concept as it relates to an example of three possible risk levels—higher, medium and lower. The suggested response to a higher risk is to reduce it to at least the medium level which in most cases would generally not be considered to be as low as reasonably practicable. Lower risks would be addressed as part of the normal on-going improvement processes. Thus, at this lower end of the risk spectrum, where there is little scope for further risk reduction, the ALARP test is typically intuitive. So it is at the medium level where there may be less obvious decisions to be made about further risk reduction, and therefore may warrant a more rigorous demonstration of ALARP.

As it applies to evaluating the risk of a MODU mooring failure, the ALARP process requires an operator evaluate the efforts and benefits associated with alternative mooring system designs (e.g., catenary vs. semi-taut configurations, steel vs. fiber rope, various anchor designs, etc.) and alternative drilling locations and schedules (e.g., can the location be drilled outside of the peak of hurricane season, is there a lower risk location in another region of the GOM, is it feasible to trade MODU slots or share the MODU with another operator, etc.) to demonstrate that the risk of mooring at the location is ALARP.

### K.14.10.3 Risk Reduction

If a tolerable risk level is not achieved in the risk assessment process, the next step is to identify risk reducing measures and evaluate their potential to reduce the risks to a tolerable level. Risk assessment is an iterative process, i.e., it needs to be repeated considering the changes in the system until a tolerable risk level is achieved.

Some of the methods that can be used to reduce the risks include:

- drilling the well during a more environmentally benign time of the year to reduce probability of mooring failure (prevention);
- strengthening the mooring system to reduce the probability of mooring failure (prevention);
- using different mooring components to reduce the consequences of mooring failure (mitigation).

The consequential effects of risk reduction measures should always be considered: there is little point in reducing the risks of one operation only to find that the comparable risks of another have been increased.

## K.14.11 DOCUMENTATION

It is important to document the basis, and any assumptions inherent within the analysis regardless of the level of risk analysis that is being undertaken. Normally for a simple "spreadsheet" type assessment this will be easy to document since the simple sets of answers should always be available. The important point is that anyone auditing the process at a later date can determine the fundamental information for the analysis. One important piece of information is a document register that gives the number, date and revision number of all documents used in the assessment. There should also be clear documentation of personnel involved.

## K.14.12 OUTLINE METHODOLOGY FOR QUASI-QUANTIFIED MOORING FAILURE FINANCIAL RISK ASSESSMENT

The following approach assumes that the probability of mooring failure has been determined. This methodology is provided for guidance only and is not intended to be all-inclusive. When calculating the probability of a mooring failure it is important to ensure that a suitable factor has been used on the mooring line strength.

1. Obtain a map of area showing all the surface and subsurface infrastructure.
2. Divide it up into homogenous sectors. A homogenous sector is one in which there are no changes in infrastructure: the damage caused by a drifting MODU will be independent of the direction it drifts off location within the sector. It may be reasonable to exclude point sources (e.g., subsea wells or distant surface facilities) if angle they subtend is small. These would then be handled separately.
3. Determine the likelihood that the MODU drifts in that direction (based on either mooring analysis or metocean extremes, or both).
4. Determine likelihood of dragging each of the components in the mooring system (based on line break statistics). This will necessitate building a table of components, failure likelihood, and potential for dragging across the seabed.
5. Determine the likelihood and extent of damage to subsea facilities (based on water depth at subsea infrastructure and dragged components, etc.).
6. Determine the consequences of that damage, including direct costs to repair, and delayed or lost production.
7. Develop a weighted sum of the product of consequence and likelihood for each type of subsea infrastructure that it is desired to keep separate (so they can be plotted on a risk matrix). Alternatively, determine a weighted average for expected loss for all subsea equipment lumped together.
8. Determine the likelihood that dragged mooring components damage subsurface structures.
   - All: consider potential for dragged mooring line interaction with SCRs, well risers, subsea tieback flowlines, etc.;
   - For TLPs: consider pile, tendon, porch, and hull;
   - For jackets: consider jacket structural members;
   - For spread moored floaters: consider interaction with mooring system components based on what is dragged, layout of permanent facility mooring, and type of moorings (synthetic or steel). A grouped synthetic mooring system is expected to have the highest likelihood of suffering catastrophic damage due to the potential of damaging an entire group of lines.
9. Determine consequences of drifting into, or otherwise interacting with, surface facility, including direct repair cost and delayed or lost production.
10. Sum the various types of surface facility consequences so that, with the related probabilities, so they can be plotted onto a risk matrix.
11. Present the results in both tabular form, and plotted on a suitable risk matrix.

### K.14.13 REFERENCES

1. MODU Mooring Strength and Reliability JIP, "Post Mortem Failure Assessment of Semi-Submersible MODUs during Hurricanes Katrina & Rita." Report No. ABSC/1514096/LB-08, 2007
2. Sharples, Malcom "Post Mortem Failure Assessment of MODUs During Hurricane Ivan," U.S. Minerals Management Service Report No. 0105PO39221, 2006
3. Coyne, Mike "2005 Hurricane Season Impact," API Offshore Hurricane Readiness and Recovery Conference Proceedings, Nov 2006.
4. Det Norske Veritas, "Pipeline Damage Assessment from Hurricanes Katrina and Rita in the Gulf of Mexico," U.S. Mineral Management Service Report No. 44814183, 2007.
5. Floating Production Mooring Integrity JIP, (UK HSE Report 444), 2006.

## K.15 Storm Reporting Sheet for Semi-submersible Rig Status Report

This reporting sheet can be used as required for documenting information on the MODU mooring after an event. This form may also be useful documenting the as-installed mooring system information as per K.10.2. Every effort has been taken to make the form cover all situations. However, there will be cases where it does not exactly ask the right questions, thus flexibility is requested of the person completing it. The intent is to capture the impact of a storm on the MODU and its mooring.

The reporting sheet is divided into four sections:

1. General Description of the MODU and the Mooring Location;
2. As-installed MODU Mooring Information;
3. As-abandoned MODU Mooring Information;
4. Post-storm MODU Condition.

Please use consistent units throughout (either feet and kips or tonnes and meters, line diameters in in. or mm).

## Section 1—General Description of the MODU and the Mooring Location

| | | |
|---|---|---|
| **Date Form Completed:** | | |
| **Contact Information:** | | |
| Drilling contractor: | | |
| Contact name: | Telephone: | E-Mail: |
| **General MODU Characteristics:** | | |
| MODU name: | | |
| Designer: | Designer class description: | |
| Classification society and notation: | | |
| MODU modified since delivery? | Yes | No |
| Brief description of modifications: | | |
| **Location Description:** | | |
| Operator of well: | | |
| Block name and no. | Latitude: | Longitude: |
| Water depth: | MODU heading: | |
| Soils data available? | Yes | No |
| If soils data is available, please supply brief description and strength profile. | | |

## Section 2—As-installed MODU Mooring Information

| | | | | |
|---|---|---|---|---|
| **General Information:** | | | | |
| Was mooring MODU's own system or preset? | Own | Preset | Both own and preset | |
| If preset, whose equipment was used? | | | | |
| Number of mooring lines (details requested in table below): | | | | |
| What is mooring line 1 (e.g. Starboard; Bow): | | | | |
| What is numbering sequence? (Add drawing if required.) | clockwise from above | | anti-clockwise from above | |
| **Anchor Information:** | | | | |
| Type of anchors | Drag | Plate anchor | Pile | Other |
| Anchor description (e.g. weight, size, manufacturer, model, etc.): | | | | |
| For drag embedment anchors: | | | | |
| Manufacturer | Type | | Weight and fluke angle | |
| Anchor test load and duration | | | | |
| Were any anchor legs run short? | Yes | | No | |
| If any were run short, why? | | | | |
| **Tension and Length Information:** | | | | |
| Method of payout measurement during installation: | | | | |
| Method of measuring pretension and operating tension: | | | | |
| Are anchor scopes known or estimated? | known | | estimated | |

## Please Complete for AS-INSTALLED Condition

| | Mooring Line Number | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Line azimuth (True North) | | | | | | | | | | | | |
| Fairlead-anchor horizontal distance | | | | | | | | | | | | |
| Fairlead component type, size, outboard length | | | | | | | | | | | | |
| Component type, size, length | | | | | | | | | | | | |
| Component type, size, length | | | | | | | | | | | | |
| Anchor component type, size, length | | | | | | | | | | | | |
| Buoy/clump size and location | | | | | | | | | | | | |
| Type of anchor size, etc. | | | | | | | | | | | | |
| Fluke angle or shear pin size (force to break) | | | | | | | | | | | | |
| Test load at fairlead or AHV stern roller or AHV bollard pull | | | | | | | | | | | | |
| Line arrangement during test load | | | | | | | | | | | | |
| Test load at anchor shackle | | | | | | | | | | | | |
| Anchor test load duration (min) | | | | | | | | | | | | |
| Anchor drag distance during installation | | | | | | | | | | | | |
| Location of MODU during test load | | | | | | | | | | | | |
| Nominal operating pretension | | | | | | | | | | | | |

| **Mooring Line Component Information** | | | Lines ______ to ______ | | | |
|---|---|---|---|---|---|---|
| | Type | Construction | Diameter | Break Strength | Manufacturer | Age |
| At Fairlead | | | | | | |
| Intermediate Line 1 | | | | | | |
| Intermediate Line 2 | | | | | | |
| At Anchor | | | | | | |
| **Mooring Line Component Information** | | | Lines ______ to ______ | | | |
| | Type | Construction | Diameter | Break Strength | Manufacturer | Age |
| At Fairlead | | | | | | |
| Intermediate Line 1 | | | | | | |
| Intermediate Line 2 | | | | | | |
| At Anchor | | | | | | |
| Note: The table above has been developed to ease information flow. If mooring system cannot easily be described through use of this table, please attach a separate and full description. | | | | | | |

## Section 3—As-Evacuated MODU Mooring Information

| | | |
|---|---|---|
| **Mooring System Details:** | | |
| Was rig position modified for evacuation? | Yes | No |
| How modified? (distance and direction) | | |
| Were line tensions modified prior to evacuation? | Yes | No |
| Was line slackening complicated by high currents, high winds, etc. that made it difficult to accurately establish the line tensions on evacuation? | | |
| | Yes | No |
| Prevailing weather conditions at time of mooring adjustment | | |
| Seas | Height | Direction |
| Wind (1 minute average) | Speed | Direction |
| Current | Speed | Direction |

## Please Complete for AS-EVACUATED Condition

| | Mooring Line Number | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Line azimuth (True North) | | | | | | | | | | | | |
| Fairlead-anchor horizontal distance | | | | | | | | | | | | |
| Fairlead component outboard length at evacuation | | | | | | | | | | | | |
| Nominal survival pretension (at zero environment) | | | | | | | | | | | | |
| Evacuation tension (measured) | | | | | | | | | | | | |
| Note: The table above has been developed to ease information flow. If mooring system cannot easily be described through use of this table, please attach a separate and full description. | | | | | | | | | | | | |

## Section 4—Post-storm MODU Condition

| | | |
|---|---|---|
| **Storm Name and Date:** | | |
| **Hull and Structural Condition Summary** | | |
| Did the unit suffer any damage during the hurricane? | Yes | No |
| Can repairs be effected on site? | Yes | No |
| Major: Description of hull or structural damage requiring third party or shipyard repair? | | |
| Significant: Description of hull or structural damage that must be completed prior to restarting drilling operations: | | |
| Minor: Description of hull or structural damage that can be repaired during normal operations. Please include whether there was green water damage. | | |
| What surprised you when you got back on the MODU? (either damage or indications of things) | | |
| **Mooring System Condition Summary** | | |
| Did the unit suffer any mooring related failures? | Yes | No |
| Did any anchors drag, and if so, how far? | Yes | No |
| | How far? | |

| | Mooring Line Number | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Failed at fairlead | | | | | | | | | | | | |
| Failed at intermediate | | | | | | | | | | | | |
| Failed at anchor | | | | | | | | | | | | |
| Dragged anchor | | | | | | | | | | | | |
| Other (anchor broke, brake failure, etc.) | | | | | | | | | | | | |
| Component(s) recovered for inspection? | | | | | | | | | | | | |
| Inspection results and availability? | | | | | | | | | | | | |
| Length of any “dangling” mooring component below the keel | | | | | | | | | | | | |

| | | |
|---|---|---|
| **MODU Recovery and Reboarding Operations Summary** | | |
| How far did unit drift? | | |
| Where was the unit after the storm when found? | | |
| Was the unit grounded? | Yes | No |
| Were tugs dispatched to recover the unit? | Yes | No |
| Did the tugs prevent additional drift? | Yes | No |
| Any comments on effectiveness of tugs? | | |
| Is course of unit drift known (e.g., through transponder)? | Yes | No |
| Did the transponder operate properly during the storm, and if not, why (e.g., batteries failed, etc.)? | | |
| Is the plot of location against time (to help with hindcasting and drift prediction) available? | | |
| Other comments on MODU recovery and reboarding operations | | |

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