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Petroleum and natural gas industries — Offshore structures —

Part 2:

Fixed steel structures

Industries du pétrole et du gaz naturel — Structures en mer —

Partie 2: Structures fixes en acier



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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental in liaison with ISO, also take part in the work.

Draft International Standards adopted by the technical committees are circulated to the member bodies for approval before their acceptance as International Standards by the ISO council. They are approved in accordance with ISO procedures requiring at least 75 % approval by the member bodies voting.

International Standard ISO 13819-2 was prepared by Technical Committee ISO/TC 67, *Materials equipment and offshore structures for petroleum and natural gas industries*, Sub-committee SC 7, *Offshore structures*. Annex A was prepared as ANSI/API RP2A-LRFD-93 by the American Petroleum Institute.

ISO 13819 will consist of the following parts, under the general title *Petroleum and natural gas industries — Offshore structures*:

- Part 1: General requirements
- Part 2: Fixed steel structures
- Part 3: Fixed concrete structures
- Part 4: Floating systems
- Part 5: Arctic structures
- Part 6: Site specific assessment of MODUS

Annexes A and B of this part of ISO 13819 are for information only.

Introduction

It is important to recognize that structural integrity is an overall concept comprising models for describing actions, structural analyses, design rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modifications of one aspect of design in isolation can disturb the balance of reliability inherent in the overall concept or structural system. The implications involved in modifications, therefore, need to be considered in relation to the overall reliability of all offshore structural systems.

International Standard ISO 13819 constitutes a common basis covering those aspects that address design requirements and assessments of all structures used by the petroleum and natural gas industries worldwide. Through its application, the intention is to achieve reliability levels appropriate for manned and unmanned offshore structures, whatever the nature or combination of the materials used.

The standard is intended to provide a wide latitude in the choice of structural configurations, materials and techniques and to allow for incorporation of technological advances without hindering innovation. It is, therefore, essential that it is used in conjunction with sound engineering judgement.

The primary objectives of Part 2 of International Standard ISO 13819 are to provide a safe place of work for personnel and to protect the environment.

The detailed provisions given in this part of ISO 13819 implement the general principles in ISO 13819-1 and are specific to fixed steel structures. While some of the provisions of this part of ISO 13819 are applicable worldwide, others are applicable to specific geographic regions only. Where this is appropriate, ISO/TC 67/SC 7 is actively striving to harmonize the differing regional technical provisions. Once harmonized, these provisions will be incorporated in an updated version of this standard.

International Standard ISO 13819-2 includes the following informative annexes:

- a) Annex A, which reproduces API RP2A-LRFD, First Edition, July 1, 1993, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor design*.

The API document in Annex A is reproduced in this standard by permission of the American Petroleum Institute, 1200 L Street, Northwest, Washington DC 2005 U.S.A.

ISO, in endorsing this API document as an informative annex, recognizes that in certain respects the latter does not comply with all current ISO rules on the presentation and content of an International Standard.

Information given in the POLICY is relevant to the API publication only.

ISO/TC 67/SC 7 is however currently engaged in updating this API document in conjunction with the American Petroleum Institute. An updated version, in a form complying with ISO rules, will be issued when practicable.

- b) Annex B, which identifies regionally applicable provisions that supplement those of ISO 13819-1 and/or annex A of this part of ISO 13819.

Petroleum and natural gas industries — Offshore structures —

Part 2:

Fixed steel structures

1 Scope

This part of ISO 13819 contains provisions for the design and assessment of fixed offshore structures constructed of steel, including jackets, towers, and free standing caissons. The provisions for member resistance also apply to elements of compliant towers and possibly other types of structures to the extent that these provisions are relevant.

This part of ISO 13819 is applicable to the design of complete structures including substructures, topside structures and foundations.

It specifies design principles that are also applicable to the successive stages in construction (namely fabrication, transportation and installation), to the use of the structure during its intended life, and to its abandonment. Generally, the principles are also applicable to the reassessment or modification of existing structures. Aspects relating to quality control are also addressed.

2 Definitions

For the purposes of this International Standard, the following definition applies:

2.1 Fixed structure: Structure that is bottom founded and transfers all actions that act upon it to the sea floor.

3 Informative references

Standards referenced in Annex A that primarily address structural design of nontubular components may be replaced by equivalent ISO, CEN or national standards that can be shown to meet or exceed the requirements of the referenced standards.

4 Design recommendations

4.1 Technical provisions

Technical provisions and criteria are given in Annex A which also contains requirements and criteria specific to United States offshore regions, including partial factors.

The resistance factors that are provided in Annex A are consistent with the load factors in Annex A and should be applied to the nominal resistances for nontubular components. Equivalent International, CEN or National Standards may also be used provided consistent load and resistance factors are used.

When establishing partial factors for other geographic areas, differences in practices or in the nature of the environmental conditions should be considered to ensure that required reliability levels are achieved. The regional information given in Annex B should be consulted for guidance.

4.2 Supplementary information

In some cases, Annex A might need to be supplemented to allow for regional environmental conditions, governmental requirements and local design, construction and operating practices. Information on these aspects is contained in Annex B.

Each section of Annex B contains technical provisions and criteria for a specific geographic region, that should be used for that region in preference to the recommendations of Annex A.

NOTE 1: Depending on the legislative system of a country, some of these provisions might be mandatory within the jurisdiction of the country concerned.

NOTE 2: Further regionally-specific information will be added to Annex B as and when required.

4.3 Other regions

For regions where there is no specific information given in Annex B, the technical provisions and criteria of Annex A may be substituted by corresponding technical provisions and criteria providing these are appropriate and consistent. The user is, however, warned that certain regional provisions and criteria, for example environmental criteria and partial factors for loads, generally require development on a case-by-case basis. In such cases, reference may be made to appropriate national standards where these have been developed specifically for offshore conditions.

Annex A

(informative)

Design recommendations

The Informative Annex A of this part of ISO 13819 contains guidelines for the design of steel, fixed offshore platforms. Annex A is the American Petroleum Institute Recommended Practice for Planning, Designing and Constructing fixed Offshore Platforms - Load and Resistance Factor Design, (API RP2A-LRFD), First Edition, July 1, 1993. Annex A is supplemented by Annex B, which includes provisions for specific regional areas. The regional provisions for United States territorial waters are included in Annex A.

Annex A provisions are based on a reliability-based calibration of the American Petroleum Institute *Recommended Practice for Planning, Designing and Constructing fixed Offshore Platforms — Working Stress Design* (API RP2A - WSD), which has served widely as the de facto basis for design of offshore platforms over the last 15 years. This calibration process was carried out on a component-type by component-type basis. The load and resistance factors were developed for each component-type such that target reliabilities for Annex A are on the average consistent with those for the API RP2A-WSD.

Annex A is based on limit states design principles and the general format is consistent with ISO 13819-1.

Annex A relies on the use of the nominal resistances for nontubular components provided in the American Institute for Steel Construction entitled *Load and Resistance Factor Design Specification for Structural Steel Buildings*.

The partial factors in Annex A reflect the practices and the target reliability levels in the Gulf of Mexico and other locations where the API RP2A-WSD has served as a reference design standard.

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Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design

**API RECOMMENDED PRACTICE 2A-LRFD (RP 2A-LRFD)
FIRST EDITION, JULY 1, 1993**

American National Standard
ANSI/API RP2A-LRFD-93
Approved: April 13, 1994

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ERRATA
(OCTOBER, 1993)

Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design

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Page 2, Table of Contents. *Change A.3.7 to read:*

A.3.7 Deck Elevation

Page 33, Section C.3.4.2.3 Turbulence Intensity.
Change Equation C.3-8 to the following:

$$I(z) \equiv \sigma(z)/V(1 \text{ hr}, z) = \begin{cases} 0.15(z/z_g)^{0.125} & \text{for } z \leq z_g \\ 0.15(z/z_g)^{0.275} & \text{for } z > z_g \end{cases} \dots\dots (\text{C.3-8})$$

Page 52, Section D.3.2.1 Cylindrical Members.
Change Equation D.3.2-1 to the following:

$$\frac{f_c}{\phi_c F_{cn}} + \frac{1}{\phi_b F_{bn}} \left\{ \left[\frac{C_{my} f_{by}}{\phi_c F_{ey}} \right]^2 + \left[\frac{C_{mz} f_{bz}}{\phi_c F_{ez}} \right]^2 \right\}^{0.5} \leq 1.0 \dots\dots (\text{D.3.2-1})$$

Change Equation D.3.2-3 to the following:

$$f_c < \phi_c F_{xc} \dots\dots\dots (\text{D.3.2-3})$$

Page 55, Section E.1 Connections of Tension and Compression Members. *Add the following note after the definition of F_y for Equation E.3-1:*

Note: The tensile strength limitation on F_y is intended to apply throughout Section E.

Page 83, Table I.1 Structural Steel Plates. *Change information shown under Group II, Class C, to the following:*

GROUP	CLASS	SPECIFICATION & GRADE	YIELD STRENGTH		TENSILE STRENGTH	
			MPa	ksi	MPa	ksi
II	C	ASTM A572 Grade 42 (to 2" thick)*	290	42	415 min.	60 min.
		ASTM A572 Grade 50 (to 2" thick;				
		ASTM S91 required over 1/2")*	345	50	450 min.	65 min.

Page 84, Table I.2 Structural Steel Shapes. *Change information shown under Group II, Class C to the following:*

GROUP	CLASS	ASTM SPECIFICATION & GRADE	YIELD STRENGTH		TENSILE STRENGTH	
			MPa	ksi	MPa	ksi
II	C	A572 Grade 42 (to 50 mm (2 in) thick) ¹	290	42	415 min.	60 min.
		A572 Grade 50 (to 50 mm (2 in) thick;	345	50	450 min.	65 min.
		S91 required over 13 mm (1/2 in) thick) ¹				

Page 127, Commentary Comm.C.3.1 Strength Requirements, Fourth Paragraph, Third Sentence.
Change L_2 to L_1 .

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FOREWORD

This "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design" (LRFD) contains the engineering design principles and good practices that have been the basis of the API RP2A working strength design (WSD) recommended practice, now in its 20th Edition. The LRFD provisions have been developed from the WSD provisions using reliability based calibration.

This recommended practice is based on sound engineering principles, extensive testing and field application experience. In no case is any specific recommendation included which could not be accomplished by presently available techniques and equipment. Consideration is given in all cases to the safety of personnel, compliance with existing regulations, and prevention of pollution.

This is the First Edition of the "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design." This practice has been approved by the API as an alternative to the 20th Edition of the RP2A "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms."

The LRFD was first issued in December 1989 in draft form. It was consistent in most respects with the 18th Edition of the RP2A. The draft was open to comment for a period of two years. All of the comments received were carefully considered in the development of the First Edition. In general, the provisions are consistent with the 20th Edition of the RP2A.

The main changes in the First Edition from the draft are as follows:

- Sections A.6 and P have been inserted covering platform reuse. Other sections have been reworded to address reuse.
- The wave force section (C.3.2) has been totally revised and is consistent with the changes introduced into the WSD. Recalibration of the provisions was not felt to be necessary, since the same change is introduced into the calibration reference, e.g., the WSD.
- The wind force section (C.3.4) is now consistent with the WSD.

- The section on earthquake design of deck appurtenances and equipment (C.4.4.2) has been expanded.
- The section covering loads to be used in fabrication and installation (C.5) has been revised to address the problem of factoring various load types that inherently are self equilibrating. The solution is to use a common load factor in such cases.
- Section C.6 has been added that covers accidental loads.
- The scope of the recommendation for design of tubular members has been limited to $D/t < 300$ and $F_y \leq 414$ mPa (60 Ksi). See Section D.1. This is consistent with WSD.
- Section G.7 has been inserted that covers soil reactions for axially loaded piles.
- Section P covering platform reuse has been added.
- Section Q covering minimum structures has been added.
- The prime units are now SI, with English units the secondary set.

In this practice, reference is made to ANSI/AWS D1.1-92 Structural Welding code — Steel. While use of this edition is endorsed, the primary intent is that the AWS code be followed for the welding and fabrication of fixed Offshore Platforms. Chapters 8, 9 and 10 of the AWS Code give guidance that may be relevant to the design of fixed Offshore Platforms. This recommended Practice makes specific reference to Chapters 9 and 10 for certain design considerations. Where specific guidance is given in this API document, as in Sections E and F, this guidance should take precedence.

This standard shall become effective on the date printed on the cover but may be used voluntarily from the date of distribution.

As with all other API documents, this practice will be reviewed and changed periodically as required. Comments or suggestions on the LRFD should be sent to: American Petroleum Institute, 1201 Main Street, Suite 2535, Dallas TX 75202-3994. Attention: RP2A-LRFD

NOTE: This 1st edition supersedes the Draft RP dated December 15, 1989. It includes changes adopted at the 1992 Standardization Conference. This Recommended Practice is under jurisdiction of the API Committee on Standardization of Offshore Structures.

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SECTION A PLANNING

A.1 GENERAL

A.1.1 Planning. This section serves as a guide for those who are concerned with the design and construction of platforms for drilling, production, and storage of hydrocarbons in offshore areas. Adequate planning should be done before actual design is started in order to obtain a workable and economical offshore platform to perform the given function. The initial planning should include the determination of all criteria upon which the design of the platform will be based.

A.1.2 Design Criteria. Design criteria as used herein include all operational requirements and environmental criteria which could affect the design of the platform.

A.1.3 Codes and Standards. This publication has incorporated and made maximum use of existing codes and standards which have been found acceptable for engineering design and practices from the standpoint of public safety.

A.2 PLATFORM TYPES

A.2.1 Fixed Platforms.

A.2.1.1 Template. A template-type platform consists of:

1. A jacket or welded tubular space frame which is designed to serve as a template for pile driving, and as lateral bracing for the piles.
2. Piles which permanently anchor the platform to the ocean floor and carry both lateral and vertical loads.
3. A superstructure consisting of the necessary trusses and deck for supporting operational and other loads.

A.2.1.2 Tower. A tower platform is one which has relatively few large diameter, e.g., 5 m (15 ft), legs. The tower may be floated to location and placed in position by selective flooding. Tower platforms may or may not be supported by piling.

A.2.1.3 Minimum Structures. Minimum structures include one or more of the following attributes:

1. Structural framing which provides less redundancy than a typical four-leg template type platform.
2. Free-standing caisson platform, which consists of one large tubular member supporting one or more wells.
3. Well conductor(s) or free-standing caisson(s) which are utilized as structural and/or axial foundation elements by means of attachment using welded, nonwelded or nonconventional welded connections.
4. Threaded, pinned or clamped foundation elements (piles or pile sleeves).

A.2.1.4 Gravity. A gravity platform relies on the weight of the structure rather than piling to resist environmental loads. This practice does not cover the design of gravity platforms except as included in Section G.13.

A.2.2 Other Platforms.

A.2.2.1 Guyed Tower. A guyed tower is a structure with a tubular steel frame supported vertically by piles or by a shallow bearing foundation. Primary lateral support is provided by a guyline system. Guy towers are covered in this practice only to the extent that the provisions are applicable.

A.2.2.2 Tension Leg Platform. A tension leg platform is a buoyant platform connected by vertical tethers to the seafloor. Tension leg platforms are covered in the API RP2T.

A.2.2.3 Compliant Platform. A compliant platform is a bottom-founded structure having substantial flexibility. It is flexible enough that applied forces are resisted in significant part by inertial resistances to platform motion. The result is a reduction in forces transmitted to the platform and the supporting foundation. Guyed towers are normally compliant, unless the guying system is very stiff. Compliant platforms are covered in this practice only to the extent that the provisions are applicable.

A.2.2.4 Others. Other structures, such as underwater oil storage tanks, bridges connecting platforms, etc., are covered in this practice only to the extent to which the provisions are applicable.

A.3 OPERATIONAL CONSIDERATIONS

A.3.1 Function. The functions for which a platform is to be designed are usually categorized as drilling, producing, storage, materials handling, living quarters, or some combination of these. When sizing the platform, consideration should be given to equipment operational requirements, such as access, clearances, and safety.

A.3.2 Location. The location of the platform should be specific before the design is completed. Design conditions vary with geographic location. Within a given geographic area the foundation conditions may vary, as may such parameters as design wave heights, periods, tides, currents, marine growth, and earthquake induced ground motion.

A.3.3 Orientation. The orientation of the platform refers to its position in plan referenced to a fixed direction such as true north. Orientation is usually governed by the direction of prevailing seas, winds, and currents, and operational requirements.

A.3.4 Water Depth. The water depth and tides at the site and surrounding area are needed to select appro-

appropriate oceanographic design parameters. The water depth should be determined as accurately as possible so that elevations can be established for boat landings, fenders, decks, and corrosion protection.

A.3.5 Access and Auxiliary Systems. The location and number of stairways and access boat landings on the platform should be governed by safety considerations. A minimum of two accesses to each manned level should be provided, and they should be located so that escape is possible under varying wind conditions. Operating requirements should also be considered in locating stairways.

A.3.6 Fire Protection. Personnel safety and possible damage to or loss of the platform require that attention be given to fire protection methods. The selection of the system depends upon the function of the platform. Procedures should conform to all Federal, state, and local regulations where they exist.

A.3.7 Deck Elevation. Unless the platform has been designed to resist wave and current forces on the lowest deck, the elevation of this deck should provide adequate clearance above the crest of the design wave. An additional generous air gap (see Section C.3.6) should be provided to allow the passage of extreme waves larger than the design wave. The clearance between other decks is governed by operational restrictions.

A.3.8 Wells and Risers. Well conductors and riser pipes will result in additional environmental loads on the platform when they are supported by the platform. Their number, size, and spacing should be known early in the planning stage. Conductor pipes may or may not assist in resisting the wave force. Consideration should be given to the possible need for future wells and risers.

A.3.9 Equipment and Material Layouts. Layouts and weights associated with gravity loads as defined in Section C.2 are needed in the development of the design. Heavy concentrated loads on the platform should be located so that proper framing for supporting these loads can be planned. Consideration should be given to future operations.

A.3.10 Personnel and Material Transfer. Plans for transferring men and materials should be developed at the start of the platform design. This planning should consider the type and size of supply vessels and the anchorage system required to hold them in position at the platform; the number, size, and location of the boat landings and fenders; and the type, capacity, number, and location of the deck cranes. If portable equipment or materials are to be placed on a lower deck, then adequately sized hatches should be provided and conveniently located on the upper decks. The possible use of helicopters should be established and the appropriate facilities provided.

A.3.11 Spillage and Contamination. Provision for handling spills and potential contaminants should be provided. A deck drainage system that collects and

stores liquids for subsequent handling should be provided. The drainage and collection system should meet applicable government regulations.

A.3.12 Exposure. Design of all systems and components should anticipate normal as well as extremes in environmental phenomena which may be experienced at the site.

A.4 ENVIRONMENTAL CONSIDERATIONS

A.4.1 General. The following subsections present a general summary of the environmental information that could be required.

1. Normal oceanographic and meteorological environmental conditions (conditions which are expected to occur frequently during the life of the structure) are needed to plan field operations such as installation and to develop the operational environmental load. See Section C.3.1.4.
2. Extreme oceanographic and meteorological environmental conditions (conditions which recur with a return period of typically 100 years) are needed to develop the extreme environmental load. See Section C.3.1.2.
3. Two levels of earthquake environmental conditions are required to develop the loading described in Section C.4 (1) ground motion which has a reasonable likelihood of not being exceeded at the site during the platform's life and (2) ground motion from a rare intense earthquake.

A.4.2 Winds. Wind forces are exerted upon that portion of the structure that is above the water as well as any equipment, deck houses, and derricks which are located on the platform. Wind velocities for both extreme and normal conditions are required.

A.4.3 Waves. Wind-driven waves are a major source of environmental forces on offshore platforms. Such waves are irregular in shape, can vary in height and length, and may approach a platform from one or more directions simultaneously. For these reasons, the intensity and distribution of the forces applied by waves are difficult to determine. Wave criteria for both extreme and normal conditions are required.

A.4.4 Tides. Tides are important in the design of platforms as they affect (1) the forces on the platform and (2) the elevations of boat landings, fenders, and deck.

A.4.5 Currents. Currents are important in the design of platforms as they affect (1) the forces on the platform and (2) the location and orientation of boat landings and fenders.

A.4.6 Marine Growth. In most offshore areas, marine growth on submerged platform members is a design consideration. The effects of increased surface roughness, increased member diameter and increased mass on wave and earthquake loadings should be considered.

A.4.7 Floating Ice. If the structure is to be located in an area where ice may develop or drift, ice conditions and associated ice loads should be considered in the design.

This recommended practice does not provide specific guidance on designing against ice forces. A more complete review of ice load design considerations is given in Reference A1.

A.4.8 Other Oceanographic and Meteorological Information. Other environmental information of differing value, depending on the platform site, includes records and/or predictions of precipitation, fog, wind chill, and air and sea temperatures.

A.4.9 Active Geologic Processes.

A.4.9.1 General. In many offshore areas, geologic processes associated with movement of the near-surface sediments occur within time periods that are relevant to fixed platform design. The nature, magnitude, and return intervals of potential seafloor movements should be evaluated by site investigations and judicious analytical modeling to provide input for determination of the resulting effects on structures and foundations. Due to uncertainties associated with definition of these processes, a parametric approach to studies may be helpful in the development of design criteria.

A.4.9.2 Earthquakes. Seismic forces should be considered in platform design for areas that are determined to be seismically active. Areas are considered seismically active on the basis of previous record of earthquake activity, both in frequency of occurrence and in magnitude. Seismic activity of an area for purposes of design of offshore structures is rated in terms of possible severity of damage to these structures. Seismicity of an area should be determined on the basis of detailed investigation.

Seismic considerations for such areas should include investigation of the subsurface soils at the platform for instability due to liquefaction, submarine slides triggered by earthquake activity, proximity of the site to faults, the characteristics of both levels of ground motion described in Section A.4.1(3) expected during the life of the platform and the acceptable seismic risk for the type of operation intended. Platforms in shallow water that may be subjected to tsunamis should be investigated for the effects of resulting forces.

A.4.9.3 Faults. In some offshore areas, fault planes may extend to the seafloor with the potential for either vertical or horizontal movement. Fault movement can occur as a result of seismic activity, removal of fluids from deep reservoirs, or long-term creep related to large-scale sedimentation or erosion. Siting of facilities in close proximity to fault planes intersecting the seafloor should be avoided, if possible. If circumstances dictate siting structures nearby potentially active features, the magnitude and time scale of expected movement should be estimated on the basis of geologic study for use in the platform design.

A.4.9.4 Seafloor Instability. Movements of the seafloor may be caused by ocean wave pressures, earthquakes, soil self-weight, or combinations of these phenomena. Weak, underconsolidated sediments occurring in areas where wave pressures are significant at the seafloor are most susceptible to wave induced movement and may be unstable under negligible slope angles. Earthquake induced forces can induce failure of seafloor slopes that are otherwise stable under the existing self-weight forces and wave conditions.

Rapid sedimentation (such as actively growing deltas), low soil strength, soil self-weight, and wave induced pressures are believed to be the controlling factors for the geologic processes that continually move sediment downslope. Important platform design considerations under these conditions include the effects of large-scale movement of sediment in areas subjected to strong wave pressures, downslope creep movements in areas not directly affected by wave-seafloor interaction, and the effects of sediment erosion and/or deposition on platform performance.

The scope of site investigations in areas of potential instability should focus on identification of metastable geologic features surrounding the site and definition of the soil engineering properties required for modeling and estimating seafloor movements.

Analytical estimates of soil movement as a function of depth below the mudline can be used with soil engineering properties to establish expected forces on platform members. Geologic studies employing historical bathymetric data may be useful for qualifying deposition rates during the design life of the facility.

A.4.9.5 Scour. Scour is removal of seafloor soils caused by currents and waves. Such erosion can be a natural geologic process or can be caused by structural elements interrupting the natural flow regime near the seafloor. From observation, scour can usually be characterized as some combination of:

1. **Local scour.** Steep sided scour pits around such structure elements as piles and pile groups, generally as seen in flume models.
2. **Global scour.** Shallow scoured basins of large extent around a structure, possibly due to overall structure effects, multiple structure interaction or wave/soil/structure interaction.
3. **Overall seabed movement.** Movement of sandwaves, ridges and shoals which would occur in the absence of a structure. This can be bed lowering or accumulation.

Scour can result in removal of vertical and lateral support for foundations, causing undesirable settlements of mat foundations and overstressing of foundation elements. Where scour is a possibility, it should be accounted for in design and/or its mitigation should be considered.

A.4.9.6 Shallow Gas. The presence of either biogenic or petrogenic gas in the porewater of near-surface soils is an important consideration to the engineering of the foundation. In addition to being a potential drilling hazard for both site investigation soil borings and oil well drilling, the effects of shallow gas may be important to engineering of the foundation. The importance of assumptions regarding shallow gas effects on interpreted soil engineering properties and analytical models of geologic processes should be established during initial stages of the design.

A.4.10 Site Investigation — Foundations.

A.4.10.1 Objectives. Knowledge of the soil conditions existing at the site of construction on any sizeable structure is necessary to develop a safe and economical design. Onsite soil investigations should be performed to define the various soil strata and their corresponding physical and engineering properties.

Previous site investigations and experience at the site may permit the installation of additional structures without additional studies.

The initial step for a site investigation is a review of available geophysical and soil boring data, as might be available in engineering files, literature, or government files. The purposes of this review are to identify potential problems and to aid in planning subsequent data acquisition phases of the site investigation.

Soundings and any required geophysical surveys should be part of the on-site studies and generally should be done before borings. These data should be combined with an understanding of the shallow geology of the region to develop the required foundation design parameters. The on-site studies should extend throughout the depth and areal extent of soils that will affect or be affected by installation of the foundation elements.

A.4.10.2 Seabottom Surveys. The primary purpose of a geophysical survey in the vicinity of the site is to provide data for a geologic assessment of foundation soils and the surrounding area that could affect the site. Geophysical data provide evidence of slumps, scarps, irregular or rough topography, mud volcanoes, mud lumps, collapse features, sand waves, slides, faults, diapirs, erosional surfaces, gas bubbles in the sediments, gas seeps, buried channels, and lateral variations in strata thicknesses. The areal extent of shallow soil layers may sometimes be mapped if good correspondence can be established between the soil boring information and the results from the sea-bottom surveys.

The geophysical equipment used includes (a) subbottom profiler (tuned transducer) for definition of bathymetry and structural features within the near-surface sediments, (b) side-scan sonar to define surface features, (c) boomer or mini-sparker for definition of structure to depths up to a few hundred feet below the seafloor, and (d) sparker, air gun, water gun, or sleeve-exploder for definition of structure at deeper depths and tying together with deep seismic data from reservoir studies.

Shallow sampling of near-surface sediments using drop, piston, grab samplers or vibrocoring along geophysical tracklines may be useful for calibration of results and improved definition of the shallow geology.

For more detailed description of commonly used seabottom survey systems, see Reference A2.

A.4.10.3 Soil Investigation and Testing. If practical, the soil sampling and testing program should be defined after review of the geophysical results. On-site soil investigation should include one or more soil borings to provide samples suitable for engineering property testing and a means to perform in situ testing, if required. The number and depth of borings will depend on the soil variability in the vicinity of the site and the platform configuration.

The foundation investigation for pile-supported structures should provide, as a minimum, the soil engineering property data needed to determine the following parameters: axial capacity of piles in tension and compression, load-deflection characteristics of axially and laterally loaded piles, pile drivability characteristics, and mudmat bearing capacity.

The required sophistication of the soil sampling and preservation techniques, in situ testing, and laboratory testing programs are a function of the platform design requirements and the need to characterize active geologic processes that may affect the facility. For novel platform concepts, deep-water applications, platforms in areas of potential slope instability, and gravity-base structures, the geotechnical program should be tailored to provide the data necessary for pertinent soil-structure interaction and pile capacity analyses.

When performing site investigations in frontier areas or areas known to contain carbonate material, the investigation should include diagnostic methods to determine the existence of carbonate soils. Typically, carbonate deposits are variably cemented and range from lightly cemented with sometimes significant void spaces to extremely well cemented. Therefore, in planning a site investigation program, there should be enough flexibility in the program to switch between soil sampling, rotary coring, and in situ testing as appropriate. Qualitative tests should be performed to establish the carbonate content. In a soil profile which contains carbonate material (usually in excess of 15 to 20 percent of the soil fraction) engineering behavior of the soil could be adversely affected. In these soils additional field and laboratory testing and engineering may be warranted.

A.5 SELECTING THE DESIGN CONDITIONS

Selection of the environmental conditions to which platforms are designed is the prerogative of the owner. As a guide, the recurrence interval for oceanographic design criteria should be several times the planned life of the platform. Experience with major platforms in the Gulf of Mexico supports the use of 100-year oceanographic design criteria. Risk analyses may justify

either longer or shorter recurrence intervals for design criteria. However, not less than 100-year oceanographic criteria should be considered where the design event may occur without warning while the platform is manned and/or when there are restrictions, such as great flying distances, on the speed of personnel evacuation.

Other factors to be considered in selecting design criteria are:

1. Intended use of platform.
2. Platform life.
3. Time and duration of construction, installation, and environmental operational loading conditions.
4. Probability of personnel being quartered on the platform under extreme design loading conditions.
5. Possibility of pollution.
6. Requirements of regulatory agencies.
7. Ability to predict loads for specific environmental and operating conditions and the ability to predict the platform's resistance to the loads.
8. The probability of occurrence of extreme oceanographic loads accounting for the joint frequency of occurrence of extreme winds, waves and currents (both magnitude and direction).
9. The probability of occurrence of extreme earthquake loads.
10. The probability of occurrence of extreme ice loads.

A.6 PLATFORM REUSE.

Existing platforms may be removed and relocated for continued use at a new site. When this is to be considered, the platform should be inspected to ensure that it is in (or can be returned to) an acceptable condition. In addition, it should be reanalyzed and reevaluated for the use, conditions and loading anticipated at the new site. In general, this inspection, reevaluation, and any required repairs or modifications should follow the procedures and provisions for new platforms which are stated in this recommended practice. Additional special provisions regarding platform reuse are included in Section P.

A.7 REGULATIONS

Each country has its own set of regulations concerning offshore operations. Listed below are some of the typical rules and regulations which may be applicable and, if applicable, should be considered when designing and installing offshore platforms in U.S. territorial waters. Other regulations, not listed, may also be in effect. It is the responsibility of the operator to determine which rules and regulations are applicable and should be fol-

lowed, depending upon the location and type of operations to be conducted.

1. U.S. Coast Guard, DOT, *Outer Continental Shelf Activities* (Title 33, CFR, Chapter N, Parts 140 to 147 inclusive).

These regulations stipulate requirements for identification marks for platforms, means of escape, guard rails, fire extinguishers, life preservers, ring buoys, first aid kits, etc.

2. U.S. Coast Guard, DOT, *Aids to Navigation on Artificial Islands and Fixed Structures* (Title 33, CFR, Part 67).

These regulations prescribe in detail the requirements for installation of lights and foghorns on offshore structures in various zones.

3. The Minerals Management Service (formerly U.S. Geological Survey) OCS Regulations (Title 30, CFR, 250).

These regulations govern the marking, design, fabrication, installation, operation, and removal of offshore structures and related appurtenances.

4. *Occupational Safety and Health Act of 1970* (Title 29, CFR, 1910).

This act specifies requirements for safe design of floors, handrails, stairways, ladders, etc. Some of its requirements apply to components of offshore structures.

5. U.S. Corps of Engineers *Permits for Work in Navigable Waters*.

Title 33, CFR, Part 330. Nationwide Permit describes requirements for making application for permits for work (e.g., platform installation) in navigable waters. Section 10 of the River and Harbour Act of 1899 and Section 404 of the Clean Water Act apply to state waters.

6. Federal Aviation Administration *Obstruction Marking and Lighting*.

This booklet sets forth requirements for marking towers, poles, and similar obstructions. Platforms with derricks, antennas, etc., are governed by the rules set forth in this booklet. Additional guidance is provided by American Petroleum Institute RP 2L, *Recommended Practice for Planning, Designing, and Constructing Heliports for Fixed Offshore Platforms*.

7. Various state and local agencies (e.g., Department of Wildlife and Fisheries) require notification of any operations which may take place under their jurisdiction.

Other regulations, not listed above, concerning offshore pipelines, facilities, drilling operations, etc., may be applicable and should also be consulted.

SECTION B DESIGN REQUIREMENTS

B.1 SCOPE

This section presents the overall requirements for design of the platform and indicates which other sections apply. These requirements are a minimum.

B.2 DESIGN FOR IN-PLACE CONDITIONS

The platform should be designed to resist gravity loads; wind, wave and current loads; earthquake loads; and accidental loads occurring during its service life. The nominal values of these loads are given in Sections C.2 to C.6. Each mode of operation of the platform, such as drilling, production, workover, or combinations thereof, should be considered. In addition, the fatigue requirements of Section F should be satisfied.

B.3 DESIGN FOR CONSTRUCTION CONDITIONS

The platform should be designed to resist loads occurring during fabrication, transportation, and installation as specified in Section C.5.

For platforms which are to be relocated to new sites, loads resulting from removal, onloading, transportation, upgrading and reinstallation should be considered in addition to the above construction loads.

B.4 THE STRENGTH AND STABILITY CHECK

B.4.1 A load factor is applied to each of the nominal external loads in the combinations given in Sections C.2 to C.6. The combination of factored nominal loads causes amplified internal forces; these amplified internal forces are labeled Q in Section C.

B.4.2 A resistance factor is applied to the nominal strength of each member, joint, and foundation component to determine its factored strength. Each component should be proportioned to have sufficient factored strength to resist any amplified internal force, Q . The appropriate strength and stability checking equations are given in Sections D, E, G, and H. These checking equations are comprised of the formulas for the nominal strength of the component and the resistance factors.

B.4.3 In some construction and installation conditions, the internal forces should be computed from unfactored

nominal loads and then the load factors applied to the internal forces to arrive at Q . See Section C.5.3.

B.5 STRUCTURAL ANALYSIS

Internal forces in members should be determined by an indeterminate, 3-dimensional structural analysis. In general, a linear elastic model of the structure should be sufficient. The nonlinear behavior of axial and lateral pile-soil support should be modeled explicitly to ensure load-deflection compatibility between the structure and pile-soil system. For pile members that are highly stressed, the effects of material nonlinearity (so called $M-P-\Phi$ moment-axial load-rotation relationship) should be accounted for within the structure-pile-soil system.

In no case should load factors be used as a substitute for a rational analysis to determine internal forces.

Other aspects of structural analysis are discussed in relevant sections on wave dynamics, fatigue, and earthquakes.

B.6 REDUNDANCY

Consideration should be given to providing redundancy in the structure. Framing patterns that provide substitute load paths are to be preferred.

The resistance and load factors in this recommended practice apply only to pile-founded jacket structures typical of those installed in the Gulf of Mexico and offshore California. Modifications of these factors or development of new factors suitable for a different type of structural framing should be based on a rational system reliability analysis.

B.7 CORROSION PROTECTION

Unless specified otherwise by the owner, the systems for corrosion protection should be designed in accordance with NACE RP-01-76.

B.8 DEFORMATION LOADS

Consideration should be given to the stresses induced by deformation loads such as caused by temperature change, creep, relaxation, prestressing or uneven settlement. See Commentary.

SECTION C LOADS

C.1 SCOPE

For each category of load — gravity; wind, wave and current; earthquake; and fabrication and installation — the following topics are covered in Section C: (1) nominal values for the load, (2) procedures for determining the external load (applied force), (3) load factors to apply to the load and to other loads acting in combination, and (4) methods for computing internal forces due to the action of the load. Load factors should be applied for each load category before combining the loads and before structural analysis.

C.2 GRAVITY LOADS

C.2.1 Factored Gravity Loads. Each member, joint, and foundation component should be strength checked for the internal force Q caused by the following factored gravity loads:

$$Q = 1.3 D_1 + 1.3 D_2 + 1.5 L_1 + 1.5 L_2 \dots \dots \dots (C.2-1)$$

where D_1 , D_2 , L_1 , and L_2 are defined below.

C.2.2 Dead Load 1, D_1 . Dead Load 1 is the self weight of the structure including:

1. Weight of the structure in air, including where appropriate the weight of piles, grout, and solid ballast.
2. Weight of equipment and other objects permanently mounted on the platform that do not change with the mode of operation.
3. Hydrostatic forces acting on the structure below the waterline including internal and external pressure and resulting buoyancy.
4. The weight of water enclosed in the structure, whether permanently installed or temporary ballast. See Section C.2.6 for unintentional flooding.

The nominal value of D_1 is the value computed from nominal dimensions and densities.

C.2.3 Dead Load 2, D_2 . Dead Load 2 is the load imposed on the platform by weight of equipment and other objects. These loads may change from one mode of operation to another or during a mode of operation but otherwise remain constant for long periods of time. Dead Load 2 should include the following:

1. The weight of drilling and production equipment that can be added or removed from the platform.
2. The weight of living quarters, heliport and other life-support equipment, diving equipment and utilities equipment, which can be added or removed from the platform.

The nominal value of D_2 should be the estimated lift weight of the object plus any field installed appurtenances.

C.2.4 Live Load 1, L_1 . Live Load 1 includes the weight of consumable supplies and fluids in pipes and tanks. The nominal value of L_1 is computed from the nominal weight of the heaviest material and the largest capacity under the mode of operation considered.

C.2.5 Live Load 2, L_2 . Live Load 2 is the short duration force exerted on the structure from operations such as lifting of drill string, lifting by cranes, machine operations, vessel mooring and helicopter loadings. The nominal value should be the load caused by the rated maximum capacity of the equipment involved and should include dynamic and impact effects.

C.2.6 Unintentional Flooding. All unflooded members should be checked with the members also flooded. All flooded members should be strength checked for the unflooded state or positive means of assuring full flooding should be provided.

In addition, when computing the aggregate weight of the submerged portion of the platform, some reasonable fraction of the volume of unflooded members should be assumed flooded if this creates a more severe load condition. The weight should be included as part of the dead load, D_1 .

C.2.7 Position and Range of Gravity Loads. Variations in supply weights and locations of movable equipment should be considered to find the maximum internal force Q in each component. Maximum and minimum values of D_2 , L_1 and L_2 should be considered for each mode of operation and in conjunction with each environmental load.

C.2.8 Carry Down Factors. In the case of gravity loads which may not be present over the entire influence area, a reduction in the resulting internal force may be used. Such a reduction should only be applied if the operating practices provide adequate safeguards to prevent loads from exceeding the reduced values.

C.2.9 Area Loads. Uniform or specified area loads may be used to represent the normal gravity loads on the platform deck. The possibility of concentrated loads should be checked. Load factors for area loads should be based on the proportion of dead and live loads represented by the area load.

C.3 WIND, WAVE AND CURRENT LOADS

C.3.1 Strength Requirements

C.3.1.1 Factored Loads. Each member, joint and foundation component should be strength checked for the internal force Q caused by the action of these factored loads:

$$Q = 1.1D_1 + 1.1D_2 + 1.1L_1 + 1.35(W_e + 1.25D_n) \dots (C.3-1)$$

where W_e is defined below and D_1 , D_2 , and L_1 are defined in Section C.2 and include those parts of each

mode of operation that might reasonably be present during an extreme ocean storm. D_n is defined in Section C.3.3.

When internal forces due to gravity loads oppose the internal forces due to wind, wave and current loads, the gravity load factors should be reduced so that:

$$Q = 0.9D_1 + 0.9D_2 + 0.8L_1 + 1.35(W_e + 1.25D_n) \dots (C.3-2)$$

For this check, the D_2 and L_1 should exclude any parts of that mode of operation that cannot be assured of being present during an extreme storm.

C.3.1.2 Extreme Wind, Wave and Current Load, W_e . W_e is the force applied to the structure due to the combined action of the extreme wave (typically 100-yr return period) and associated current and wind, accounting for the joint probability of occurrence of winds, waves and currents (both magnitude and direction). For some structures whose fluid loading is not strongly dominated by waves, some other combination of wind, wave and current may be appropriate. This definition applies only to storm generated events. In some areas special consideration must be given to effects of tidal and general circulation currents in calculating W_e . See Commentary.

C.3.1.3 Direction of Wind, Wave and Current. Wind, wave and current loads should be anticipated from any direction unless specific conditions make a different assumption more reasonable.

C.3.1.4 Operating Wind, Wave and Current Load. An operating wind, wave and/or current condition can also be specified for design. Each member, joint, and foundation component should be strength checked for the internal force Q caused by the action of the following factored loads:

$$Q = 1.3D_1 + 1.3D_2 + 1.5L_1 + 1.5L_2 + 1.2(W_o + 1.25D_n) \dots (C.3-3)$$

where W_o is the owner defined operating wind wave and current load; D_2 , L_1 , and L_2 are maximum values for each mode of operation; and the inertial load, D_n , corresponds to the static load, W_o .

C.3.2 Static Wave Analysis. The sequence of steps in the calculation of deterministic static design wave forces on a fixed platform (neglecting platform dynamic response and distortion of the incident wave by the platform) is shown graphically in Figure C.3.2-1. The procedure, for a given wave direction, begins with the specification of the design wave height and associated wave period, storm water depth, and current profile. Values of these parameters for U.S. waters are specified in Section C.3.7. The wave force calculation procedure follows these steps:

- An apparent wave period is determined, accounting for the Doppler effect of the current on the wave.
- The two-dimensional wave kinematics are determined from an appropriate wave theory for the specified wave height, storm water depth and apparent period.

- The horizontal components of wave-induced particle velocities and accelerations are reduced by the wave kinematics factor, which accounts primarily for wave directional spreading.
- The effective local current profile is determined by multiplying the specified current profile by the current blockage factor.
- The local current profile is combined vectorially with the wave kinematics to determine locally incident fluid velocities and accelerations for use in Morison's equation.
- Member dimensions are increased to account for marine growth.
- Drag and inertia force coefficients are determined as functions of wave and current parameters; and member shape, roughness (marine growth), size, and orientation.
- Wave force coefficients for the conductor array are reduced by the conductor shielding factor.
- Hydrodynamic models for risers and appurtenances are developed.
- Local wave/current forces are calculated for all platform members, conductors, risers, and appurtenances using Morison's equation.
- The global force is computed as the vector sum of all the local forces.

The discussion in the remainder of this section is in the same order as the steps listed above. There is also some discussion on local forces (such as slam and lift) that are not included in the global force.

C.3.2.1 Apparent Wave Period. A current in the wave direction tends to stretch the wave length, while an opposing current shortens it. For the simple case of a wave propagating on a uniform in-line current, the apparent wave period seen by an observer moving with the current can be estimated from Figure C.3.2-2, in which T is the actual wave period (as seen by a stationary observer), V_1 is the current component in the wave direction, d is storm water depth (including storm surge and tide), and g is the acceleration of gravity. This figure provides estimates for $d/gT^2 > 0.01$. For smaller values of d/gT^2 , the equation $(T_{app}/T) = 1 + V_1/\sqrt{gd}$ can be used. While strictly applicable only to a current that is uniform over the full water depth, Figure C.3.2-2 provides acceptable estimates of T_{app} for "slab" current profiles that are uniform over the top 50 m (164 ft) or more of the water column. For other current profiles, a system of simultaneous nonlinear equations must be solved iteratively to determine T_{app} (see Commentary). The current used to determine T_{app} should be the free-stream current (not reduced by structure blockage).

C.3.2.2 Two-Dimensional Wave Kinematics. For the apparent wave period, T_{app} , specified wave height, H , and storm water depth, d , two-dimensional regular

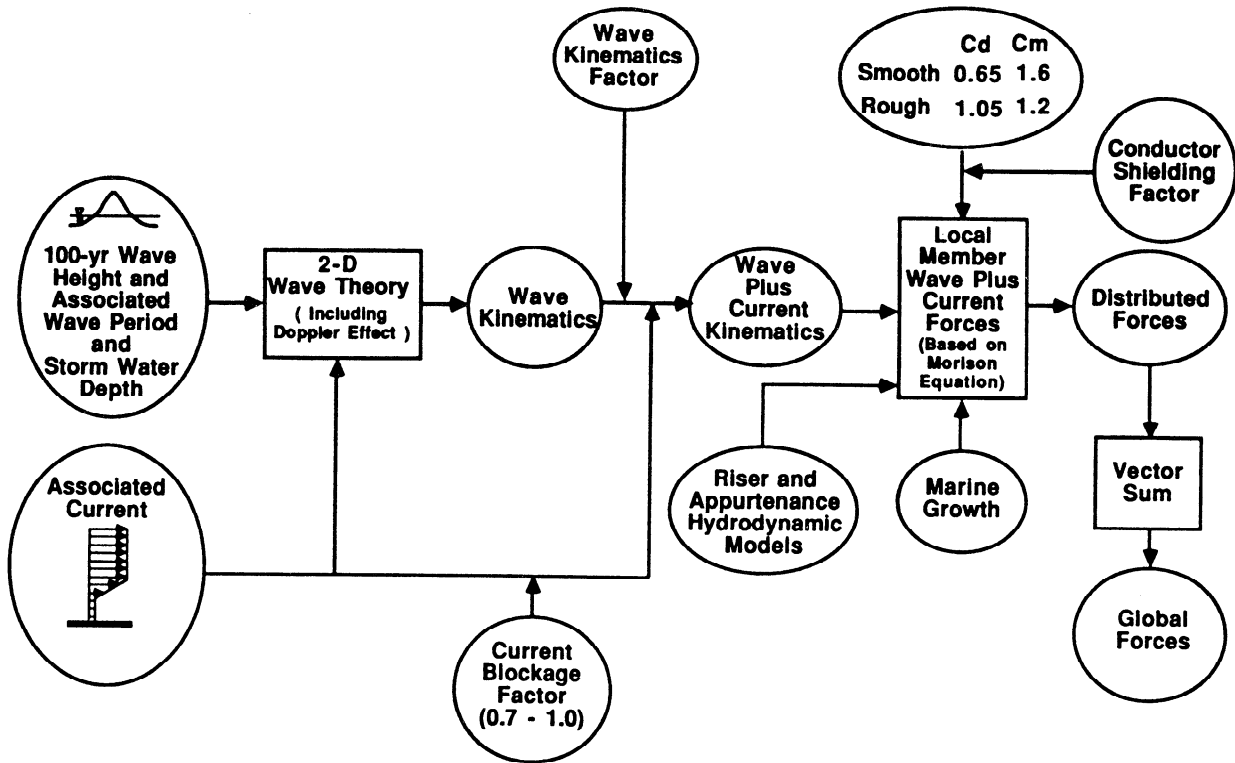


FIG. C.3.2-1
PROCEDURE FOR CALCULATION OF WAVE PLUS CURRENT FORCES FOR STATIC ANALYSIS

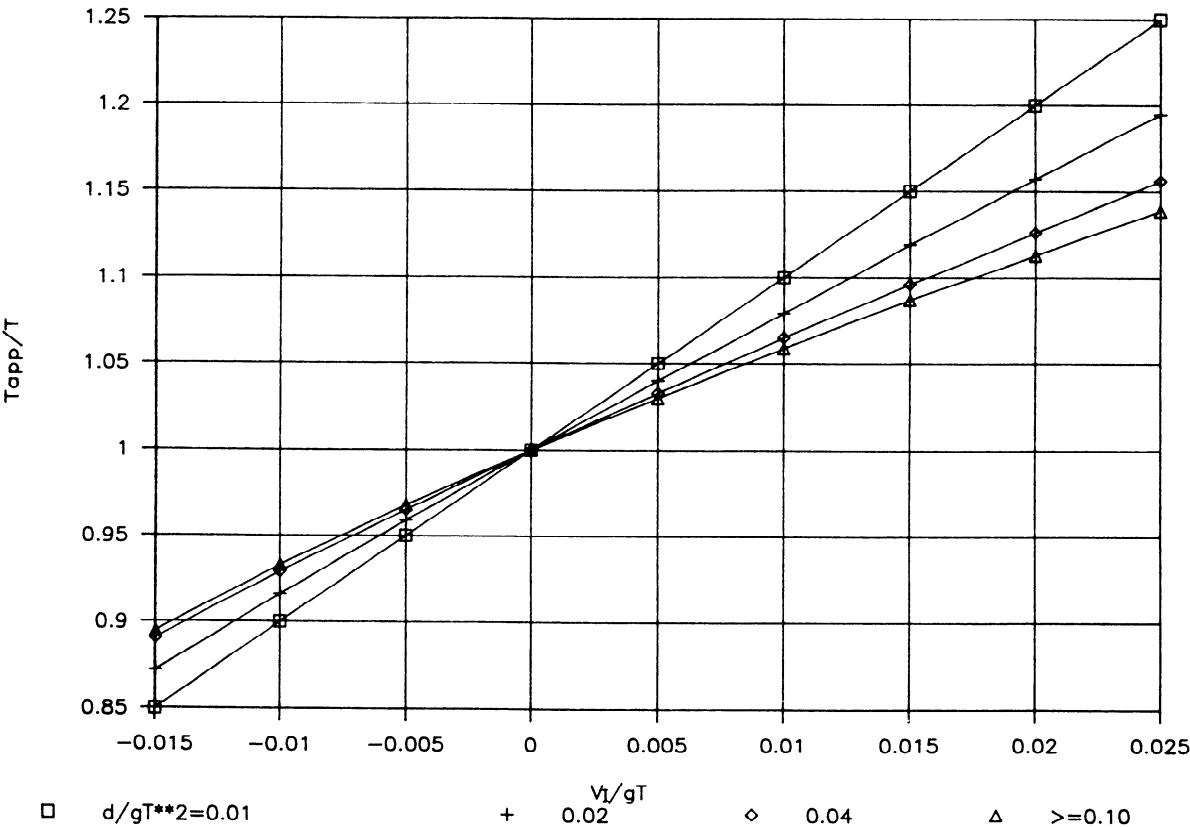


FIG. C.3.2-2
DOPPLER SHIFT DUE TO STEADY CURRENT

wave kinematics can be calculated using the appropriate order of Stream Function wave theory. In many cases, Stokes V wave theory will produce acceptable accuracy. Figure C.3.2-3 (Reference C252) shows the regions of applicability of Stokes V and various orders of Stream Function solutions in the H/gT_{app}^2 , d/gT_{app}^2 plane. Other wave theories, such as Extended Velocity Potential and Chappellear, may be used if an appropriate order of solution is selected.

C.3.2.3 Wave Kinematics Factor. The two-dimensional regular wave kinematics from Stream Function or Stokes V wave theory do not account for wave directional spreading or irregularity in wave profile shape. These “real world” wave characteristics can be approximately modeled in deterministic wave analyses by multiplying the *horizontal* velocities and accelerations from the two-dimensional regular wave solution by a wave kinematics factor. Wave kinematics measurements support a factor in the range 0.85 to 0.95 for tropical storms and 0.95 to 1.00 for extratropical storms. Particular values within these ranges that should be used for calculating guideline wave forces are specified for the Gulf of Mexico in Section C.3.7.4.1 and for other U.S. waters in Section C.3.7.6.1. The Commentary provides additional guidance for calculating the wave kinematics factor for particular seastates whose directional spreading characteristics are known from measurements or hindcasts.

C.3.2.4 Current Blockage Factor. The current speed in the vicinity of the platform is reduced from the specified “free stream” value by blockage. In other words, the presence of the structure causes the incident flow to diverge; some of the incident flow goes around the structure rather than through it, and the current speed within the structure is reduced. Since global platform loads are determined by summing local loads from Morison's equation, the appropriate local current speed should be used.

Approximate current blockage factors for typical Gulf of Mexico jacket-type structures are as follows:

# of Legs	Heading	Factor
3	all	0.90
4	end-on	0.80
	diagonal	0.85
	broadside	0.80
6	end-on	0.75
	diagonal	0.85
	broadside	0.80
8	end-on	0.70
	diagonal	0.85
	broadside	0.80

For structures with other configurations or structures with an atypical number of conductors, a current blockage factor can be calculated with the method described in the Commentary. Calculated factors less than 0.7 should not be used without empirical evidence to

support them. For free-standing or braced caissons and tripod structures, the current blockage factor should be 1.0.

C.3.2.5 Combined Wave/Current Kinematics. Wave kinematics, adjusted for directional spreading and irregularity, should be combined vectorially with the current profile, adjusted for blockage. Since the current profile is specified only to storm water level in the design criteria, some way to stretch (or compress) it to the local wave surface must be used. As discussed in the Commentary, “nonlinear stretching” is the preferred method. For slab current profiles such as those specified for U.S. waters in Section C.3.7, simple vertical extension of the current profile from storm mean water level to the wave surface is a good approximation to nonlinear stretching. For other current profiles, linear stretching is an acceptable approximation. In linear stretching, the current at a point with elevation z , above which the wave surface elevation is η (where z and η are both positive above storm mean water level and negative below), is computed from the specified current profile at elevation z' . The elevations z and z' are linearly related, as follows:

$$(z' + d) = (z + d) d / (d + \eta)$$

where d is storm water depth.

C.3.2.6 Marine Growth. All structural members, conductors, risers, and appurtenances should be increased in cross-sectional area to account for marine growth thickness. Also, elements with circular cross-sections should be classified as either “smooth” or “rough,” depending on the amount of marine growth expected to have accumulated on them at the time of the loading event. Specific marine growth profiles are provided for U.S. waters in Section C.3.7.

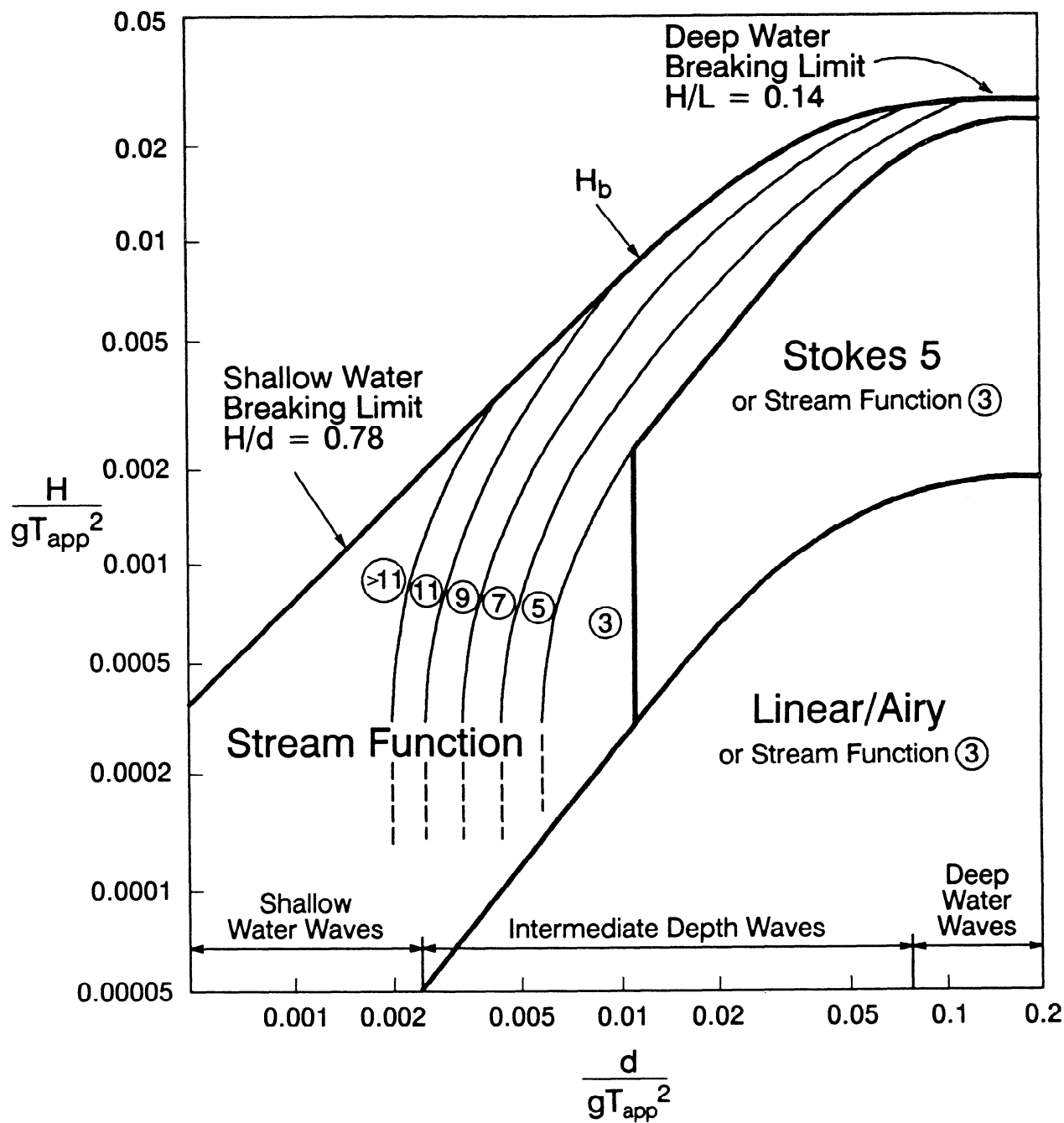
C.3.2.7 Drag and Inertia Coefficients. Drag and inertia coefficients are discussed in detail in the Commentary. For typical design situations, global platform wave forces can be calculated using the following values for unshielded circular cylinders:

$$\text{smooth} \quad C_d = 0.65, C_m = 1.6$$

$$\text{rough} \quad C_d = 1.05, C_m = 1.2$$

These values are appropriate for the case of a steady current with negligible waves or the case of large waves with $U_{mo} T_{app}/D > 30$. Here, U_{mo} is the maximum horizontal particle velocity at storm mean water level under the wave crest from the two-dimensional wave kinematics theory, T_{app} is the apparent wave period, and D is the platform leg diameter at storm mean water level.

For wave-dominant cases with $U_{mo} T_{app}/D < 30$, guidance on how C_d and C_m for nearly vertical members are modified by “wake encounter” is provided in the Commentary. Such situations may arise with large-diameter caissons in extreme seas or ordinary platform members in lower seastates considered in fatigue analyses.



H/gT_{app}^2 : Dimensionless wave steepness

d/gT_{app}^2 : Dimensionless relative depth

H : Wave height

H_b : Breaking wave height

d : Mean water depth

T_{app} : Wave period

g : Acceleration of gravity

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FIG. C.3.2-3
REGIONS OF APPLICABILITY OF STREAM FUNCTIONS, STOKES V, AND LINEAR WAVE THEORY
(FROM ATKINS, 1990; MODIFIED BY API TASK GROUP ON WAVE FORCE COMMENTARY)

For members that are not circular cylinders, appropriate coefficients can be found in Reference C301.

C.3.2.8 Conductor Shielding Factor. Depending upon the configuration of the structure and the number of well conductors, the wave forces on the conductors can be a significant portion of the total wave forces. If the conductors are closely spaced, the forces on them may be reduced due to hydrodynamic shielding. A wave force reduction factor, to be applied to the drag and inertia coefficients for the conductor array, can be estimated from Figure C.3.2-4, in which S is the center-to-center spacing of the conductors in the wave direction and D is the diameter of the conductors, including marine growth. This shielding factor is appropriate for either (a) steady current with negligible waves or (b) extreme waves, with $U_{mo} T_{app}/S > 5\pi$. For less extreme waves with $U_{mo} T_{app}/S < 5\pi$, as in fatigue analyses, there may be less shielding. The Commentary provides some guidance on conductor shielding factors for fatigue analyses.

C.3.2.9 Hydrodynamic Models for Appurtenances. Appurtenances such as boat landings, fenders or bumpers, walkways, stairways, grout lines, and anodes should be considered for inclusion in the hydrodynamic model of the structure. Depending upon the type and number of appurtenances, they can significantly increase the global wave forces. In addition, forces on some appurtenances may be important for local member

design. Appurtenances are generally modeled by non-structural members which contribute equivalent wave forces. For appurtenances such as boat landings, wave forces are highly dependent on wave direction because of shielding effects. Additional guidance on the modeling of appurtenances is provided in the Commentary.

C.3.2.10 Morison Equation. The computation of the force exerted by waves on a cylindrical object depends on the ratio of the wavelength to the member diameter. When this ratio is large (>5), the member does not significantly modify the incident wave. The wave force can then be computed as the sum of a drag force and an inertia force, as follows:

$$F = F_D + F_I = C_d \frac{w}{2g} A U|U| + C_m \frac{w}{g} V \frac{\delta U}{\delta t} \quad \text{..... (C.3-4)}$$

where:

F = hydrodynamic force vector per unit length acting normal to the axis of the member

F_D = drag force vector per unit length acting normal to the axis of the member in the plane of the member axis and U

F_I = inertia force vector per unit length acting normal to the axis of the member in the plane of the member axis and $\delta U/\delta t$

C_d = drag coefficient

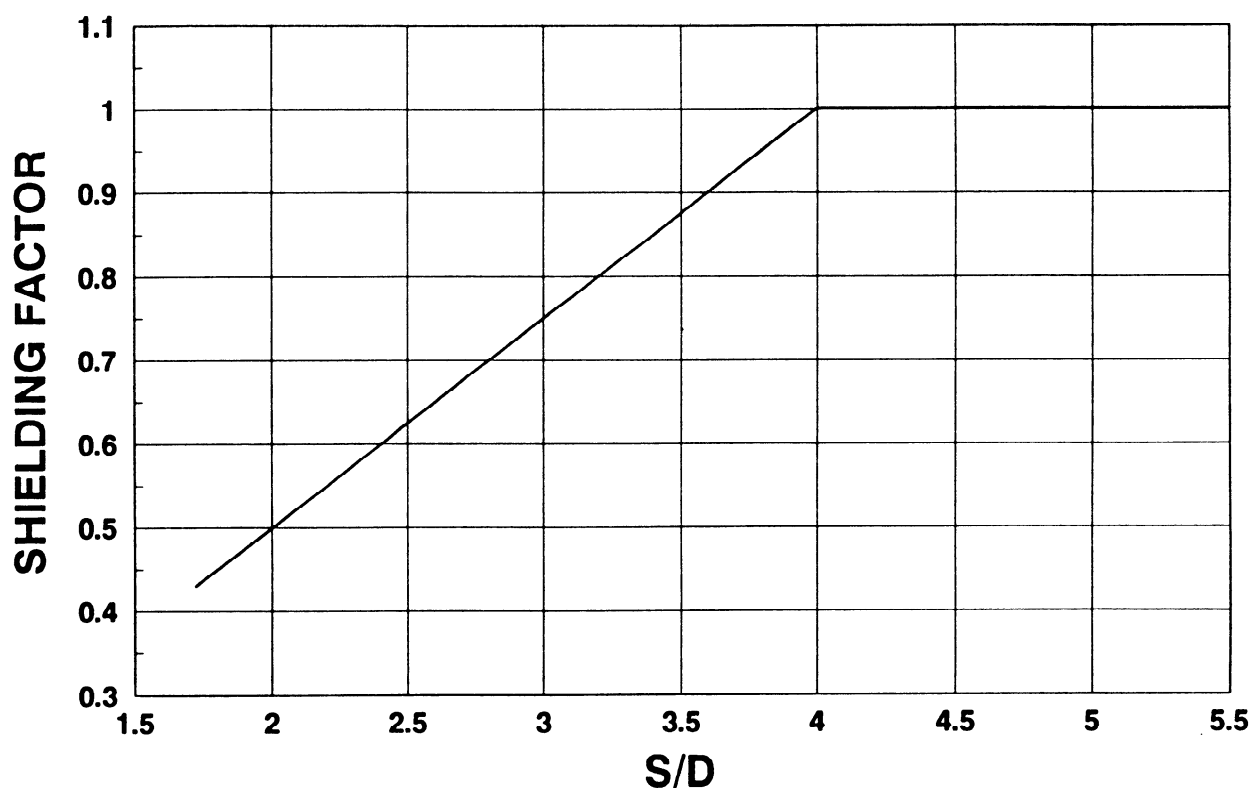


FIG. C.3.2-4
SHIELDING FACTOR FOR WAVE LOADS ON CONDUCTOR ARRAYS
AS A FUNCTION OF CONDUCTOR SPACING.

- w = weight density of water
 g = gravitational acceleration
 A = projected area normal to the cylinder axis per unit length ($= D$ for circular cylinders)
 V = displaced volume of the cylinder per unit length ($= \pi D^2/4$ for circular cylinders)
 D = effective diameter of circular cylindrical member including marine growth
 U = component of the water velocity vector (due to wave and/or current) normal to the axis of the member
 $|U|$ = absolute value of U
 C_m = inertia coefficient
 $\delta U / \delta t$ = component of the local acceleration vector of the water normal to the axis of the member

Note that the Morison equation, as stated here, ignores the convective acceleration component in the inertia force calculation (see Commentary). It also ignores lift forces, slam forces, and axial Froude-Krylov forces.

When the size of a structural body or member is sufficiently large to span a significant portion of a wavelength, the incident waves are scattered, or diffracted. This diffraction regime is usually considered to occur when the member width exceeds a fifth of the incident wave length. Diffraction theory, which computes the pressure acting on the structure due to both the incident wave and the scattered wave, should be used, instead of the Morison equation, to determine the wave forces. Depending on their diameters, caissons may be in the diffraction regime, particularly for the lower sea-states associated with fatigue conditions. Diffraction theory is reviewed in Reference C306. A solution of the linear diffraction problem for a vertical cylinder extending from the sea bottom through the free surface (caisson) can be found in Reference C248.

C.3.2.11 Global Structure Forces. Total base shear and overturning moment are calculated by a vector summation of (a) local drag and inertia forces due to waves and currents (see Section C.3.2.10), (b) dynamic amplification of wave and current forces (see Section C.3.3), and (c) wind forces on the exposed portions of the structure (see Section C.3.4). Slam forces can be neglected because they are nearly vertical. Lift forces can be neglected for jacket-type structures because they are not correlated from member to member. Axial Froude-Krylov forces can also be neglected. The wave crest should be positioned relative to the structure so that the total base shear and overturning moment have their maximum values. It should be kept in mind that: (a) maximum base shear may not occur at the same wave position as maximum overturning moment; (b) in special cases of waves with low steepness and an opposing current, maximum global structure force may occur near the wave trough rather than near the wave

crest; and (c) maximum local member stresses may occur for a wave position other than that causing the maximum global structure force.

C.3.2.12 Local Member Design. Local member stresses are due to both local hydrodynamic forces and loads transferred from the rest of the structure. *Locally generated forces* include not only the drag and inertia forces modeled by Morison's equation (Equation C.3-4), but also lift forces, axial Froude-Krylov forces, and buoyancy and weight. Horizontal members near storm mean water level will also experience vertical slam forces as a wave passes. Both lift and slam forces can dynamically excite individual members, thereby increasing stresses (see Commentary). *Transferred Loads* are due to the global fluid-dynamic forces and dynamic response of the entire structure. The fraction of total stress due to locally generated forces is generally greater for members higher in the structure; therefore, local lift and slam forces may need to be considered in designing these members. The maximum local member stresses may occur at a different position of the wave crest relative to the structure centerline than that which causes the greatest global wave force on the platform. For example, some members of conductor guide frames may experience their greatest stresses due to vertical drag and inertia forces, which generally peak when the wave crest is far away from the structure centerline.

C.3.3 Extreme-Wave Dynamic Analysis.

C.3.3.1 Extreme Inertial Load, D_n . D_n is the nominal inertial load computed by applying the provisions of this section. D_n is the inertial load at the time when the total global dynamic response (static and inertial) is a maximum. The return period of this total global dynamic response should be approximately the same as the quasi-static response to W_e alone. See Commentary.

For platforms with sufficiently short natural periods (less than three seconds), D_n may be neglected. See Commentary.

C.3.3.2 Global Dynamic Wave Analysis. The load D_n is established by a global dynamic analysis. The magnitude of D_n can be a direct result of the global dynamic wave analysis or can be derived from W_e and the Dynamic Amplification Factors (DAF) from the global dynamic analysis. This section describes the global dynamic analysis; Section C.3.3.3 describes the load set D_n .

C.3.3.2.1 Dynamic Analysis Methods. The wave and other time varying loads should be realistic representations of the frequency content of the loading. Time history methods using random waves are preferred. Frequency domain methods may be used for the global dynamic analysis, provided the linearization of the drag force can be justified.

C.3.3.2.2 Design Seastate. The random waves should originate from one or more wave spectra that are plausible conditions for producing the design wave defined

in Section C.3.1.2. Currents associated with the design seastate can affect dynamic response through the drag force in Equation C.3-4 and should be considered in the global dynamic analysis. Wind fluctuations with time may be neglected except for some compliant platforms.

C.3.3.2.3 Fluid Force on a Member. Equation C.3-4 may be used to compute forces on members of fixed platforms. Guidance on selection of drag and inertia coefficients for dynamic analyses is provided in the commentary Comm. C.3.2.7.

The relative velocity form of Morison's equation should not be used for fixed platforms (see Commentary). Fluid forces associated with the platform's acceleration are accounted for by the hydrodynamic added mass.

C.3.3.2.4 Mass. The mass of the dynamic model of a fixed platform should include that of the platform steel, all appurtenances, conductors, and deck loads, the mass of water enclosed in submerged tubular members, the mass of marine growth expected to accumulate on the structure and the hydrodynamic added mass of submerged members, accounting for increased member diameter due to marine growth.

C.3.3.2.5 Damping. Equivalent viscous damping values may be used in lieu of an explicit determination of damping components. In the absence of substantiating information for damping values for a specific structure, a damping value of two to three percent of critical for extreme wave analyses (and two percent of critical for fatigue analyses) may be used.

C.3.3.2.6 Stiffness. A linear elastic, stiffness model of the platform that includes the interaction of the structure and the foundation is adequate for the global dynamic wave analysis.

C.3.3.3 Member Design. The dynamic stresses in the members may be computed from a constant-in-time load set that represents the inertial forces; this load set is D_n . The scalar magnitude of D_n is defined in Section C.3.3.1 and is a direct result of the Global Dynamic wave analysis described in Section C.3.3.2. D_n needs to be distributed to the lumped mass points to model the inertial force at each mass point. For free vibration motion, these inertial forces are proportional to the mass times the mode shape at that mass. Consideration should be given to the number of modes that are being excited and to the possible phasing of the dynamic modes of vibration with the static wave force.

C.3.4 Wind Force.

C.3.4.1 General. The wind criteria for design should be determined by proper analysis of wind data collected in accordance with Section Comm. A.4.2. As with wave loads, wind loads are dynamic in nature, but some structures will respond to them in a nearly static fashion. For conventional fixed steel templates in relatively shallow water, winds are a minor contributor to global loads (typically less than 10 percent). Sustained

wind velocities should be used to compute global platform wind loads, and gust velocities should be used for the design of individual structural elements. In deeper water and for compliant designs, wind loads can be significant and should be studied in detail. A dynamic analysis of the platform is indicated when the wind field contains energy at frequencies near the natural frequencies of the platform. Such analyses may require knowledge of the wind turbulence intensity, spectra, and spatial coherence. These items are addressed below.

C.3.4.2 Wind Properties. Wind speed and direction vary in space and time. On length scales typical of even large offshore structures, statistical wind properties (e.g., mean and standard deviation of velocity) taken over durations of the order of an hour do not vary horizontally, but do change with elevation (profile factor). Within long durations, there will be shorter durations with higher mean speeds (gust factor). Therefore, a wind speed value is only meaningful if qualified by its elevation and duration. A reference value $V(1 \text{ hr}, z_R)$ is the one hour mean speed at the reference elevation, z_R , 10m (33 ft.). Variations of speed with elevation and duration, as well as wind turbulence intensity and spectral shape have not been firmly established. The available data show significant scatter, and definitive relationships cannot be prescribed. The relationships given below provide reasonable values for wind parameters to be used in design. Alternative relationships are available in the public domain literature or may be developed from careful study of measurement.

C.3.4.2.1 Mean Profile. The mean profile for the wind speed average over one hour at elevation z can be approximated by

$$V(1 \text{ hr}, z) = V(1 \text{ hr}, z_R) (z/z_R)^{0.125} \dots\dots\dots (\text{C.3-5})$$

C.3.4.2.2 Gust Factor. The gust factor $G(t, z)$ can be defined as

$$G(t, z) \equiv V(t, z) / V(1 \text{ hr}, z) = 1 + g(t)I(z) \dots\dots\dots (\text{C.3-6})$$

where $I(z)$ is the turbulence intensity described below and t is the gust duration with units of seconds. The factor $g(t)$ can be calculated from

$$g(t) = 3.0 + \ln[(3/t)^{0.6}] \text{ for } t \leq 60 \text{ sec} \dots\dots\dots (\text{C.3-7})$$

C.3.4.2.3 Turbulence Intensity. Turbulence intensity is the standard deviation of wind speed normalized by the mean wind speed over one hour. Turbulence intensity can be approximated by

$$I(z) \equiv \sigma(z) / V(1 \text{ hr}, z) = \begin{cases} 0.15(z/z_s)^{-0.125} & \text{for } z \leq z_s \\ 0.15(z/z_s)^{-0.275} & \text{for } z > z_s \end{cases} \dots\dots\dots (\text{C.3-8})$$

where $z_s = 20\text{m}$ (66 ft.) is the thickness of the "surface layer."

C.3.4.2.4 Wind Spectra. As with waves, the frequency distribution of wind speed fluctuations can be described by a spectrum. Due to the large variability in measured wind spectra, there is no universally accepted spectral

shape. In the absence of data indicating otherwise, the simple shape given by the following equation is recommended:

$$\frac{fS(f)}{\sigma(z)^2} = \frac{f/f_p}{[1 + 1.5f/f_p]^{5/3}} \quad \text{..... (C.3-9)}$$

where $S(f)$ is the spectral energy density at elevation z , f is the frequency in Hertz, and $\sigma(z)$ is the standard deviation of wind speed, i.e., $\sigma(z) = I(z)V(1 \text{ hr}, z)$. Measured wind spectra show a wide variation in f_p about an average value given by

$$f_p z / V(1 \text{ hr}, z) = 0.025 \quad \text{..... (C.3-10)}$$

Due to the large range of f_p in measured spectra, analysis of platform sensitivity to f_p in the range

$$0.01 \leq f_p z / V(1 \text{ hr}, z) \leq 0.10 \quad \text{..... (C.3-11)}$$

is warranted. It should be noted that f_p is not at the peak of the dimensional wind energy, since Equation C.3-9 gives the reduced spectrum.

C.3.4.2.5 Spatial Coherence. Wind gusts have three dimensional spatial scales related to their durations. For example, three second gusts are coherent over shorter distances and therefore affect smaller elements of a platform superstructure than fifteen second gusts. The wind in a three second gust is appropriate for determining the maximum static wind load on individual members; five second gusts are appropriate for maximum total loads on structures whose maximum horizontal dimension is less than 50m (164 ft.); and fifteen second gusts are appropriate for the maximum total static wind load on larger structures. The one minute sustained wind is appropriate for total static superstructure wind loads associated with maximum wave forces for structures that respond dynamically to wind excitation but which do not require a full dynamic wind analysis. For structures with negligible dynamic response to winds, the one-hour sustained wind is appropriate with maximum wave forces. In frequency domain analyses of dynamic wind loading, it can be conservatively assumed that all scales of turbulence are fully coherent over the entire superstructure.

C.3.4.3 Wind Velocity and Force Relationship. The wind force on an object should be calculated by using an appropriate method such as:

$$F = (\rho/2) (V)^2 C_s A \quad \text{..... (C.3-12)}$$

where:

F = wind force,

V = wind speed,

C_s = shape coefficient,

A = area of object,

ρ = mass density of air (at standard temperature and pressures
or
= 1.226 kg/m³
= 0.00238 lb. sec²/ft)

C.3.4.4 Local Wind Force Considerations. For all angles of wind approach to the structure, forces on flat surfaces should be assumed to act normal to the surface and forces on vertical cylindrical objects should be assumed to act in the direction of wind. Forces on cylindrical objects which are not in a vertical attitude should be calculated using appropriate formulas that take into account the direction of the wind in relation to the attitude of the object. Forces on sides of buildings and other flat surfaces that are not perpendicular to the direction of the wind shall also be calculated using appropriate formulas that account for the skewness between the direction of the wind and the plane of the surface. Where applicable, local wind effects such as pressure concentrations and internal pressures should be considered by the designer. These local effects should be determined using appropriate means such as the analytical guidelines set forth in Section 6 of Reference C303.

C.3.4.5 Shape Coefficients. In the absence of data indicating otherwise, the following shape coefficients, C_s , are recommended for perpendicular wind approach angles with respect to each projected area.

Beams	1.5
Sides of buildings	1.5
Cylindrical sections	0.5
Overall projected area of platform	1.0

C.3.4.6 Shielding Coefficients. Shielding coefficients may be used when, in the judgment of the designer, the second object lies close enough behind the first to warrant the use of the coefficient.

C.3.4.7 Wind Tunnel Data. Wind pressures and resulting forces may be determined from wind tunnel tests on a representative model.

C.3.5 Current Force.

C.3.5.1 Current Force Only. Where current is acting alone (i.e., no waves) the drag force should be determined by Equation C.3-4 with $\delta U/\delta t = 0$.

All slender members exposed to the current should be investigated for the possibility of vibration due to periodic vortex shedding as discussed in commentary Comm. C.3.2.12.

C.3.5.2 Current Associated with Waves. Consideration should be given to the possible superposition of current and waves. In those cases where this superposition is necessary, the current velocity should be added vectorially to the wave particle velocity before the total force is computed as described in Section C.3.2. If the current is a substantial fraction of the wave orbital velocity, the effects of wave-current interaction may be considered before superposing the two. Where there is sufficient knowledge of wave/current joint probability, it may be used to advantage in the choice of design current.

C.3.6 Deck Clearance. Large forces result when waves strike a platform's deck and equipment. To avoid this,

the bottom of the lowest deck should be located at an elevation which will clear the calculated crest of the design wave with adequate allowance for safety. Omnidirectional guideline wave heights with a nominal return period of 100 years, together with the applicable wave theories and wave steepnesses should be used to compute wave crest elevations above storm water level, including guideline storm tide. A safety margin, or air gap, of at least 1.5 m (5 ft.) should be added to the crest elevation to allow for unexpected platform settlement, water depth uncertainty, and for the possibility of extreme waves in order to determine the minimum acceptable elevation of the bottom beam of the lowest deck to avoid waves striking the deck. An additional airgap should be provided for any known or predicted long term seafloor subsidence.

In general, no platform components, piping or equipment should be located below the lower deck in the designated air gap. However, when it is unavoidable to position such items as minor subcellars, sumps, drains or production piping in the air gap, provisions should be made for the wave forces developed on these items. These wave forces may be calculated using the crest pressure of the design wave applied against the projected area. These forces may be considered on a "local" basis in the design of the item. These provisions do not apply to vertical members such as deck legs, conductors, risers, etc., which normally penetrate the air gap.

C.3.7 Hydrodynamic Force Guidelines for U.S. Waters

C.3.7.1 General. Design parameters for hydrodynamic loading should be selected in the manner described in Section A.5, using environmental data collected and presented as outlined in Comm. A.4. This section presents guideline design hydrodynamic force parameters which should be used if the studies described in Commentary A.4 are not performed.

C.3.7.2 Intent. The provisions of this section provide for the analysis of static wave loads for platforms in areas designated in Figure C.3.7-1. The resulting guideline wave forces may be taken as satisfying the definition of W_e in Section C.3.1.2. Depending upon the natural frequencies of the platform and the predominant frequencies of wave energy in the area, it may be necessary to perform dynamic analyses following the provision of Section C.3.3. Further, the general wave conditions in certain of these areas are such that consideration of fatigue loads may be necessary.

Use of the guidelines should result in safe but not necessarily optimal structures. Platform owners may find justification for designing structures for conditions more or less severe than indicated by these guidelines. As discussed in Comm. A.5, design criteria depend upon the overall loading, strength, and exposure characteristics of the installed platform. The guidelines should not be taken as a condemnation of platforms designed by different practices. Historical experience, loading, and strength characteristics of these structures

may be used for such evaluations. The provisions of this section are intended to accommodate such considerations. The actual platform experience and exposure and the amount of detailed oceanographic data available vary widely among the areas shown in Figure C.3.7-1. The Gulf of Mexico is characterized by a substantial amount of experience, exposure, and data. For other areas, there is less experience and data. The amount of wave information available for the different areas is indicated by the quality rating in Table C.3.7-1. The guidelines presented herein represent the best information available at this time and are subject to revision from time to time as further work is completed.

C.3.7.3 Guideline Design Metocean Criteria for the Gulf of Mexico North of 27° N Latitude and West of 86° W Longitude. The metocean criteria for this region of the Gulf of Mexico represent a consensus reached during 1991 by the API Task Group on Wave Force Commentary. The criteria are based on the 100-yr wave height and associated variables that result from hurricanes. The criteria are defined in terms of the following results:

- Omnidirectional wave height vs water depth
- Principal direction associated with the omnidirectional wave height
- Wave height vs direction
- Currents associated with the wave height by direction
- Associated wave period
- Associated storm tide
- Associated wind speed

For locations affected by strong tidal and/or general circulation currents, such as the Loop current and its associated detached eddies, special metocean criteria need to be defined to take into account the possibility of large forces caused by a combination of extreme currents and waves smaller than the 100-yr hurricane wave.

The metocean criteria are intended to be applied in combination with other provisions of Section C.3.7 to result in a guideline design level of total base shear and overturning moment on a structure.

The criteria apply for Mean Lower Low Water (MLLW) greater than 7.5m (25 ft.) and outside of barrier islands, except in the immediate vicinity of the Mississippi Delta (denoted by the cross-hatched area in Figure C.3.7-2). In this area the guidelines may not apply because the Delta may block waves from some directions, and there are some very soft seafloor areas that may partially absorb waves. Wave heights lower than the guideline values may be justified in these areas.

C.3.7.3.1 Omnidirectional Wave Height vs Water Depth. The guideline omnidirectional wave height vs MLLW with a nominal return period of 100 years is given in Figure C.3.7-3.

TABLE C.3.7-1
GUIDELINE EXTREME WAVE, CURRENT, AND STORM TIDE VALUES*
FOR TWENTY AREAS IN UNITED STATES WATERS
(Water depth > 91m (300 ft.) except as noted)

	U _i , kt		H _m , ft.		S	X, ft.		Quality
	"Open Shelf"	Range	"Open Shelf"	Range	Range	"Open Shelf"	Range	
1. Gulf of Mexico (N of 27° N & W of 86° W) (See Section C.3.7.3 for Guideline Extreme Metocean Criteria)								
2. Gulf of Mexico (E of 86° W)	2	(1-3)	70	(60-80)	$\frac{1}{11} - \frac{1}{15}$	3	(2-5)	2
3. Southern California (Santa Barbara & San Pedro Ch)	2	(1-3)	45	(35-55)	$\frac{1}{11} - \frac{1}{30}$	6	(5-7)	1
4. California Outer Bank	2	(1-3)	60	(50-65)	$\frac{1}{13} - \frac{1}{25}$	5	(4-6)	2
5. Central California	2	(1-3)	60	(50-70)	$\frac{1}{13} - \frac{1}{25}$	7	(6-8)	2
6. Washington/Oregon	2	(1-4)	85	(70-100)	$\frac{1}{13} - \frac{1}{19}$	8	(7-10)	3
7. Gulf of Alaska (Icy Bay)	3	(2-4)	100	(90-120)	$\frac{1}{13} - \frac{1}{17}$	11	(10-13)	2
8. Gulf of Alaska (Kodiak)	3	(2-4)	90	(80-110)	$\frac{1}{13} - \frac{1}{17}$	10	(9-12)	2
9. Lower Cook Inlet	4	(3-6)	60	(45-70)	$\frac{1}{10} - \frac{1}{11}$	16	(13-20)	2
10. North Aleutian Shelf	3	(2-4)	70	(60-90)	$\frac{1}{12} - \frac{1}{16}$	8	(6-12)	1
11. St. George Basin	3	(2-4)	85	(75-95)	$\frac{1}{12} - \frac{1}{16}$	5	(3-7)	1
12. Navarin Basin	2	(1-3)	85	(75-95)	$\frac{1}{12} - \frac{1}{16}$	4	(3-5)	1
13. Norton Sound (d=60 ft.)	3	(1-4)	45	(35-50)	$\frac{1}{11} - \frac{1}{18}$	11	(8-14)	2
14. Chukchi Sea (d>60 ft.)	2	(1-3)	50	(40-60)	$\frac{1}{11} - \frac{1}{15}$	6	(4-8)	3
15. Chukchi Sea (d<60 ft.)	3	(2-5)	0.78(d+X)		**	9	(6-12)	3
16. Beaufort Sea (d>50 ft.)	2	(1-3)	40	(35-50)	$\frac{1}{13} - \frac{1}{17}$	4	(2-7)	2
17. Beaufort Sea (d<50 ft.)	4	(3-6)	0.78(d+X)		**	8	([-2] -12)	2
18. Georges Bank	2	(1-3)	85	(75-95)	$\frac{1}{10} - \frac{1}{16}$	5	(4-6)	2
19. Baltimore Canyon	3	(2-4)	90	(80-100)	$\frac{1}{10} - \frac{1}{14}$	5	(4-6)	2
20. Georgia Embayment	5	(2-8)	75	(65-85)	$\frac{1}{11} - \frac{1}{15}$	5	(3-7)	2

U_i = inline current at storm water levelH_m = 100-yr. maximum individual wave heightS = deep water wave steepness from linear theory = $(2\pi H_m)/(gT_m^2)$

g = acceleration of gravity

T_m = zero-crossing period associated with H_m, which can be calculated from SX = storm tide (Section A.4.4) associated with H_m (mean higher high water plus storm surge)

d = datum water depth

Quality

1 = based on comprehensive hindcast study verified with measurements.

2 = based on limited hindcasts and/or measurements.

3 = preliminary guide.

* wind speeds, significant wave height, and spectral peak period associated with H_m are discussed in Sections C.3.7.5.1 and C.3.7.5.2.** Use the same range for T_m as in deeper water.