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Fatigue Design Review of Offshore Wind Turbine Generator Structures

Reference: 30087.FR (Rev. 01)
Date: 21 October 2013

30087.FR (Rev. 01)

FATIGUE DESIGN REVIEW OF OFFSHORE WIND TURBINE GENERATOR STRUCTURES

FINAL REPORT

21 October 2013

Submitted to:

United States Department of the Interior
Bureau of Safety and Environmental Enforcement
Acquisition Operations Branch, MS2101
381 Elden Street
Herndon, VA 20170

ATTENTION: Altan Aydin

Submitted by:

BMT FLEET TECHNOLOGY LIMITED
311 Legget Drive
Kanata, ON
K2K 1Z8

BMT Contact: Dale Braun, PEng.
Tel: 613-592-2830, Ext. 333
Fax: 613-592-4950
Email: dbraun@fleetech.com

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REPORT: Fatigue Design Review of Offshore Wind Turbine Generator Structures – Final Report

DATE: 21 October 2013

PREPARED BY:

Dale Braun, M.Eng., P.Eng.
Senior Engineer

REVIEWED AND APPROVED BY:

Aaron Dinovitzer, MASc, MBA, P.Eng.
Vice President, Engineering and Technology

PROJECT TEAM:
Aaron Dinovitzer
Dale Braun
George Comfort (G. Comfort Ice Engineering Ltd.)

REVISION HISTORY RECORD

Revision No.	Date of Issue	Description of Change
00	20 Sept. 2013	Initial draft submission.
01	21 Oct 2013	Final report submission.

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ACRONYMS AND ABBREVIATIONS

3D	Three-dimensional
ABS	American Bureau of Shipping
AISI	American Iron and Steel Institute
ALS	Accidental limit state
API	American Petroleum Institute
CC	Continuous brittle crushing
DEN	Denmark
DNV	Det Norske Veritas
FEA	Finite element analysis
FLI	Frequency lock-in
FLS	Fatigue limit state
GL	Germanischer Lloyd
H2S	Hydrogen Sulfide
HAT	Highest astronomical tide
HAWT	Horizontal axis wind turbine
HSLA	High-strength low-alloy
HSS	Hollow structural section
IACS	International Association of Classification Societies
IC	Intermittent crushing
IEC	International Electrotechnical Commission
IIW	International Institute of Welding
ISO	International Organization for Standardization
JONSWAP	JOint North Sea WAve Project sea spectrum
LAT	Lowest astronomical tide
LRFD	Load and resistance factored design
MIT/ NREL	Massachusetts Institute of Technology / National Renewable Energy Laboratory
MSL	Mean sea level
NaCl	Sodium Chloride
NSS	Normal sea state
NTM	Normal wind turbulence model
NWH	Normal wave height
NWLR	Normal water level range
NWP	Normal wind profile
OWTG	Offshore wind turbine generator
POR	Portugal
PSD	Power spectral density
RNA	Rotor-nacelle assembly
SCE	Standard calomel electrode
SCF	Stress concentration factor
SLS	Serviceability limit state
SWE	Sweden
TIG	Tungsten inert gas weld
TLP	Tension leg platform
UK	United Kingdom

ULS	Ultimate limit state
US	United States
US OCS	United States Outer Continental Shelf
W.E.S.T.	Wind Energy Systems Technology (Galveston, Texas)

LIST OF SYMBOLS

1P	First period of rotor rotation
3P	Third period of rotor rotation
a	A constant relating to the mean S-N curve
A	Fatigue strength coefficient for the design S-N curve determined empirically (by ABS)
A	Area of section
$\bar{a}$	Fatigue strength coefficient (intercept parameter) for the design S-N curve
C	Fatigue strength coefficient for the design S-N curve determined empirically (by ABS)
cm	Centimeters
C_R	Strength parameter for calculation of the static or quasi-static ice load by ISO 19906
Cr	Chromium
C_{R0}	Strength parameter for the reference area in the calculation of the static or quasi-static ice load by ISO 19906
D	Total damage accumulated (or, cumulative damage ratio)
D	Structure diameter for ice calculations
D_C	Characteristic cumulative fatigue damage
D_D	Design fatigue damage
D_f	Factored cumulative damage ratio
DFF	Design fatigue factor
f	Frequency of the ice forcing function
$\mathrm{f_b}$	Blade passing frequency
FDDs	Freezing degree days
FDF	Fatigue design factor
F_{max}	Maximum ice force during a loading cycle
F_{min}	Minimum ice force during a loading cycle
ΔF	Variation in ice load during a cycle
f_N	Natural frequency of the eigenmode
F_o	Static horizontal ice load by DNV-OS-J101
$\mathrm{f_o}$	Fundamental eigenfrequency
$\mathrm{f_R}$	Rotor frequency
ft	Feet
h	Thickness of sea ice
$\mathrm{h_{50}}$	Thickness of sea ice with a recurrence period of 50 years
H_d	Quasi-static ice load
$\mathrm{h_m}$	Ice thickness equal to the long term mean value of the annual maximum ice thickness for winters with ice
$\mathrm{H_{NWH}}$	Height of the normal deterministic design wave
H_s	Significant wave height
Hz	Hertz
i	Index of stress range interval
I	Moment of inertia (second moment of area)
I_{15}	Characteristic value of the turbulence intensity at hub height for

	a 10-minute mean wind speed of 15 m/s
I_{mk}	Indentation formula factor
J	Number of stress range intervals
k_1	Shape factor for ice calculations by IEC 61400
k_2	Contact factor for ice calculations by IEC 61400
k_3	Aspect ratio factor for ice calculations by IEC 61400
K_g	Geometric stress concentrations
K_{ISSC}	Stress corrosion cracking threshold
$K_{t\alpha}$	Angular misalignment stress concentrations
K_{te}	Eccentric misalignment stress concentrations
K_w	Notch stress concentrations
ΔK	Stress intensity factor range
m	Meters
m	Slope parameter for the design S-N curve for N less than N_Q
m	Empirical coefficient for calculation of the static or quasi-static ice load by ISO 19906
M	Applied bending moment acting on section
MN	Mega Newton
M_n	True modal mass in kilograms
n	Empirical coefficient for calculation of the static or quasi-static ice load by ISO 19906
N	Number of cycles for the S-N (i.e. stress-life) approach
n	Applied number of cycles
N_f	Calculated number of cycles at the fatigue life (by ABS)
n_i	Number of cycles endured at stress range $\Delta\sigma_i$
N_i	Number of cycles to failure at stress range $\Delta\sigma_i$
N_Q	The limiting number of cycles at which the slop of the design S-N curve changes
N_t	Total number of stress cycles for the service life (by ABS)
P	Applied axial force acting on section
q	Empirical coefficient ranging from 0.1 to 0.5 to calculate the variation in ice load during a cycle by ISO 19906
r	Slope parameter for the design S-N curve for N greater than N_Q
R	Fatigue loading ratio
s	Standard deviation of log N for the design S-N curve
S	Applied stress range for the S-N (i.e. stress-life) approach
T	Period
t_{limit}	Maximum ice thickness at which movements fo not occur (for coastal waters), taken to be the long-term mean value of the annual maximum ice thickness
T_p	Peak spectral period
U_{ice}	Ice movement speed during interaction with the structure
V	Volt
V_1 and V_{ref}	The mean wind speeds associated with the significant wave height, peak spectral period and direction for each normal sea state for the design situation under consideration

V_{ave}	Annual average wind speed
V_{hub}	Average wind speed at the elevation of the hub
V_{in}	Lowest 10-minute mean wind speed at hub height during power production (steady wind without turbulence)
V_{out}	Highest 10-minute mean wind speed at hub height during power production (steady wind without turbulence)
w	Structure width
y	Elevation of neutral axis of section for bending
z	Elevation above the still waterline
Z_{hub}	Elevation of the hub above the mean sea level
α	Power law exponent
σ	A measured or inferred strength index for the area of interest
φ	Non-normalized modal amplitude at the ice action point
θ	Coefficient used in the energy inflow criterion
$\Delta\sigma_i$	The applied stress range at index i
σ_1	Total stress field
σ_b	Bending stress
σ_c	Ice crushing strength
σ_c	A strength index for the reference area
σ_m	Membrane stress
σ_{max}	Maximum stress
σ_{min}	Minimum stress
ξ_n	Relative damping coefficient of the structure
σ_p	Peak stress
σ_r	Residual stress
γ_m	Material factor for fatigue

1 INTRODUCTION

1.1 Overview

Offshore wind turbine generators (OWTGs) are subjected to fatigue loads arising from several random conditions associated with the demanding environments in which these systems are designed to operate. The design of an OWTG attempts to maximize the power production by optimizing the rotor blade diameter and the rotor/nacelle control system to capture as much of the wind force as possible. However, the design of the tower structure also should be optimized, which is achieved generally by making it as light as possible. For floating OWTGs, an additional challenge is introduced in that the system on which the tower is supported must be designed with efficient consideration of the mooring response and damping effects. Thus, a satisfactory tower design is achieved when the competing interests are balanced between:

(i) maximizing the wind force captured; and

(ii) providing a structurally efficient design.

Subsequently, the projected power production from the OWTG can be evaluated against the cost to implement the design and to ensure the level of return appropriate for the investment.

In all cases, the tower structures specifically require due consideration of the design of fatigue critical details and the potential for crack initiation and crack progression. Consistent with any fatigue design, all sources potentially contributing to the accumulation of fatigue damage need to be identified for design. For OWTGs, the fatigue loading is dominated by the wind loads, wave loads and operational loads, which may act alone or in combination to produce the long-term stress history used to determine the structural response. The behavior of the OWTG tower structure in response to the random loads is complex and must consider the static and dynamic response and the aerodynamic damping produced by the turbine rotor operating in the environment. Then, the damage attributed to the load cycles in the long-term stress history needs to be characterized and applied consistent with a design philosophy, such as safe life; damage tolerant; or, fail-safe to estimate the fatigue performance.

A reliable estimation of the fatigue life is fundamental to ensuring the design of the OWTG will function throughout the expected service life and allow the opportunity to optimize power production.

Given the expanding interest in OWTGs, both technical and commercial, several pilot projects and prototype designs are now installed alongside productive wind farms. Of significance, the development of standards and guidelines has accelerated in parallel. The work associated with this project has involved reviewing the available standards and guidelines, along with technical papers and conference proceedings, related to the development of the fatigue design methodologies and the certification of OWTG tower structures.

The approach taken, and which is described in this report, has been to identify the primary standards and guidelines relevant to the design of OWTG tower structures and to describe the methodologies for certification, which may be based on limit states design or working stress design methods; and with application of safe life, damage tolerant or fail-safe philosophies. Subsequently, the loads and load combinations used to develop the long-term stress range history for the fatigue life estimation are defined based on the external conditions associated with the environment, including wind, wave, ice and operation of the turbine rotor.

The fatigue design methodologies account for the various factors affecting the fatigue resistance and the life estimation. Specifically, material behavior considerations and load effects are described. Further, the approaches by which the standards and guidelines complete the estimation of fatigue life are presented and include conventional damage accumulation or crack growth methods. An overview is given regarding some design solutions applicable to enhancing the fatigue performance of OWTG tower structures.

Significant factors affecting the fatigue life of the OWTG tower structure are discussed, with demonstration of the key issues, including calculation of the nominal, hot-spot and notch stresses for design; the environmental effects on fatigue and the difference between the fatigue life of a corroded structure and corrosion fatigue; and, the effect of fabrication quality on fatigue life.

1.2 Fatigue Design of OWTG Tower Structures

Evaluation of the fatigue performance represents but one significant aspect of the design process, made along with designing for strength, serviceability and accidents that may occur. Fatigue failure typically is manifested by a crack having propagated into or through the thickness of a structural component. In a heavy steel structure such as the OWTG tower, which is subject to highly random and cyclic environmental loading, fatigue damage can lead to failures of the tower (usually, a non-redundant component) such that loss of power production could result. As for any large steel structure, the consequences of fatigue failure may result in loss of the asset, loss of safety or worse.

The standards and guidelines have established the philosophies by which fatigue design and evaluation is completed. Effectively, these philosophies are based on assumptions regarding the initiation of a crack, or the propagation of a crack once initiated, and can be related to safe life, damage tolerant or fail-safe approaches.

Undertaking a fatigue assessment of the OWTG structure is a complex process. Characterization of the wind and wave environments, and the simulation of an integrated structural model, requires significant resources and planning. The OWTG will be subject consistently to variable and cyclic loading in harsh environments, with an expectation that an operational performance will be achieved and a level of safety will be preserved throughout the service life. To achieve an efficient design, it is recommended that the response of the OWTG tower structure be characterized using an integrated analysis in the time-domain; i.e. one that considers the whole of the structure and nacelle components subject to the simultaneous loading due to wind and wave actions.

Following from the definition provided in ABS "Guide for the Fatigue Assessment of Offshore Structures", a fatigue assessment establishes the fatigue demand on a structure or structural component by calculating the fatigue damage. The fatigue demand on a structure is described in terms of the stress ranges resulting from the applied variable loading. The calculation of the stress ranges for design may be accomplished using various methods, depending on the level of complexity chosen for the desired efficiency of the design. ABS identifies a spectral (or, frequency domain) method, a deterministic method and a simplified method. An assessment based on the results of an integrated time domain analysis is identified also as a detailed option, and this is described by GL and DNV in detail. In general, the latter approach may be considered most appropriate for analyzing systems subject to non-linear loading and which exhibit a non-linear structural response.

The method used to calculate the fatigue damage (i.e., the stress ranges) will reflect the level of detail required for the analysis. The structural model for the OWTG will include a representation of the global behavior when subject to the various loads associated with the environmental conditions. The global response may need to consider the excitation of resonant frequencies, which can be accounted for using dynamic amplification factors. Depending on the extent of local refinement and modeling, the effects of stress concentrations may be addressed in the calculation of the fatigue damage. Methods such as the nominal stress approach and the hot-spot stress approach can account for these effects, with application of the appropriate S-N curves.

Once the stress range distribution or history for each component of the structure is described analytically, a comparison is made to the predicted fatigue strength. The fatigue strength may be characterized based on any of the safe life, damage tolerant or fail-safe design philosophies. The standards and guidelines in effect describe a safe life methodology in which the fatigue strength is based on a stress-life (or, S-N) approach that relates the number of cycles (N) and the constant stress range (S) likely to cause fatigue failure for specific categories of welded connection details or components. The cumulative fatigue damage resulting from the full history of variable stress ranges is estimated using a linear Miner-Palmgren summation of the individual damage resulting from each stress range interval.

Design fatigue factors (DFFs) may be prescribed for determining the estimated design life for the OWTG. In other cases, the minimum design life is specified or otherwise taken as equal to 20 years. Thus, the cumulative fatigue damage is calculated for each fatigue critical detail and then compared to the calculated fatigue life. When the fatigue life of every component achieves the required design life, then the fatigue assessment is completed successfully.

1.2.1 Importance of Fatigue Design

Aside from the structural considerations of the fatigue assessment, the importance of a good fatigue design is to realize a structural configuration that is optimized for weight and costs. For OWTG designs to achieve a competitive advantage as compared to conventional power generation (such as offshore oil and gas operations), the structural design must be as light and as simple as possible while providing the required resistance with respect to strength, fatigue, accidental loads and serviceability. For typical steel structures, conservative designs tend to rely on thicker and heavier components, using more robust connections and details, which arise from the application of simplifying assumptions, intended to reduce the computational effort and costs. However, for OWTGs, the expenses associated with a detailed fatigue assessment may be justified based on achieving a design that respects the requirements of the local climate while not being overly conservative, and hence heavier, more expensive and more costly to operate.

1.2.2 Fatigue Related Issues and OWTGs

With regards to the steel supporting structure of an OWTG (the blades being composed of materials other than steel and subject to complex load effects), the issues related to fatigue design and requiring consideration include:

- Complex loading resulting from the environmental and operational conditions, support stiffness and aerodynamic effects;
- Geometric discontinuities, such as may occur in way of access openings or changes in section/thickness and abrupt changes in connection geometry;
- Special requirements for tubular members and connections;
- Steel material properties (include grade and hardness);
- Flaws, imperfections or inclusions in the material;
- Welds and attachments affecting the fatigue life;
- Welding residual stress effects;
- Mean stress effects (compressive and tensile);
- Corrosion effects and protection schemes, including cathodic protection;
- Quality assurance and quality control programs to address fabrication practices, including tolerances:
    - Fabrication methods as applied in practice will differ from those used to fabricate the test specimens, which have been used to develop the design S-N curves; and,
- For monopile types, the grouted connections between the tower base and the piles (though failure of the grout is not considered herein; discussion being limited to steel components).

1.3 Objectives

The objective of this project is to identify the methodologies applied in the existing standards and guidelines to provide safe life and damage tolerant designs for the OWTG support structure, including those that make use of criteria such as the Miner-Palmgren cumulative damage hypothesis and fracture mechanics. The available design methods and criteria applicable to OWTG design are reviewed with respect to being:

- Self-consistent;
- Applicable to all of the relevant structural applications;
- Comprehensive and with consideration of all relevant load effects;
- Easily understood and applied; and,
- Capable of providing an acceptable level of safety and reliability.

The application of the recommended fatigue design methodology for offshore wind turbine generator (OWTG) tower structures regarding (i) characterization of loads related to the ice-structure interaction; and, (ii) fatigue life estimation for typical fixed OWTG tower structures are also described.

A demonstration of the fatigue life estimation methodologies is provided with focus on the issues associated with the stress analysis techniques, including determination of the nominal or hot-spot stresses; design for the effects of environment (corrosion); and, the effect of fabrication quality on fatigue design.

1.4 Scope of Work

The project scope of work has been undertaken in two parts, as follows.

1.4.1 Review of Fatigue Design Philosophies and Methodologies

The first section of the project provides an overview of the design philosophies described in the standards and guidelines, being applicable to the design of OWTG structures with respect to the materials, connections, arrangements and loads, and regarding the following aspects:

1. Philosophy used to define the design criteria:
    a. Safe Life, for which components are designed to survive a specific design life (e.g., Miner-Palmgren summation and the S-N approach);
    b. Damage Tolerant, for which a component is designed to operate with damage (possibly at a reduced level) and not fail completely (e.g., fracture mechanics);
    c. Fail-Safe, for which a structure is designed such that the system continues to operate without harm in the event of component failure (e.g., redundancy combined with other safe life or damage tolerant approaches);
2. Complexity of the methodology:
    a. Level 1: Rule of thumb guidelines or non-analytical techniques for fatigue life enhancement.
        – Considers fabrication quality and inspection standards.
    b. Level 2: Analytical closed-form solutions based on regression or semi-empirical methods
        – These analytical approaches use conservative idealisations of load and structural response. The assumptions regarding loading, structural configuration and materials-related issues must be consistent with the design intent.
    c. Level 3: Numerical or spectral methods (computer simulation and analysis)
        – The strengths, weaknesses and assumptions associated with detailed modeling techniques must be consistent with the design intent.

3. Treatment of uncertainty in the methodology:

    a. Deterministic, including safety factors (often based on experience from proven designs);

    b. Calibrated Implicit Reliability, such as load and resistance factored design (LRFD);

    c. Probabilistic, including statistical distribution of uncertain design inputs (i.e. load data).

Further, the methods are described by which each of the following issues affecting the fatigue design and performance are addressed by the current standards and guidelines:

1. Characterization of the environmental and other conditions from which the loads, load combinations and load cases associated with the fatigue limit state are derived, including:

    a. Wind;

    b. Aerodynamic;

    c. Wave;

    d. Marine, including those associated with interaction of the structure and sea ice (fluctuation of global or local loads due to the ice-structure interactions); and,

    e. Operational.

2. Characterization of factors affecting the fatigue resistance:

    a. Material behavior considerations such as strength, thickness effects, mean stress effects, corrosion protection and the effects of surface treatments;

    b. Load effects, including those associated with stress concentrations (i.e. large openings or member discontinuities; local discontinuities in geometry or stiffness; misalignments; and, weld shape);

    c. Welding issues and details, including defects and residual stresses.

The generalized methodology for the fatigue design and assessment of OWTG tower structures is described, with discussion regarding the calculated structural response, application of design S-N curves, fatigue life estimation and improvement.

1.4.2 Fatigue Methodology Demonstration and Calculation

The second part of the project demonstrated the fatigue methodology using discussion and calculations corresponding to a sample monopile tower application. The load characterization and fatigue life estimation are described with respect to the following aspects of OWTG tower structure design:

1.4.2.1 Detailed Ice Load Characterization

The cyclic loads associated with the interaction of moving ice sheets with the tower structure above the waterline, similar to that as shown in Figure 1.1 for the European Union's (EU) LOLEIF lighthouse project in Norway, are identified. The existing methodologies described in a subset of the standards and guidelines (specifically, IEC 61400, ISO 19906 and DNV-OS-J101) are discussed in detail. Gaps in the understanding and opportunities for further development are identified.

Figure 1.1: Ice-Structure Interaction for the LOLEIF Lighthouse in the Gulf of Bothnia (by M. Bjerkås)

1.4.2.2 Fatigue Life Estimation

The procedure involved in estimating the fatigue life, or the probability of fatigue failure over the design life of an OWTG tower structure, is considered generally in the following steps:

a. Define the long-term statistical distribution of the cyclic loads for the design life;

b. Determine the corresponding long-term statistical distribution of stress history at each detail to be considered; and,

c. Calculate the incremental fatigue damage associated with the stress ranges.

Consistent with the standards and guidelines reviewed, the fatigue design methodology is based on the principles of limit states design for a given class of structure; the safety index/target reliability; and, using the prescribed partial factors on the load combinations and cases associated with the fatigue limit state. Failure of a component is considered to occur when a fatigue crack is initiated. In this way, a safe-life design philosophy is applied to ensure that the OWTG tower structure will perform adequately throughout the full design life.

The methodology by which the fatigue life of the OWTG tower structure is estimated is described. In addition, some significant aspects involved in the stress analysis are detailed. The effects on fatigue of corrosion (corrosion fatigue versus fatigue of a corroding structure) and the effects of fabrication quality are demonstrated.

2 DESIGN GUIDELINES AND CERTIFICATION

2.1 Introduction

As compared to other structures, such as offshore platforms in the oil and gas industry, offshore wind turbine generators are more affected by fatigue. It has been estimated that an OWTG operating for 20 years will be subjected to more than 10^9 load cycles, based only on operation of the turbine blades (Dalhoff, Argyriadis and Klose 2007). When additional load cycles are considered, including those associated with the wind and wave loadings, the total number of load cycles over the design life is significant and approaches the maximum number of cycles typically reported on S-N curves for steel.

Since OWTGs are subject to complex loading associated with stochastic wind and wave combinations, the standards and guidelines have established various methodologies and requirements for the design, analysis and assessment of these structures with respect to fatigue and other limit states. The guidance has been established over many years with input from within the industry and with adaptation of the existing (and well-defined) standards and requirements from the offshore oil and gas industry, in particular. An overview of the guidance available to designers is provided subsequently.

2.2 Standards and Guidelines

The standards and the guidelines applicable for the design and certification of OWTGs are listed in Table 2.1. Here, standards are distinguished from guidelines in that the former are considered to have been developed by following a recognized standards development process and with consensus established among committee members, representing industry and researchers (among others); the latter are developed by a group or company, who may not be subject to a vote or a formal protocol.

Some of these references for OWTGs are based on assumptions associated with monopile installations, or other fixed-types, which are valid (or, feasible) only up to some maximum water depth. Floating types, which are more recently under development, draw from available offshore structures design standards, mainly the API guidance or others as listed in Table 2.2. Both GL and DNV cross-reference their own standards or guidelines and recommended practices for design or certification of offshore structures.

2.3 Types of OWTG Considered

In the recent past, the development of wind turbine generators has progressed from onshore designs to offshore designs. Initial offshore installations were located in relatively shallow waters; however, as the expertise and the commercial interest has become established, designs for larger wind turbine generators has grown and so solutions have been sought for deeper water applications.

In this section, the different types of OWTG applications are introduced, while noting that the current standards and guidelines do not yet address all manner of OWTG design concepts, specifically floating OWTGs. As will be shown, the different types of OWTGs may be limited by practical issues and depending on the fixity of the support structure or system, and the water depth. Table 2.3 lists some of the different types of OWTG structural designs based on the water depth.

Table 2.1: Standards for Design or Certification of OWTGs

Type	Organization	Reference
Standard	International Electrotechnical Commission (IEC)	IEC 61400-1 "Wind Turbines – Part 1: Design Requirements" (2005) Outlines minimum design requirements for wind turbines and is not intended for use as a complete design specification or instruction manual. The standard is not intended to give requirements for wind turbines installed offshore, in particular for the support structure.
		IEC 61400-3 "Wind Turbines – Part 3: Design Requirements for Offshore Wind Turbines" (2009) Outlines minimum design requirements for offshore wind turbines and is not intended for use as a complete design specification or instruction manual. It specifies additional requirements for assessment of the external conditions at an offshore wind turbine site and essential design requirements to ensure the engineering integrity of offshore wind turbines. Its purpose is to provide an appropriate level of protection against damage from all hazards during the planned lifetime. IEC 61400-3 specifically addresses the issues associated with the stochastic wind and wave loads acting on OWTGs.
		IEC 61400-22 "Wind Turbines – Part 22: Conformity Testing and Certification" Identifies the requirements relevant to the wind turbine itself (including the RNA and blades) and the control systems.
Guideline	Germanischer Lloyd (GL)	"Guideline for the Certification of Offshore Wind Turbines" (2005) It is noted the GL Guidelines precede (and form the basis of) IEC 61400-3.
	Det Norske Veritas (DNV)	DNV-OS-J101 "Design of Offshore Wind Turbine Structures" (2011) Provides basis for design principles; loads and load effects; and, load and resistance factors, and covering the design of steel offshore structures, using limit states design and including the fatigue limit state.
		DNV-OS-J103 "Design of Floating Wind Turbine Structures" (2013)
	American Bureau of Shipping (ABS)	"Guide for Building and Classing Offshore Wind Turbine Installations" (2010) This guide requires that the fatigue resistance of structural details be evaluated in accordance with ABS "Guide for Fatigue Assessment of Offshore Structures".
		"Guide for Building and Classing Floating Offshore Wind Turbine Installations" (2013)
		"Guide for Building and Classing Bottom-Founded Offshore Wind Turbine Installations" (2013)
	Federal Maritime and Hydrographic Agency (BSH) (Germany)	"Standard - Design of Offshore Wind Turbines" (2007)
	Bureau Veritas (BV)	"Classification and Certification of Floating Offshore Wind Turbines" (2010)

Table 2.2: Standards and Guidelines for Design or Certification of Offshore Structures

Type	Organization	Reference
Standard	International Organization for Standardization	ISO 2394:1998 "General principles on reliability for structures" Presents a limit states design approach and the principles of probability based design. The application of partial safety factors for the evaluation of structural behavior is described.
		ISO 19900:2002; ISO 19901; ISO 19902:2007; ISO 19903:2006; ISO 19904:2006; ISO 19905:2012; ISO 19906:2012 Represents the suite of ISO standards detailing the assessment and design of offshore structures (fixed and floating), with focus on those structures associated with the offshore oil and gas industry.
	American Petroleum Institute (API)	API RP-2A-WSD (22nd Edition) "Recommended practice for planning, designing and constructing fixed offshore steel platforms – Working Stress Design" The API standards have been developed as the primary reference for offshore structures in the oil and gas industry. The API standards do not address the concerns unique to OWTGs; however elements have been adapted (in particular) for floating OWTGs.
		API RP-2A-LRFD "Recommended practice for planning, designing and constructing fixed offshore steel platforms – Load and Resistance Factor Design" API has withdrawn this standard subsequent to collaborating with ISO on the development of ISO 19902.
	Standards Norway (NORSOK)	N-001 Structural design (Rev. 4, February 2004) N-003 Actions and action effects (Edition 2, September 2007) N-004 Design of steel structures (Rev. 2, October 2004) N-005 Condition monitoring of loadbearing structures (Rev. 1, Dec. 1997) N-006 Assessment of structural integrity for existing offshore load-bearing structures (Edition 1, March 2009)
Guideline	Det Norske Veritas (DNV)	DNV-RP-C203 "Fatigue Strength Analysis of Offshore Steel Structures"
	Germanischer Lloyd (GL)	Germanischer Lloyd Rules and Guidelines, IV – Industrial Services, Part 6 – Offshore Installations
		Germanischer Lloyd Rules and Guidelines, IV – Industrial Services, Part 6 – Offshore Installations, Chapter 6 – Guidelines for the Construction and Classification/Certification of Floating Production, Storage and Off-Loading Units, Edition 2000
		Germanischer Lloyd Rules and Guidelines, IV – Industrial Services, Part 6 – Offshore Installations, Chapter 7 – Guideline for the Construction of Fixed Offshore Installations in Ice Infested Waters
	American Bureau of Shipping (ABS)	ABS-115 "Guide for the Fatigue Assessment of Offshore Structures" (updated 2010)
		"Commentary on the Guide for the Fatigue Assessment of Offshore Structures" (2003)

Table 2.3: Types of Offshore Wind Turbine Generator Installations

Type	Depth	Installation	Description
Fixed	Shallow Depth < 30 m	Monopile	A monopile OWTG tower structure consists of a steel pipe supported by a single large diameter pile with a wall thickness of up to approximately 150 mm. A grouted transition piece connects the two primary components. The monopile may be driven into the seabed or grouted into sockets drilled into rock. A monopile is a simple design and therefore is the most common foundation in service. However, there are known issues affecting the performance at the interaction between the transition piece and the monopile and due to failure of the grout.
Fixed	Shallow Depth < 30 m	Monopod (Gravity-based foundation or suction bucket)	Gravity-based foundations may be appropriate when installation of piles is costly or complicated by the seabed conditions. The environmental conditions and requirements to resist overturning loads may limit the use of gravity-based foundations.
Fixed	Transitional 30 m < Depth < 50 m	Multipod (Tripod or Quadpod)	Tripod foundations, such as those installed at Alpha Ventus in Germany and planned for W.E.S.T. at Galveston (Texas), consist of a central column supported by the braced base and founded with suction piles or mud mats (quadpods simply extend the braced foundation to four supports at the base). The large diameters of the primary column and bracing members increase the wave interaction, introducing unique issues for ensuring structural integrity, especially with respect to fatigue.
Fixed	Transitional 30 m < Depth < 50 m	Jacket (Piled or gravity-base)	Jacket support structures have been used extensively with offshore platforms in the oil and gas industry. In a form adapted for OWTG towers, the jacket consists of a platform to support the tower and which is supported by tubular members configured as a set of four legs with “X” and “K” bracing. The connections between main members and bracing may be welded or made using cast nodes. Of significant importance to the design of jacket structures is the fatigue performance of the numerous joints in the configuration.
Floating	Deep 50 m < Depth < 200 m	Spar	The arrangement of a spar corresponds to a moored single cylinder, floating due to the entrapped air and kept upright due to ballast. The appropriate ballast must be used in order that the operating turbine does not tilt excessively.
Floating	Deep 50 m < Depth < 200 m	Semi-submersible (“barge”)	Also taken from the offshore oil and gas industry, semi-submersibles use a framework/truss structure with a combination of buoyant sections and ballast to provide dynamic stability. However, wave interaction may affect performance critically.
Floating	Deep 50 m < Depth < 200 m	Tension Leg Platform (TLP)	A TLP consists of a submerged buoyant platform, moored in place with tensioned lines. The submerged platform helps to reduce the global motions, while the reduced cross-section at the waterline minimizes the wave interaction forces.

This discussion of fatigue design for OWTGs in this report considers only Horizontal Axis Wind Turbines (HAWT) having 3-blades. Further, the subset of floating OWTGs considered is taken as consisting of one unit as opposed to a platform arrangement of multiple units. The discussion of fatigue related design issues also is limited to those affecting the design of the tower structure and supporting system. Fatigue issues regarding blades, machinery, electrical systems or connections to the grid (including components such as J-tubes) are not considered.

2.3.1 Fixed OWTGs

For the fixed (or, founded) OWTGs, the various foundations used in practice include gravity caissons, piles (which may include grouted connections to the tower support structure), or suction anchors. The selection of the foundation type may depend on the water depth and the soil conditions at the site. Other factors, including environmental impact, construction costs and the availability of equipment such as cranes or jack-ups will influence the selection.

The behavior of fixed OWTGs is related to the foundation stiffness and the dynamic characteristics of the support structure, each being either “soft” or “stiff”. The design regimes can be classified according to the ratio between the fundamental eigenfrequency (f_o) and the rotor frequency (f_R) and the blade passing frequency (f_b). With respect to the following combinations of foundation stiffness to support structure stiffness, the design regimes correspond to:

1. Soft-Soft, with $f_o < f_R$;
2. Soft-Stiff, with $f_R < f_o < f_b$;
3. Stiff-Stiff, with $f_b < f_o$.

The dynamic behavior of fixed type OWTGs must be well understood in order that resonant frequencies are not excited during operation or certain environmental conditions. The aerodynamic and structural damping also should be determined for subsequent characterization of the combined wind and wave loading effects for design. Over the full service life of the fixed OWTG, the transient soil-stiffness interaction is an important consideration. Changes in the dynamic behavior need to be considered in the evaluation for the long-term.

2.3.2 Floating OWTGs

Floating structures must consider stability and rigid body motions, which can be described by the methodologies applied in the offshore oil and gas industry. For the floating types, the moorings may correspond to catenary or taut arrangements. Floating turbines must minimize the pitch rotation in order that the turbine operates properly. Therefore, mooring, stability and station-keeping are key aspects for the global design.

2.3.3 Water Depth

The size of the OWTG itself, and the size of the farm site, will be designed with consideration of the environmental conditions, metocean conditions (i.e. meteorological and oceanographic), seabed conditions and the economic factors including the potential optimal power production.

However, the decision to install one design type versus another is largely determined by the economics, with certain types being feasible within certain ranges of water depth.

- Shallow water applications correspond to those having a water depth less than about 30 m (100 ft) and now are common and well-understood. Several farms have been operating in Europe and the United Kingdom for several years.
- Deep water applications, having water depths between approximately 50 m (165 ft) and 200 m (655 ft), are seeing increased development with few prototype designs now operating. For these applications, significant experience is drawn from the offshore oil and gas industry.
- Transitional applications are designed for water depths between approximately 30 m (100 ft) and 50 m (165 ft). Again, these designs are based largely on experience in the offshore oil and gas industry with the application of jacket designs. Some new applications have been installed.

A structural design guide, addressing fatigue design, must consider the range of applications for all of the types of OWTGs and environmental conditions, based (in part) on the water depth.

2.4 Philosophies for Design Criteria

When addressing fatigue design of structures subject to random, cyclic loading, the end result by which a safe design is achieved can adhere to any of three main fatigue design philosophies. These include:

(i) a safe life approach;

(ii) (ii) a damage tolerant approach; or,

(iii) (iii) a fail-safe approach.

These approaches each are based in different industries, such as traditional civil structural design or the design of aircraft. The concepts seek to ensure adequate fatigue performance (i.e., a safe, reliable design) while considering the costs and level of effort associated with operations and maintenance; with lost production; and, the consequences of failure with respect to safety or economics.

2.4.1 Safe Life

The safe life design philosophy requires that the OWTG tower structure be designed to perform adequately throughout the design life (taken as 20 years in the standards and guidelines, unless specified otherwise) with some additional capacity included in the design. The recent standards and guidelines apply methodologies that are probability-based and that use the limit states design approach. An OWTG tower structure designed in accordance with these standards and guidelines will be consistent with the concepts of safe life design.

With respect to the fatigue limit state, the standards and guidelines achieve a safe life design through following approach.

- Defining the service life required for the design;
- Defining the nominal acceptable probability of structural failure;
- Partial factors for resistance taken as values equal to or less than 1.0;
- Consideration of a reduced material thickness to account for corrosion effects over the service life;
    - The amount of the reduction is related to the component being a primary or secondary member, or if the component is in the splash zone.
- Application of characteristic design S-N curves, developed based on extensive experimental testing, consistent with the intended level of safety. The design S-N curves are classified to reflect the behavior with respect to:
    - Plates versus hollow-structural steel sections (HSS);
    - In air versus in seawater, with and without cathodic protection;
    - Weld effects;
    - Thickness effects; and,
    - Classification of joints, being welded members or cast nodes.
- Calculation of the design cumulative damage based on the number of cycles to failure at a design stress range (from the corresponding design S-N curves) and with application of a design fatigue factor (DFF).
    - The value for the DFF is determined with consideration of the type of detail under consideration; the accessibility of the component for inspection and maintenance; and, the susceptibility to corrosion.

The risk of overdesign (i.e. being overly conservative) is mitigated by:

- Defining the appropriate class for the wind turbine, based on the site and in consideration of the consequences of failure (i.e., economic-only consequences as compared to the possibility of loss of life or safety).
- Applying specified loads (i.e., partial factors for loads are taken as equal to 1.0);
- Integrated analyses to characterize the environmental conditions from which the wind and wave loading is derived, and with consideration of the aerodynamic damping;
- Determining the loads from the environmental conditions based on long-term measurements and hindcast studies, and using the appropriate statistical analysis to define the corresponding probability distributions.

As part of a safe life design philosophy, it is important to recognize that the structure should be removed from service upon achieving the design life safely. At a minimum, the performance of the structure should be evaluated through inspection to determine the residual capacity (if any) remaining as the end-of-life is approached. This end of life assessment or life extension process often will include damage tolerance analysis, similar to that described in the section that follows, to consider the impact of the existence of defects not detected by the inspection process.

2.4.2 Damage Tolerant

A damage tolerant philosophy explicitly considers degradation or damage and its growth to ensure that the design will continue to function adequately between inspections. This approach considers the inspection regime in the design process because the design philosophy requires that degradation or damage smaller than the failure inducing degradation or damage will be identified during planned inspections and remediated before failure is promoted. The application of a fracture mechanics philosophy, assuming the existence of a crack or other defect that was not detected by inspection, is consistent with a damage tolerant approach. Based on assumptions made regarding the crack size, in-service loading and environment, the fatigue life or recommended inspection interval can be estimated.

As noted, the standards and guidelines allow for the fatigue life estimation by a fracture mechanics approach. As defined in DNV-OS-J101, the fatigue life calculation based on a fracture mechanics approach may be completed separately or as a supplement to the S-N based fatigue calculations. In general, the standards and guidelines reference BS 7910 for guidance regarding the completion of a fracture mechanics fatigue life calculation.

When considering fatigue as the damage accumulation mechanism, a key component in developing an effective damage tolerant design is to implement an inspection, maintenance and repair regime. Since the design is predicated on the basis of assuming it is already damaged (i.e., a crack exists), then the interval between inspections must be defined to provide a minimum level of safety. Further, the inspection activities should include means of identifying small cracks in the structural components, including non-destructive techniques such as ultrasound and magnetic particle inspection. A holistic fatigue damage tolerant design approach would commence with the definition of two crack-like flaw sizes:

(1) the largest flaw that would not be reliably detected; and,

(2) the smallest flaw, or family of flaw sizes (e.g., lengths and depths) that would compromise structural integrity.

Determination of the detectible flaw size is a function of the inspection technology deployed (e.g., visual versus ultrasonic inspection) and is considered as the initial flaw size used in the analysis process when considering fatigue crack growth. The latter flaws, those considered to compromise structural integrity, are used to define the end of the fatigue crack growth process and thus the end of life. In defining an inspection or maintenance interval, the ability of the inspection process to detect flaws before they grow to a critical size is essential. Generally, aside from other conservatisms or safety factors built into the structural or inspection process design, the inspection time interval should be considered to be more than two times shorter than the fatigue life to ensure that defects not detected at the end of one interval do not grow to a critical state prior to the next scheduled inspection.

2.4.3 Fail-Safe

A fail-safe philosophy to design of OWTGs is based on the assumption that failure of structure or component will occur eventually. However, once failure occurs, it will happen in such a manner as to ensure that safety is preserved. Safety may correspond to personnel safety and the prevention of harm or loss of life; or, to a level of economic safety in which a loss of production is mitigated. In this way, failures that are identified during inspection can be managed.

Safety factors can be applied in the design to account for the component as part of a non-fail-safe structure, for which a local failure would lead to global failure (i.e., non-redundant structure); or, part of a fail-safe structure for which a local failure will not extend to a global failure of the structure. Minimum material safety factors are prescribed in the standards and guidelines, with consideration given to components that are accessible for inspection and monitoring and on the basis of the structural redundancy. Further, a fail-safe design may be ensured as follows:

- Using redundant structural arrangements to avoid a single-point of failure;
  - In general, multiple load paths will help to ensure that applied loads can be redistributed effectively as the result of failure of one or more members. The goal is to retain the overall load-carrying capacity even when failure of some components has occurred. The damaged structure can then be rehabilitated or removed from service safely and in a controlled manner.
  - Regarding OWTG, monopiles by definition do not have redundant arrangements for the tower structure; the tower provides only a single load path. By comparison, jackets, multipods or semi-submersible structures can be designed to provide redundancies.
- Designing for ductile failures by avoiding the use of brittle materials, which includes some high strength steels, and ensuring certain buckling modes are avoided;
  - Second-order effects or elastic-plastic analyses may contribute to achieving a ductile structural design.
- The timely identification of failed components is critical to the design. Therefore cracks or other modes of failure must be easily identified from inspection activities.
  - Structural details, particularly at connections, should be designed such that the propagation of cracks will not extend beyond a limit at which the overall integrity is impacted.
- An effective quality control/quality assurance program; proper weld details and techniques; applying surface treatments such as weld profiling or peening; and, specifying fatigue-insensitive details and large-radius corners (for example) will contribute to ensuring a fail-safe design.

2.5 Design and Inspection Philosophy

In order that the OWTG structure can achieve the design service life (at a minimum), an inspection philosophy must be established in parallel with the design philosophy from the outset. The level of inspection required and the type of inspection (i.e., visual versus non-destructive examination) employed will be dependent on the goals defined by the design philosophy.

For example, a safe life design may consider that structural components and details are checked visually for damage, corrosion or distortion to ensure the structural capacity or behavior remains consistent with the design. The residual structural capacity of the system can be calculated based on the condition of the structure and on the period of service. For a damage tolerant approach, NDE methods with known probabilities of detection (such as magnetic particle inspection) can be used, based upon an assumed defect depth to length aspect ratio, to identify cracks or defects that could reduce the structural fatigue life. The residual capacity of the component or detail can be calculated with a fracture mechanics approach based on the assumed (or measured) defect. In both cases, a life extension assessment may be undertaken to confirm the residual fatigue life available in the structure.

The inspection program defined for each OWTG design should identify key components and details that may be subject to increased fatigue damage accumulation, and therefore would warrant detailed inspection. As described in Section 10, various types of weld faults may be associated with the welded connections. Typically, welded connections completed in the field may be associated with more faults than those completed in a controlled environment (i.e., manufacturing facility). Further, areas of the design should be identified for detailed inspection where the continuity of the load path acting through a component or detail (i.e., abrupt cross sections or loading eccentricities) may be associated with stress concentrations.

Continuous primary support structure without attachments will be less susceptible to fatigue damage accumulation. Damage to coatings, or any damage which may have introduced a notch or localized change in stiffness, may lead to corrosion or other effects, including stress concentrations that reduce the fatigue life. In general, the inspection program should be configured to identify the main potential sources of fatigue damage, with emphasis on all welded connections and details, material discontinuities and geometric discontinuities. Guides supporting structural inspection prioritization have been developed for various structural geometries including ship and offshore structures. These types of guides are freely available from various organizations, including the US Ship Structure Committee and American Bureau of Shipping, amongst others.

2.6 Limit States Design

Historically, steel structures designed in the US are based on the working stress design method and this is reflected in the earlier approaches adopted by the API Standards and ABS Guides. However, aligning with more recent practice, the ABS Guides apply the limit states design method. API has published the “Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms” in both limit states design and working stress design formats. Further, API has collaborated in the development of recent international standards, including the ISO standards for offshore structures: series ISO 19900 to ISO 19906. This report focuses on the application of the limit states design method, consistent with recent standards development.

Load and Resistance Factor Design (LRFD), as an implementation of the method of limit states design, prescribes the application of partial (safety) factors to ensure that the probability of exceeding a type of failure (i.e., a limit state) is maintained at an acceptably small value. The limit states are defined based on the various conditions that may occur during the lifetime of the OWTG structure and which are associated with collapse or other modes of failure, including those corresponding to serviceability, accidents or fatigue.

OWTG structures are designed based on the loads primarily associated with wind and waves and including the effects of aerodynamic and structural damping. The limit states design method identifies the environmental and operating conditions and the corresponding load cases, including those loads that may act simultaneously and with consideration of directionality. For the fatigue limit state, the load cases are defined such that all anticipated contributions to fatigue damage are included in the design for the life of the structure.

The determination of the partial factors for design are based on probabilistic evaluations and design studies with consideration for the requirements of the structure to function throughout the design service life most appropriate for the class of structure and the prescribed safety index, described subsequently.

2.6.1 Wind Turbine Class Definitions

In general, OWTGs are classified according to the wind speed and turbulence intensity at a site. In accordance with IEC 61400-3, an OWTG is to be designed either to a normal safety class; or, to a special safety class[1]. The former applies to designs for which a failure would result in risk of personal injury or other social or economic consequence. The latter refers to designs for which the safety requirements are determined by local regulations and/or the safety requirements are agreed between the manufacturer and the customer. However, it is typical for OWTG designs to proceed on the basis of a normal safety class as they are unmanned structures operating far from populated areas and where the consequence of failure is limited, more or less, to the economics. (Note the design service life for the OWTG is taken typically to be equal to 20 years for the normal safety class.)

With the safety class defined, the standards and guidelines provide basic parameters, including the reference wind speed and the annual average wind speed of many years at hub height. Parameters used subsequently in the calculation of the turbulence models are also provided. The assignment of wind turbine safety class influences the load factors associated with the limit states not related to fatigue; however, the resistance factors are unchanged.

2.6.2 Safety Index/Target Reliability

The target safety level defines the nominal acceptable probability of structural failure. By DNV-OS-J101, the design of support structures and foundations for OWTGs with a normal safety class

[1]By comparison DNV-OS-J101 defines three types of safety classes: low, normal and high. OWTGs are normally unmanned and are designed typically for the normal safety class, as above. However, design to an elevated safety class may be motivated by other concerns regarding economics or safety. Consequently, DNV suggests the OWTG assets are best protected by designing the support structure to the high safety class.

assumes a nominal annual probability of failure of 10^{-4} (or, a probability of failure of 1-in-10,000). The target safety level of 10^{-4} represents DNV's interpretation of the safety level inherent in the normal safety class for wind turbines defined in accordance with IEC61400-1. For OWTG structures on which personnel may be present during storms, for example, it may be warranted to designate the asset as a high safety class having a corresponding nominal annual probability of failure of 10^{-5}.

2.6.3 Partial Factors

The partial safety factors (or simply, partial factors) applied in design are determined based on consideration of the separate assessment of the load effects in the structure due to the applied loads for the limit state under evaluation. This is required since both the loads acting on a structure and the resistance of the structure or component can be described statistically. In limit states design, load factors are applied to the specified loads to account for this statistical distribution and to reflect uncertainties in the approximation of load effects. Similarly, factors are applied to the structural resistances or calculated fatigue lives to reflect variability in the material properties, fabrication tolerances, workmanship, accessibility for inspection and other uncertainties related to the prediction of resistances.

The standards and guidelines for the design and certification of OWTGs prescribe the load factors and resistance factors to be applied for each limit state. Resistance factors may take the form of material resistance factors or, as given in DNV-OS-J101 for example, design fatigue factors (DFF) applied to the characteristic cumulative fatigue damage.

For the design of OWTGs, consideration of the combined load effects due to wind and waves (specifically accounting for the effects of aerodynamic damping), may require design by direct simulation. Guidance is provided in order that the intent of the partial factor method is retained.

2.6.3.1 Partial Factors for Loads

As applied to the fatigue limit state, the partial factors for the loads are taken as equal to 1.0 for all normal and operational design situations. At the fatigue limit state, specified (i.e., unfactored) loads must be used to obtain the stress ranges directly. Subsequently, the calculated fatigue life or the fatigue stress ranges may be adjusted using partial factors for resistance to ensure the design service life is achieved.

Instead of applying factors greater than 1.0 to the loads, conservatism is introduced into the calculation of fatigue life by, for example, S-N curves which represent a lower bound on life (corresponding to the mean less two standard deviations, typically). Due to the non-linear impact on fatigue life, since the S-N curves are logarithmic, a load factor greater than 1.0 will have a significant effect on the calculated life. For example, a factor of 2.0 applied to the calculated fatigue stress range would reduce the fatigue life by a factor of approximately 10.

2.6.3.2 Partial Factors for Fatigue Life Calculation

The various design standards and guidelines describe the approaches to establishing material and component resistances, summarized briefly in Table 2.4 along with the DFFs where applicable.

Table 2.4: Overview of Partial Factors for Fatigue Life Calculation

Standard/Guideline	Description
IEC 61400-3	IEC 61400-3 defines the fatigue design load cases and associated load factors to be applied but defers to the ISO standards for the fatigue design of the support structure, including application of the resistance factors. Alternatively, the design resistance of the tower may be determined according to IEC 61400-1.
DNV-OS-J101	Two methods are defined for the calculation of the design cumulative damage, in which a resistance factor is applied, as follows. • By Method (1), the characteristic cumulative damage is calculated using a Miner-Palmgren summation and subsequently scaled by a design fatigue factor (DFF). The DFF is related to the accessibility of the component under evaluation and to the S-N curve corresponding to exposure to seawater, with or without cathodic protection. Values for the DFF range for 1.0 for components accessible to inspection and exposed only to air; to 3.0 for inaccessible components in seawater. • By Method (2), the design stress range (taken from the long term distribution of stress ranges) is scaled by a material resistance factor. The design cumulative damage is calculated directly using a Miner-Palmgren summation. Values for the material resistance factor are related to the DFF values from Method (1) and vary between 1.0 and 1.25.
GL "Guideline for the Certification of Offshore Wind Turbines" (2005)	The partial material factor to be used as a basis for metallic components of all load case groups is taken equal to 1.10. However, the partial material safety factors on stress ranges for the fatigue assessment are defined with respect to the component being part of a non-fail-safe structure, where local failure of a component leads rapidly to a catastrophic failure of the structure; or, part of a fail-safe structure with reduced consequences of failure (i.e. local failure of a component does not result in a catastrophic failure of the structure). Further consideration is given to the inspection and accessibility of the component. For non-fail-safe structures, to which the tower support structures belong, the minimum partial safety factor for materials is taken equal to 1.15; the maximum (for non-monitored, non-accessible components) is taken equal to 1.25.
ABS "Guide for Building and Classing Offshore Wind Turbine Installations" (2010)	The calculated fatigue life of structural members and joints shall exceed the design life multiplied by the safety factors for fatigue life; or, fatigue design factors (or, effectively design fatigue factors). The DFFs are prescribed with consideration of the importance of the component, with respect to the consequence of failure of that component, being either "Critical" or "Non-critical". A further distinction is made regarding the accessibility of the component for inspection and maintenance. Values for the DFF ranges from 1.0 for non-critical and accessible components; to 5.0 for critical and inaccessible components.
API 2A-WSD	The design fatigue life of each joint and member should not be less than the intended service life of the structure multiplied by a fatigue safety factor. For the design fatigue life, the cumulative fatigue damage ratio, obtained from a Miner-Palmgren summation, should not exceed 1.0. The fatigue safety factor is defined based on the criticality of the component (i.e. the consequence of failure) and the accessibility for inspection. Values range from 2.0 for non-critical and accessible components; to 10 for critical and inaccessible components.

2.7 Fatigue Design and Assessment Methodology

As introduced in Section 1.2, the fatigue assessment of a structure or component is used to establish the fatigue demand (i.e., stress ranges), which is then compared to the fatigue strength. Calculating the load effects resulting from the application of the design load cases, arising from the environmental conditions, to produce a stress history at each location represents a significant effort for the complete fatigue assessment.

With respect to the fatigue limit state, the assessment is concerned with the damage accumulated from the repeating and varying load history. The loads acting on the OWTG are produced from the environmental and operating conditions, which are stochastic, time-varying and non-linear. The resulting stress range histories at critical sections through the OWTG structure are evaluated in terms of the incremental damage imparted. For the required service life, the structure is designed such that the total damage accumulated by each cycle of incremental stress range at every critical section does not exceed the requirements for the overall damage accumulation.

The various standards and guidelines apply the following methodology, in general, for the fatigue design and assessment of OWTG tower structures:

1. Define the long-term statistical distribution of the cyclic loads:
    a. Using the established stochastic modeling techniques, the environmental conditions associated with the wind, marine and aerodynamic conditions are characterized with consideration of the effects of the control system operating within the environment.
2. Determine the corresponding long-term statistical distribution of stress history at each detail to be considered:
    a. The load effects are estimated from the results of structural analyses describing the global and local behaviors. Whether the structural analysis has been completed in the time domain or in the frequency domain, the stress history is developed as a stress range histogram using techniques to filter the stress ranges and count the number of cycles.
    b. Typically, the rainflow counting method is used to determine the distribution of stress ranges defining the response; however there are various methods that can be used to account for full and partial cycles in the evaluation. Thus, the calculated stress history provides the number of cycles experienced for each stress range associated with the environmental conditions, as given by the design load cases.
    c. Subsequently, the distributions of stress ranges are modified to account for the thickness (size) effect; mean stress effect; or the effect of surface treatments, as applicable.
    d. Depending on the classification of each fatigue detail to be evaluated, stress concentration factors may be applied in accordance with (i) the nominal stress approach; or, (ii) the hot-spot stress approach. (A notch-stress approach is also available however the detail required for its application is such that it is not used typically for similar fatigue assessments). In general, the two

approaches are equivalent in that a reference stress range is modified using a factor to account for the increased stresses occurring at weld toes and other potential crack initiation points. However they are distinguished with respect to the calculation of the reference stress range from the results of the modeling and simulation and in the determination of the stress concentration factors.

e. Another simplifying method, as described in GL "Guideline for the Certification of Offshore Wind Turbines", can be applied in the fatigue assessment by defining the permissible stress range based on the standard distributions of the long term stress ranges. This approach assumes a two-parameter Weibull cumulative distribution to represent statistically the distribution of stress ranges. Empirical factors are then applied, based on a limited scope of details for welded joints and plate edges and based on a conservative design S-N curve.

3. Calculate the incremental fatigue damage associated with the stress ranges:

   a. Once the stress ranges have been calculated and adjusted for material or local effects, the fatigue life of each component or detail can be estimated using either a damage accumulation or crack growth method. Subsequently, the design can be detailed to incorporate fabrication quality and inspection standards such that fatigue-related issues may be minimized.

   b. Depending on the classification of each fatigue detail to be evaluated, stress concentration factors may be applied in accordance with (i) the nominal stress approach; or, (ii) the hot-spot stress approach. (A notch-stress approach is also available however the detail required for its application is such that it is not used typically for similar fatigue assessments). In general, the two approaches are equivalent in that a reference stress range is modified using a factor to account for the increased stresses occurring at weld toes and other potential crack initiation points. However they are distinguished with respect to the calculation of the reference stress range from the results of the modeling and simulation and in the determination of the stress concentration factors.

4. Compare the estimated fatigue life to the design service life:

   a. Typically, the fatigue design for OWTG tower structures is consistent with a safe-life approach, based on a minimum design life of 20 years.

   b. Design fatigue factors (DFFs) may be applied to increase the required design life or to decrease the permissible damage. DFF values are dependent on the location of the structural detail, of the accessibility for inspection and repair, and of the type of corrosion protection. Effectively, the design fatigue factor modifies the calculated fatigue life for each individual structural detail to account for uncertainties in the fatigue assessment and to further account for the consequences of failure.

Another simplifying method, as described in GL "Guideline for the Certification of Offshore Wind Turbines", can be applied in the fatigue assessment by defining the permissible stress range based on the standard distributions of the long term stress ranges. This approach assumes a two-parameter Weibull cumulative distribution to represent statistically the distribution of stress ranges. Empirical factors are then applied, based on a limited scope of details for welded joints and plate edges and based on a conservative design S-N curve.

The general methodology for the fatigue design and assessment of OWTG tower structures is illustrated schematically in Figure 2.1.

2.8 Complexity of Fatigue Design and Assessment Methodologies

An overview of the fatigue design and assessment methodologies applied to OWTG tower structures is illustrated in Figure 2.1. The levels of complexity that may be assigned to each part of the process have been introduced in Section 1.4, and are repeated below. Table 2.5 identifies the general level of complexity associated with each major activity in the methodology.

- Level 1: Rule of thumb guidelines or non-analytical techniques for fatigue life enhancement;
- Level 2: Analytical closed-form solutions based on regression or semi-empirical methods; or,
- Level 3: Numerical or spectral methods (computer simulation and analysis).

As shown, and as described in the following sections, the fatigue design and assessment is completed mainly using analytical closed-form solutions and empirical or semi-empirical methods to ensure the design intent is maintained. The highest level of complexity is associated with developing numerical solutions to describe the integrated response of the OWTG tower structure within the operating environment. Determination of the stress range histories is achieved using advanced numerical techniques developed in the time or frequency domains. In comparison, the design S-N curves are based on extensive test data and have several associated approximations, addressed in part by the use of the nominal or hot-spot stress methods. Methods to improve the fatigue life of the structural components are, to some extent, based on the observed effectiveness of shot peening or grinding.

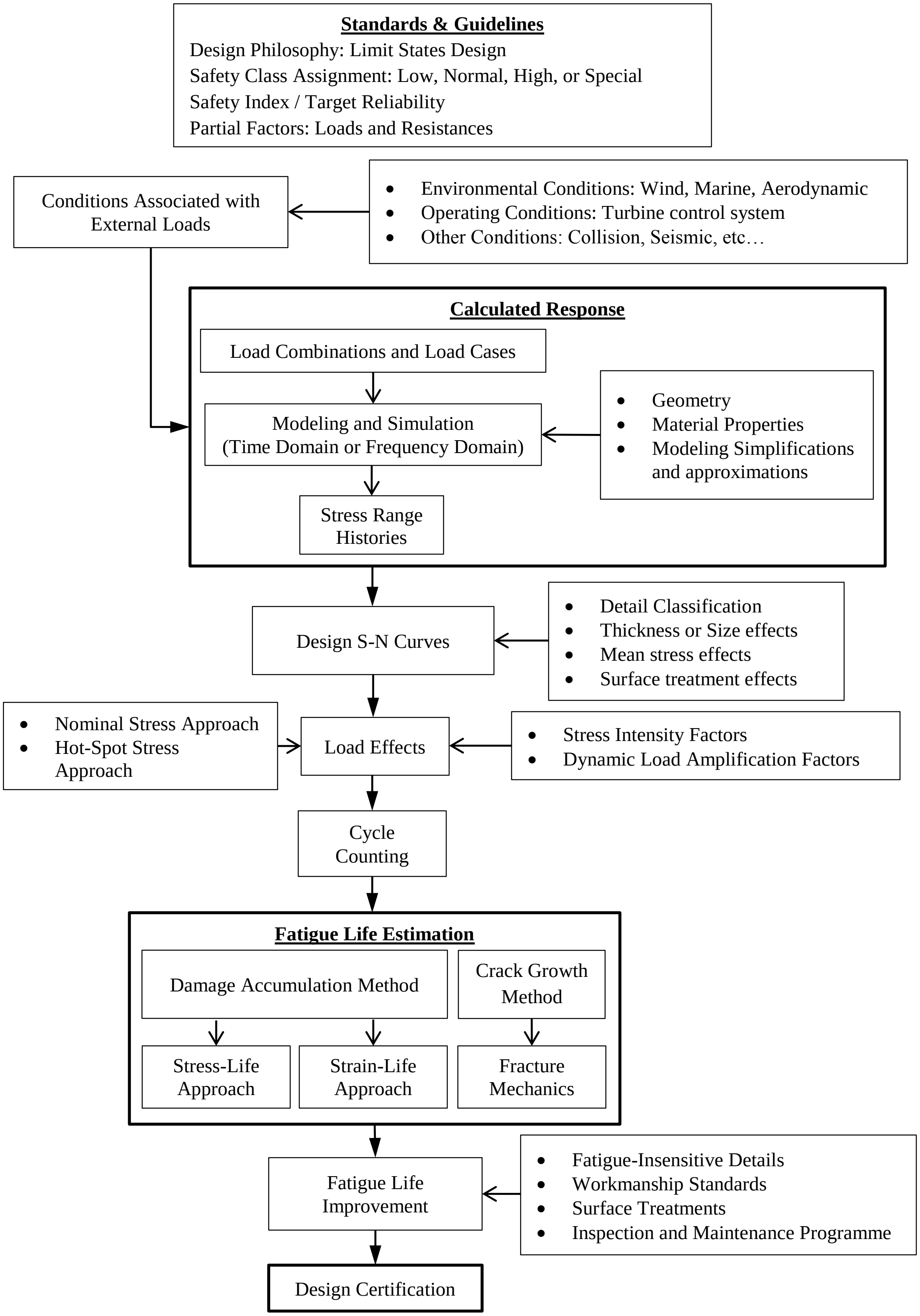


Figure 2.1: Fatigue Design and Assessment Methodology

Table 2.5: Levels of Complexity Associated with Fatigue Design and Assessment

Process	Activity	Overview	Level of Complexity
Conditions Associated with External Loads	Load Characterization	Although simplified or deterministic methods are available, accurate analytical representation of the external loads (specifically, aerodynamic effects) requires detailed analysis.	3
Calculated Response	Load Combinations and Load Cases	Definitions for the fatigue load cases are based on design situations for location and anticipated behavior. Determinations are made based on experience and analytical methods.	2
	Modeling and Simulation	Requires detailed numerical or spectral methods (computer simulation and analysis).	3
	Stress Range Histories	The results of integrated structural analyses in the time or frequency domains are used to develop the stress range histories.	3
Design S-N Curves	Detail Classification	Classification of structural details for fatigue design is based primarily on empirical testing and formulae.	2
	Material effects	Adjustments or corrections are made for various effects associated with the material behavior. The parameters related to the adjustments may be determined empirically.	2
Load Effects	Nominal Stress or Hot-Spot Stress Approach	These analytical approaches use conservative idealisations of load and structural response to maintain the design intent.	2
	Cycle Counting	Cycle counting may be completed using various practical approaches, including the Rainflow method, for example.	1
Fatigue Life Estimation	Damage Accumulation Methods	A linear Miner-Palmgren summation of the accumulated fatigue damage (whether using a stress-life or strain-life approach) is an approximate method, verified by testing.	1
	Crack Growth Method	The fatigue life estimated using fracture mechanics is related directly to the assumed initial crack size. As such, the accuracy of the life estimation is entirely dependent on the accuracy of the initial assumption.	2
	Design Fatigue Factors	The values of design fatigue factors (DFFs) are based on achieving the desired safety index for the class of structure. The determination of the DFFs should be made based on a probabilistic approach and limit states design.	2
	Fatigue Life Improvement	Addresses fabrication quality and inspection standards.	1

2.9 Treatment of Uncertainty

As noted, standards and guidelines based on a limit states design approach mitigate uncertainty and assure design reliability during the design process through the application of partial factors (for loads and resistances) and by defining a target safety index for the intended wind turbine class. The loads and resistances are evaluated using probabilistic methods, including the statistical distribution of the variability associated with the environmental loads, load combinations and material properties or fabrication tolerances. Some aspects of the fatigue life estimation apply approximations or empirical results and so require design fatigue factors (DFF) to ensure the fatigue life estimations remain conservative. Other potential sources of uncertainty in the design include the following:

- More than for most conventional structures, the live loads acting on OWTGs are uncertain. Wind and wave actions impart loads on the structure that cannot be determined fully for the entire service life. At best, each environmental condition can be characterized using probability distributions, detailed analyses and testing. The effects of the combined loads, considering the influence of the aerodynamic damping effects of the OWTG, are significant and can only be defined probabilistically. The resulting stresses in the structure therefore are uncertain due to the uncertainties of the loading and the calculation of the response, including modeling assumptions. Modeling simplifications may be applied for the geometry (including misalignment) and at welded connections, which will affect the stress range near the location of critical details.
- The type of analysis to be completed for the fatigue design depends on the load case under consideration and may correspond to a static, frequency, dynamic, or stability/buckling analysis. The analysis, in some cases, may be sufficiently accurate based on linear-elastic behavior. Some analyses will require that the effects associated with plasticity, geometric non-linearities and material non-linearities are considered. In general, conservatism is increased as with increasing analytical simplifications to ensure that the loading uncertainties are considered adequately. Due to the increasing structural complexity of some designs (floating OWTGs, in particular) and the increasing capabilities of computing and available design software, industry practice is moving towards fully-integrated analytical design in the time domain.
- When calculating the incremental fatigue damage, the design S-N curves are used to estimate the number of cycles associated with each stress range (the latter being estimates). The design S-N curves correspond to a log-log relationship such that small changes in the stress range may result in large changes in the estimated fatigue life. Therefore, the application of the nominal stress or hot-spot stress methods should be carefully considered.
- It is noted that the design S-N curves have been developed based on empirical data. Uncertainties not apparent to the designer may be contained in the data and application of the curves to a particular detail must be made appropriately. Further, there are analytical approximations associated with the development of stress concentration factors.

- Ultimately, the fabrication of the OWTG tower structure and components requires the development and application of appropriate quality assurance and quality control programs to ensure the design is implemented correctly and accurately. Fabrication tolerances and techniques need to be monitored closely to avoid creating stress risers or other fatigue related issues that may not be accounted for in the design.

3 CONDITIONS ASSOCIATED WITH EXTERNAL LOADS

3.1 Load Characterization

The standards and guidelines define methods by which the loads are characterized based on the effects associated with the environment and operating conditions, as listed in Table 3.1 and described subsequently.

Table 3.1: External Conditions and Corresponding Characteristic Loads

External Condition	Load Effect	Characteristic Load
Permanent	Self-weight and eccentricities	Gravitational and Inertial Loads
	Hydrostatic and buoyancy loads	
Environmental	Distribution of mean wind speed	Wind Loading
	Normal wind profile model	
	Normal wind turbulence model with wake effects	
	Extreme wind profile and turbulence	
	Damping	Aerodynamic Loading
	Effect of air density	
	Wave modeling, including diffraction	Wave Loading
	Breaking waves and wave slamming	
	Sea current effects	Other Marine Loading
	Sea level (hydrostatics)	
	Sea ice	
	Ancillary conditions not related to fatigue (scour, salinity, temperature of air and seawater, marine growth)	
Operational	Dynamic behavior, including resonance and foundation stiffness considerations.	Dynamic Loads
	Actuation loads, including torque control from a generator or inverter, yaw and pitch actuator loads and mechanical braking loads.	
	Gyroscopic effects due to yawing, or other effects, as transmitted to the support structure.	
Other	Collisions with ships or service vessels	Ship Impact
	Earthquake loads, as appropriate for the founding system.	Seismicity
	Loads on access platforms and internal structures such as ladders and platforms	Personnel Loads
	Temporary loads may arise during fabrication and installation, for example from lifting, transporting or launching.	Installation and Construction

3.2 Permanent Conditions

Loads arising from the permanent conditions correspond to those associated with the mass of the structure, components and attachments. The hydrostatic and buoyancy forces affecting members below the waterline also are included.

3.3 Environmental Conditions

The standards and guidelines present different methods of characterizing the environmental conditions from which the loads are derived for subsequent application in determining the structural response and corresponding to wind, marine and aerodynamic effects. Typically, the fatigue loads are based on the effects of "normal" environmental conditions, corresponding to those having a probability of being exceeded at least once a year. By comparison, the extreme conditions used for design to the ultimate limit states are those related to events having a probability of being exceeded once in 50 years or once in 100 years, depending on the standard or guideline referenced.

Since the design of an OWTG must achieve a structural efficiency, and since the characterization of the resulting load effects is complex and sophisticated, the design must consider the specific conditions at each site. In some cases, the OWTGs within a site at a wind farm may be subject to a range of metocean (i.e., meteorological and oceanographic) conditions and so will require different designs.

In this section, more detail is provided to describe the complete loading environment used to develop the specified loads, as applicable to the fatigue limit state.

3.3.1 Wind

An OWTG tower is required to withstand safely the full range of wind conditions at the local site. The load cases identified for the fatigue strength evaluation include the effect of distribution of wind speed over the rotor swept area to account for the deterministic behavior (including the vertical wind-speed gradient and tower shadow) and the stochastic influences of the wind climate (such as partial gusts and turbulence). The parameters, conditions and modeling associated with defining the wind environment are summarized in Table 3.2.

For the fatigue load cases, the wind climate is defined by the normal wind condition and the normal turbulence model (as differentiated from the extreme wind conditions). The normal wind conditions are specified in accordance with IEC 61400-1 in terms of an air density; a long term distribution of the 10-minute mean wind speed; a wind shear gradient of the mean wind speed with respect to height above the sea surface; and, turbulence. However, the normal wind conditions (and certainly the extreme wind conditions) may not adequately represent areas subject to hurricanes, as for the United States Outer Continental Shelf (US OCS). In regions susceptible to hurricanes, the effects of sustained wind speeds and sea level changes require special consideration for the fatigue evaluation.

A detailed description of the wind conditions, data, modeling and theory is provided in other references; specifically, the DNV recommended practice DNV-RP-C205 "Environmental Conditions and Environmental Loads".

Table 3.2: Wind Conditions and Modeling

Condition	Description
Basic Wind Parameters	• Reference wind speed, V_{ref} • Annual average wind speed, V_{ave} • Wind speed distribution, described in Section 3.3.1.1 • Wind direction distribution (i.e. wind rose) • Turbulence intensity, I_{15} at V_{hub} = 15 m/s, where I_{15} is the characteristic value of the turbulence intensity at hub height for a 10-minute mean wind speed of 15 m/s. • Wind shear.
Normal Wind Conditions	The wind speed distribution at the site (described by Weibull or Rayleigh distributions) determines the frequency of occurrence of the individual load components. The wind profile V(z), described by the power law (refer to Section 0), describes the average wind speed as a function of the height above the still waterline. The assumed wind profile is used to define the average vertical wind shear across the rotor swept area.
Normal Turbulence Model (NTM)	The values of the turbulence intensity at the hub height may be extrapolated for other heights based on the assumption that the standard deviation of the wind speed remains constant as the wind speed varies with height. In general, 3D turbulence models are employed to account for both the longitudinal transversal and lateral components of wind speed. Refer to Section 3.3.1.3.
Wind Farm Influence	For OWTGs installed in wind farms, the effects of wind field perturbations in the wake must be considered in characterizing the loads. The influence is described in terms of the shadow effect and the superimposed wake interaction, and including the effects of terrain roughness or environmental turbulence, as appropriate.

3.3.1.1 Mean Wind Speed Distribution

The normal wind conditions are represented by the long term distribution of the wind speed, considered as a stationary process over a 10-minute period, and described by a probability distribution having a mean value equal to the 10-minute mean wind speed and a standard deviation of the 10-minute mean wind speed. In this respect, the 10-minute mean wind speed is a measure of the intensity of the wind climate and the standard deviation is a measure of the variability of the wind speed about the mean.

Characterization of the long term distribution of the 10-minute mean wind speed for a site determines the frequency of occurrence of load components on the OWTG. Depending on the site measurements, the distribution is described either using a Weibull distribution verified with long term site measurements; or, using a Rayleigh distribution, which is applicable to the standard wind turbine classes.

3.3.1.2 Normal Wind Profile Model (NWP)

The normal wind profile model represents the variation with elevation of the wind speed (or, wind shear across the rotor swept area) and is given by the power law, as follows.

$$V(z) = V_{hub}\left(\frac{z}{z_{hub}}\right)^{\alpha} \qquad \text{Eqn. 3.1}$$

For the normal wind profile model, *V(z)*, denotes the average wind speed as a function of height, *z*, above the still waterline up to the hub height, z_{hub}. In the case of the standard wind turbine classes, the normal wind speed profile is given by Equation 3.1 with the power law exponent, α, taken equal to 0.14 in accordance with IEC 61400-3.

It is noted that the design loads are based typically on an assumed air density equal to 1.225 kg/m^3. Significant variations in air density such as may occur in arctic areas during the winter season or due to hurricanes and tropical storms, require consideration with respect to characterizing the wind loads. Further, the operating conditions of the OWTG will be affected since drag on the wind turbine will increase with air density. Decreasing air density will decrease the lift on the blades and thus reduce power production.

3.3.1.3 Normal Wind Turbulence Model (NTM) including Wake Effects

As noted, the turbulence is characterized by the standard deviation about the 10-minute mean wind speed distribution. The long term distribution of the turbulence, capturing the natural variability from successive 10-minute periods, can itself be characterized by a probability distribution.

The normal turbulence model represents the turbulent wind speed in terms of the 90% quantile of the probability distribution of the standard deviation of the longitudinal mean speed conditioned on the 10-minute mean wind speed at the hub height. For wind turbines within large wind farms, the influence of other OWTGs on the wind speed at an individual OWTG is considered using a characteristic standard deviation of the ambient wind speed. Other site-specific influences leading to increased turbulence also are to be taken into account, including the shore topology, if appropriate.

3.3.1.4 Hurricane Winds

The various ABS guides are recognized as having been developed with a focus on Gulf of Mexico installations and the ABS Guide for "Building and Classing of Offshore Wind Turbine Installations" (2010) provides additional guidance regarding the wind profile modeling for hurricane conditions in the Gulf of Mexico. With reference to the recommendations of API Bulletin 2INT-MET, the wind speed profile is expressed using a logarithmic wind shear law, instead of a power law as described in Section 0. However, other wind profile models may be used as appropriate, based on local site measurement data. Other standards and guidelines, while having been developed for European installations, have identified wind models applicable to hurricane conditions.

With application of the appropriate hurricane wind profile model, the component of fatigue damage associated with this relatively infrequent environmental loading can be calculated based on the corresponding stress ranges. Further, dynamic analyses are recommended to confirm resonance is avoided at the hurricane load level.

3.3.2 Aerodynamics

The action of the wind turbine operating within the environment creates aerodynamic load effects to be considered in the evaluation. These static and dynamic loads result from airflow around both the stationary and moving parts of the turbine and the support structure. The interaction of the moving turbine creates aerodynamic damping as a function of the wind speed, turbine design (i.e., the aerodynamics and control systems) and the support structure. The effects of aerodynamic damping are significant in designing for the dynamic response, including the natural frequencies of the structural system.

The standards and guidelines note that aero-elastic analyses are required (in part) to provide an assessment of the aerodynamic damping provided by the turbine control system. The performance of the OWTG structure in response to the wind and waves (specifically, with respect to the misalignment of those loads) will be affected by the characteristics of the turbine control systems. For example, the yawing strategies and cut-in/cut-out wind speed profile will determine the associated loads. The results of aero-elastic analyses can inform the design of the control system and provide the aerodynamic damping profile of the support structure.

3.3.3 Waves

The wave climate dominates for fatigue design of the tower structure and the complex characterization must be completed based on site-specific data; i.e., measurements or hindcast studies. Waves are random in nature, having irregular shapes and variable heights and lengths, moving with non-uniform directions and speeds. For the fatigue analysis, the normal stochastic fluctuating wave series are used as the basis for characterizing the marine conditions. Further, the wave climate is affected by other factors including the fetch and water depth and is influenced by the wind climate. Table 3.3 summarizes the conditions and modeling associated with defining the marine environment.

In order that the wave loads are identified adequately for design, the wave climate must be defined. However, the wave climate is a naturally occurring and effectively random process. Analytical representations of the wave climate seek to simplify the complex behavior such that a time-series of loads can be applied to calculate the structural response.

The approach applied to achieve the wave loads follows as:

1. Apply a time-series approximation of the wave surface using a wave theory, such as:
    a. Second-order;
    b. Airy; or,
    c. Stream function.

2. Characterize the wave kinematics by extending the wave theory to provide predictions of fluid velocity and acceleration at points above the mean waterline (i.e. extrapolation to the free surface).
    a. Extrapolated Airy model (or, vertical stretching);
    b. Non-linear Stokes expansion (5^{th} order, typically); or,
    c. Wheeler stretching.
3. Use the Morrison equation to estimate the corresponding wave loads from the given wave theory and kinematics.
4. Calculate the structural response, including fatigue damage estimations.

Note that the choice of wave model must consider the local environmental characteristics (based on long-term site measurements) with respect to the wave height. The mean water depth (being deep, finite or shallow) may influence also the choice of wave model.

As part of characterizing the wave climate for the design of an OWTG, the standards and guidelines state that the action of breaking waves should be considered, which can include spilling, plunging or surging. However, the influence of breaking waves with respect to a fatigue evaluation and dynamic amplification is to be determined as appropriate for the environment and the structure.

3.3.3.1 Wave Modeling (NSS and NWH)

For the fatigue limit state, the wave model should correspond to the normal marine conditions occurring more frequently than once per year during operation. The wave models relevant to a fatigue evaluation include the normal sea state (NSS) and the normal wave height (NWH). Other factors associated with the marine climate and influencing the fatigue life estimation include sea current, sea ice, marine growth and scour (for founded structures). Subsequently, the wave condition is adjusted using wave diffraction approximations to account for the size of the structure and the interacting wavelength.

The wave climate in a region is described by a sea state (or set of sea states), which is characterized by a stochastic wave model. A stochastic representation considers the wave climate as a superposition of many periodic waves having individual characteristics, but with random phase relationships to each other. The sea state thus can be described based on the selection of a wave spectrum along with the significant wave height, H_s; a peak spectral period; and a mean wave direction. The wave conditions to be used for design are based on the long term distribution, determined from site data or hindcast studies. The correlation of these metocean parameters is to be considered in terms of their long term joint probability distribution.

A standard, irregular wave spectrum is selected (likely the JONSWAP-spectrum or Pierson-Moskowitz) to relate the wind data to the wave data and to describe the wave height and wave direction. The spectral model effectively provides a mathematical representation of the distribution of wave energy as a function of the spectral parameters, including the significant wave height and the mean wave period.

Table 3.3: Marine Conditions and Modeling

Condition	Description
Basic Wave Parameters	• The sea state is assumed to be a Gaussian process and can be described by the superposition of an infinite number of harmonic waves with a different height, period and direction and random phase. • The basic quantity characterizing the severity of the sea state is the significant wave height, H_s.
Normal Sea State (NSS)	The normal sea state is described by short-term wave statistics, long-term wave statistics, breaking waves and swell. The significant wave height, peak spectral period and direction for each normal sea state is selected, based on the long term joint probability distribution of metocean parameters for the site. • Wave breaking and plunging breakers may induce very high impact forces on a slender structure. A detailed analysis of the forces from breaking waves or plunging breakers may be required for slender structures due to the associated high impact forces, despite the very short duration. • Swells may result when waves from a wind driven sea travel far from where the waves were generated. Thus, swells are not associated directly with the local wind conditions. In areas where swell may be expected, special consideration may be required.
Equivalent Design Wave	As an alternative to defining the natural sea state, a quasi-static analysis may be completed based on an equivalent design wave. The long-term statistics of wave heights and periods, including the relative frequencies of occurrence, can be used to develop a marginal probability distribution function (e.g., Weibull) for F(H_s) derived from a least square fit of the data.
Current	The types of current relevant to the design of OWTGs include sub-surface currents and wind generated currents. Note that current may only be a significant design consideration for fixed structures in shallow waters. Flow modeling or site measurements may be used with statistical data to estimate the currents acting on the structure. • Sub-surface currents result from tidal motions and the influence of the site topography. Storm surges and barometric pressure differentials may produce sub-surface currents. • Wind generated currents are caused by the wind stress and atmospheric pressure gradient throughout a storm. For design, the maximum loads may occur when the direction of the sub-surface current is aligned with the wave direction.
Sea Level	The highest still water level for design corresponds to the highest sea level with a recurrence period of 50 years considering tidal and storm surge effects as well as seasonal variations.
Sea Ice	If appropriate, sea ice occurring within the operating area must be taken into consideration. More detail is provided in Section 5 of this report.
Marine Growth	For OWTGs located in some areas, the wave and current loads acting on the submerged structure will be affected by marine growth in terms of the increased weight and changing profile. If data is unavailable, a thickness of 50 mm may be assumed based on normal climatic conditions.

3.3.3.2 Wave Modeling (NSS and NWH)

For the fatigue limit state, the wave model should correspond to the normal marine conditions occurring more frequently than once per year during operation. The wave models relevant to a fatigue evaluation include the normal sea state (NSS) and the normal wave height (NWH). Other factors associated with the marine climate and influencing the fatigue life estimation include sea current, sea ice, marine growth and scour (for founded structures). Subsequently, the wave condition is adjusted using wave diffraction approximations to account for the size of the structure and the interacting wavelength.

The wave climate in a region is described by a sea state (or set of sea states), which is characterized by a stochastic wave model. A stochastic representation considers the wave climate as a superposition of many periodic waves having individual characteristics, but with random phase relationships to each other. The sea state thus can be described based on the selection of a wave spectrum along with the significant wave height, H_s; a peak spectral period; and a mean wave direction. The wave conditions to be used for design are based on the long term distribution, determined from site data or hindcast studies. The correlation of these metocean parameters is to be considered in terms of their long term joint probability distribution.

A standard, irregular wave spectrum is selected (likely the JONSWAP-spectrum or Pierson-Moskowitz) to relate the wind data to the wave data and to describe the wave height and wave direction. The spectral model effectively provides a mathematical representation of the distribution of wave energy as a function of the spectral parameters, including the significant wave height and the mean wave period.

3.3.3.3 Breaking Waves and Wave Slamming

In elevated sea states (corresponding to the extreme and severe wave climates), breaking waves will occur at the tower structure and support system. These breaking waves are characterized as spilling, plunging or surging. Breaking waves can potentially cause significant dynamic amplification of the structural response for OWTG on monopile foundations (Rogers 1998).

However, breaking waves are dynamic phenomena resulting in transient impact loads. The corresponding structural response is characterized by frequencies of vibration that are considerably higher than those associated with wave loading in the normal sea state. It can be assumed that the number of breaking waves acting on the tower structure over the service life will be negligible with respect to fatigue (Glen, et al. 1999). Therefore, the load cases identified as relevant to the fatigue evaluation and with respect to wave loading only consider the actions associated with the normal sea state and wave height.

3.3.4 <u>Other Marine Conditions</u>

3.3.4.1 Sea Current Effects

In general, hydrodynamic fatigue loads are not influenced by sea currents. (However, loads resulting from sea current effects are considered for other load cases and those associated with the ultimate limit states). Exceptions might include vortex shedding or fatigue loads associated with ice interactions, depending on the specific site conditions.

If required, a normal current model can be applied, which considers the combination of wind generated currents and breaking wave surf currents. The tide and storm-generated sub-surface currents are not considered in the normal current model. The return period chosen for the extreme current conditions is generally not to be less than 100 years.

When sea current effects are included, the current speed can be added to the wave particle speed from the applied wave theory. The Morrison equation used to determine the wave loads subsequently can consider the current speed in the application of the drag coefficients.

3.3.4.2 Sea Level (Hydrostatics)

If there is a significant fluctuation in the mean water level at the site, then the effect on the hydrodynamic loading should be considered. With respect to fatigue loading, the normal water level range (over the 1-year recurrence period) is relevant and can be based on the variation between the highest astronomical tide (HAT) and the lowest astronomical tide (LAT).

For the calculation of the hydrodynamic fatigue loads, the influence of water level variation may be demonstrated to be negligible through analysis or can be accounted for conservatively by assuming a constant water level greater than or equal to the mean sea level.

In general, analyses of the structural stability should demonstrate the adequacy with respect to hydrostatic collapse at the given water depth. ABS recommends checking the performance in accordance with either the “Guide for Buckling and Ultimate Strength Assessment for Offshore Structures” or the API RP-2A.

3.3.4.3 Sea Ice

The influence of ice may be associated with mass imbalances when ice covers the rotating blades. However, with respect to sea ice, the focus of this report is on the interaction between moving ice and the tower or support system.

In such areas or where moving ice floes may be a recurring issue, the action of ice interacting with the tower structure or support system at the waterline (eccentrically or otherwise) requires special consideration. Note that it is generally assumed that sea ice actions are not combined with any wave actions; but are combined with wind and current.

The ABS “Guide for Offshore Wind Installations” requires local statistical ice data to be used to determine the ice thickness, ice crushing strength and pack ice concentration, which are required for determining the ice loads. Ice loads acting on the OWTG support structure can be determined in accordance with Annex E of IEC 61400-3 (2009) and with reference to API RP-2N and ISO 19906. A more detailed description of the methodologies applied in the standards and guidelines is presented in Section 5 of this report.

3.3.4.4 Ancillary Marine Conditions

Ancillary environmental conditions that may require consideration in the design (though perhaps not directly with respect to the fatigue limit state) include salinity, which influences the requirements for cathodic protection and will therefore impact the material S-N curve definitions; and, temperature for both air and seawater.

For founded structures, scour may be an issue whereby the action of sea currents removes or displaces material at the seabed around the foundation. The implications are not considered significant for a fatigue evaluation of the tower structure, but must be examined for design of the foundations.

Marine growth on the support structure of an OWTG will affect the hydrodynamic behavior due to the influence of the added mass, changes to the geometry (with accumulation) and surface roughness. While marine growth may affect the hydrodynamic loading, the effects may also extend to accessibility of the structure or a component for inspection and maintenance and so the safety index should reflect accordingly.

3.4 Operating Conditions

The OWTG is subjected to the external loads associated with the environment; however, the operation of the OWTG will itself create loads requiring consideration. The dynamic interaction of the OWTG operating within the environment can be assessed using aero-elastic hydro-elastic analyses. Such analyses may be appropriate when issues arise relating to dynamic amplification or resonance; and, effects associated with the turbine control system.

Table 3.4 describes the operational conditions to be included in the modeling.

Table 3.4: Operational Conditions and Modeling

Condition	Description
Dynamic Loading	Severe cyclic loads and therefore fatigue damage may result from exciting the natural frequency of the structure or components. Dynamic analyses identify the natural frequencies of vibration (and the higher harmonics) for the OWTG that may occur generally between those of the local waves and the rotor operation. An evaluation of the dynamic response is required to assess the risk of resonance of global or local modes of vibration that may result from the periodic excitations associated with turbine operation. The dynamic behavior of fixed OWTGs must be such that the resonant frequencies are not excited during operation and with consideration of the foundation stiffness. The foundation stiffness is to be designed such that resonant frequencies are not excited about the 1P and 3P frequencies of the rotor rotations.
Turbine Control System	The operating scheme of the wind turbine nacelle and rotor blades can affect the development of wind loading and the aerodynamic damping profile for the structure. Actively controlled turbine systems may produce a non-linear aerodynamic profile and introduce hysteresis, which should be included in the turbine response calculations.

3.4.1.1 Dynamic Loading

Severe cyclic loads and therefore fatigue damage may result from exciting the natural frequency of the structure or components. Dynamic analyses identify the natural frequencies of vibration (and the higher harmonics) for the OWTG that may occur generally between those of the local waves and the rotor operation. By comparison, platforms in the oil and gas industry are heavier and are designed with a much stiffer response such that the natural frequencies of vibration occur above the wave frequencies (Henderson and Zaaijer 2004).

As identified in Section 2.3.1, the dynamic behavior of fixed OWTGs must be such that the resonant frequencies are not excited during operation and with consideration of the foundation stiffness. The foundation stiffness is to be designed such that resonant frequencies are not excited about the 1P and 3P frequencies of the rotor rotations (Gerven 2011). (For OWTGs, the 3P frequency corresponds to the blade passing frequency for three bladed turbines; the 1P frequency corresponds to that for simple rotational frequency). The results of aero-elastic analyses can be used to determine the performance of the OWTG rotor, structure and foundation in response to the cyclic loads. It is noted that although waves contain elements of cyclic loading, they are irregular and random in nature so a frequency range should be evaluated (Andersen 2008).

3.4.1.2 Turbine Control System

The operating scheme of the wind turbine nacelle and rotor blades can affect the development of wind loading and the aerodynamic damping profile for the structure. This integrated response will be determined by the type of turbine control system, with some responding passively and others behaving actively. For the former, the rotor nacelle assembly (RNA) effectively may be fixed such that the performance is not optimized with respect to the intensity or the direction of the wind. For the latter, the operating profile is configured to optimize the power production of the turbine while minimizing the risk of structural or other damage.

Actively controlled wind turbines may be configured to cut-in and cut-out at specified wind speeds such that the power production is regulated across the various load conditions to achieve a uniform output. These actively controlled turbines can also adjust the RNA yaw or pitch to suit the wind environment. However, actively controlled turbine systems may produce a non-linear aerodynamic profile and introduce hysteresis, which should be included in the turbine response calculations (Al-Bahadly 2011).

3.5 Other Conditions

Other scenarios are described in the standards and guidelines regarding conditions such as ship collisions or seismic considerations. However, these and other load conditions, described in Table 3.5, are not considered significant with respect to fatigue life estimation and design.

Table 3.5: Other Conditions and Modeling

Condition	Description
Ship Impact	Ship impact loads are used for the design of primary support structures and foundations and for design of some secondary structures at an accidental limit state.
Seismicity	In this report, geotechnical issues (including seismicity) and movement of the seabed are not considered with regards to the fatigue limit state.
Personnel Loads	Loads on access platforms and internal structures are used only for local design of these structures.
Installation and Construction	Some temporary loads during installation and construction, specifically during marine transport, may lead to fatigue-related load conditions or incremental fatigue damage accumulation and should be considered as appropriate.

4 CALCULATED STRUCTURAL RESPONSE

4.1 Load Combinations and Load Cases

The OWTG structure is subject to the time-varying and random loading associated with the conditions in which it operates. For calculation of the structural response, the loads to be applied in design are determined from the external conditions. Consistent with the limit states design approach, the different situations to be considered for the fatigue assessment are identified and characterized by a set of load cases and combinations. The fatigue limit state is related to those loads associated with normal operating conditions and which occur with some frequency such that incremental contributions to fatigue damage may accumulate.

The load combinations are associated with the range of design situations expected to occur throughout the service life of the OWTG. Within each design situation, the environmental conditions (dominated by the wind and waves) are defined, including the combinations and directionality. The wind and wave conditions are determined based on the parameters described in Table 3.2 and Table 3.3, respectively, and include the mean annual wind speed; the parameters of the wind speed and wave height distribution as well as their correlation; and, the operating life. The wind-wave combination may be determined from load analysis scatter diagrams corresponding to the long-term statistics and including wave height, wave period and wind speed. In cases for which the long-term statistics of wind and waves are known, but not their combination, reasonable and conservative combinations may be applied.

In general, calculation of the loads acting on the support structure can be based on the assumption of co-directionality of the wind and waves. For some applications, the directional distribution of the wind and waves (including any misalignment) may also be included in the evaluation. The design of fixed support structures in particular may require special consideration for any misalignment of wind and wave directions in the parked condition.

4.1.1 Wind Loads

The wind loads acting on the rotor blades and the tower include those loads produced by the wind directly as well as the loads that result indirectly from the aerodynamic effects (i.e. drag forces) and operation of the wind turbine (including those during normal operation; parking and idling; braking; and, start-up), with consideration also of the wind farm effects (i.e., tower shadow). As noted previously, aero-elastic analyses are required in order that the aerodynamic wind loads on the tower are determined.

Load cases for the fatigue limit state are defined such that all contributions to fatigue damage are considered. With respect to wind loading and the design of the OWTG tower structure, DNV-OS-J101 recommends that analyses are completed using time histories of the loads corresponding to the combined wind and wave environments. Alternatively, appropriate dynamic amplification factors may be applied to a constant wind speed or individual wave height. (It is noted that such analytical simplifications may result in more conservative designs and thus heavier structures such that costs are increased.)

4.1.2 Wave Loads

With the application of the appropriate wave theory to represent the wave climate and kinematics, the wave loads acting on the structure can be calculated. In general, the Morrison equation is suggested for slender structures such as jacket and monopile towers. However, there is consideration for large-volume structures and additional wave diffraction analysis, when the size of the structure disturbs the wave kinematics. For floating structures, wave radiation forces must be included.

4.1.3 Combined External Conditions

The external conditions, and the corresponding characteristic loads, are combined to develop the design load cases for the fatigue limit state. The combination of external conditions requires consideration of return periods (typically, 50-year or 1-year, depending on the event). Time series stochastic simulations and joint probability distributions are required. GL prescribes the development of scatter diagrams to identify the directional distribution of wind and waves and their misalignment.

Alternatively, separate analyses may be completed for each load (wind or wave) and then the results combined by some method of superposition. However, although this may enable simplifications as compared to an integrated analytical approach, the results may be overly conservative, which impacts the cost of the project in terms of material, transportation and erection costs (Kühn 2001).

4.1.4 Fatigue Design Load Cases

Any load or combination of loads that may occur with a reasonable probability must be considered for design. The design situations and load cases are prescribed in the standards and guidelines corresponding, in general, to the set of normal and fault design operating situations; and, the normal and extreme external conditions, as listed in Table 4.1. Further, consideration is made for design situations associated with transportation, erection and maintenance.

Load cases relevant to the fatigue analysis are contained as a subset within the overall suite of cases required for strength analysis. Those load cases described by IEC 61400-3 and included in the fatigue limit states evaluation are listed in Table 4.2. Note that sea currents are not considered in the fatigue design load cases. The load cases defined in IEC 61400-3 generally are equivalent to those defined in the other standards and guidelines referenced and the load case numbering scheme is consistent generally for all reviewed. The GL guideline defines the same load cases, but also defines some additional load cases, as listed in Table 4.3.

Loads for the fatigue limit state are those associated with normal operating conditions. For these load combinations, the Normal Sea State (NSS) conditions are assumed with the significant wave height, H_s, for each individual sea state taken as the expected value of the significant wave height conditioned on the relevant mean wind speed.

A detailed description for each load case is provided in Table 4.4.

Table 4.1: General Design Situations

Design Situation	Description
Power Production	This design situation corresponds to normal operation of the OWTG, including consideration of the Rotor Nacelle Assembly (RNA) maximum mass and aerodynamic imbalances (e.g., blade pitch and twist deviations). Yaw misalignment and control system tracking errors are also considered. This also applies to transient cases including ice-structure interaction (moving ice floes at the waterline or ice formation on the blades), and grid failure
Power Production (fault occurrence)	This design situation considers possible sources of fatigue damage resulting from faults that cause an immediate shutdown.
Start-Up	This design situation applies to loads associated with transitioning from a standstill or idling situation to power production. The number of occurrences is estimated based on the control system behaviour.
Normal Shut-Down	This design situation applies to loads associated with transitioning from power production to a standstill or idling condition. The number of occurrences is estimated based on the control system behaviour.
Parked (standing or idling)	In this design situation, the rotor of a parked wind turbine is either in a standstill or idling condition.
Parked (fault conditions)	This design situation considers any deviations from the normal behaviour of a parked wind turbine, resulting from faults on the electrical network or in the wind turbine.
Transport, assembly, maintenance and repair	Fatigue loads that may result during the transport, assembly or maintenance and repair of the turbine are considered in the evaluation.

Table 4.2: Fatigue Design Load Case Descriptions by IEC 61400-3

Design Situation	Wind Condition	Wave Condition	Wind/Wave Directionality	Ice Condition	Water Level	Load Case
Power Production	NTM $V_{in} < V_{hub} < V_{out}$	NSS Joint probability distribution of H_s,T_p,V_{hub}	Co-directional; Multi-directional	None	NWLR or ≥ MSL	1.2
	$V_{in} < V_{hub} < V_{out}$	None	None	Horizontal load from moving ice floe at relevant velocities $h = h_{50}$ in open sea $h = h_m$ for land-locked waters	NWLR	E4
Power Production (fault occurrence)	NTM $V_{in} < V_{hub} < V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	Co-directional; Uni-directional	None	NWLR or ≥ MSL	2.4
Start-Up	NWP $V_{in} < V_{hub} < V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	Co-directional; Uni-directional	None	NWLR or ≥ MSL	3.1
Normal Shut-Down	NWP $V_{in} < V_{hub} < V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	Co-directional; Uni-directional	None	NWLR or ≥ MSL	4.1
Parked (standing or idling)	NTM $V_{hub} < 0.7\ V_{ref}$	NSS Joint probability distribution of H_s,T_p,V_{hub}	Co-directional; Multi-directional	None	NWLR or ≥ MSL	6.4
	NTM $V_{hub} < 0.7\ V_{ref}$	None	None	Horizontal load from moving ice floe at relevant velocities $h = h_{50}$ in open sea $h = h_m$ for land-locked waters	NWLR	E7
Parked (fault conditions)	NTM $V_{hub} < 0.7\ V_1$	NSS Joint probability distribution of H_s,T_p,V_{hub}	Co-directional; Multi-directional	None	NWLR or ≥ MSL	7.2
Transport, assembly, maintenance and repair	NTM $V_{hub} < 0{,}7\ V_{ref}$	NSS Joint probability distribution of H_s,T_p,V_{hub}	Co-directional; Multi-directional	None	NWLR or ≥ MSL	8.3

Table 4.3: Fatigue Design Load Case Descriptions by GL Wind

Design Situation	Wind Condition	Wave Condition	Wind/Wave Directionality	Ice Condition	Water Level	Load Case
Power Production	NTM $V_{in} < V_{hub} < V_{out}$	Irregular sea state with H_s(V) or acc. to scatter diagram	Not Specified	None	Not Specified	1.2
	NWP $V_{in} < V_{hub} < V_{out}$	Regular wave $H = H_s(V)$		Ice formation on blades and structure		1.10
	NWP $V_{hub} = V_r$ or V_{out}	Regular wave $H = H_s(V)$		None		1.13
	NWP $V_{hub} = V_r$ or V_{out}	-		Sea Ice		1.14
Power Production (fault occurrence)	NTM $V_{in} < V_{hub} < V_{out}$	Irregular sea state with H_s(V) or acc. to scatter diagram		None		2.3
Start-Up	NWP $V_{in} < V_{hub} < V_{out}$	Regular wave $H = H_s(V)$		None		3.1
Normal Shut-Down	NWP $V_{in} < V_{hub} < V_{out}$	Regular wave $H = H_s(V)$		None		4.1
Parked (standing or idling)	NTM $V_{hub} < 0.7\ V_{ref}$	Irregular sea state with H_s(V)		None		6.4
Parked (fault conditions)	NTM $V_{hub} < 0.7\ V_1$	Irregular sea state with H_s(V)		None		7.2
Transport, assembly, maintenance and repair	Vortex induced transverse vibrations			None		8.3
	NTM $V_{hub} < 0.7\ V_1$	Irregular sea state with H_s(V)		None		8.4

Table 4.4: Fatigue Design Load Case Details

Load Case	Reference	Description
1.2	IEC; GL	A single value of significant wave height, H_s, may be considered for each relevant mean wind speed. The requirements for loads resulting from atmospheric turbulence are considered. GL assumes 700 generator switching operations per year, if applicable. In the case of HAWT turbines with active yaw control, operation of the yaw system during the entire service life shall be considered for certain conditions. GL also assumes 300 changes per year in the mean wind speeds from V_{in} to V_r and back to V_{in}; and, 50 changes per year in the mean wind speeds from V_r to V_{out} and back to V_r.
E4	IEC	Forces from large moving ice floes may be estimated according to the methods prescribed for vertical cylindrical shapes or for sloping shapes. Refer to Section 5.
1.10	GL	Humid weather conditions with ice formation on the blades are considered. Typically, the duration of operation with ice formation is defined by the manufacturer.
1.13	GL	The transient switching operations of the offshore wind turbine triggered by grid failure are considered, with the frequency of grid failures per year dependent on the local grid stability.
1.14	GL	Potentially critical transient events in the OWTG life are considered for this GL load case.
2.3	GL	GL includes consideration of the potential fatigue damage associated with a fault and immediate shutdown. The probable number of shut-downs and the duration of the extraordinary design situation are considered. At least 10 shut-downs per year due to overspeed and 24 hours of operation with extreme yaw error are required. This load cases assumes irregular sea state conditions consistent with the fault situation.
2.4	IEC	Similar to the GL load case 2.3, the IEC load case considers the loads that may result following an immediate shutdown or a transient event caused by a fault of loss of electrical connection while producing power. If significant fatigue damage may result, the likely duration of the event is evaluated assuming the normal turbulence conditions (NTM). However, normal sea state conditions (NSS) are assumed and the significant wave height for each individual sea state is taken as the expected value of the significant wave height conditioned on the relevant mean wind speed.
3.1	IEC; GL	This load case includes (per year) at least 1000 start-up procedures at V_{in}; 50 start-up procedures at V_r; and, 50 start-up procedures at V_{out}.
4.1	IEC; GL	Shutdown procedures are included as (per year) at least 1000 shut-down procedures at V_{in}; 50 shut-down procedures at V_r; and, 50 shut-down procedures at V_{out}.
6.4	IEC; GL	While in the parked (standing or idling) condition, for components subject to significant fatigue damage (e.g. from weight of idling blades), the expected number of hours of non-power production time at a fluctuating load appropriate for each wind speed is considered. This load case includes consideration of the resonant loads acting on the OWTG tower support structure resulting from the waves and effects of reduced aerodynamic damping associated with the rotor in a standstill or idling condition. Normal sea state conditions (NSS) are applied along with either the steady wind model or the turbulent wind model using a full dynamic simulation or a quasi-steady analysis with corrections for dynamic response.
E7	IEC	This load case corresponds to the situation for which the turbine is parked and moving ice at the waterline causes fatigue loads on the support structure and the tower.
7.2	IEC; GL	In the parked condition due to fault, the expected number of hours of non-power production time is considered for each wind speed and sea state where significant fatigue damage can occur to any component. Resonant loading of the support structure due to excitation by the waves is an important consideration.
8.3	IEC; GL	For this load case, consideration is made for the fatigue damage that may be accumulated during construction while the OWTG is partially installed (e.g. standing without the RNA installed). Further, vortex shedding due to current and wave loading may also be considered.
8.4	GL	This load case includes maintenance operations or other long periods with an incomplete OWTG or without grid connection. (GL suggests a period of 3 months as a guideline).

4.2 Modeling and Simulation

Consideration of the dynamic effects; aerodynamic damping; and, material or geometric nonlinearities may require the use of numerical procedures to predict the load effects with sufficient accuracy for design. When simplified calculations cannot be completed for the fatigue evaluation of a structure or structural component, then other detailed methods are to be used, such as finite element analysis. Such analyses may be completed in the time domain or in the frequency domain, with each approach having benefits and limitations. An effective model includes representations of the OWTG structure and the environment (corresponding to the wind and waves) and captures the joint interaction.

4.2.1 Material Properties

The characteristic mechanical properties for steel, including the stress-strain relationship and the modulus of elasticity are required. The effects of material nonlinearities (i.e., plasticity) may be included with the analysis by defining the minimum specified yield strength and the appropriate material model (such as bilinear or multi-linear stress-strain relationships).

4.2.2 Modeling Simplifications and Approximations

The type of analysis to be completed for the fatigue design depends on the load case under consideration and may correspond to a static, frequency, dynamic, or stability/buckling analysis. The analysis, in some cases, may be sufficiently accurate based on linear-elastic behavior. Some analyses will require that the effects associated with plasticity, geometric non-linearities and material non-linearities are considered.

Other important modeling issues identified in the standards and guidelines include:

1. The modeling of welds, specifically at the weld toe regions as relevant to the evaluation of hot-spot stress ranges, requires special consideration. Guidance is provided in DNV-RP-C203, for example, regarding the modeling and analysis by finite element methods of the local stress distributions at hot-spot regions.
2. For fixed type structures, consideration of the elastic behavior of the support structure is required. The behavior of the support structure and the natural frequencies will be influenced by the elastic foundation stiffness. Thus, these fixed type OWTGs have been categorized as soft-soft, soft-stiff or stiff-stiff based on the response.
3. Stress concentrations associated with the geometry or other aspects:
    a. Including sufficient detail in finite element models so as to capture the localised effects;
    b. Applying stress concentration factors, as commonly used for tubular joints with Efthymiou parametric equations;
    c. S-N curves with implicit stress concentration factors.

4.2.3 Solution Domain

As introduced in Section 2.3, the type of structure can influence the type of analysis to be completed. Shallow water OWTGs generally correspond to monopile concepts; mid-depth water installations use jacket structures; and, deep water installations (>50 m) use floating structures of some configuration. The fatigue behavior associated with each arrangement differs, not just because of the structural configurations but also because of the loads associated with the environment (i.e., the hydrodynamic and aerodynamic profiles) and due to the interaction of the structure with those loads (including aerodynamic damping and rotor control).

In general, solutions to the structural analysis of OWTGs are often too complex to be completed in the time domain. Instead, experience is taken from the design of offshore platforms in the oil and gas industry where analyses are completed in the frequency domain. However, characterizing the response of OWTGs depends highly on the accurate representation of the aerodynamic effects. While computing power has matured such that time domain solutions are becoming more reasonable, frequency domain solutions may remain desirable. Table 4.5 provides a brief overview of the different solution domain methodologies.

Effectively, structural analysis in the time domain calculates directly the effects of fatigue loading resulting from the wind and wave combinations. For monopile towers, the time domain analyses may be most appropriate, especially since computing power and the available software tools allow direct, mostly efficient calculation for a large number of load cases.

By comparison, for jacket structures rapid fatigue analysis techniques have been developed in the frequency domain, based on the experience in the design of offshore platforms. The large number of load combinations and the size and complexity of the models previously prohibited transient dynamic analyses in the time domain.

Table 4.5: Overview of Solution Domain Methodologies

Solution Domain	Overview
Static Analysis with Dynamic Response Factors	• Calculate static response for several loading conditions with separate consideration of wind, wave and gravity loads; • Estimate a dynamic response factor for each condition, typically between 1.2 and 1.5; and, • Superimpose results, including partial safety factors per loading type.
Frequency Domain (Spectral) Analysis	Due to non-linearity in the system, this procedure must be repeated for different environmental conditions and wind/wave combinations, but generally follows as: • Determine transfer function per load source; • Linearize system or use small harmonic loads; • Multiply spectrum of load source with transfer function; and, • Superimpose response spectra of different sources.
Time Domain Analysis	Generate realizations of external conditions: • Integrate equations of motion numerically; • Analyze fatigue response ; and, • Repeat until statistically sound information is obtained.

4.3 Long-Term Distribution of Cyclic Loads

With the environmental and operating conditions characterized and the relevant fatigue load cases identified, the resulting long-term distribution of cyclic loads is developed. In general, the different methods by which the response is developed include the following:

- Time history analyses (or, direct integration) are required for analysis of dynamic, non-linear behavior.
- Response spectral analyses are required, and may be appropriate for the determination of dynamic amplification effects, for structures having a linear-elastic response to random loading but only if the non-linear load effects can be linearized.
- As a simplification, quasi-static analyses are appropriate only for specific systems and require the application of the appropriate dynamic amplification factors (DAF) to account for the dynamic effects.

As described, consideration of the interaction of the OWTG structure within the operating environment is essential in order that conservatism is maintained and that dynamic effects are accounted for adequately. Therefore, advanced analytical tools may be warranted in order that the application of wind, wave and operating loads with both aero-elastic and hydro-elastic effects are considered simultaneously for design. Considerable research and progress has been invested in such tools in recent years and several are used in practice, analyses being based in the frequency or time domains. Some involve superposition of loads, with various levels of simultaneity available (i.e., concurrent application or interaction).

Through detailed structural analysis, the distribution of cyclic loads is used to develop the long-term stress history at each component or structural element required for the fatigue design. The calculated stresses, described in Section 6, are related to the applicable S-N curves for each assigned detail to estimate the limiting number of cycles (N) for a given stress range ($\Delta\sigma$) and applied number of cycles (n). Note that a damage tolerance approach (i.e., fracture mechanics) is permitted also, but is not discussed herein. Typically, the stresses are evaluated on the basis of:

a. Nominal stress approach with existing detail classifications;

b. Nominal stress approach with parametric equations accounting for local stress concentrations (such as holes in plates, for example); or,

c. Hot-spot stress approach using testing or Finite Element Analysis (FEA).

Once the stress history is determined for the entire service life, the stress range histogram is developed using various techniques depending on whether the stresses are given in the time domain or in the frequency domain. Typically, a rainflow counting technique is applied whereby the applied cyclic stress ranges are given in terms of the number of applications. The fatigue life of a detail or connection is then estimated using a linear Miner-Palmgren summation of the incremental damage associated with each applied stress range, as described in Section 7.

5 DETAILED CHARACTERIZATION OF ICE LOADS FOR FATIGUE

5.1 Introduction

In this section, the methods and approaches are compared for evaluating dynamic ice loads and their contribution to the fatigue of an OWTG tower foundation. A number of general approaches were considered initially such as:

a. Existing codes – several codes are available which address dynamic ice loadings. These include: (i) IEC 61400 (2009); (ii) DNV-OS-J101 (2013); and (iii) ISO 19906 (2010). The advantages of using existing codes are that:

   i. They tend to be conservative, while still representing the state-of-the-art. It should be noted that the three codes above are all recent.

   ii. Generally, the use of existing codes requires less specific investigations to be conducted (compared to the other approaches below) with respect to the site and ice conditions; and the expected ice loads. Other approaches may probably be more uncertain unless significant amounts of data were available or collected.

   iii. Typically, a wind farm design would need to be approved by a certifying or regulatory authority. The use of an existing code facilitates the acceptance process.

b. An empirical approach, in which dynamic ice loads over the life of the structure are evaluated using site-specific data and analyses. The main limitation with this approach is that typically, long-term site-specific or project-specific data are not available.

c. A numerical approach in which the ice loading function is defined based on mathematical models which could be run to predict the ice loading time history given appropriate inputs (e.g., ice properties and conditions). This approach was rejected because it would have significant uncertainties. There are inaccuracies in the models themselves. Also, their accuracy would be limited by a user's ability to define the inputs to the models.

The use of existing codes was considered to be the preferred approach; and the one that would most likely be used in a practical case. As a result, this project was focused on the use of existing codes. The three codes listed above (i.e., IEC 61400, DNV-OS-J101, and ISO 19906) were each used as a basis for evaluating dynamic ice loadings. They were first compared in general with respect to the approaches that they recommend.

Then, a test case was selected and attempts were made to apply the three codes to it. The results were compared, and gaps or issues with the existing codes were identified. It should be recognized that because the analyses were only a test case (to investigate the application of the codes) rather than a detailed engineering investigation, a number of simplifying assumptions were made with respect to defining the inputs required. These assumptions were identified and noted.

5.2 Design Codes Used as a Basis for Evaluation

5.2.1 Overview

Dynamic ice loadings are considered in IEC 61400, DNV-OS-J101, and ISO 19906. Each code advises that the structure's characteristics (shape and modal properties) affect the ice loads. As a result, ice load evaluations must be done in relation to a specific structure type. For each code, the general steps for an evaluation of ice-induced fatigue are the same:

a. Define the ice environment;

b. Define the static or quasi-static ice load; and,

c. Define the dynamic ice loadings: None of the codes contain specific requirements in their mandatory (normative) sections although they do identify ice-induced fatigue as a limit state that must be considered. However, each code contains guidance for practising engineers in the informative sections as advisory material.

5.2.2 Summary Assessment of IEC 61400

5.2.2.1 Step 1 – Define the Ice Environment

IEC 61400 is prescriptive as it specifies loading cases that must be considered. The only ice conditions specified for the fatigue limit state are a "moving ice floe at relevant velocities". IEC 61400 further specifies the ice thickness to be used for fatigue limit analyses as follows:

a. "Open sea": 50-year ice thickness; and,

b. "Landlocked waters": "ice thickness equal to the long-term mean value of the annual maximum ice thickness for winters with ice".

5.2.2.2 Step 2 – Define the Static or Quasi-Static Ice Load

IEC 61400 only provides guidance material.

For vertical structures, it advises that the quasi-static ice load should be calculated using an indentation formula as given by Equation 5.1 and it provides guidance regarding the inputs as listed in Table 5.1. IEC 61400 recommends that the static ice load produced by ice crushing against a vertical structure, H_d, be calculated using Equation 5.1.

$$H_d = k_1 k_2 k_3 h D \sigma_c \qquad \text{Eqn. 5.1}$$

Where:

- k_1 = a shape factor – guidance is provided in IEC 61400 for selecting k_1;
- k_2 = a contact factor – guidance is provided in IEC 61400 for selecting k_2;
- k_3 = an aspect ratio factor – guidance is provided in IEC 61400 for k_3;
- h and D = the ice thickness and structure diameter respectively; and,
- σ_c = the ice crushing strength – guidance is provided in IEC 61400 for σ_c.

Table 5.1: Inputs for Static Ice Loads for Vertical Structures Using IEC 61400

Parameter	Recommended by IEC 61400
Shape Factor, k_1	1.0: rectangular shape 0.9: circular shape
Contact Factor, k_2	0.5: ice continuously moving 1.0: ice adfrozen to structure 1.5: ice bustle formed around structure
Indentation Factor, k_3	Defined by the formula: $k_3 = (1+5*h/D)^{0.5}$
Ice Crushing Strength, σ_c in MPa	3.0: ice movements during the coldest time of year 2.5: slow ice movements (e.g., caused by thermal expansion) 1.5: ice movements in spring with ice temperature near melting point 1.0: partly deteriorated ice with ice temperature near melting point 0.5: moving saline 1st year ice in the open sea

For structures with sloped faces, IEC 61400 advises the approach by (Ralston 1977).

With respect to the fatigue limit state, IEC 61400 advises that, for the purpose of evaluating ice load time histories, the maximum ice force be calculated for both vertical and conical structures using the indentation formula advised for vertical structures.

5.2.2.3 Step 3 – Define Dynamic Ice Loadings

IEC 61400, DNV-OS-J101, and ISO 19906 are compared in Table 5.2. In summary, IEC 61400 tends to be more prescriptive than ISO 19906 while it is generally similar to DNV-OS-J101.

IEC 61400 outlines two possible approaches for defining the ice load time history as it may either be presumed to:

(a) be sinusoidal; or

(b) have a saw-tooth shape, as shown in Figure 5.1.

Algorithms are defined in IEC 61400 for:

(a) vertical structures which are presumed to induce ice crushing failures; and

(b) conical structures which are presumed to induce ice bending failures.

It is noteworthy that IEC 61400 does not distinguish between the various crushing modes that are observed during ice-structure interactions, i.e., intermittent crushing; frequency lock-in; and continuous crushing. (These are described in more detail in a subsequent section). It would output the same variation in force (Equation 5.2) during a loading cycle regardless of the crushing mode. Although this makes IEC 61400 easier to apply, it limits its accuracy.

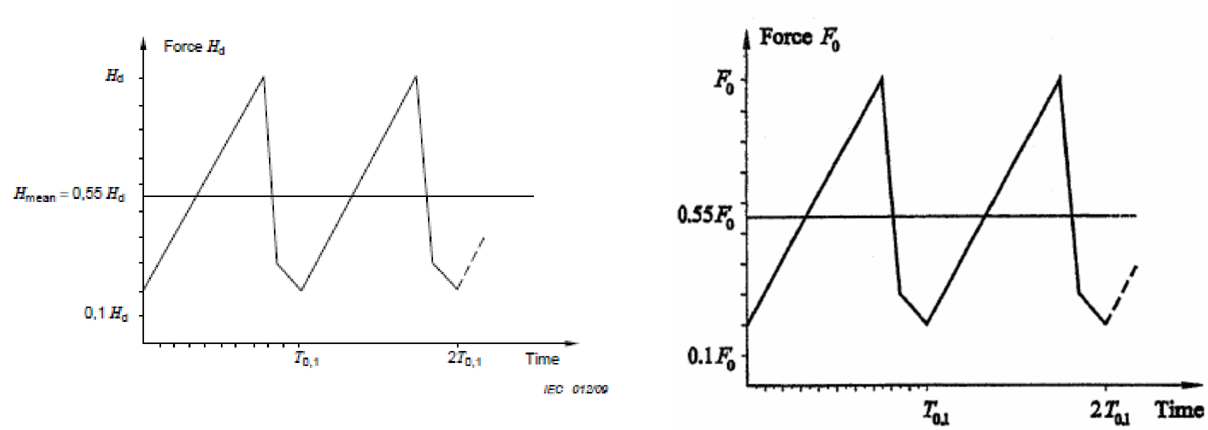
$$\Delta F = F_{max} - F_{min} \qquad \text{Eqn. 5.2}$$

Where,

- F_{max} = the maximum force during a loading cycle; and
- F_{min} = the minimum force during a loading cycle.

Table 5.2: Suggested Approaches in Various Codes for Defining Dynamic Ice Loadings

Code	Structure	Summary of Suggested Approach
IEC 61400	Vertical	Approach 1 – The ice load time history may be presumed to be sinusoidal. An algorithm is defined in IEC 61400; and the variation in load (ΔF) is thus an output of the load algorithm. The key inputs to the algorithm are: (a) the quasi-static ice load; and, (b) the natural frequency of the structure.
IEC 61400	Vertical	Approach 2 (Figure 5.1) – Alternatively, the ice load time history may be assumed to have a serrated or saw-tooth shape. The maximum load is defined as the quasi-static ice load (H_d). The minimum load is defined as $0.1H_d$. The period of the saw-tooth is defined based on the natural frequency of the structure.
IEC 61400	Conical	Approach 1 (using a sinusoidal ice load time history) - An algorithm is defined in IEC 61400; and the variation in load (ΔF) is thus an output of the equation. The key inputs are: (a) the quasi-static ice load; and (b) the ice breaking length, which depends on the ice velocity and thickness.
IEC 61400	Conical	Approach 2 (using a saw-tooth ice load time history shown in Figure 5.1) - Alternatively, the ice load time history may be assumed to have a saw-tooth shape. The maximum load is defined as the quasi-static ice load (H_d). The minimum load is defined as $0.1H_d$. The period of the saw-tooth is defined based on the icebreaking length, which depends on the ice velocity and the ice thickness.
DNV-OS-J101	Vertical	DNV-OS-J101 advises that the ice load time history may be presumed to have a saw-tooth shape (Figure 5.1). The maximum ice load is taken as the quasi-static ice load (H_d); and the minimum load is taken as $0.2H_d$. The period of the saw-tooth depends on whether the structure is vertical or conical, as follows: • vertical - period defined based on the structure's natural frequency. • conical structures – the period is defined based on the icebreaking length, which depends on the ice velocity and the ice thickness.
ISO 19906	Vertical; or Conical	ISO 19906 has a more complete, but more complex treatment of dynamic ice-structure interactions. The approach in ISO 19906 is described in Section 5.2.4.



(a) Source: IEC 61400 **(b) Source: DNV-OS-J101**

Figure 5.1: Saw-Tooth Ice Load Time Histories Specified in IEC 61400 and DNV-OS-J101

5.2.3 Summary Assessment of DNV-OS-J101

5.2.3.1 Step 1 – Define the Ice Environment

DNV-OS-J101 is prescriptive, in that it identifies loading cases, although it does not require that they be considered. "Moving ice floes" are the only ice conditions identified for the fatigue limit state. DNV-OS-J101 specifies the ice thickness for fatigue limit analyses as follows:

a. "open sea" conditions: 50-year ice thickness; and

b. "land-locked waters": DNV-OS-J101 advises that the "t_{limit}" is to be used. This is generally defined as the maximum ice thickness at which ice movements do not occur (for coastal waters). DNV-OS-J101 further advises that "unless data indicate otherwise", "t_{limit}" can be taken to be the "long-term mean value of the annual maximum ice thickness".

5.2.3.2 Step 2 – Define the Static or Quasi-Static Ice Load

DNV-OS-J101 is not prescriptive and only offers guidance regarding the calculation of ice loads. For vertical structures, it advises that ice loads may be determined using the methodology in API RP-2N, and it provides guidance regarding key ice load inputs. API RP-2N recommends that an indentation formula (of the same general type as Equation 5.1) be used to calculate static ice loads for a vertical structure, and it also provides guidance regarding the key inputs.

For conical structures, DNV-OS-J101 offers the guidance that ice loads may be calculated using the formula by (Ralston 1977).

DNV-OS-J101 also cautions the user that ice loads are complex and advises the user to consult ISO 19906. Hence, the user would be allowed to use the methodology in ISO 19906 should this be desired.

5.2.3.3 Step 3 – Define Dynamic Ice Loadings

The approach in DNV-OS-J101 is essentially the same as in IEC 61400 except that DNV-OS-J101 advises that the ice load time history be presumed to have only a saw-tooth shape.

5.2.4 Summary Assessment of ISO 19906

5.2.4.1 Step 1 – Define the Ice Environment

ISO 19906 identifies fatigue as a limit state that must be considered. ISO 19906 states that ice loads and ice actions "shall be determined for each relevant ice loading scenario"; and, then proceeds to list ice-structure loading scenarios to be considered "where applicable".

ISO 19906 contains extensive information regarding ice conditions, ice features and ice properties. However, ISO 19906 does not provide direction or guidance regarding the types of ice conditions to be considered for the fatigue limit state.

5.2.4.2 Step 2 – Define the Static or Quasi-Static Ice Load

For vertical structures, ISO 19906 provides an ice load algorithm (Equation 5.3) as guidance material that depends on: (a) the ice thickness; (b) the aspect ratio; and, (c) the ice strength, as follows:

$$P_G = C_R h^n \left(\frac{w}{h}\right)^m \text{ [MPa]} \qquad \text{Eqn. 5.3}$$

Where,

- C_R is a strength parameter;
- n and m are empirical coefficients; and,
- h and w are the ice thickness and structure width respectively.

It should be noted that Equation 5.4 is applicable to rigid structures with aspect ratios (w/h) greater than 2.0; and where the waterline displacement, obtained as a static response to the representative ice action is typically less than 10 mm.

ISO 19906 advises that C_R can be scaled based on the ice strength as follows:

$$C_R = C_{RO} \frac{\sigma}{\sigma_0} \qquad \text{Eqn. 5.4}$$

Where:

- C_R = the strength parameter for the area of interest;
- C_{R0} = the strength parameter for the reference area (given in ISO 19906 as 2.8 MPa for Arctic areas, and 1.8 MPa for the Baltic Sea);
- σ = a measured or inferred strength index for the area of interest; and,
- σ_0 = a strength index for the reference area.

The ice strength used as a scaling basis should be representative of the type of ice-structure interaction that occurs. For a vertical structure, the ice will generally fail by crushing so the ice compressive strength is a reasonable scaling basis for the strength parameter, C_R.

For conical structures, ISO 19906 offers the guidance that ice loads may be calculated using the formula by (Ralston 1977).

5.2.4.3 Step 3 – Define Dynamic Ice Loadings

ISO 19906 advises that three types of dynamic ice-structure interactions may occur, as illustrated in Figure 5.2.

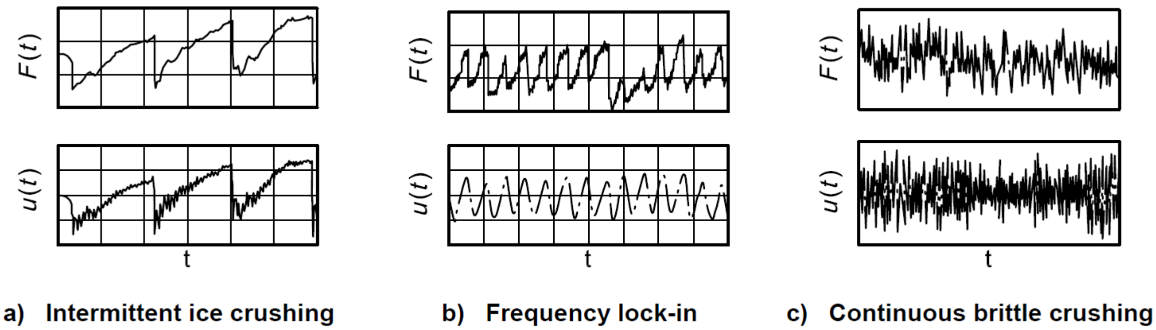
a. Intermittent Crushing (IC): this occurs at slow speed in which the ice load period is much greater than the structure's natural frequency range. IC is typical of scenarios where ice flexural failures occur. ISO 19906 advises that F_{max} can be presumed equal to the quasi-static ice load for IC.

b. Frequency Lock-In (FLI): FLI is of most concern as it generates the largest ice loads as well as the largest structural displacements and accelerations. It generally occurs at intermediate ice speeds of about 0.03 m/s to 0.10 m/s, although this depends on both the structure and the ice. ISO 19906 offers the following guidance:

    i. Vertical structures: ISO suggests that a saw-tooth ice load time history be presumed, as illustrated in Figure 5.3, and that the ice load time history can be assumed to be constant with time. ISO 19906 offers guidance regarding how F_{max}, and F_{min} can be evaluated as well as the period of the ice load time history. ISO 19906 warns that FLI, and dynamic ice loadings in general, are not well understood. Hence, ISO 19906 provides ranges of values for critical parameters in the algorithms. It is noted that the parameters may vary greatly.

    ii. Conical structures: FLI has been observed for conical structures as well although because the ice failure process includes bending, the ice load time history has randomness. ISO 19906 offers guidance regarding ice load time histories and their distribution. Again, it is noted that the parameters may vary greatly.

c. Continuous Crushing (CC): ISO 19906 does not offer specific guidance for evaluating CC other than to conduct analyses in the frequency domain.



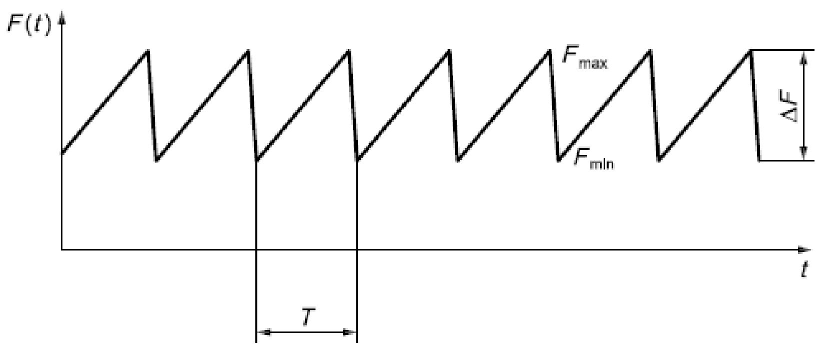
Key

t time

F ice action

u structure displacement

Figure 5.2: Ice Load Time Histories during Ice Interactions with Structures (ISO 19906)



Key

t	time
F	ice action
F_{max}	maximum value of ice action
F_{min}	minimum value of ice action
ΔF	difference between maximum and minimum values of ice action
T	period of ice action

Figure 5.3: Simplified Ice Forcing Function Given by ISO 19906 for Vertical Structures

In summary, ISO 19906 has a more complete, but more complex treatment of dynamic ice-structure interactions. As a result, many more parameters must be specified in order to apply ISO 19906. As the required input parameters are uncertain and may vary greatly, this will of course introduce uncertainty and/or conservatism in the results. Furthermore, an added uncertainty is the conditions (i.e., ice environment, structure properties) at which each type of ice loading (i.e., IC, FLI, and CC) will take place are not fully understood, and may vary substantially.

Thus, it is not clear whether or not the added complexity of the approach in ISO 19906 will lead to better accuracy than the simpler approaches in IEC 61400 or DNV-OS-J101. This was explored using sample calculations for a test case.

5.3 Wind Turbine Foundation Characteristics

5.3.1 Effect of Structure Properties on Ice Loads

It is well-known that the shape of the structure affects the ice failure process, and as a result, the ice loads. Vertical structures cause the ice to fail in crushing. For conical structures, the process is more complex. Initially, the ice fails in bending. However, continued ice movements create a necessity for the ice to clear around the structure which leads to a mixed-mode ice failure process (flexure, ride-up, "punching" through ice rubble in front of the structure, etc.). Thus, evaluations of ice-induced fatigue must be performed in relation to a specified structure shape.

Furthermore, significant ice load magnification (of a factor of about 2.0) has been measured in some cases; e.g., (Blenkarn 1970) and (Jeffries, Karna and Loset 2008). Frequency Lock-In (FLI) is generally most problematic for compliant, vertical structures, for situations where a stationary crushing process has been set up. However, FLI has occurred in some cases for conical structures as well, although the process is more random. As a result, one must be careful in specifying the peak ice load for a loading cycle as in some cases, the peak ice load would exceed the static ice load; or the quasi-static ice load exerted at ice speeds above the range where FLI occurs.

In summary, the structure's properties (i.e., shape and modal properties) both affect the ice loads.

Each of the codes (i.e., IEC 61400, DNV-OS-J101, and ISO 19906) contains cautionary comments regarding the range of conditions where FLI, or "tuning" may occur. IEC 61400 and DNV-OS-J101 both contain the criterion of Equation 5.5 for "tuning", or the occurrence of FLI.

$$\frac{U_{ice}}{(h \times f_N)} > 0.3 \qquad \text{Eqn. 5.5}$$

Where,

- U_{ice} = the ice movement speed during interaction with the structure;
- h = the ice thickness;
- f_N = the natural frequency of the structure; and,

IEC 61400 does not contain further guidance regarding the occurrence of FLI. DNV-OS-J101 provides cautionary comments in relation to the damping of the structure, as shown in Figure 5.4.

As long as the total structural damping is not too small, the following method for analysis of dynamic ice loading can be applied:

Guidance note:

In assessing whether the structural damping is too small, it may be helpful to consider that, based on field experience, structures with natural frequencies in the range 0.4 to 10 Hz have experienced lock-in vibrations when the total structural damping has been lower than 3% of critical damping.

Figure 5.4: Cautionary Comments and Limitations Given in DNV-OS-J101

ISO 19906 also contains a criterion for the susceptibility of a structure to FLI. ISO 19906 advises that to ensure dynamic stability, of a natural mode, *n*, the relative damping coefficient, ξ_n, of the structure should be larger than the opposite contribution of ice action to dynamic instability. The damping coefficients, ξ_n on the lowest natural modes of the structure should be large enough to prevent energy inflow due to ice action by satisfying the criterion of Equation 5.6.

$$\xi_n \geq \left[\frac{\varphi_{nc}^2}{4\pi f_n M_n}\right] \times h \times \theta \qquad \text{Eqn. 5.6}$$

Where,

- ξ_n = the total damping of the eigenmode as a fraction of critical;
- φ = the non-normalized modal amplitude at the ice action point;
- M_n = the true modal mass, expressed in kilograms;
- f_n = the natural frequency of the eigenmode, expressed in hertz;
- h = the ice thickness, expressed in meters; and,
- θ = a coefficient, the suggested value of which is 40310^{6} kg/m/s.

As well, ISO 19906 provides the following general guidance:

a. The potential for dynamic ice loads should be investigated if the structure's waterline displacement, obtained as a static response to the representative ice action, is typically more than 10 mm.

b. A static design should be supplemented with a dynamic analysis if there is a risk of frequency lock-in for a particular structure. Based on field experience, structures with a fundamental frequency in the range of 0.4 Hz to 10 Hz have experienced self-excited vibrations when the total structural damping (as a fraction of critical) has been lower than about 3%.

5.3.2 Impact of the Structure's Properties on the Analyses for This Project

5.3.2.1 Structure Shape and Type

Recognizing that the structure's properties affect the ice loads, it is therefore important to select an appropriate test case for analysis in this project. In practice, the most appropriate foundation structure varies with the water depth, as illustrated in Figure 5.5. For shallow water, the most suitable foundation types are monopiles, gravity base structures and suction bucket structures. (Musial and Butterfield 2006) pointed out that the largest wind turbine constructed to date (as of 2006) was a near-vertical monopile, illustrated in Figure 5.6.

It is also noteworthy that the wind turbine platforms in the Lake Vanern Wind Farm (Sweden) are near-vertical monopiles, as illustrated in Figure 5.6 after (Wizelius and Frykberg 2012). This is considered to be of particular interest for this project as the Lake Vanern wind farm "sees" ice each year.

It was further noted that the waterline diameter was about 5 m to 6 m for all of these monopiles.

(Eranti, et al. 2011) described the development of a novel, gravity-based platform structure with an upward-breaking conical shape to minimize ice forces, illustrated in Figure 5.7. That structure is intended to be installed as a pilot project in the Baltic Sea near Pori, Finland. It is noteworthy that the structure was intended for use in "heavy" ice conditions.

In summary, most of the wind turbine platforms in shallow water are near-vertical monopiles, especially those intended to operate in icy waters. For that reason, the analyses in this project were focused on defining ice loading time histories for vertical structures, which cause the ice to fail in crushing.

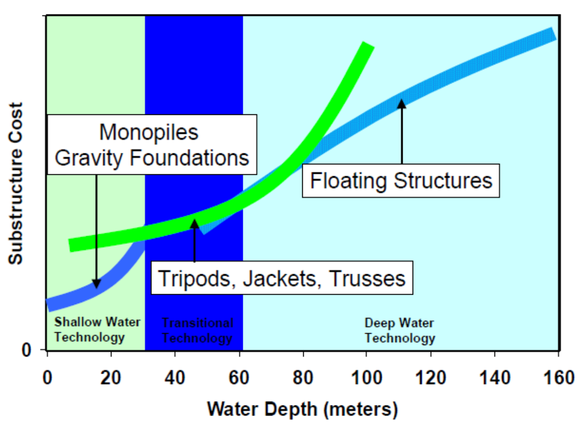


Figure 6 – Cost of Offshore Wind Turbine Substructures with Water Depth [10]

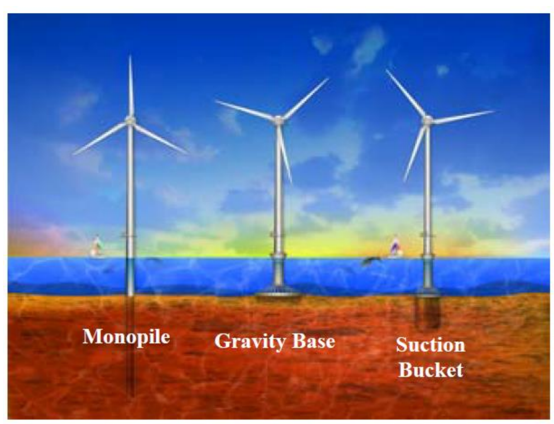


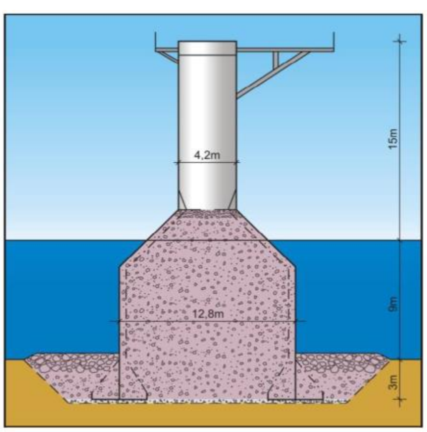
Figure 7 – Shallow Water Foundation Technology – Current Options

Figure 5.5: Effect of Water Depth (Musial and Butterfield, 2006)

(a) Iris Sea 25.2 MW Arklow Banks Wind Farm
(Musial and Butterfield 2006)

(b) Monopile Foundations in Lake Vanern
(Wizelius and Frykberg 2012)

Figure 5.6: Examples of Monopile Foundations for Wind Turbine Platforms



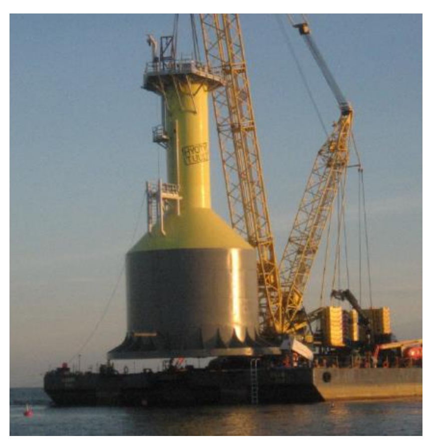


Figure 5.7: Upward-Breaking Conical Structure for the Baltic Sea (Eranti, et al. 2011)

5.3.2.2 The Structure's Compliance, Natural Frequencies and Modal Properties

As described previously, the ice loads may be magnified under certain conditions. The structure-related properties that affect this include:

a. The structure's shape – Frequency Lock-In (FLI), or "tuning" is more likely to occur for vertical, or near-vertical, structures;

b. The compliance of the structure – indices have been developed to describe a structure's susceptibility to FLI based on the ratio of its waterline displacements to the ice thickness;

c. The damping of the structure; and,

d. The natural frequency of the structure for the lower modes.

FLI is undesirable as it leads to large structural displacements and accelerations, resulting in increased fatigue. It is beyond the scope of this project to select or specify structural properties that would avoid FLI.

For this project, dynamic ice loads have been calculated presuming that the structural designers for a given wind farm project would use prudence to produce a design that would avoid FLI.

5.3.2.3 Summary of Structural Properties Assumed for Demonstration Ice Load Analyses

The demonstration analyses described herein have been conducted based on the following assumptions:

a. The structure is a near-vertical monopile with a waterline diameter of 5 m.

b. The structure's compliance, damping and natural frequency have been selected such that FLI or "tuning" does not occur. Because this requires that the structure be stiff, a natural frequency of 10 Hz was assumed for the analyses done in this project.

5.4 Ice Environment Parameters Used as a Test Case

5.4.1 Caveat

It should be noted that this study was entirely hypothetical; and a test case was only selected because this was necessary to demonstrate the methodology and results of the work. No inferences are intended, nor should be made, regarding the likelihood that an offshore wind farm might be constructed at the test case location based on its selection as a test case here.

5.4.2 Selection

A test case was selected for this project based on the following criteria:

a. It should be realistic. Because ice conditions are typically complex, a test case needs to include sufficient complexity to allow realistic assessments of the candidate codes. Furthermore, the test case should be a site where ice occurs regularly. Marginal ice zones where ice is an unusual occurrence were not considered suitable for this project. For these locations, ice-induced fatigue would be only a small component of the overall accumulated fatigue in the structure, compared to that resulting from other environmental loads (winds, waves, currents, etc.). For this reason, the Cape Wind Project in Nantucket Sound was rejected as a possible test case for this project.

b. It should not be an extreme region with respect to ice conditions as it is doubtful that wind turbines would be located in such areas. For example, the Beaufort Sea (where structures may "see" multiyear ice) was eliminated from consideration on this basis.

c. Information should be available to define the ice environment for the site.

d. The site should be located in American waters.

Cook Inlet, Alaska was selected as the test case here as it fulfills the selection criteria above.

5.4.3 Ice Input Data Requirements for an Assessment of Ice-Induced Fatigue

Although the ice input data requirements for the codes vary in detail, they all need input regarding:

(a) the ice thickness and its variation over the winter;

(b) the ice strength; and

(c) the ice movements in relation to the ice type, the ice thickness and the period during the winter.

5.4.3.1 Ice Thickness

It is well known that the ice thickness varies with the ice type (e.g., floe ice, rafted ice, ridges, etc.), as well as with time over the winter. Thus, the ice thickness must be specified in relation to these factors.

For IEC 61400, the only ice type that needs to be considered for the fatigue limit state is a "moving ice floe at relevant velocities". Thus, only floe ice thicknesses need to be specified for IEC 61400. However, an ice floe may contain rafted ice or ridges so variations in ice thickness caused by non-uniformities in ice type (e.g., the presence of rafted ice in a floe) need to be specified, as well as the lateral extent of the various ice types within a floe.

The same comments apply for DNV-OS-J101 as it specifies that "moving ice floes" are the only ice type that needs to be considered for the fatigue limit state.

ISO 19906 is more general as it requires that all ice types capable of producing ice-induced fatigue be considered.

5.4.3.2 Ice Strength

This varies over the winter of course, and hence, the ice strength must be specified over the duration of the winter.

A range of ice strengths are specified in IEC 61400, depending on the ice conditions (i.e., general temperature, rate of movement, etc.). See Equation 5.1 and Table 5.1. Thus, the ice strength must be specified for the time of the winter being considered.

The ice strength input requirements for DNV-OS-J101 vary somewhat depending on whether the calculation methodology is based on API RP-2N, which contains an indentation formula; or ISO 19906, which contains an empirical formula based on the ice thickness and aspect ratio. However, in either case, the ice strength must be specified for the given time of the winter.

In ISO 19906, the ice load varies with the ice strength based on an empirical factor (Equation 5.2). Thus, the ice strength must be specified for the time period being considered.

5.4.3.3 Ice Movements

The comments here apply to all three codes (i.e., IEC 61400, DNV-OS-J101, and ISO 19906). Ice movements are required to create ice-induced fatigue. It is important to establish the ice movement magnitudes and patterns as they affect the number of loading events that occur in a given winter. Of course, this affects the accumulation of ice-induced fatigue.

The critical parameters include the ice movement rate, frequency and duration. Because ice movements usually vary over the winter, and the ice loads change over the winter (due to variations in ice thickness and strength), ice movements need to be specified over the full winter; and, in relation to the ice type.

The loading cycle period will be controlled by the properties of either:

(a) the structure (e.g., its natural frequencies for lower modes); or

(b) the ice (i.e., the ice velocity and breaking length).

It is evident that the number of loading cycles during a winter will be controlled by the period of a loading cycle and the number of cycles that occur.

In essence, this makes it necessary to define the amount of ice movement (in terms of distance, e.g., km) that the structure will "see" in relation to:

(a) the ice type (e.g., floe ice, rafted ice, ridges, etc.); and

(b) the period during the winter.

5.4.4 Cook Inlet Ice Environment

5.4.4.1 Purpose

This section provides a preliminary description of the ice regime in Upper Cook Inlet. It should be noted that this work was not intended to produce definitive ice design criteria, but only to define the ice conditions in sufficient detail to allow realistic demonstrations of the use of the three codes being considered (i.e., IEC 61400, DNV-OS-J101 and ISO 19906). This was sufficient to meet the objectives of this project.

An engineering design analysis would require an ice investigation in significantly more depth, and quite possibly, it would include the collection of site-specific data. A number of simplifying assumptions were made throughout this study which would require verification for an engineering design investigation.

This ice investigation was focused on defining mean ice conditions because this project was focused on the long-term accumulation of ice-induced fatigue. Over a period of several years, ice conditions can be expected to tend towards the mean values on average.

5.4.4.2 Overview

The ice environment in Cook Inlet has been studied by many groups; e.g., (Mulherin, et al. 2001); (Brower, et al. 1988); and, (Nelson 1995). Table 5.3 summarizes ice information regarding Cook Inlet.

Table 5.3: Cook Inlet Sea Ice Conditions (ref: ISO 19906)

Parameter		Average Annual Values	Range of Annual Values
Occurrence	First Ice	25 November	17 Oct. to 17 Dec.
	Last Ice	7 April	10 March to 15 May
Level First Year	Landfast Ice Thickness, m	0.5	0.2 to 0.6
Ice	Floe Thickness, m	0.8	0.6 to 0.9
Rafted Ice	Thickness, m	1.3	1.2 to 1.5
Rubble Fields	Sail Height, m	1.0	0.5 to 2
	Length, m	hundreds	hundreds
Ridges (1st year)	Sail Height, m	1	1 to 2
	Keel Depth, m	4 to 6	4 to 10
Stamukhi	Water Depth Range, m	0 to 8	0 to 8
	Sail Height, m	5	4 to 12
Ice Movement	Speed, m	4	3 to 4.5

Oil and gas production platforms have been in place in Upper Cook Inlet since the 1960’s, as illustrated in Figure 5.8 and Figure 5.9, and ice loads have been measured on them for many years (Blenkarn 1970); (Bhat and Cox 1995)). Furthermore, significant ice-induced vibrations have occurred at some of these platforms; e.g., (Blenkarn 1970).

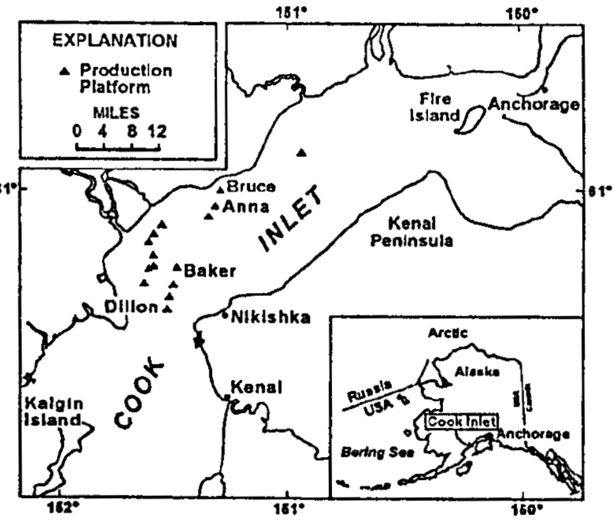


Figure 1. Location map of Cook Inlet platforms (from Visser, 19921.

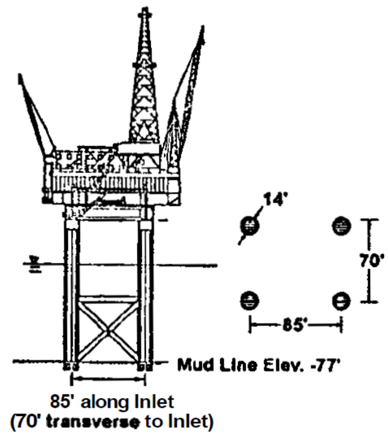


Figure 2 A typical Cook Inlet platform (Anna).

Figure 5.8: Cook Inlet Platforms (Bhat and Cox 1995)

Fig. 3—Ice failure mechanism at a platform leg.

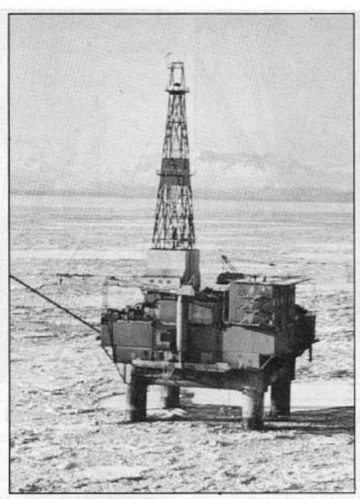


Fig. 4—Four-legged Platform C in moderate ice cover.

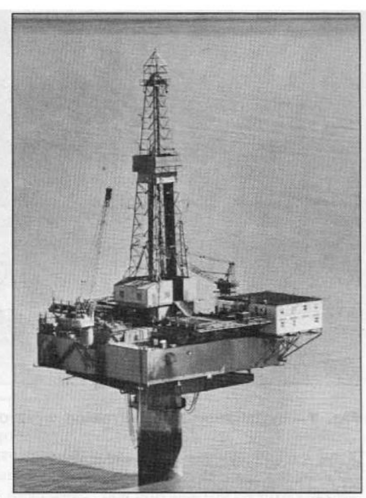


Fig. 5—Monopod platform in the Trading Bay field.

Figure 5.9: Cook Inlet Platforms in Ice (Visser 1992)

Recognizing that the ice conditions vary within Cook Inlet, the analyses here were based on a test case in Upper Cook Inlet as this is the general location for oil and gas platforms placed there.

Table 5.4 and Table 5.5 summarize ice design criteria that have been established for some oil and gas platforms in Upper Cook Inlet.

Table 5.4: Cook Inlet Sea Ice Design Criteria (Visser 1992)

TABLE 2—ENVIRONMENTAL CONDITIONS IN COOK INLET

	Customary Units	SI Metric Units
Average annual temperature	35°F	2°C
Minimum ambient temperature	−40°F	−40°C
Maximum tide	30 ft	9 m
Maximum current	10 to 12 ft/sec	3 to 3.6 m/s
Maximum water temperature	55°F	13°C
Minimum water temperature	29°F	−2°C
Maximum wind	80 miles/hr	130 km/h
Maximum wave	15 ft	4.5 m

TABLE 3—ICE DESIGN CRITERIA FOR COOK INLET PLATFORMS

	Design Criteria	
	Original*	API *RP 2N*
Design ice thickness, ft [m]	3.5 [1.1]	2 to 3 [0.6 to 0.9]
Rafted ice, ft [m]	NA	4 to 5 [1.2 to 1.5]
Unconfined compressive strength, psi [MPa]	550 [3.8]	500 to 600 [3.4 to 4.1]
Shape factor	0.55	0.55
Compressive strength, psi [MPa]	300 [2.1]	275 to 330 [1.9 to 2.3]
Load condition	All four legs	Three legs
Maximum load,** kips [MN]	9,400 [42×10^6]	9,000 [40×10^6]

*As used by one operator.
**For 15.5-ft diameter legs.

Table 5.5: Ice Design Criteria after (Visser 1992) as cited by (Mulherin, et al. 2001)

	Design Criteria	
Design parameter	*Original design*	*API (1988) Recommended*
Ice thickness, level ice (m)	1.1	0.6–0.9
Ice thickness, rafted ice (m)	NA	1.2–1.5
Compressive strength, unconfined (MPa)	3.8	3.4–4.1
Compressive strength, confined (MPa)	300	275–330
Maximum load (MN)	42×10^6	40×10^6

5.4.4.3 Ice and Thermal Regime

On average, the air temperature starts to fall below freezing in October and it reaches a minimum during the period from December to February. The air temperature is regularly above freezing in April. See Figure 5.10.

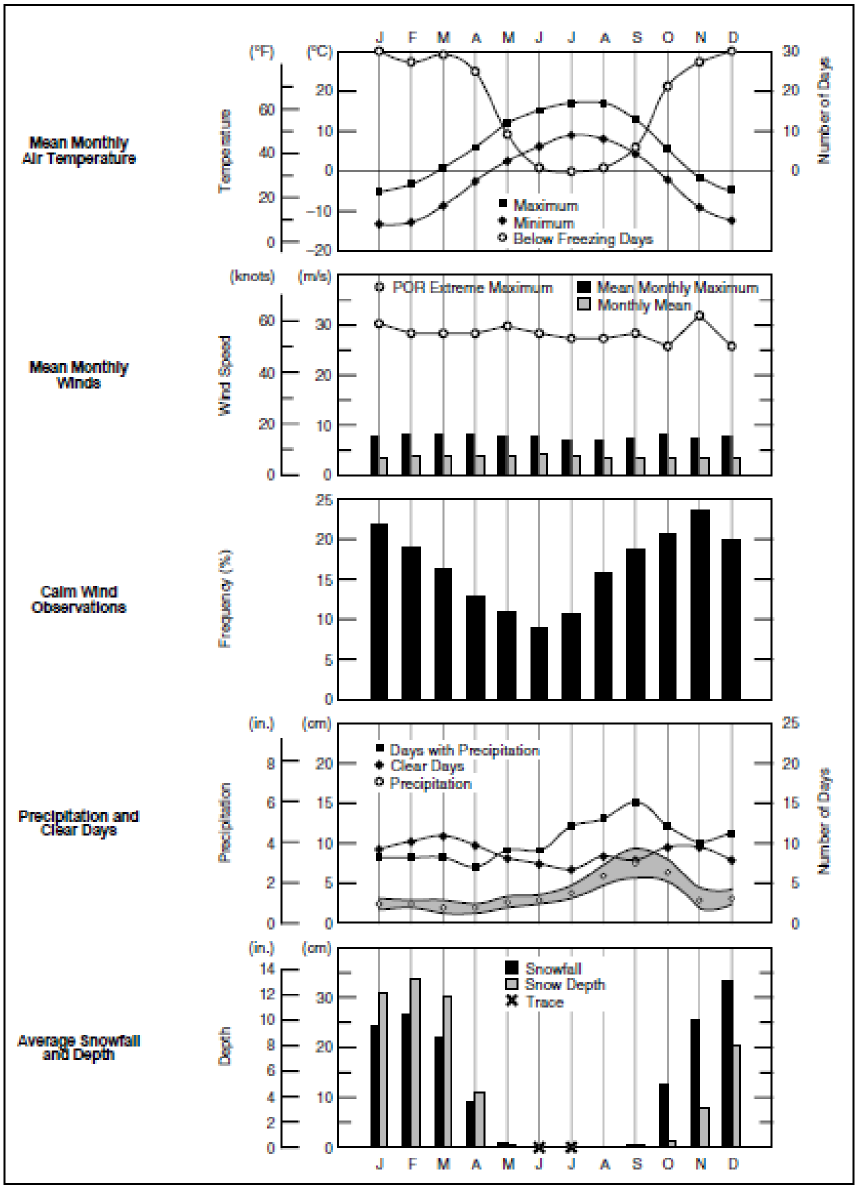


Figure 5.10: Climatological Summary for Kenai, Alaska (Mulherin, et al. 2001)

It is well-known that thermal ice growth is related to the number of Freezing Degree Days (FDDs). The FDDs over the winter are shown in Figure 5.11, subdivided by month. The FDDs for Kenai, Alaska were used as a basis here as this site is closest to the selected location for this project. On average, the FDDs start to accumulate in October and they peak during the month of January. The cumulative FFDs reach about 1200 °C3days on average.

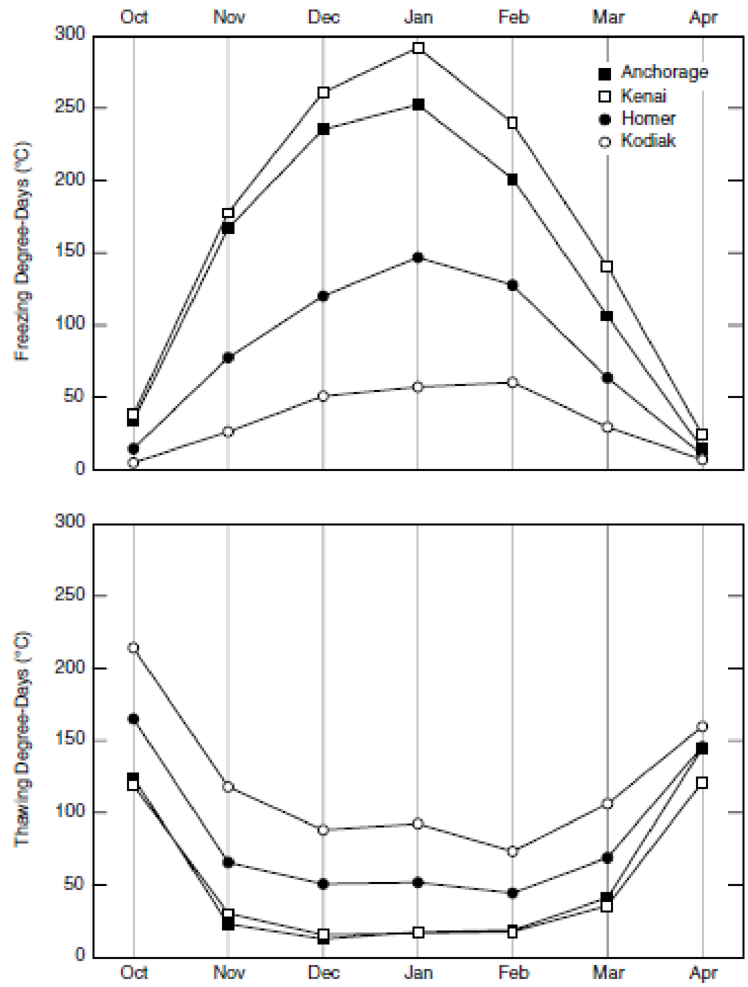


Figure 5.11: FDDs in Cook Inlet Subdivided by Month (Mulherin et al, 2001)

The ice regime in Upper Cook Inlet is dynamic as the ice moves in and out of the inlet due to tidal currents. There are two tide cycles each day (i.e., two floods and two ebbs). This causes much more ice to be formed in a winter than would be expected from purely thermal growth; e.g., (Whitney 2002).

Ice usually forms by early December and it persists until about April, although there is significant variability from winter to winter, as listed in Table 5.6. Over the period from 1969-70 to 1985-86, the ice season duration has ranged from 94 days to 202 days, with an average of 135 days.

Table 5.6: Ice Season Duration cited by (Whitney 2002)

Year	First Ice	Ice Free
69-70	Nov 18	Mar 23
70-71	Oct 17	May 7
71-72	Nov 23	May 15
72-73	Nov 13	Apr 10
73-74	Nov 18	Apr 6
74-75	Nov 24	Apr 9
75-76	Nov 12	Apr 10
76-77	Dec 17	Apr 9
77-78	Nov 20	Mar 18
78-79	Dec 16	Mar 31
79-80	Dec 12	Mar 26
80-81	Dec 6	Mar 10
81-82	Nov 20	Apr 19
82-83	Nov 29	Mar 21
83-84	Dec 14	Mar 20
84-85	Dec 17/1st rep.	Feb 13, 85*
		Apr 17
85-86	Nov 5	Apr 18
Average	Nov 24	Apr 8
Median	Nov 20	Apr 9

* Ice disappeared; then reappeared 2/13/85

Obviously, the ice cover varies with respect to both the ice type and the concentration over the winter. Annex A provides summary maps showing the mean ice coverage over the winter by ice concentration and ice type. Table 5.7 summarizes the data for Upper Cook Inlet near Kenai, Alaska.

Table 5.7: Summary of Mean Ice Conditions near Kenai (Information Source: Ice coverage maps in Annex A)

Timing	Mean Ice Concentration and Ice Type
Dec. 16-31	No ice
Jan. 1-15	Ranges from 2/10ths New Ice (0-10 cm) to 5/10ths Young Ice (10-30cm)
Jan. 16-31	Ranges from 4/10ths New Ice (0-10 cm) to 5/10ths Young Ice (10-30cm)
Feb. 1-15	Ranges from 5/10ths Young Ice (10-30cm) to 6/10ths Young Ice (10-30cm)
Feb. 16-28/29	Ranges from 5/10ths Young Ice (10-30cm) to 7/10ths First Year Ice (>30cm)
Mar. 1-15	Ranges from 5/10ths Young Ice (10-30cm) to 7/10ths First Year Ice (>30cm)
Mar. 16-31	Ranges from 3/10ths New Ice (0-10 cm) to 4/10ths Young Ice (10-30cm)

5.4.4.4 Ice Types

A wide range of ice types may occur. (Mulherin, et al. 2001) categorized the ice types as: (a) pack ice; (b) shorefast ice; (c) stamukhi; and (d) estuarine and river ice. Only pack ice and shorefast ice are considered to be capable of creating ice loadings of concern for ice-induced fatigue in Upper Cook Inlet at the general location being used as a test case.

Pack ice is usually variable. In general, pack ice can be considered to consist of a mix of ice floes, usually with a range of sizes, containing features resulting from dynamic ice interactions (rafting, ridges, etc.). A structure exposed to moving pack ice would primarily "see" floe ice, which would have a thickness that is generally uniform within a floe although it would likely vary from floe to floe. This is seen in Figure 5.12 where ice floes of variable size, thickness and surface roughness are evident. Dynamic ice processes would probably cause the formation of ridges and rafted ice within or at the edges of the floes, causing a structure to interact with these ice types too. Furthermore, the floes may be conglomerates of several small pans (seen in some floes in Figure 5.12) which would lead to non-uniformity.

Figure 5.12: Example of Ice Conditions in Upper Cook Inlet (Mulherin et al, 2001)

Shorefast ice may reach substantial thicknesses as it is created by surface flooding caused by successive tidal cycles. Shorefast ice may interact with an offshore structure should it float free and get carried out by the tides. (Blenkarn 1970) reported that shorefast ice will usually float free (due to buoyancy) after it reaches about 0.5 m in thickness. (Blenkarn 1970) also reported that "beach ice pieces 30 inches thick have been observed when ice grown afloat in the Inlet was less than 16 inches thick". (Whitney 2002) reported that shorefast ice up to about 10 m thick could get formed and floated out into the shipping lanes.

5.4.4.5 Ice Thickness

The ice thickness is difficult to estimate because it is affected by many factors, and the thickness varies within the ice pack. Thermal growth generally causes the ice thickness to increase over the winter. However, the relationship is not direct for Upper Cook Inlet. For example, measurements showed that the ice thickness at the platform location over the 1964-65 winter peaked at about 0.7 m in early January and it declined after that, as illustrated in Figure 5.13.

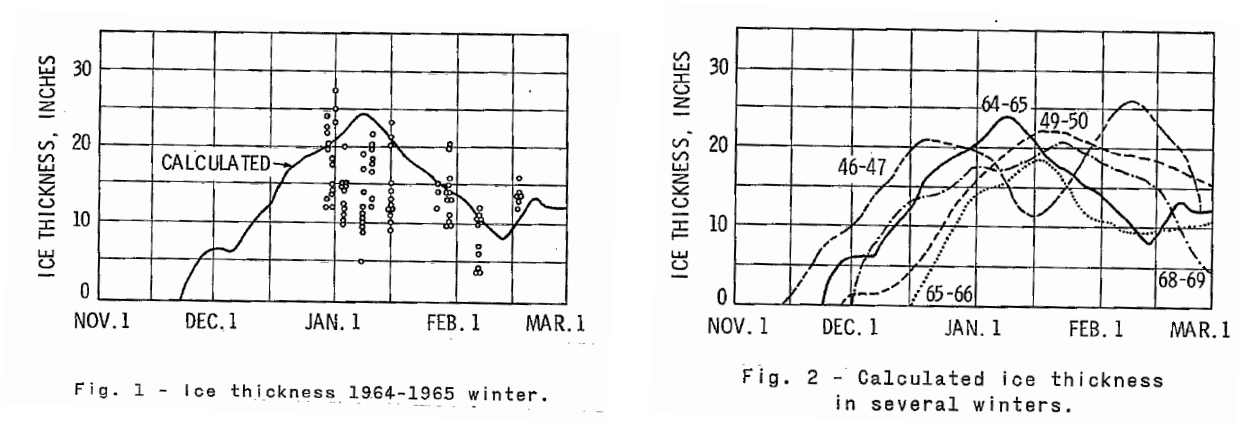


Figure 5.13: Ice Thickness at the Platform (Blenkarn 1970)

(Blenkarn 1970) compared the ice thickness data to the FDDs using a base reference of -6.7 °C (or, 20 F), based on the fact that ice does not generally form in Cook Inlet until the air temperature drops below about 20 F. Although this provided some correlation to the measured thicknesses, substantial variation still remained, as illustrated in Figure 5.13. Nevertheless, (Blenkarn 1970) found that this index provided a useful means of comparison for various winters.

These results show that the relationship is complex. Tidal action causes shoreline flooding as well as extensive ice movements, which act to flush ice in and out of Cook Inlet. This causes significant variability in ice thicknesses in Cook Inlet, as ice may continually re-form, depending on the air temperatures. The ice thickness is also affected by the ice dynamics which causes features such as rafting and ridging to form, thereby producing thicker zones in the ice pack.

As a result, one must take care in order to establish an appropriate ice thickness for the purpose of ice-induced fatigue analyses. As noted in (Bhat and Cox 1995) structures in Upper Cook Inlet may "see" rafted ice, ridges, and stamukhi as well as floe ice. (Blenkarn 1970) reported that ice ridges up to 6 m thick have been seen in Cook Inlet. For a detailed ice engineering investigation of ice-induced fatigue, one would need to evaluate the contribution of each ice type to the total accumulated ice-induced fatigue. That has not been done here because this is a demonstration project only; and, extensive analyses including probably the collection of site-specific data would be required.

A subjective approach has been used here to bridge this gap in order to allow the analyses to proceed. It was assumed that:

a. The majority of the ice "seen" by the structures in a winter would be floe ice of uniform thickness. It was assumed that 80% of the total ice movement length "seen" by a structure would be floe ice.

b. The remaining 20% of the total ice "seen" by the structure would be rafted ice.

c. Ridges would not be a significant factor causing ice-induced fatigue. This assumption was primarily made on the premises that:

   i. ridging is expected to be less extensive than the other ice types above;

   ii. ridges will mainly be present at the edges of the floes (as seen in Figure 5.12), which will mitigate against steady-state crushing loads being developed by them during an interaction with a structure; and,

   iii. ridges are expected to be mainly unconsolidated given the dynamic nature of the ice conditions. This would reduce the ridge loads that are possible.

Of course, these assumptions would need to be verified for an engineering design project.

Floe ice thicknesses were established as listed in Table 5.8 and using the empirical data and analyses of (Blenkarn 1970) illustrated in Figure 5.13.

Table 5.8: Floe Ice Thicknesses for Upper Cook Inlet

Period	Mean Thickness[1], m	Ice Thickness Range[1], m
Dec. 1 -15	0.20	0 – 0.30
Dec. 16-31	0.38	0.20 – 0.50
Jan. 1-15	0.45	0.35 – 0.60
Jan. 16-31	0.45	0.35 – 0.60
Feb. 1-15	0.40	0.25 – 0.60
Feb. 16-28/29	0.30	0.20 – 0.45
Mar. 1-15	0.30	0.20 – 0.45
Mar. 16-31	0.20	0 – 0.30

Notes:

1. These values are not intended to include rafted ice or ridges within a floe.

Rafted ice thicknesses were established taking into account the fact that rafting is a process that is limited to thin ice. In general, rafting is not considered possible for ice thicknesses exceeding about 15 cm (CIS 2005), although recent observations in the North Caspian Sea have shown that extensive rafting can occur for thicknesses up to about 0.5 m. Rafted ice thicknesses were established for the analyses using judgment in combination with the summary values in Table 5.3, and the design criteria established for the oil and gas platforms in Upper Cook Inlet (Table 5.4 to Table 5.6). Rafted ice thicknesses are presented in the respective sections for each code. Clearly, these assumptions would need to be verified for an engineering design project.

5.4.4.6 Ice Strength

Because the ice input data requirements regarding ice strength vary among the three codes, this section only presents site-specific information for Upper Cook Inlet. Ice strength inputs for the various codes were established individually and are presented in the respective sections.

1. Site-Specific Investigations: Ice Compressive Strength

For an evaluation of extreme ice loads on platforms in Upper Cook Inlet, (Bhat and Cox 1995) evaluated the ice compressive strength profile based on the ice salinity and temperature. Their analyses were done for a 0.65 m thick ice floe, as this is the expected thickness that an ice floe would reach based on its maximum residence time in the Inlet (i.e., about 30 days by (Blenkarn 1970); and the accumulated Freezing Degree Days (FDDs) within that time.

The analyses by (Bhat and Cox 1995) were done based on an ice surface temperature of -3 °C as the peak ice loads measured on the platforms generally occurred for this ice surface temperature. Furthermore, the analyses were done for a strain rate of 10^{-3}, as the ductile-brittle transition occurs at this approximate strain rate, leading to the maximum ice compressive strength.

They determined a depth-averaged value of 2.22 MPa for the ice compressive strength for 0.65 m thick ice, as illustrated in Figure 5.14.

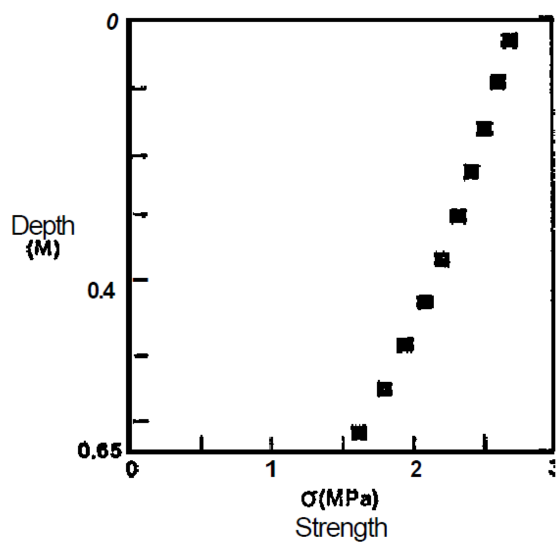


Figure 5.14: Strength Profile of an Ice Sheet in Upper Cook Inlet (Bhat and Cox 1995)

2. Site-Specific Investigations: Indentation Formula Factors

Various factors must be specified in order to use the ice load algorithm in IEC 61400 (i.e., k_1, k_2, and k_3 by Equation 5.1). It is of interest to compare the values suggested in IEC 61400 with those determined by (Bhat and Cox 1995) and by (Blenkarn 1970) based on the ice loads measured on Cook Inlet platforms, as listed in Table 5.9. (Bhat and Cox 1995) determined a value of 0.697 for the product of the indentation factors (i.e., $k_1k_2k_3$, using IEC 61400's terminology; or I_{mk} using their terminology). (Blenkarn 1970) determined a value of 0.55 for I_{mk} based on other ice load measurements on Cook Inlet platforms.

Table 5.9: Comparison of Values for Indentation Formula Factors

Parameter and Recommendation by IEC 61400		Terminology and Results from (Bhat and Cox 1995)	
Shape Factor, k_1	1.0: rectangular shape 0.9: circular shape	Shape Factor, m	0.9
Contact Factor, k_2	0.5: ice continuously moving 1.0: ice adfrozen to structure 1.5: ice bustle formed around structure	Contact Factor, k	0.5
Indentation Factor, k_3	Defined by the formula: $k_3 = (1+5*h/D)^{0.5}$	Indentation Factor, I	Defined by the formula: $I = (1+5*h/D)^{0.5}$; calculated to be 1.327

5.4.4.7 Ice Movements and the Total Length of Ice Travel Distance at the Structure

The total length of ice travel distance at the structure may be limited by various factors such as:

a. The ice movement rate and the environmental forces causing ice movements (winds, currents): Cook Inlet is a tidal estuary which leads to extensive ice movements. There are two tide cycles each day (i.e., two floods and two ebbs), which produces tidal currents up to about 1 m/s to 2 m/s, as shown in Figure 5.15. For the Upper Cook Inlet, the currents are "rectilinear or reversing"; and, as a result, they go slack during the period when the tide reverses (Mulherin, et al. 2001).

   As a result, ice in the Inlet is quite mobile, and it regularly moves in and out of the inlet due to tidal currents. In conclusion, this is considered unlikely to limit the total length of ice travel distance at the structure.

   However, as a general statement, this limit should be checked, and an engineering design project would probably assess this potential limit as well.

b. The likelihood that ice will contact the structure and fail against it: This is controlled by many factors, such as the probabilities that:

   (i) ice is present in Upper Cook Inlet in sufficient concentration to allow ice impacts;

   (ii) ice moves and contacts the structure; and

   (iii) the environmental driving forces are sufficient to allow enough force to be built up to cause the ice to fail mechanically (in crushing for a vertical structure).

This limit will probably control the total length of ice travel distance at the structure for Upper Cook Inlet. This is supported by past experience with ice field measurement programs at the Cook Inlet structures. (Visser 1992) reported that "little useful data" were obtained from several ice force measurement programs because the conditions were mild.

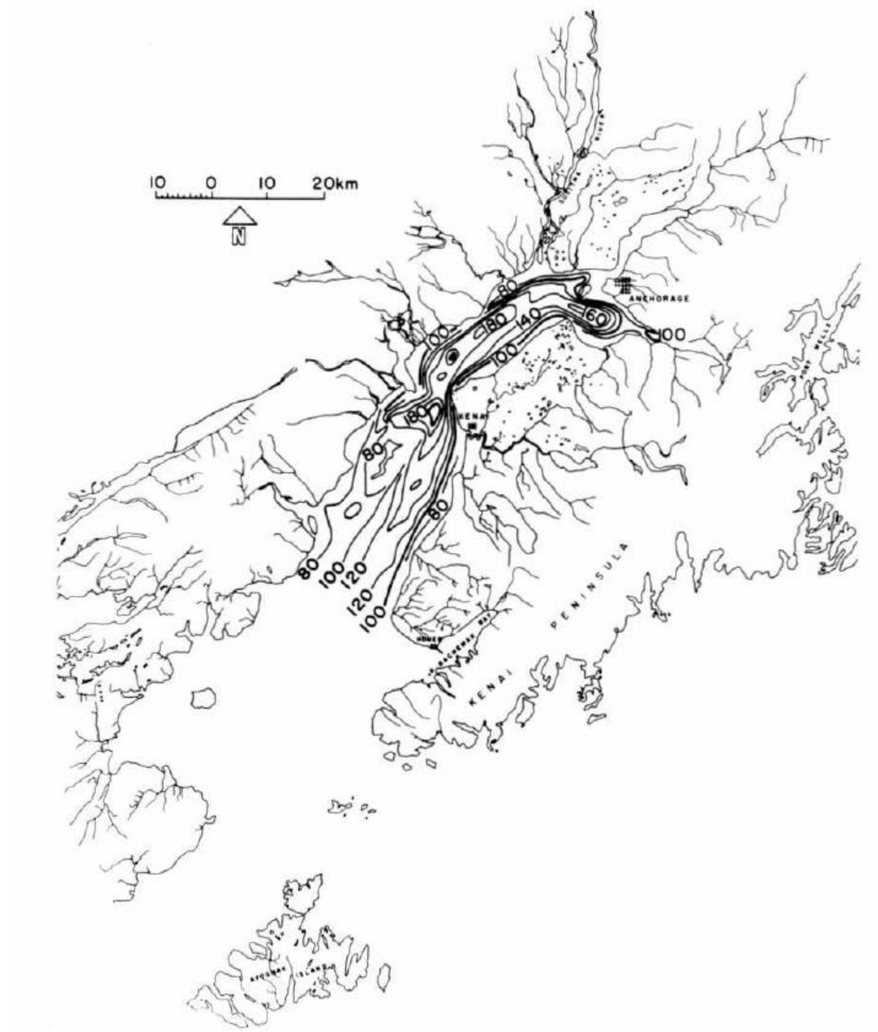


Figure 5.15: Tidal Currents (cited by Gallo, 1976)

A detailed analysis of ice movement patterns and magnitudes would be required in the conduct of an engineering design study aimed at investigating ice-induced fatigue accumulation. Because the analyses here were only aimed at demonstrating the application of the three codes, detailed ice movement analyses were not carried out; and preliminary assessments were made augmented by judgment where necessary. However, it should be recognized that this would be the subject of substantial investigation during an engineering design study.

5.5 Ice Load Time History Obtained Using IEC 61400

5.5.1 Overview of the Method in IEC 61400 for Calculating Dynamic Ice Loads

5.5.1.1 Ice Load Cases

IEC 61400 specifies various load cases depending on the ice feature, the ice interaction scenario and the limit state, as listed in Table 5.10. IEC 61400 states these load cases must be considered.

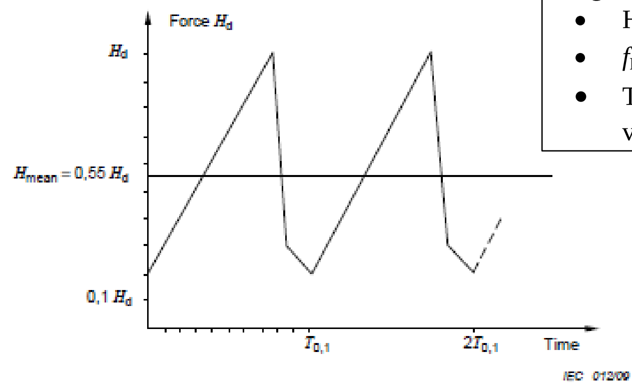
Only load cases E4 and E7 are applicable to the fatigue limit state. Both of them refer to "horizontal loads from moving ice floe at relevant velocities". Thus, it is only necessary to specify floe ice conditions for a given site in order to apply IEC 61400 with respect to ice-induced fatigue loadings.

5.5.1.2 Ice Load Time History for Dynamic Loadings

IEC 61400 specifies that the ice load time history may be presumed to have either a sinusoidal or a saw-tooth shape, as illustrated in Figure 5.16. A saw-tooth shape was assumed here as this is considered to be more realistic.

Table 5.10: Ice Loading Cases Specified in IEC 61400

Design situation	DLC	Ice condition	Wind condition	Water level	Type of analysis	Partial safety factor
Power production	E1	Horizontal load from temperature fluctuations	NTM $V_{hub} = V_r \pm 2$ m/s and V_{out} Wind speed resulting in maximum thrust	NWLR	U	N
	E2	Horizontal load from water fluctuations or arch effect	NTM $V_{hub} = V_r \pm 2$ m/s and V_{out} Wind speed resulting in maximum thrust	NWLR	U	N
	E3 For extrapol-ation of extreme events	Horizontal load from moving ice floe at relevant velocities $H = H_{50}$ in open sea $H = H_m$ for land-locked waters	NTM $V_{hub} = V_r \pm 2$ m/s and V_{out} Wind speed resulting in maximum thrust	NWLR	U	N
	E4	Horizontal load from moving ice floe at relevant velocities $H = H_{50}$ in open sea $H = H_m$ for land-locked waters	$V_{in} < V_{hub} < V_{out}$	NWLR	F	*
	E5	Vertical force from fast ice covers due to water level fluctuations	No wind load applied	NWLR	U	N
Parked	E6	Pressure from hummocked ice and ice ridges	EWM Turbulent wind model $V_{hub} = V_1$	NWLR	U	N
	E7	Horizontal load from moving ice floe at relevant velocities $H = H_{50}$ in open sea $H = H_m$ for land-locked waters	NTM $V_{hub} < 0,7\ V_{ref}$	NWLR	F	*



Legend:

- H_d, static, or quasi-static ice load
- f_N, Natural frequency of structure
- Time scale is based on f_N for vertical structures

Figure E.2 – Serrated load profile ($T_{0,1}$ = 1/f_N or 1/f_b)

Figure 5.16: Saw-Tooth Shaped Ice Load Time History Recommended by IEC 61400

5.5.1.3 The Static Ice Load, "H_d" for a Vertical Structure

IEC 61400 advises that the static ice load may be calculated using Equation 5.1 using the inputs listed in Table 5.11.

Table 5.11: Inputs for Static Ice Loads for Vertical Structures Using IEC 61400

Parameter	Recommended by IEC 61400	Values Used Here
Shape Factor, k_1	1.0: rectangular shape 0.9: circular shape	0.9 – circular monopile assumed
Contact Factor, k_2	0.5: ice continuously moving 1.0: ice adfrozen to structure 1.5: ice bustle formed around structure	0.5 – ice may be presumed to be continuously moving in Cook Inlet
Indentation Factor, k_3	Defined by the formula: $k_3 = (1+5*h/D)^{0.5}$	Varied depending on "h"
Ice Crushing Strength, σ_c in MPa	3.0: ice movements during the coldest time of year 2.5: slow ice movements (e.g., caused by thermal expansion) 1.5: ice movements in spring with ice temperature near melting point 1.0: partly deteriorated ice with ice temperature near melting point 0.5: moving saline 1st year ice in the open sea	Varied with time during the winter from 2.2 to 1.0 MPa

5.5.1.4 The Time Scale for a Loading Cycle

IEC 61400 advises that for a vertical structure, the period of a loading cycle should be taken as 1/f_N, where f_N is the wind turbine structural eigenfrequency, for either the first or the second modes. This is advised in IEC 61400 regardless of whether or not the criterion in it for tuning is met.

The analyses were based on a value of f_N equal to 10 Hz as this is the presumed natural frequency of the hypothetical structure analyzed here (as identified in Section 5.3).

5.5.2 Ice Inputs

5.5.2.1 Ice Floe Thickness

IEC 61400 specifies that for "open sea" conditions, the 50-year ice thickness should be used for fatigue limit analyses. Because Upper Cook Inlet may be considered to have "open sea" conditions, the 50-year floe ice thickness should be used here.

The ice floe thickness for Upper Cook Inlet has been considered in Section 5.4. It depends on the time in the winter as well as whether or not the floe contains rafted ice or ridges. Analyses showed that the floe ice thickness varied from winter to winter, and the return period associated with the values in Section 5.4 is not clear. In practice, probabilistic analyses would be required to determine this, and quite possibly, this would include the collection of site-specific data. These were not done as the objective of the analyses here was solely to demonstrate the application of IEC 61400. Instead, ice thicknesses were established for the analyses using judgment, based on the data and information in Section 5.4. Table 5.12 summarizes the ice thicknesses used for the analyses here. It should be noted that for consistency, the same ice thicknesses were used for evaluations of IEC 61400, DNV-OS-J101 and ISO 19906.

Table 5.12: Floe Ice Thicknesses Used for Analyses

Period	Sheet Ice Thickness[1,2], m	Rafted Ice Thickness[3], m
Dec. 1 -15	0.30	0.80
Dec. 16-31	0.50	1.4
Jan. 1-15	0.60	1.5
Jan. 16-31	0.60	1.5
Feb. 1-15	0.60	1.5
Feb. 16-28/29	0.45	1.2
Mar. 1-15	0.45	1.2
Mar. 16-31	0.30	0.80

Notes:

1. These values are not intended to include rafted ice or ridges within a floe.
2. The sheet ice thicknesses are the upper-range values listed in Table 5.8. These were used in an effort to produce an upper-range thickness.

Rafted ice thicknesses were established using judgment. Efforts were made to bring the rafted ice thicknesses into general agreement with the rafted ice thicknesses in Table 5.3, as well as with the design ice thicknesses used for the oil and gas platforms in Upper Cook Inlet (Table 5.4 to Table 5.6).

5.5.2.2 Ice Strength

The ice load algorithm in IEC 61400 specifies a range of ice strengths to be used as inputs depending on the ice conditions. See Table 5.11. Of course, the ice strength varies over the winter, and this trend should be included in the ice load analyses.

A detailed analysis would be required to establish the ice strength variation over the winter for an engineering design study aimed at investigating ice-induced fatigue accumulation. Because that is not the objective here (which is only aimed at demonstrating the application of IEC 61400), detailed ice strength analyses were not carried out.

Instead, simple assessments were made using judgment based on the experience and analyses to date, as described in Section 5.4. It is noted that the highest loads were measured on the platforms when the ice was relatively warm with a surface temperature of about -3 °C. (Bhat and Cox 1995) determined an ice compressive strength of 2.22 MPa for that condition (Section 5.4). It is further noted that the ice season extends from about December to March.

The following values were selected for the ice strength for use in Equation 5.1, taking into account the recommended ice strength values for by Table 5.11:

(a) December: 1.0 MPa;

(b) January: 2.2 MPa;

(c) February: 2.2 MPa; and,

(d) March: 1.0 MPa.

5.5.2.3 Ice Movements and the Number of Loading Events

For IEC 61400, the period of a loading cycle for a vertical structure is governed by the structure's properties (e.g., Figure 5.16). In this case, the number of ice load cycles will be entirely controlled by the length of time that the structure is loaded by ice over the winter.

As discussed in Section 5.4, extensive ice movements occur in Upper Cook Inlet. As a result, the length of time that the structure will be loaded by ice will be governed by the contact probabilities for ice with the structure. Extensive analyses would be required to establish this, and detailed investigations would be undertaken in an engineering design project, including probably, the collection of site-specific data.

Extensive analyses were not done here as these analyses were only intended to demonstrate the application of IEC 61400. The problem was simplified by assuming that the contact probability for ice with the structure during a given time period in the winter would be governed by the mean ice concentration in Upper Cook Inlet for that time. This led to the results summarized in Table 5.13.

Table 5.13: Length of Time That Ice is in Contact with the Structure

Period	No. of Days	Mean Ice Concentration[1] (rounded up)	Prorated No. of Days with Ice Contact
Dec. 1 -15	15	0	0
Dec. 16-31	16	0	0
Jan. 1-15	15	4/10	6
Jan. 16-31	16	5/10	8
Feb. 1-15	15	6/10	9
Feb. 16-28/29	13	6/10	7.8
Mar. 1-15	15	6/10	9
Mar. 16-31	16	4/10	6.4

Notes:

1. The mean ice concentration is given in Table 5.7.

It should be noted that this is an oversimplification as experience shows that the contact probabilities are sensitive to the ice concentration in a more complex manner. However, the assumption made here allowed the results to proceed, and helps to highlight the issues that would be involved in an engineering design investigation.

5.5.3 Ice Load Calculations

5.5.3.1 Static Ice Load

This was calculated for nominal 15-day periods from December 16 to March 31 using the Equation 5.1 by IEC 61400. The static ice loads for sheet ice varied from 0.8 MN to 3.8 MN, as listed in Table 5.14; and, from 2.4 MN to 11.7 MN for rafted ice as listed in Table 5.15.

Table 5.14: Static Ice Loads Given by IEC 61400 for Sheet Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Ice Thickness, m	0.50	0.60	0.60	0.60	0.45	0.45	0.30
Conductor Diameter, m	5.00	5.00	5.00	5.00	5.00	5.00	5.00
Ice Crushing Strength, MPa	1.00	2.20	2.20	2.20	2.20	1.00	1.00
Shape Factor, K_1	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Contact Factor , K_2	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Aspect Ratio Factor , K_3	1.22	1.26	1.26	1.26	1.20	1.20	1.14
Ice Load, MN	1.38	3.76	3.76	3.76	2.68	1.22	0.77

Table 5.15: Static Ice Loads Given by IEC 61400 for Rafted Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Ice Thickness, m	1.40	1.50	1.50	1.50	1.20	1.20	0.80
Conductor Diameter, m	5.00	5.00	5.00	5.00	5.00	5.00	5.00
Ice Crushing Strength, MPa	1.00	2.20	2.20	2.20	2.20	1.00	1.00
Shape Factor, K_1	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Contact Factor , K_2	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Aspect Ratio Factor , K_3	1.55	1.58	1.58	1.58	1.48	1.48	1.34
Ice Load, MN	4.88	11.74	11.74	11.74	8.81	4.00	2.41

5.5.3.2 Dynamic Ice Load Time History

This was established based on the saw-tooth load cycle profile given in IEC 61400 and illustrated in Figure 5.16, and a natural frequency for the structure of 10 Hz. The parameters for the time history for a single loading cycle are summarized in Table 5.16 and Table 5.17 for sheet ice and rafted ice, respectively.

Table 5.16: Dynamic Ice Load Time History Given by IEC 61400 for Sheet Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Maximum Load, MN	1.38	3.76	3.76	3.76	2.68	1.22	0.77
Minimum Load, MN	0.28	0.75	0.75	0.75	0.54	0.24	0.15
Load Variation Per Cycle, MN	1.10	3.01	3.01	3.01	2.15	0.98	0.62
Period of Load Cycle, sec	0.1	0.1	0.1	0.1	0.1	0.1	0.1

Table 5.17: Dynamic Ice Load Time History Given by IEC 61400 for Rafted Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Maximum Load, MN	4.88	11.74	11.74	11.74	8.81	4.00	2.41
Minimum Load, MN	0.98	2.35	2.35	2.35	1.76	0.80	0.48
Load Variation Per Cycle, MN	3.90	9.39	9.39	9.39	7.05	3.20	1.93
Period of Load Cycle, sec	0.1	0.1	0.1	0.1	0.1	0.1	0.1

Of course, the load cycle varied between sheet ice and rafted ice as rafted ice is substantially thicker than sheet ice. Also, the load cycle changed over the winter as both the ice thickness and the ice strength varied over the winter.

5.5.3.3 Accumulated Ice Load Reversals During a Winter

The number of events during a winter was established based on the amount of time that the ice is in contact with the structure (as listed in Table 5.13), as well as the period of each load cycle. The numbers of loading cycles in a winter are summarized in Table 5.18 and Table 5.19 for sheet ice and rafted ice, respectively.

Table 5.18: Number of Load Cycles Given by IEC 61400 for Sheet Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Prorated No. Of Days with	0	6	8	9	7.8	9	6.4
Ice Contact (Table 5.4)							
Proportion of Total Ice	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Movement That is Sheet Ice							
Time per Load Cycle, sec	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Number of Events	0	4147200	5529600	6220800	5391360	6220800	4423680
Load Variation per Cycle, MN	1.10	3.01	3.01	3.01	2.15	0.98	0.62

Table 5.19: Number of Load Cycles Given by IEC 61400 for Rafted Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Prorated No. Of Days with	0	6	8	9	7.8	9	6.4
Ice Contact (Table 5.4)							
Proportion of Total Ice	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Movement That is Rafted Ice							
Time per Load Cycle, sec	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Number of Events	0	1036800	1382400	1555200	1347840	1555200	1105920
Load Variation per Cycle, MN	3.90	9.39	9.39	9.39	7.05	3.20	1.93

The composite S-N curve for sheet ice and rafted ice is shown in Figure 5.17. There is a clear distinction between the rafted ice and the sheet ice events. The rafted ice events produced larger load reversals although there were fewer cycles. The sheet ice events produced lower load reversals with a larger number of cycles. This variation resulted from the facts that:

a. Rafted ice loads were much higher than those for sheet ice; and,

b. Rafted ice constituted a smaller proportion of the total ice movement "seen" by the structure. (It was assumed that rafted ice constituted only 20% of the total ice "seen" by the structure).

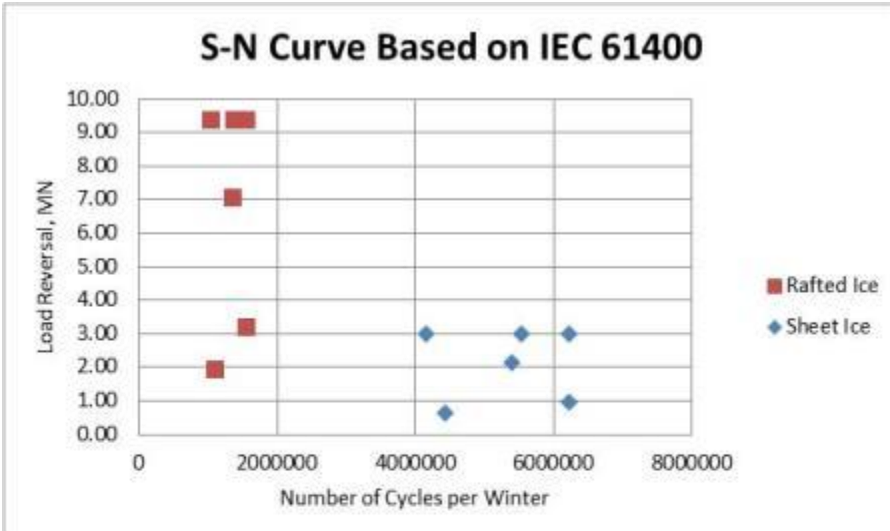


Figure 5.17: Number of Load Reversals for a Winter

5.5.4 Assessments Regarding the Application of IEC 61400

5.5.4.1 Ice Inputs and Their Availability

The key ice inputs for IEC 61400 are:

(a) the ice thickness and its variation over the winter;

(b) the ice strength; and

(c) the length of time that ice will be in contact with the structure during a winter period.

1. Ice Thickness and Variation Over the Winter

The ice type to be considered for the fatigue limit state is prescribed in IEC 61400 (i.e., a moving ice floe). This reduces the amount of investigation that is required in order to define the ice thickness although, as demonstrated here, it still requires the user to consider the possibility that the moving ice floe may contain rafted ice and/or ridges. Judgments were required here in order to proceed, such as the significance of ridges, and the relative proportions of sheet ice and rafted ice.

However, experience shows that this information can be obtained and synthesized into a set of design criteria, although it can require significant investigation, which often includes the collection of site-specific data. It is our assessment that this information can be obtained and thus will not limit the application of IEC 61400.

2. Ice Strength

The ice strengths to be used in IEC 61400 are prescribed based on general environmental conditions. While judgment is required to apply the recommendations in IEC 61400, it is our assessment that this information requirement will not limit the application of IEC 61400.

3. Length of Time that Ice is in Contact With the Structure During a Winter Period

This controls the number of loading cycles for a vertical structure, and is hence a very important ice input. Of all the ice inputs for IEC 61400, this would merit and require the greatest amount of investigation. As demonstrated here, several simplifying assumptions were required in order to proceed. In an engineering design project, these would be the subject of substantial investigation. Nevertheless, it is our assessment that this information requirement will not limit the application of IEC 61400.

5.5.4.2 Results

The resulting S-N curve spanned a wide range due to the inclusion of both rafted ice and sheet ice. The rafted ice events produced larger load reversals although there were fewer cycles. The sheet ice events produced lower load reversals with a larger number of cycles. This trend is considered to be realistic.

5.6 Ice Load Time History obtained using DNV-OS-J101

5.6.1 Overview of Method in DNV-OS-J101 for Calculating Dynamic Ice Loads

5.6.1.1 Ice Load Cases

DNV-OS-J101 defines various load cases that should be considered depending on the ice feature, the ice interaction scenario and the limit state, as listed in Table 5.20

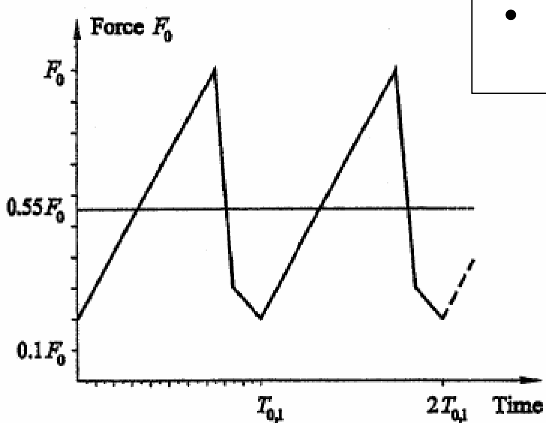
Only load cases E4 and E7 are applicable to the fatigue limit state. Both of them refer to "horizontal loads from moving ice floe". Thus, it is only necessary to specify floe ice conditions for a given site in order to apply DNV-OS-J101 with respect to ice-induced fatigue loadings.

5.6.1.2 Ice Load Time History for Dynamic Loadings

DNV-OS-J101 advises that for dynamic loadings, the ice load time history may be presumed to have a saw-tooth shape, as illustrated in Figure 5.18.

Table 5.20: Ice Loading Cases Specified in DNV-OS-J101

Table E3 Proposed load cases combining ice loading and wind loading						
Design situation	*Load case*	*Ice condition*	*Wind condition: Wind climate (U_{10hub})*	*Water level*	*Other conditions*	*Limit state*
Power production	E1	Horizontal load due to temperature fluctuations	$v_{in} < U_{10hub} < v_{out}$ + NTM 10-minute mean wind speed resulting in maximum thrust	1-year water level		ULS
	E2	Horizontal load due to water level fluctuations or arch effects	$v_{in} < U_{10hub} < v_{out}$ + NTM 10-minute mean wind speed resulting in maximum thrust	1-year water level		ULS
	E3	Horizontal load from moving ice floe Ice thickness: $t_C = t_{50}$ in open sea $t_C = t_{limit}$ in land-locked waters	$v_{in} < U_{10hub} < v_{out}$ + ETM 10-minute mean wind speed resulting in maximum thrust	50-year water level	For prediction of extreme loads	ULS
	E4	Horizontal load from moving ice floe Ice thickness: $t_C = t_{50}$ in open sea $t_C = t_{limit}$ in land-locked waters	$v_{in} < U_{10hub} < v_{out}$	1-year water level		FLS
	E5	Vertical force from fast ice covers due to water level	No wind load applied	1-year water level		ULS
Parked (standing still or idling)	E6	Pressure from hummocked ice and ice ridges	Turbulent wind $U_{10hub} = U_{10,50\text{-}yr}$ + characteristic standard deviation of wind speed $\sigma_{U,c} = 0.11 \cdot U_{10hub}$	1-year water level		ULS
	E7	Horizontal load from moving ice floe Ice thickness: $t_C = t_{50}$ in open sea $t_C = t_{limit}$ in land-locked waters	$U_{10hub} < 0.7U_{10,50\text{-}yr}$ + NTM	1-year water level		FLS



Legend:
- F_o, Static or quasi-static horizontal ice load

Figure 5.18: Saw-Tooth Shaped Ice Load Time History Advised by DNV-OS-J101

DNV-OS-J101 states that the above load time history is applicable should the tuning criterion be met. The tuning criterion is defined in Equation 5.7 (repeated from Equation 5.5 for clarity):

$$\frac{U_{ice}}{(h \times f_N)} > 0.3 \qquad \text{Eqn. 5.7}$$

Where,

- U_{ice} = the ice movement speed during interaction with the structure;
- h = the ice thickness; and,
- f_N = the natural frequency of the structure.

DNV-OS-J101 does not explicitly state what ice load time history should be presumed should the tuning criterion not be met. It is our interpretation of DNV-OS-J101 that no allowance for ice-induced fatigue would need to be made in that case, and ice-induced fatigue has been evaluated here on that basis.

5.6.1.3 The Static Ice Load, "F_o" for a Vertical Structure

DNV-OS-J101 states that F_o shall be taken as the "static horizontal ice load". It advises that the static horizontal ice load may be calculated using the approach in API RP-2N. It also advises the user to consult ISO 19906 regarding ice loads. Thus, it is our interpretation that DNV-OS-J101 can be satisfied by calculating static horizontal ice loads using either API RP-2N or ISO 19906.

In this project, static ice loads have been calculated using the approach in ISO 19906, which advises that global pressures may be calculated using Equation 5.3.

5.6.1.4 The Time Scale for a Loading Cycle for a Vertical Structure

In the case where the tuning criterion is met, DNV-OS-J101 advises that the "load is applied with a frequency that corresponds to the natural frequency of the structure".

5.6.2 <u>Ice Inputs</u>

5.6.2.1 Ice Floe Thickness

DNV-OS-J101 advises that for "open sea" conditions, the 50-year ice thickness should be used for fatigue limit analyses. Because Upper Cook Inlet may be considered to have "open sea" conditions, the 50-year floe ice thickness should be used here.

The ice floe thickness for Upper Cook Inlet has been considered in Section 5.4. It depends on the time in the winter as well as whether or not the floe contains rafted ice or ridges. Analyses showed that the floe ice thickness varied from winter to winter, and the return period associated with the values in Section 5.4 is not clear. In practice, probabilistic analyses would be required to determine this, and quite possibly, this would include the collection of site-specific data. These were not done as the objective of the analyses here was solely to demonstrate the application of DNV-OS-J101.

Instead, ice thicknesses were established for the analyses using judgment, based on the data and information in Section 5.4. Table 5.21 summarizes the ice thicknesses used for the analyses.

Table 5.21: Floe Ice Thicknesses Used for Analyses

Period	Sheet Ice Thickness[1,2], m	Rafted Ice Thickness[3], m
Dec. 1 -15	0.30	0.80
Dec. 16-31	0.50	1.4
Jan. 1-15	0.60	1.5
Jan. 16-31	0.60	1.5
Feb. 1-15	0.60	1.5
Feb. 16-28/29	0.45	1.2
Mar. 1-15	0.45	1.2
Mar. 16-31	0.30	0.80

Notes:

1. These values are not intended to include rafted ice or ridges within a floe.
2. The sheet ice thicknesses are the upper-range values listed in Table 5.8. These were used in an effort to produce an upper-range thickness.

Rafted ice thicknesses were established using judgment. Efforts were made to bring the rafted ice thicknesses into general agreement with the rafted ice thicknesses in Table 5.3, as well as with the design ice thicknesses used for the oil and gas platforms in Upper Cook Inlet (Table 5.4 to Table 5.6).

It should be noted that for consistency, the same ice thicknesses were used for evaluations of IEC 61400, DNV-OS-J101 and ISO 19906.

5.6.2.2 Application of Tuning Criterion

The application of the tuning criterion was evaluated for the range of ice movement rates and thicknesses likely to occur in Upper Cook Inlet. It was presumed that the ice movement rate could vary from 0.3 m/s to 2 m/s, as this generally covers the range of tidal currents that occur in Upper Cook Inlet (refer to Section 5.4). Ice thicknesses covering the range in Table 5.21 were evaluated for both sheet ice and rafted ice. The natural frequency of the structure was taken as 10 Hz in keeping with the values in Section 5.3.

For sheet ice, it was found that the tuning criterion was only met for certain combinations of ice velocity and thickness, illustrated in Figure 5.19.

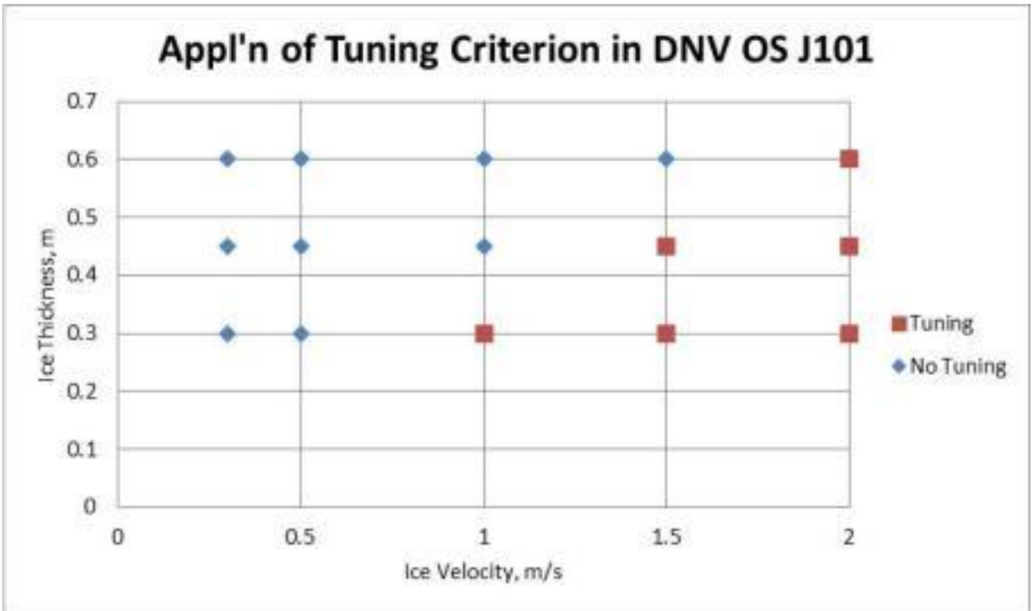


Figure 5.19: Application of Tuning Criterion in DNV-OS-J101 for Sheet Ice

For rafted ice, the tuning criterion was not met for any of the cases established here (i.e., ice velocities ranging from 0.3 m/s to 2 m/s; and ice thicknesses ranging from 0.8 m to 1.5 m as listed in Table 5.21). This indicates (in our opinion) that DNV-OS-J101 advises that ice-induced loads need not be considered for rafted ice for the test case being used here.

5.6.2.3 Ice Strength

Static horizontal ice loads were calculated using the algorithm in ISO 19906 (Equation 5.3) which is one of the cases allowed by DNV-OS-J101. The ice strength is included implicitly in this predictor through the parameter, C_R. ISO 19906 specifies C_R to be 2.8 MPa for Arctic conditions and 1.8 MPa for Baltic conditions. Adjustments can also be made using the methodology in ISO 19906 to account for other ice strengths by adjusting C_R based on the ratio of reference strengths (Equation 5.4).

For the analyses here, the Baltic C_R coefficient (of 1.8) was used as the FDDs for the Baltic are similar to those for Cook Inlet. As a result, the strength reduction factor (Equation 5.4) was assigned a value of 1.0.

The same strength coefficient was used for all periods of the winter because the ice loads measured on the Cook Inlet platforms showed that the peak loads occurred for relatively warm conditions, with an ice surface temperature of about -3^{o}C (Bhat and Cox 1995).

5.6.2.4 Ice Movements

Of course, the ice movement magnitudes and patterns affect the number of loading events that occur in a given winter. However, the analyses done using the tuning criterion in DNV-OS-J101 (in the previous section) have shown that only certain combinations of sheet ice thickness and ice velocity will cause the tuning criterion to be satisfied. Furthermore, the calculations indicated that tuning would not occur for rafted ice.

Hence, ice movement magnitudes and patterns need to be quantified for these specific combinations of sheet ice thickness and ice velocity. Furthermore, it is noted that the tidal currents in Upper Cook Inlet vary significantly within the Inlet (Section 5.4) so evaluations must be site-specific. This requires extensive analysis and the required information is not readily available for Upper Cook Inlet. For a detailed engineering design project, site-specific data would probably need to be collected in combination with analyses of existing data and imagery (e.g., satellite imagery).

Extensive analyses were not done as the analyses here were only intended to demonstrate the application of DNV-OS-J101. Thus, the problem was simplified by making the following assumptions:

a. The contact probability for ice with the structure during a given time period in the winter would be governed by the mean ice concentration in Upper Cook Inlet for that time. (Mean ice concentrations over various winter periods are summarized in Table 5.8).

b. The probability of tuning can be estimated based on the tide cycle (and the currents generated) as given below:

   i. 0.3 m ice thickness – tuning could over 50% of the tidal cycle

   ii. 0.45 m ice thickness – tuning could over 25% of the tidal cycle

   iii. 0.6 m – tuning could over 10% of the tidal cycle

This led to the results summarized in Table 5.22. The analyses suggested that ice-induced fatigue would only be possible for a total of 9.7 days over the winter.

Of course, the above approach is an oversimplification for various reasons. However, the assumptions made here allowed the results to proceed, and they help to highlight the issues that would be involved in an engineering design investigation.

Table 5.22: Length of Time For Which Ice-Induced Fatigue is Possible

Period	No. of Days	Mean Ice Conc'n (rounded up)	Sheet Ice Thickness m	Fraction of Tide Cycle with Tuning	Prorated No. of Days with Ice Contact & Tuning
Dec. 1 -15	15	0.00	0.3	0.5	0
Dec. 16-31	16	0.00	0.5	0.25	0
Jan. 1-15	15	0.40	0.6	0.1	0.6
Jan. 16-31	16	0.50	0.6	0.1	0.8
Feb. 1-15	15	0.60	0.6	0.1	0.9
Feb. 16-28/29	13	0.60	0.45	0.25	1.95
Mar. 1-15	15	0.60	0.45	0.25	2.25
Mar. 16-31	16	0.40	0.3	0.5	3.2

5.6.3 Ice Load Calculations

5.6.3.1 Static Ice Load

This was calculated for nominal 15-day periods from December 16 to March 31 using the algorithm of Equation 5.3 by ISO 19906. The static ice loads for sheet ice varied from 2.9 MN to 4.7 MN, as listed in Table 5.23.

Table 5.23: Static Ice Loads for DNV-OS-J101 for Sheet Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-3
Ice Thickness, m	0.50	0.60	0.60	0.60	0.45	0.45	0.30
Conductor Diameter, m	5.00	5.00	5.00	5.00	5.00	5.00	5.00
Cr, MPa	1.80	1.80	1.80	1.80	1.80	1.80	1.80
Strength Reduction Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Coefficient m	-0.16	-0.16	-0.16	-0.16	-0.16	-0.16	-0.16
Coefficient n	-0.40	-0.38	-0.38	-0.38	-0.41	-0.41	-0.44
Global Pressure, MPa	1.64	1.56	1.56	1.56	1.70	1.70	1.95
Ice Load, MN	4.11	4.67	4.67	4.67	3.82	3.82	2.92

Rafted ice loads were calculated for completeness although the analyses done using the tuning criterion in DNV-OS-J101 indicated that ice-induced fatigue need not be considered for rafted ice, for the test case here. Rafted ice loads varied from 5.8 MN to 9.9 MN, as listed in Table 5.24.

Table 5.24: Static Ice Loads Given for DNV-OS-J101 for Rafted Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Ice Thickness, m	1.40	1.50	1.50	1.50	1.20	1.20	0.80
Conductor Diameter, m	5.00	5.00	5.00	5.00	5.00	5.00	5.00
Cr, MPa	1.80	1.80	1.80	1.80	1.80	1.80	1.80
Strength Reduction Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Coefficient m	-0.16	-0.16	-0.16	-0.16	-0.16	-0.16	-0.16
Coefficient n	-0.30	-0.30	-0.30	-0.30	-0.30	-0.30	-0.34
Global Pressure, MPa	1.33	1.31	1.31	1.31	1.36	1.36	1.45
Ice Load, MN	9.29	9.86	9.86	9.86	8.14	8.14	5.79

5.6.3.2 Dynamic Ice Load Time History

This was established based on the saw-tooth load cycle profile given in IEC 61400 and illustrated in Figure 5.18, and a natural frequency for the structure of 10 Hz. The parameters for the time history for a single loading cycle are summarized in Table 5.25 for sheet ice. The load cycle changed over the winter as the ice thickness varied over the winter.

Table 5.25: Dynamic Ice Load Time History for DNV-OS-J101 for Sheet Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Maximum Load, MN	4.11	4.67	4.67	4.67	3.82	3.82	2.92
Minimum Load, MN	0.82	0.93	0.93	0.93	0.76	0.76	0.58
Load Variation Per Cycle, MN	3.29	3.74	3.74	3.74	3.06	3.06	2.34
Period of Load Cycle, sec	0.1	0.1	0.1	0.1	0.1	0.1	0.1

5.6.3.3 Accumulated Ice Load Reversals During a Winter

The number of events during a winter was established based on the amount of time that tuning would be possible, listed in Table 5.22 as well as the period of each load cycle. The numbers of loading cycles in a winter are summarized in Table 5.26 for sheet ice.

Table 5.26: Number of Load Cycles Given by DNV-OS-J101 for Sheet Ice

Parameter	Dec. 16-31	Jan. 1-15	Jan. 16-31	Feb. 1-15	Feb. 16-28	Mar. 1-15	Mar. 16-31
Prorated No. Of Days with	0	0.6	0.8	0.9	1.95	2.25	3.2
Ice Contact (Table 6.3)							
Proportion of Total Ice	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Movement That is Sheet Ice							
Time per Load Cycle, sec	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Number of Events	0	414720	552960	622080	1347840	1555200	2211840
Load Variation per Cycle, MN	3.29	3.74	3.74	3.74	3.06	3.06	2.34

The resulting S-N curve is shown in Figure 5.20. The number of load reversals was inversely related to the magnitude of the load reversal.

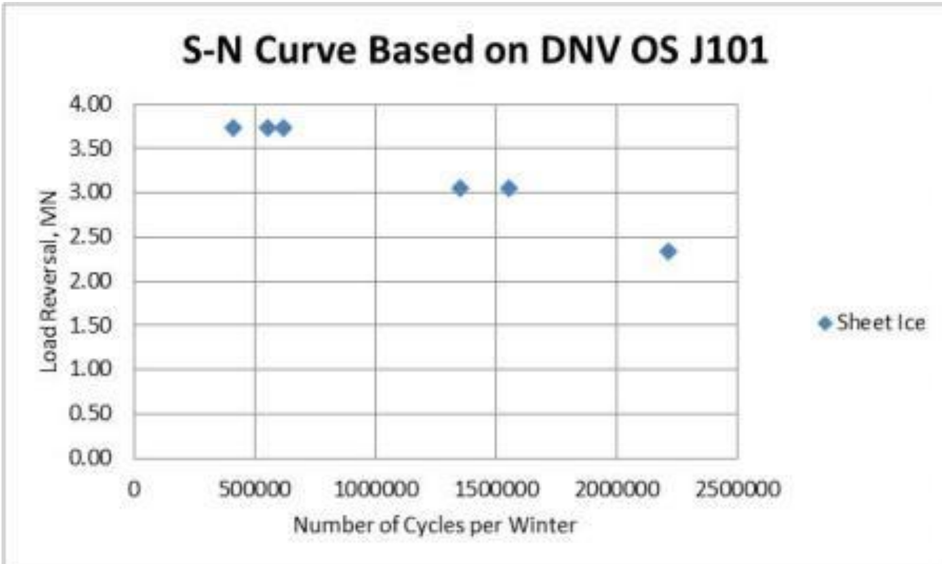


Figure 5.20: Number of Load Reversals for a Winter

5.6.4 Assessments Regarding the Application of DNV-OS-J101

5.6.4.1 Ice Inputs and Their Availability

The key ice inputs for DNV-OS-J101 are: (a) the ice thickness and its variation over the winter; (b) the ice strength; (c) the ice conditions causing the tuning criterion to be met; and, (d) the length of time that ice will be in contact with the structure during a winter period.

1. Ice Thickness and Variation Over the Winter

The ice type to be considered for the fatigue limit state is prescribed in DNV-OS-J101 (i.e., a moving ice floe). This reduces the amount of investigation needed to define the ice thickness although, as demonstrated here, it is necessary to consider the possibility that the moving ice floe may contain rafted ice and/or ridges. Judgments were required here in order to proceed, such as the significance of ridges, and the relative proportions of sheet ice and rafted ice.

However, experience shows that this information can be obtained and synthesized into a set of design criteria, although it can require significant investigation, which often includes the collection of site-specific data. It is our assessment that this information requirement will not limit the application of DNV-OS-J101.

All of the above findings and results are very similar to those for IEC 61400.

2. Ice Strength

The ice strengths required for DNV-OS-J101 depend upon the ice load algorithm that is utilized. For this demonstration, the algorithm in ISO 19906 was used. This required the ice strength parameter to be defined. While judgment is required to define this, it is our assessment that this information requirement will not limit the application of DNV-OS-J101.

A similar conclusion would be reached if static ice loads were defined using the indentation formula in API RP-2N, which is another case allowed by DNV-OS-J101.

Again, as an overall result, these findings are very similar to those for IEC 61400.

3. The Ice Conditions Causing the Tuning Criterion to be Met

The tuning criterion affects whether or not a particular case (i.e., ice movement velocity and ice thickness) is relevant for evaluations of ice-induced fatigue. This is a very significant parameter as it limits the number of cases, and hence loading events, that need to be included in an assessment of ice-induced fatigue. This difference (regarding the tuning criterion) is considered to be the most significant variation between DNV-OS-J101 and IEC 61400, as IEC 61400 required all ice loading cases to be considered for ice-induced fatigue.

While the cases meeting, and not meeting, the tuning criterion could be readily defined, several simplifying assumptions were necessary in this demonstration project in order to produce the inputs for the analyses. For an engineering design project, detailed analyses would be required, which would probably include the collection of site-specific data.

However, experience shows that this information can be obtained and synthesized into a set of design criteria. It is our assessment that this information requirement will not limit the application of DNV-OS-J101.

4. Length of Time that Ice is in Contact With the Structure During a Winter Period

This controls the number of loading cycles for a vertical structure, and is hence a very important ice input. As demonstrated here, several simplifying assumptions were required in order to proceed. In an engineering design project, these would be the subject of substantial investigation. Nevertheless, it is our assessment that this information requirement will not limit the application of DNV-OS-J101.

5.6.4.2 Results

The resulting S-N curve for DNV-OS-J101 indicated that the number of cycles was inversely related to the magnitude of the load reversal. This trend is considered to be logical.

The S-N curves for DNV-OS-J101 and IEC 61400 are compared in Figure 5.21. The S-N curve for DNV-OS-J101 spanned a narrower range than did that from IEC 61400. This is due to the fact that only certain cases for sheet ice needed to be included for DNV-OS-J101, as the others did not meet the tuning criterion. The variation is mainly due to differences with respect to rafted ice; these cases generated the highest ice loads and largest load reversals.

Furthermore, DNV-OS-J101 indicated significantly lower load reversal magnitudes were associated with a given number of cycles than did IEC 61400. Again, this variation is principally due to the fact that the rafted ice cases did not meet the tuning criterion in DNV-OS-J101, and thus they were excluded.

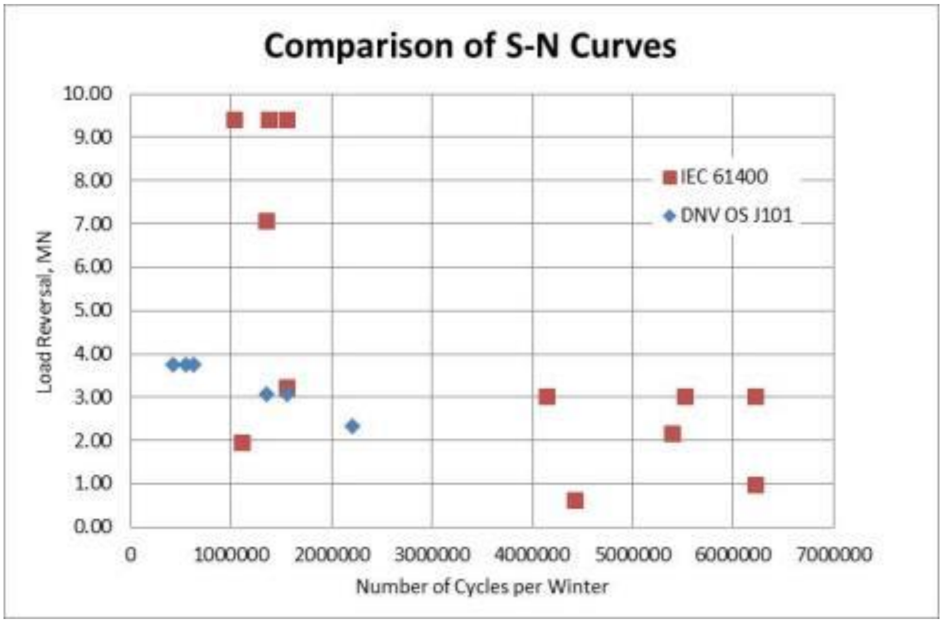


Figure 5.21: Number of Load Reversals for a Winter

5.7 Ice Load Time History obtained using ISO 19906

Only preliminary investigations could be performed for ISO 19906 for a number of reasons.

a. Detailed structural analyses would be required to apply ISO 19906. These are required for the criteria in it regarding the structure's susceptibility to frequency lock-in, as well as for an analysis of continuous brittle crushing. Of course, the analyses would have to be done in relation to a specific structure design. This was not possible here as a specific design for a wind platform structure was not available for this project.

b. Site-specific ice load data would be required to analyse a structure's response to continuous brittle crushing. Sources for such data are limited, and furthermore, their applicability to a structure in Upper Cook Inlet is questionable.

5.7.1 Overview of Method in ISO 19906 for Calculating Dynamic Ice Loads

5.7.1.1 Ice Load Cases

ISO 19906 does not specify the ice load cases that should be considered for the fatigue limit state although it does describe the types of ice-structure interactions that can occur, and the ice features of concern. The user must determine which ice load cases are relevant and include them appropriately for analyses of the fatigue limit state.

For this analysis, the ice load case of concern was presumed to be a moving ice floe, which may or may not include rafted ice and ridges. This selection was primarily made for the reasons given in Section 5.4. This selection also maintained consistency with the analyses done for IEC 61400 and DNV-OS-J101.

5.7.1.2 Ice Load Time History for Dynamic Loadings on Vertical Structures

ISO 19906 advises that three types of crushing patterns may occur during ice loadings on vertical structures as shown in Figure 5.22 (repeated from Section 5.2 for clarity).

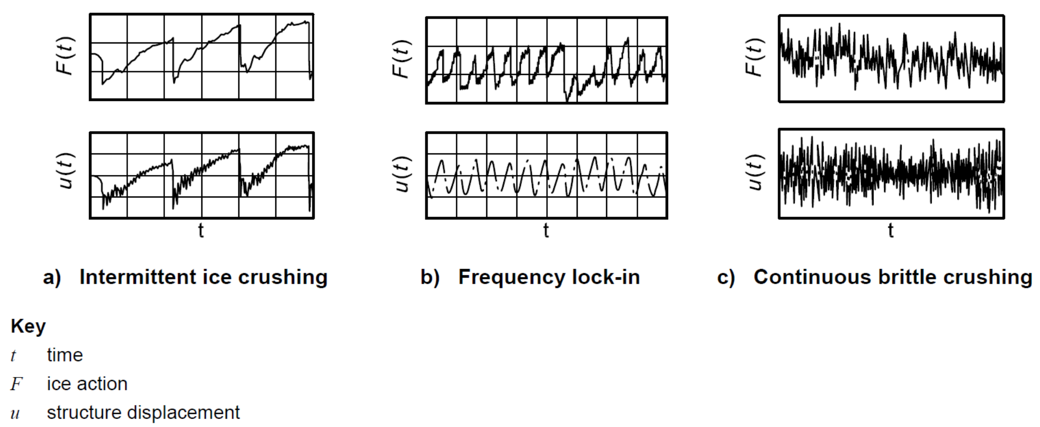


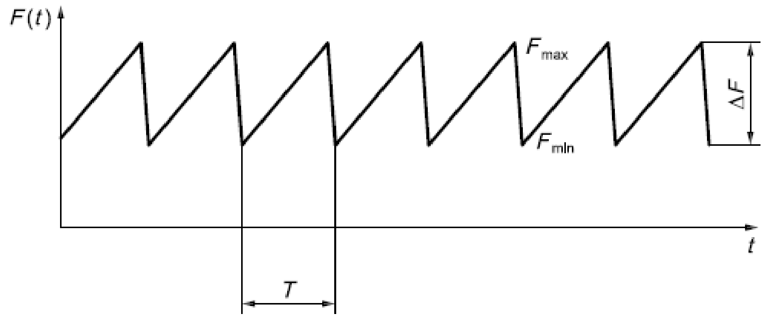
Figure 5.22: Ice Load Time Histories during Ice Interactions with Structures (ISO 19906)

1. Intermittent Ice Crushing

Intermittent ice crushing occurs for slow loadings, in which the period of the ice load is much longer than the longest natural period of the structure. This type of loading is not a practical case for structures in Upper Cook Inlet, as relatively rapid ice movement rates occur in the Inlet.

2. Frequency Lock-In

Frequency Lock-In (FLI) is potentially of concern and has been considered here. During FLI, the structure's response and the ice loading are tuned to each other. It is obvious that, of the three types of crushing, FLI is capable of producing the largest structural displacements. ISO 19906 advises that the ice load time history for FLI may be considered to have the form shown in Figure 5.23 (repeated from Section 5.2 for clarity).



Key

t	time
F	ice action
F_{max}	maximum value of ice action
F_{min}	minimum value of ice action
ΔF	difference between maximum and minimum values of ice action
T	period of ice action

Figure 5.23: Simplified Ice Forcing Function Given by ISO 19906 for Vertical Structures

3. Continuous Brittle Crushing

Continuous Brittle Crushing (CC) is also potentially of concern. It occurs at higher ice velocities than FLI (typically greater than about 0.1 m/s), and both the ice load and the structure's response are random. Thus, it is evident that many ice-structure interactions in Upper Cook Inlet will cause CC to occur. Figure 5.24 shows CC at a Cook Inlet Structure (refer to the left side of the upper trace). FLI is also evident in this same data record, in the middle section of the upper trace.

ISO 19906 does not provide quantitative guidance regarding CC other than to advise that if required, the structure's response can be calculated in the frequency domain using a power spectral density function to characterize the ice load.

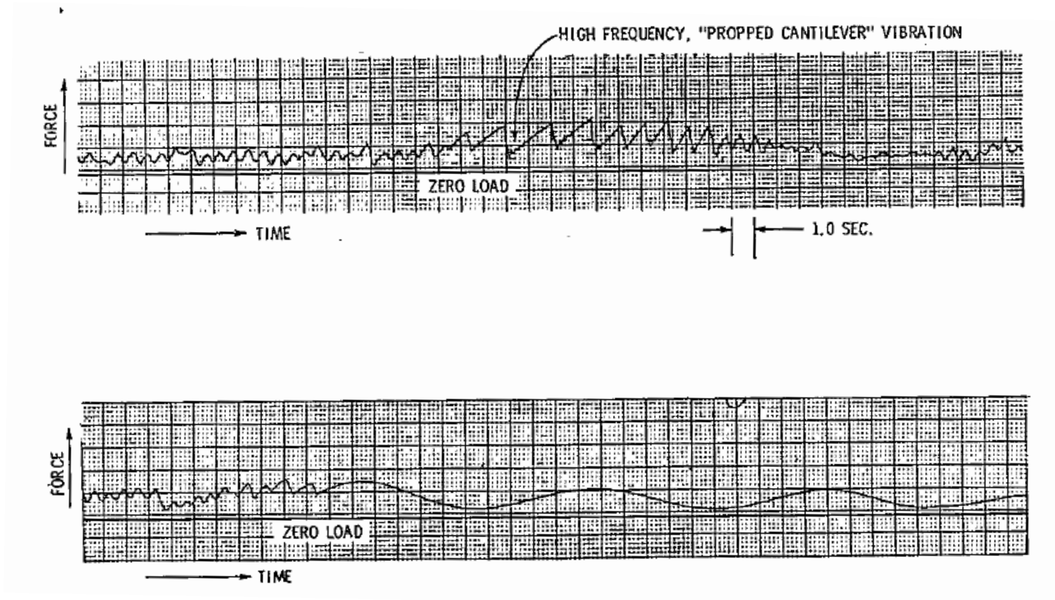


Figure 5.24: CC and FLI at a Cook Inlet Structure (Blenkarn, 1970)

CC was not included in the investigation here for a number of reasons, as follows:

a. It is evident that the largest structural displacements would occur for FLI and hence, it is believed that the majority of the accumulated fatigue would result from FLI (if it occurred). It is suspected (but not verified) that the contribution of CC to the overall accumulated ice-induced fatigue would be of lesser significance. This assumption would need to be verified in an engineering design project.

b. Detailed analyses would be required that were beyond the scope of this project. For example, (Karna, et al. 2010) pointed out that evaluations must be done based on the local ice movement rate at the structure during an interaction, as the floe may decelerate to the point where FLI occurs. This requires mechanics-based analyses on a case-by-case basis that account for the deceleration of a moving floe while in contact with the structure. Again, detailed analyses would be required for an engineering design project.

c. Site-specific ice load data would be required to analyse a structure's response to CC. Sources for such data are limited, and furthermore, their applicability to a structure in Upper Cook Inlet is questionable. Significant uncertainties would be introduced by basing analyses on data measured elsewhere, and substantial analyses would be required for an engineering design project to verify the applicability of the results.

d. Detailed structural analyses would be required, and of course, the analyses would have to be done in relation to a specific structure design. This was not possible here as a specific design for a wind platform structure was not available for this project.

5.7.1.3 The Maximum Load in a Loading Cycle for FLI

ISO 19906 advises that this can be taken as the static global ice load. Static ice loads may be calculated using Equation 5.3 provided that its applicability criteria are met.

5.7.1.4 The Minimum Load in a Loading Cycle for FLI

ISO 19906 advises that the variation in load during a cycle, ΔF, can be defined as follows:

$$\Delta F = q \cdot F_{max} \qquad \text{Eqn. 5.8}$$

ISO 19906 specifies "q" as an empirical coefficient ranging from 0.1 to 0.5. ISO 19906 further advises that "q" should be selected based on the relation between the structure's waterline response and the highest ice velocity at which lock-in can occur. Detailed structural analyses would be required to apply this criterion; and furthermore, the analyses would have to be done in relation to a specific structure design. This could not be done here as a specific design for a wind platform structure was not available for this project.

5.7.1.5 The Time Scale for a Loading Cycle with FLI for a Vertical Structure

ISO 19906 advises that the frequency of the ice forcing function (f =1/T) can be assumed equal to the frequency of one of the unstable natural modes that has a natural frequency below 10 Hz. ISO 19906 further contains a criterion for a structure's susceptibility for FLI (given in Section 5.3), and states that this criterion should be used to identify unstable natural modes.

5.7.1.6 Criterion for Susceptibility to FLI

ISO 19906 contains a criterion for a structure's susceptibility to FLI, which has been given in Equation 5.6. The governing factors include structural properties (i.e., damping, modal amplitude, modal mass and natural frequency) as well as the ice thickness. ISO 19906 also provides general criteria to assess this (given in Section 5.3) based on the structure's waterline displacement, natural frequency, and damping.

Detailed structural analyses would be required to apply these criteria; and furthermore, the analyses would have to be done in relation to a specific structure design. Unfortunately, the criteria in ISO 19906 can't be applied here as a specific design for a wind platform structure is not available for this project. Consequently, only preliminary analyses could be done using ISO 19906 here.

5.7.2 Ice Inputs

5.7.2.1 Ice Floe Thickness

Ice floe ice thicknesses were established previously, and used for IEC 61400 and DNV-OS-J101, as listed in Table 5.27. These were used as inputs for calculating static ice loads.

Table 5.27: Floe Ice Thicknesses Used for Analyses

Period	Sheet Ice Thickness[1,2], m	Rafted Ice Thickness[3], m
Dec. 1 -15	0.30	0.80
Dec. 16-31	0.50	1.4
Jan. 1-15	0.60	1.5
Jan. 16-31	0.60	1.5
Feb. 1-15	0.60	1.5
Feb. 16-28/29	0.45	1.2
Mar. 1-15	0.45	1.2
Mar. 16-31	0.30	0.80

Notes:

1. These values are not intended to include rafted ice or ridges within a floe.
2. The sheet ice thicknesses are the upper-range values listed in Table 5.8. These were used in an effort to produce an upper-range thickness.

Rafted ice thicknesses were established using judgment. Efforts were made to bring the rafted ice thicknesses into general agreement with the rafted ice thicknesses in Table 5.3, as well as with the design ice thicknesses used for the oil and gas platforms in Upper Cook Inlet (Table 5.4 to Table 5.6).

5.7.2.2 Ice Strength

Ice loads were calculated using the same basis that was employed for the analyses done for DNV-OS-J101, which is described in Section 5.6. For brevity, this is not repeated here.

5.7.2.3 Ice Movements and the Number of Loading Cycles in a Winter Period

The same considerations described in Section 5.6 for DNV-OS-J101 would apply for an analysis using ISO 19906. For brevity they are not repeated here.

5.7.3 Results

5.7.3.1 Static Ice Load

Static horizontal ice loads were calculated using the algorithm of Equation 5.3 by ISO 19906. For consistency, the loads were calculated on the same basis used for the analyses done for DNV-OS-J101. These are described in Section 5.6 and, for brevity, are not repeated here.

Obviously, this led to the same values for the static ice loads for sheet ice and rafted ice as were determined in Section 5.6 and as listed in Table 5.23 and Table 5.24, respectively. For brevity, they are not repeated here.

5.7.3.2 Dynamic Ice Load Time History and the Number of Load Cycles

This is not described here for brevity as the results would be generally the same as those obtained for DNV-OS-J101 as given in Section 5.6. This is due to the following:

a. The shapes of the load time history are identical for ISO 19906 and DNV-OS-J101. Both recommend a saw-tooth profile for FLI or the "tuned" condition, respectively.

b. The static ice loads are identical are as they are calculated on the same basis using the same inputs. Thus, the peak load in a loading cycle would be identical.

c. The minimum loads in a loading cycle will be within the same range. DNV-OS-J101 defines the minimum load as 20% of the peak value. In the absence of detailed structural analyses (which could not be done here due to the lack of a specific structure), ISO 19906 advises that the minimum load be taken as 10% to 50% of the peak load.

Comments cannot be made regarding the number of events since the required analyses to apply ISO 19906 could not be done (due to the lack of a specific structure). However, it is obvious that similar S-N curves would be obtained from DNV-OS-J101 and ISO 19906 if they had the same number of events.

5.7.4 Assessments Regarding the Application of ISO 19906

Only preliminary statements can be made for the reasons given in this section. The following observations are made:

a. ISO 19906 provides a more complete treatment of dynamic ice loadings than do IEC 61400 or DNV-OS-J101.

b. As a result, in order to apply ISO 19906, more inputs must be specified and more analyses must be carried out. Recognizing the uncertainties and issues in defining the ice input requirements, it is not clear whether or not this leads to a more accurate assessment of dynamic ice loadings.

5.8 Conclusions

5.8.1 Objectives and Basis for Conclusions

The intent of the project was to compare methods and approaches for evaluating dynamic ice loads and their contribution to the fatigue of a wind turbine foundation. These include:

a. The available codes include IEC 61400, DNV-OS-J101 and ISO 19906.

b. An empirical approach, in which dynamic ice loads over the life of the structure are evaluated using site-specific data and analyses.

c. Numerical methods defining the ice load time history using mathematical models.

The use of existing codes is considered to be the preferred approach.

Then, attempts were made to apply the three codes to a test case which was selected to be a monopile in Upper Cook Inlet. It should be recognized that because the analyses were only a test case (to investigate the application of the codes) rather than a detailed engineering investigation, simplifying assumptions were required. Nevertheless, significant efforts were made to define a realistic set of ice conditions for Upper Cook Inlet.

5.8.2 Comparison of IEC 61400, DNV-OS-J101 and ISO 19906

5.8.2.1 Overall Comments

IEC 61400 and DNV-OS-J101 were able to be applied to the test case which allowed detailed observations to be made. Only preliminary investigations could be performed for ISO 19906, primarily because a specific design for a wind platform structure was not available for this project, which prevented the criteria in it from being applied. As a result, only preliminary statements can be made regarding ISO 19906, as follows:

a. ISO 19906 provides a more complete treatment of dynamic ice loadings than do IEC 61400 or DNV-OS-J101.

b. As a result, in order to apply ISO 19906, more inputs must be specified and more analyses must be carried out. Recognizing the uncertainties and issues in defining the ice input requirements, it is not clear whether or not this will lead to a more accurate assessment of dynamic ice loadings.

5.8.2.2 Defining the Required Ice Inputs

It was concluded that the required ice inputs can be defined for both IEC 61400 and DNV-OS-J101, although significant investigation would be required for an engineering design project, including probably, the collection of site-specific data. It was concluded the availability of ice information requirements will not limit the application of IEC 61400 and DNV-OS-J101.

5.8.2.3 Results: The S-N Curve

The resulting S-N curve from IEC 61400 spanned a wide range due to the inclusion of both rafted ice and sheet ice. The rafted ice events produced larger load reversals although there were fewer cycles. The sheet ice events produced lower load reversals with a larger number of cycles. This trend is considered to be realistic.

The resulting S-N curve for DNV-OS-J101 also indicated that the number of cycles was inversely related to the magnitude of the load reversal. The S-N curve for DNV-OS-J101 spanned a narrower range of load reversal magnitudes than did the one from IEC 61400. This is due to the fact that only certain cases for sheet ice needed to be included for DNV-OS-J101, as the others did not meet the tuning criterion. The variation is mainly due to differences with respect to rafted ice, these cases generated the highest ice loads and largest load reversals.

Furthermore, DNV-OS-J101 indicated significantly lower load reversal magnitudes were associated with a given number of cycles than did IEC 61400. Again, this variation is principally due to the fact that the rafted ice cases did not meet the tuning criterion in DNV-OS-J101, and thus they were excluded.

As an overall observation, these results show the significance of differences in the methodologies in IEC 61400 and DNV-OS-J101.

6 STRESS ANALYSIS

Using the calculated structural response and the design S-N curves, the load effects (or, stress history) must be determined in order to complete the fatigue life estimation. There are several methodologies by which the local stresses are determined from the applied loads (associated with the combinations of the wind, wave, and operational conditions) and, in general, the process follows as:

1. Determine the components of the nominal stress range in the structure; and,

2. Use the stress concentration factors (associated with a simplified analysis) or transfer functions (obtained from a spectral analysis) to determine the hot-spot stress range.

The method selected to develop the load effects should be that most appropriate for the detail under consideration. The selection will depend on how well the joint and the weld profile are classified according to the fatigue tests on which the design S-N curves are based. When the joint or weld is represented directly by the defined fatigue detail categories, then the nominal stress approach should be valid. If the joint or weld is not represented adequately, or if the component corresponds to a tubular structure, the hot-spot stress approach should be used. The notch stress approach is used less often and requires detailed analysis including representation of the weld notch.

This procedure is described subsequently for relating the external load distribution or spectra to the response of the structural detail or connection under consideration.

6.1 Stresses in a Fatigue Analysis

6.1.1 Introduction

The development of the long-term stress range distributions for calculation of the fatigue life estimates may be completed in either the time or frequency domain, as follows:

1. Time Domain

   Here, the loads associated with the wind, waves and operation are combined to form a time domain record. Structural calculations are completed directly to develop the stress history using advanced, integrated analytical methods. The response history, expressed as a stress range histogram, is determined subsequently using rainflow counting techniques.

2. Frequency Domain

   In this case, the combined loads associated with the wind, waves and operation are converted from a time domain record to a frequency domain record using a Fourier Transform. Through relatively rapid structural analyses in the frequency domain, a stress spectrum is developed.

The selection of either methodology depends on the computational resources available and the degree to which the structural wind/wave interaction and damping are to be integrated. Regardless of the method, the simulation results will consist of the set of stress range histories. The calculated results, whether completed in the time domain or in the frequency domain, are represented as a histogram of the calculated response to be used in the fatigue damage estimation. However, the calculated stresses may be interpreted in different manners depending on the type of detail under consideration and the analytical method applied.

At a detail or welded connection, the fatigue life will be determined by the cyclic stresses at the point of interest. The stress field at a detail can be resolved into four components of stress, as illustrated in Figure 6.1. Depending on the method applied by the standards and guidelines, and the basis on which the design S-N curves have been developed, certain components of the stress range may be excluded from consideration. Thus, the standards and guidelines outline different requirements for the determination of the stress range to be used in the fatigue life calculations. Typically, this involves the following approaches:

a. Nominal stress approach with existing detail classifications: The stresses are calculated in the absence of any stress concentrations associated with the local structural detail and weld;

b. Nominal stress approach with stress concentration factors: The effects of local stress concentrations due to attachments or structural (geometric) discontinuities, but excluding any contribution associated with welds, are considered; or,

c. Hot-spot stress approach using testing or Finite Element Analysis (FEA): The structural geometric stress is determined with consideration of the influence of the nominal stress and local effects, but excluding the notch effect of the weld seam itself.

In rare cases, a notch stress approach may be required when considering the stress concentrations associated with significant geometric discontinuities or weld toe discontinuities, such as at a weld undercut (as shown in Figure 6.1). Note that the type of construction dictates the type of stresses required for the analysis, as prescribed by the standards and guidelines. Specifically, the design of tubular joints with full penetration welds requires the application of approaches (b) or (c) to account for the stress concentrations, which govern the design.

Residual stresses are related to the local self-equilibrating stresses associated with the cooling and restraint involved in welded fabrication. Provided that the effects of residual stresses are considered adequately during fabrication, they are not considered generally for the fatigue design calculations. Since the design S-N curves are based on empirical testing, the effect of residual stresses on the mean stress level is already accounted for in the S-N curve or fatigue life data.

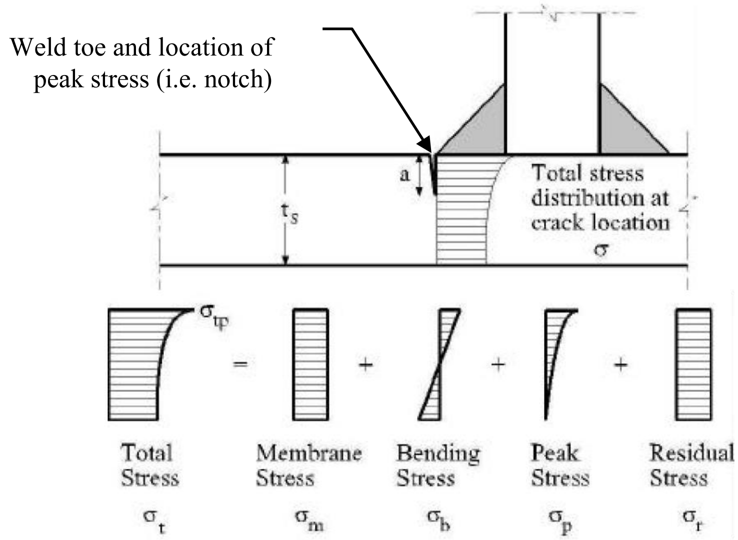


Figure 6.1: Stress Components in a Welded Joint (Glen, et al. 1999)

6.2 Stress and Stress Concentration Categories

6.2.1 Nominal Stresses

The nominal stress at a detail may be considered as the combination of

(i) the component of uniformly distributed membrane stress (tension or compression); and

(ii) the component of linearly varying through-thickness bending stress.

In simple structures (including slender bars and beams), the nominal stress may be determined using engineering calculations (such as simple beam theory, for example) and using the elastic section properties of the detail or component under consideration. Beam element models or coarsely meshed finite element models using shell elements may also be applied. Any effects resulting from the notch effect or other discontinuities are excluded from consideration for the nominal stress. Stress concentration factors (SCFs) and fatigue notch factors are instead used to modify the nominal stresses to account for the additional stress components.

A stress concentration factor (SCF) is defined as the ratio of the hot-spot stress (at a weld toe, for example) to the nominal stress, taken equal to an extrapolation of the calculated stress at some distance from the weld toe or other concentration. SCFs can be calculated from parametric equations (such as Efthymiou), model tests or from the results of finite element analyses, with specific requirements to be considered for tubular members and connections. In general, SCFs depend on the type of variable loading, the type of joint, and details of the joint geometry.

When applying the nominal stress method, the location and direction of the stress for which the stress range is to be calculated are indicated with the tables of existing detail classifications. The nominal stress range is then determined by using either the cross-sectional area of the parent metal or the weld throat thickness, depending on the detail and on the crack location. (Typically, a corrosion allowance is taken into account by decreasing the nominal thickness to ensure an adequate level of corrosion protection).

Note that the design of tubular members with full penetration butt welds may proceed on the basis of the nominal stress approach with the application of the Efthymiou parametric equations, which account implicitly for the effects of stress concentrations.

6.2.2 Stress Concentration Factors and the Peak Stress Component

Typically, the standards and guidelines define the design S-N curves for application to the nominal stress approach. For details which do not meet the requirements of the existing detail classifications, the design S-N curves are modified to account for the influence of peak stress components. The peak stress component at a detail results from stress concentrations at discontinuities and is typically considered to occur at the toe in a welded joint, as illustrated in Figure 6.1. These peak stresses are associated with various effects, including:

- Geometric stress concentrations;
- Notch stress concentrations; and
- Misalignment stress concentrations

In cases where the existing detail classifications for fatigue do not represent adequately the stress distribution at a detail, the long-term nominal stress range, determined in accordance with Section 6.2.1, is modified by a stress concentration factor (SCF), which depends on the structural geometry. SCFs can be calculated from parametric equations or using finite element analysis. Factors requiring consideration include the following:

- Thickness effect;
- Corrosion effect (when adequate corrosion protection is not included);
- Material effects;
- Effect of mean stress;
- Effect of weld shape;
- Importance of the structural member; and
- Effect of misalignment.

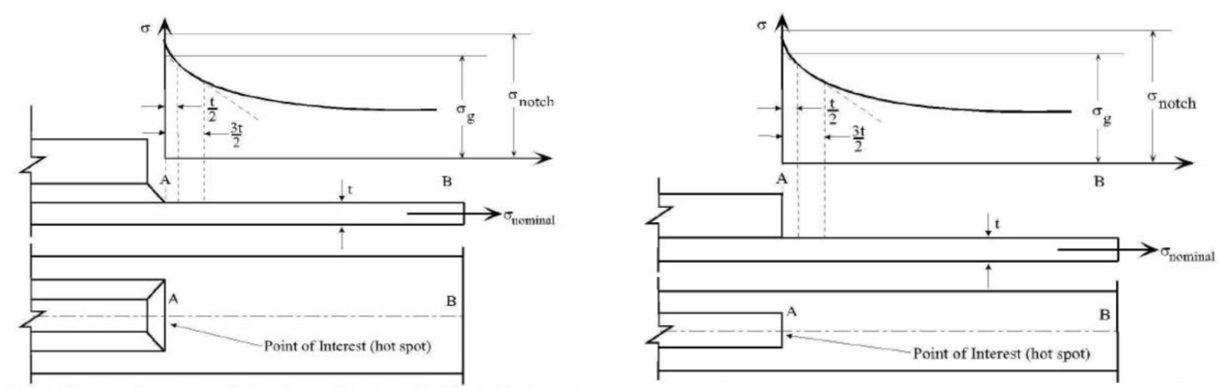
It is important to ensure that any applied SCFs are consistent with a corresponding design S-N curve. Using SCF’s and design S-N curves from different sources may lead to calculated stresses that are not directly applicable.

Note that the notch stress concentration associated with the weld toe (as illustrated in Figure 6.1) is not used directly in the application of the design S-N curves. Typically, the design S-N curves (being based on empirical testing) already account for the effects of the notch stress in the safe-life design process. Therefore, the notch stress concentration is relevant only when performing a damage tolerance (i.e., fracture mechanics).

6.2.3 Hot-Spot Stresses

For those welded joints, for which the detail classification is not possible or where additional stresses occur and are not adequately considered by the detail classification, the fatigue strength analysis may be performed on the basis of the hot-spot stresses.

Stress singularities occur at weld roots and weld toes, as illustrated in Figure 6.1. The hot-spot stress method provides for the estimation of stress at the weld for design by extrapolating the geometric stresses across the singularity of the notch stress zone. Typically, the hot-spot stress is determined using a linear extrapolation, however other techniques are available and include (i) using the stresses at a single point a given distance away from the notch; or, (ii) using a quadratic extrapolation. The hot-spot stress may be calculated from the results of detailed local finite element analyses and, as illustrated in Figure 6.2. The method by which the surface stresses are extrapolated in the finite element model (shown for the typical linear extrapolation) is dependent on the choice of element used.



(a) Solid elements **(b) Shell elements**

Figure 6.2: Hot-Spot Stress Extrapolation for Solid and Shell Elements (Glen, et al. 1999)

Guidance regarding the type of elements to be used and the level of detail required for the model are provided in the standards and guidelines and includes consideration of the:

- Extent of the model;
- Boundary conditions;
- Element type (shell or solid);
- Element size and mesh refinement; and,

- Element geometry (aspect ratio, skew, corner angles and warp).

In using the finite element analyses, the stresses to be used in the hot-spot stress calculation should correspond to those on the surfaces of the plates and should be based on averaged nodal stresses. For shell element models, this requires that the mid-thickness stresses are not applied. Stresses at the integration points should not be used directly.

6.3 Nominal Stress Approach

Nominal stress ranges are calculated based on the applied loading and on the section properties of a component or connection. The effects of localized geometries such as holes, cutouts, or tapers, for example, are excluded from the determination of the stresses. However, such discontinuities can affect significantly the results of the fatigue assessment.

The nominal stress approach uses the calculated results from generalized structural models, in which welds and other connections are not represented in detail. The nominal stresses are calculated with the model and used in conjunction with the design S-N curves (as derived from the experimental nominal stresses) to complete the fatigue assessment. Therefore, in the nominal stress approach the effects associated with weld profiles and other limited geometric effects are addressed by

(i) application of the design (nominal) S-N curves; and

(ii) the application of geometric stress concentration factors.

For this approach, the design S-N curves must correspond to those determined from testing on the basis of nominal stresses. Certain localized stress concentrations, including those attributable to the weld profile or specific geometric changes, are considered in the determination of the design S-N curves. The design S-N curve describing a particular connection or detail is determined by identifying the most suitable detail classification from the guidance. Therefore, when the component or connection matches that described by the corresponding detail classification, the calculated nominal stress range can be used with the nominal design S-N curve directly to estimate the fatigue damage.

In some cases, the component or connection will not be represented adequately by a detail classification and the nominal stress range is modified by a stress concentration factor (SCF or K-factor). Here, a stress concentration factor is calculated for each detail in the assessment to reflect the influence of holes, tapers, cutouts or other gross geometric influences not implicitly included in the determination of the nominal design S-N curve. The standards and guidelines provide guidance for calculating the stress concentration factor as described by multiple effects including the gross geometry of the detail; the weld geometry; eccentricity tolerance; angular mismatch in plate connections; and, asymmetrical stiffener arrangements.

The nominal stress approach requires the application of the nominal design S-N curves and the corresponding applied nominal stress ranges. To consider appropriately the limitations of the localized effects at a welded connection or detail, additional stress concentration factors may be applied to the nominal stress ranges for the fatigue assessment.

6.4 Hot-Spot Stress Approach

The hot-spot stress approach is applicable to situations for which the local stress and the geometry of a connection does not correspond adequately to a detail classification. Arrangements include those connections having a welded attachment such that local stress concentrations will occur at the weld toe. Other applications for the hot-spot stress approach relevant to OWTG structures include arrangements of tubular joints, as for a jacket structure.

Welded joints for which a detail classification cannot be achieved or for which additional stresses occur, may require that the fatigue strength analysis be performed on the basis of the hot-spot stress, as follows:

- For tubular joints with full penetration welds the stress to be used in the fatigue analysis should be the hot-spot stress at the weld toe.
- For other welded joints the fatigue strength analysis is normally based on the nominal stress at the structural detail considered and on an appropriate detail classification (or the fatigue strength reference value).
- For notches of free plate edges the relevant notch stress is determined for linear-elastic material behavior, normally calculated from a nominal stress and a theoretical stress concentration factor (SCF).

In the hot-spot stress approach, detailed finite element models are used to calculate the stress ranges at the local detail. The finite element models must meet requirements regarding element type; element size; element aspect ratios; and, mesh refinement and gradation. The manner by which the hot-spot stress is extracted linearly from the adjacent reference stresses is also defined. Various methods are prescribed in the standards and guidelines for obtaining the stress concentration factors associated with the different weld geometries.

As for the nominal stress approach, the calculated stress ranges by the hot-spot stress approach must be used in conjunction with the corresponding design S-N curves in order that the desired safety level is achieved. In this case, the design S-N curves similarly should be determined on the basis of hot-spot stresses, as opposed to nominal stresses.

6.5 Development of the Stress Range Distributions

As noted, time domain simulations that consider the integrated loading and structural interaction currently are preferred. The internal forces and moments, and hence the stresses at structural details, can be determined for the fatigue analysis directly. However, solutions to the structural analysis of OWTGs may be too complex to be completed in the time domain. Instead, experience is taken from the design of offshore platforms in the oil and gas industry where analyses are completed in the frequency domain. However, characterizing the response of OWTGs depends highly on the accurate representation of the aerodynamic effects. While computing power has matured such that time domain solutions are becoming more reasonable, frequency domain solutions may remain desirable. Table 6.1 provides a brief overview of the different solution domain methodologies.

Table 6.1: Solution Domain Methodologies

Solution Domain	Overview
Quasi-Static Analysis with Dynamic Response Factors	• Calculate static response for several loading conditions with separate consideration of wind, wave and gravity loads; • Estimate a dynamic amplification factor (DAF) for each condition, typically between 1.2 and 1.5; and, • Superimpose results, including partial safety factors per loading type. The loading is increased by the dynamic amplification factor to consider dynamic response to varying wind and wave loading. Different DAFs may have to be applied for wind load on the turbine rotor, the support structure and wave load on the support structure.
Frequency Domain (Spectral) Analysis	Due to non-linearity in the system, this procedure must be repeated for different environmental conditions and wind/wave combinations, but generally follows as: • Determine transfer function per load source; • Linearize system or use small harmonic loads; • Multiply spectrum of load source with transfer function; and, • Superimpose response spectra of different sources. For jacket structures rapid fatigue analysis techniques have been developed in the frequency domain, based on the experience in the design of offshore platforms. The large number of load combinations and the size and complexity of the models previously prohibited transient dynamic analyses in the time domain.
Time Domain Analysis	Full dynamic computations of the section forces take into account the control behaviour of the turbine, the stiffness and the dynamic response of the support structure: • Generate realizations of external conditions; • Integrate equations of motion numerically; • Analyze fatigue response; and, • Repeat until statistically sound information is obtained. For monopile towers, the time domain analyses may be most appropriate, especially since computing power and the available software tools allow direct, mostly efficient calculation for a large number of load cases. Dynamic analyses will also be required in general for designs that may experience large motions such floating OWTGs. For dynamic simulations, it is important to ensure that the wind/wave directional data and the modeling of aero-elastic damping is considered correctly.

6.6 Cycle Counting

Cycle counting is used on the calculated history to produce the response history, which is used subsequently for the estimation of fatigue life by the method of stress-life, strain-life or fracture mechanics.

Various methods of cycle counting can be applied to develop the stress history of constant amplitude loadings resulting from the variable stress history. The rainflow counting method or the reservoir method are used commonly in the time-domain. Other methods of cycle counting in the frequency domain may be used and include a formulations by Rayleigh, Rice and Dirlik (Tempel 2006).

Regardless of the approach taken, the method should be checked for consistency to confirm how the stress range (S) and number of cycles (n) are defined in the standards and guidelines for the damage summation. For example, the stress range may correspond to the range of the cycle; the amplitude of a cycle; the maximum (tension) or minimum (compression) of the cycle. The number of cycles may refer to the number of full cycles to failure; the number of cross-overs; or the number of reversals. GL recommends that the method of cycle counting (specifically, rainflow counting) should include consideration of the residuals (half cycles).

6.7 Design S-N Curves

6.7.1 Introduction

Design S-N curves are used with a cumulative damage approach to estimate the fatigue life of components and connections. The standards and guidelines have developed these design S-N curves differently in some cases, being based on the results of experimental testing. The application of the design S-N curves for the fatigue evaluation is described subsequently, including discussion of the modifications to those curves that are required to account for the various influencing factors. An overview of the damage summation model (i.e., Miner-Palmgren summation) is also given for the calculation of the cumulative damage and estimated design life.

6.7.2 Fatigue Design Curves

The stresses calculated from the application of the fatigue loads are evaluated:

(i) based on the material properties;

(ii) based on the selection of the design S-N curve for the appropriate detail and conditions; and,

(iii) with consideration of various modifying factors, such as mean stress effects. This section describes design S-N curves and the associated modifications to be applied in the damage accumulation calculations for the fatigue life assessment.

Design S-N curves describe the number of cycles (N) required to initiate a failure versus the applied stress range (S) and are used in the cumulative damage approach for the fatigue evaluation. The general form of the S-N curve is described by either expression in Equation 6.1.

$$N = \bar{a}S^{-m} \qquad \text{Eqn. 6.1(a)}$$

$$\log N = \log \bar{a} - m \log S \qquad \text{Eqn. 6.1(b)}$$

Here, the number of cycles (N) is related to the stress range (S) through the slope parameter (m) and the fatigue strength coefficient ($\bar{a}$), for which the value of log $\bar{a}$ corresponds to the intercept with the log N axis. Depending on the standard and guideline, as well as the specific application, the values of the slope parameter will vary. In most cases, the design S-N curves are defined as a bi-linear curve, with the value of the slope parameter changing at some fatigue limit, typically taken as approximately 10^7 cycles for non-tubular details in air. The change in slope is made to ensure the estimated life remains conservative for low-cycle fatigue.

The value applied for the fatigue strength coefficient ($\bar{a}$) relates to the interpretation of survival probabilities as obtained from the results of the characteristic testing. As described in DNV-RP-C203, Equation 6.2 calculates the value of the intercept of the log N-axis for the S-N curve using a constant (a) related to the mean S-N curve and the standard deviation (s) of log N.

$$\log \bar{a} = \log a - 2s \qquad \text{Eqn. 6.2}$$

In this respect, a survival probability can be applied based on interpretations of the probabilities associated with the generation of the S-N curves from empirical data. In the standards and guidelines, these correspond to the lower limit of the scatter band of 95% of the available test results for a 97.5% survival probability (as in GL); or, the mean less two standard deviations of the experimental data and thus a 97.6% probability of survival (by DNV-RP-C203).

The characteristic design S-N curves may be derived differently among the standards and guidelines; however, the majority of the S-N curves applied for design correspond to component S-N curves. These are based on the results of experimental tests for specific components and welded connections and so the effects of weld residual stresses and stress concentrations are included implicitly in the definition of the design S-N curves.

In general, the design S-N curves for non-tubular details and non-intersecting tubular connections are categorized according to the most applicable of several nominal joint classifications, while considering the potential crack locations in the detail and the direction of the applied loading. The components are further considered for the design S-N curves with respect to the long-term environmental conditions, referring to being in air; being in sea water and with cathodic protection; or, freely corroding (i.e., unprotected). For such details, the joint classification and corresponding design S-N curves implicitly account for local stress concentrations, such as may occur at a weld profile. Therefore, the design stress can be taken as equal to the nominal stress adjacent to the weld or detail. In cases for which the component or detail has additional complexity and does not conform to a prescribed joint classification, the maximum principle stress at the critical hot-spots can be extrapolated.

For other components, including tubular joints and cast steel members or nodes, design S-N curves are described for use with the hot-spot method. Similar to the nominal design S-N curves, these curves also consider the long-term environmental conditions and any provisions for cathodic protection. Stress concentration factors are established for the joints based on parametric equations (after Efthymiou); using those referenced in API-RP 2A; or, as determined from detailed finite element analyses.

Some standards and guides apply material S-N curves, which have been determined only from tests using base materials. This approach is non-conservative and stress concentration factors must be applied subsequently to account for the effects of weld residual stresses. Therefore, when applying an S-N curve for design, it is important to understand the basis on which the characteristic curve has been derived.

6.7.3 Modified S-N Curves

The suitability of a design S-N curve for the fatigue life estimation of a component or detail requires consideration of various factors that may affect the application, including the geometry of the component and the absence or inclusion of corrosion protection. The calculated stress ranges may need to be modified or corrected with consideration of the following:

1. Material Thickness (or, Size) Effects

The tests on which the design S-N curves are based are applicable to components up to a maximum material thickness, as prescribed by the standards and guidelines. Table 6.2 lists some reference values used in the calculation to adjust for the material thickness effect. Material thickness effects consider the through-thickness stress gradient and the geometry of the adjoining plates in a welded connection. Basically, the modification of material thickness accounts for the increased probability that the larger component (as compared to that on which the S-N design curves are based) will initiate a fatigue crack under cyclic loading.

Table 6.2: Reference Values for Material Thickness Effects

Standard or Guideline	Base Material Reference Thickness
GL "Guideline for the Certification of Offshore Wind Turbines"	25 mm
DNV-OS-J101 / DNV-RP-C203	25 mm for welded non-tubular connections; 32 mm for tubular joints; 38 mm for cast steel components.
ABS-115 "Guide for the Fatigue Assessment of Offshore Structures"	22 mm for non-tubular details; 22 mm for the S-N 'T' curve and tubular joints; 38 mm for cast steel components.

2. Mean Stress Effects

The mean stress represents the average stress between the minimum and maximum stresses of the cycle, providing an indication that the component may be in tension or in compression, partly or wholly. Components subject to compressive cycling will be less subject to fatigue crack initiations. The effect of a non-zero mean stress may be accounted for by scaling the long term stress range distribution by a reduction factor with values between 1.0 and 0.6 and representing the effect of a partial or full fatigue crack closure when the material is in compression. Components subject to fully tensile cycling are not subject to a stress range reduction; i.e., the factor is taken as equal to 1.0.

3. Effect of Surface Treatments (or, Weld Shape Effect)

When subject to cyclic loading, tensile residual stresses at the surface of a component will reduce the fatigue life significantly. In welded components, the presence of residual stresses at the welds can be improved by weld profiling by machining or grinding; tungsten inert gas weld (TIG) dressing; or, shot peening. These mechanical methods of treating the component's surface create compressive residual stresses. Further, the surface finishing is improved by smoothing out defects or potential flaws that act as crack initiators.

4. Corrosion Effect

The effect of corrosion may be considered differently depending on the approach described in the standards and guidelines. In addition to defining a set of design S-N curves for components in air (i.e., above the splash zone), DNV-OS-J101 and ABS also define S-N curves depending on the environmental conditions, including exposure to seawater with free corrosion or with cathodic protection. GL instead uses a factor of 0.7 applied to the reference value of the S-N curve for parts of a structure in a corrosive environment and without cathodic protection. (The value is otherwise taken as equal to 1.0.) The application of this factor has the effect of adjusting the characteristic design S-N curve to be approximately equivalent to those individually defined by DNV and ABS, for example, for parts exposed to seawater without corrosion protection. Additional discussion of corrosion and fatigue is provided in Section 9.

5. Material Effect

For steels having a minimum specified yield stress greater than 235 MPa, the GL guideline provides an adjustment to the reference value for the S-N curve. For yield strengths greater than 235 MPa, the material effect factor will be greater than 1.0. The DNV guideline provides a modified S-N curve for base materials consisting of high strength steel, having a yield stress exceeding 500 MPa (excluding cast nodes, which are specified uniquely).

6. Structural Importance

The GL guideline provides consideration for the effect of the radius on notched components by specifying an adjustment to the importance factor (taken usually as equal to 1.0). The importance factor is corrected for notches at plate edges and takes a value between 0.9 and 1.0, depending on the radius. The resulting factor is then applied to the characteristic reference value when defining the S-N curve.

7. Misalignment

Misalignment in axially loaded welded connections introduces additional stresses depending on the condition. The first condition relates to the differences that occur between the component as fabricated (or as tested for the basis of the S-N curves) and the component as analyzed. The second condition relates to the deviations from the design condition that may occur during fabrication (i.e., a non-conformance). The overall stress magnification that may result from either condition must be accounted for, and reference may be made to the International Institute of Welding (IIW) Recommendations. The design misalignments may otherwise be considered explicitly in the fatigue analysis of welded connections by using detailed finite element analysis and subsequent interpretation of the results using the nominal or hot-spot stress methods and the appropriate stress concentration factors.

7 FATIGUE LIFE ESTIMATION

7.1 Introduction

The objective of the fatigue assessment is to verify that the performance of the structure during its design life is satisfactory within an acceptable probability level; i.e., that fatigue failure is unlikely to occur. Subsequently, the reliability is calculated with application of safety factors or, for example, by specifying the fatigue life as a multiple of the design life.

Once the stress histories are developed from the environmental design loads, then the fatigue damage can be estimated. As commonly done for many other types of steel structures, a Miner-Palmgren linear damage summation is used. Together with the corresponding design S-N curves, the fatigue life is estimated. Calculated fatigue lives represent conservative design estimates and should be considered as indicating relative performance, rather than being relied upon as an absolute life. The fatigue life is to be estimated for all components of the structure; the reliability of the OWTG being based on the probability of achieving that fatigue life.

7.2 Damage Accumulation Methods

The principle damage accumulation methods for the design of steel structures are the stress-life method; the strain-life method; and, the crack growth method, as described subsequently (Bannantine, Comer and Handrock 1990).

7.2.1 Stress-Life Method

The stress-life method to fatigue life estimation is based on the application of characteristic S-N curves determined from load-controlled experimental testing of materials, details and connections. In general, the stress-life method is relevant to high-cycle fatigue life estimation and for structures or components that are loaded elastically.

Since the stress-life method assumes elastic behavior (i.e., elastic strains), the true nature of fatigue crack initiation, which is caused by plastic deformation, is ignored. This simplifying assumption is reasonable as long as the design of the structure or component is made such that elastic behavior is maintained. It is acknowledged that a structure designed in accordance with a stress-life method for fatigue evaluation may not be weight optimized. However, in applying this method and with the requirements of the standards and guidelines, the resulting design will be satisfactory in general, and the fatigue life can be estimated with confidence.

To ensure the design by the stress-life method remains valid, it is important the standards and guidelines prescribe the loads with some accuracy such that periodic overloading will not occur.

The design S-N curve is selected as appropriate for the detail classification and based on the approach used to account for the stress concentrations (i.e., nominal or hot-spot stress approaches). Further, the design must consider and mitigate the effects of corrosion, which will affect the fatigue performance. The curves then may be modified, as described in Section 6.7.3, to account for the effects of corrosive environments and thickness (size) effects.

The design S-N curve detail classification, modified as appropriate, is then used to estimate the permissible number of cycles for each applied stress range. The individual occurrence of damage experienced by the structural component is determined from the number of cycles for each interval of applied stress range, taken from the stress range histogram, when compared to the permissible number of cycles for that applied stress range. The cumulative fatigue damage for a component is calculated by a Miner-Palmgren linear summation of all of the individual damages.

DNV provides two methods for calculating the design cumulative damage. Method (1) computes the characteristic cumulative damage, based on the characteristic S-N curve, and then multiples by a design fatigue factor (DFF). The DFF takes a value equal to 1.0, 2.0 or 3.0 based on the location of the structural detail, on the accessibility for inspection and repair, and on the type of corrosion protection provided.

Method (2) computes the design cumulative fatigue damage based on the number of cycles to failure at the design stress range, which is based on a material factor for fatigue and on the characteristic S-N curve. The material fatigue factor is related to the DFF values, based on the type of detail, accessibility and corrosion protection.

7.2.2 Strain-Life Method

In notched components, plastic strains can develop as a result of localized stress concentrations even when a component, globally, is loaded elastically. The strain-life method to fatigue life estimation accounts for the effects of the plastic component of strain.

For notched or similar components, the local deformations may be strain dependent. Therefore, for this application the results of deformation-controlled or strain-controlled experimental testing are used. The strain-life method assumes that the fatigue damage can be represented by the results of testing smooth (i.e., un-notched) strain-controlled specimens. The stress-strain history applied in the testing is taken to be equivalent to that required to produce fatigue damage in the notched specimen. In this respect, the strain-life method represents a crack initiation approach to fatigue life estimation, for which failure occurs once a crack is started.

Similar to the stress-life method, the strain-life method applies the loading (i.e., stress-strain) history at the critical location and a damage accumulation technique such as the Miner-Palmgren summation to estimate the fatigue life. Also, mean stress effects are considered. However, being a strain-controlled method, the material response subject to the cyclic inelastic loading is used instead of the characteristic design S-N curves. The effects of cyclic strain hardening or softening (or, Bauschinger effect) are described by the material and component testing. Strain-life curves are developed to relate the total strain range to the life to failure.

The standards and guidelines do not consider the strain-life method for fatigue life estimation, as currently the stress-life method and the characteristic design S-N curves are used.

7.2.3 Crack Growth Method

A crack growth method, or fracture mechanics approach, assumes that a crack has already been initiated in the structure or component and so it is the propagation life that is estimated. If the initiation life is estimated using a strain-life approach, the total fatigue life can be estimated as the sum of these two components.

In applying the fracture mechanics approach, an initial crack size is assumed for design. The method is then used to relate the stresses to the crack size and shape. A stress intensity factor relates the local stresses and strains at the crack to the stresses and strains applied remote from the crack. In determining the stress intensity factor range, the fatigue crack growth rate can be estimated and the cycles to failure calculated. Since the crack growth rate therefore can be characterized, a structure or component can be designed with a fail-safe approach such that the performance can be monitored between inspection intervals.

Note that a fracture mechanics approach to damage tolerance estimation is based on the assumption of a pre-existing crack (of some dimension), which then propagates through the material thickness. Allowance is made for an approach based on fracture mechanics, which can be supplemented by the S-N data. (DNV-RP-C203 requires that the structural reliability based on a fracture mechanics approach will not be less than that determined from the S-N data).

7.3 Miner-Palmgren Linear Damage Summation

Once the stress histories are developed from the environmental design loads, then the cumulative fatigue damage can be estimated. As commonly done for many other types of steel structures, a Miner-Palmgren linear damage summation is used. For a detail, this summation relates the total number of cycles at each stress range to its fatigue limit (i.e., maximum number of cycles that can be endured before initiation of a through-thickness crack). The total damage accumulated is summed for all stress range intervals associated with the applied variable amplitude stresses determined from the load history. Provided the cumulative damage ratio is less than or equal to 1.0, then the fatigue design of the detail is considered to satisfy the design service life requirement for a minimum life of 20 years (or as specified). Equation 7.1 expresses the linear damage summation.

$$D = \sum_{i=1}^{J} \left(\frac{n_i}{N_i}\right) \qquad \text{Eqn. 7.1}$$

Where,

- D is the total or cumulative damage ratio at the detail under consideration;
- n_i is the number of cycles the structural detail endures at stress range $\Delta\sigma_i$;
- N_i is the number of cycles to failure at stress range $\Delta\sigma_i$, as determined by the corresponding design S-N curve (modified as appropriate by Section 6.7.3); i.e. that detail's fatigue limit; and,
- J is the number of considered stress range intervals.

Effectively, the individual occurrence of damage experienced by the structural component is determined from the number of cycles for each interval of applied stress range, taken from the stress range histogram, when compared to the permissible number of cycles for that applied stress range. The cumulative fatigue damage for a component thus is calculated by summing all individual occurrences of damage.

Since the number of stress cycles endured is estimated on the basis of the applied loads over the full design service life, the cumulative damage ratio, D, will correspond to that for the design life. Thus, the fatigue design will be satisfied for the detail under consideration when the relationship in Equation 7.2 is met:

$$D \leq 1.0 \qquad \text{Eqn. 7.2}$$

Calculated fatigue lives represent conservative design estimates and should be considered as indicating relative performance, rather than being relied upon as an absolute life. The fatigue life is to be estimated for all components of the structure; the reliability of the OWTG being based on the probability of achieving that fatigue life.

Since the stress-life method assumes elastic behavior (i.e., elastic strains), the true nature of fatigue crack initiation, which is caused by plastic deformation, is ignored. This simplifying assumption is reasonable as long as the design of the structure or component is made such that elastic behavior is maintained. It is acknowledged that a structure designed in accordance with a stress-life method for fatigue evaluation may not be weight optimized. However, in applying this method and with the requirements of the standards and guidelines, the resulting design will be satisfactory in general, and the fatigue life can be estimated with confidence.

To ensure the design by the stress-life method remains valid, it is important that the loads are defined for the entire service life with some accuracy such that periodic overloading will not occur. Further, the stress-life method is based on the application of characteristic S-N curves determined from load-controlled experimental testing of materials, details and connections. In general, the stress-life method is relevant to high-cycle fatigue life estimation and for structures or components that are loaded elastically. The design S-N curve is selected as appropriate for the detail classification and based on the approach used to account for the stress concentrations (i.e., nominal or hot-spot stress approaches). The curves then may be modified, as described in Section 6.7.3, to account for the effects of corrosive environments and thickness (size) effects.

7.4 Factored Cumulative Damage and Design Life

DNV-OS-J101 provides two methods for calculating the factored design cumulative damage. Method (1) computes the cumulative damage ratio, based on the characteristic S-N curve, and then multiples by a design fatigue factor (DFF), alternatively known as fatigue design factors (FDF) by ABS, for example. The DFF takes a value equal to 1.0, 2.0 or 3.0 based on the location of the structural detail, on the accessibility for inspection and repair, and on the type of corrosion protection provided.

Method (2) computes the design cumulative fatigue damage based on the number of cycles to failure at the design stress range, which is based on a material factor (effectively, a partial safety factor) for fatigue, γ_m, and the characteristic S-N curve. The material fatigue factor is related to the DFF values, based on the type of detail, accessibility and corrosion protection.

Note that the load factors for fatigue design are taken as equal to 1.0 for all load cases with the limit states design approach.

7.4.1 Design Fatigue Factors

Design fatigue factors (DFFs) may be applied to increase the required design life or to decrease the permissible damage, depending on the standard or guideline. In DNV-OS-J101, the design fatigue factors (DFF) values provided apply to structures or structural components having a low consequence of failure. It is noted that an OWTG is an unmanned structure, and compared to an offshore platform, for example, the consequences of failure will be reduced. Failure of the asset may occur, but there will be a very low probability for loss of life. DFF values are prescribed for other consequence levels. The value of the DFF depends on the type of structural detail; the type of component (tubular or non-tubular, for example); and, the criticality of the component. The latter is influenced in part by the accessibility of the component for inspection and maintenance. The DFF is interpreted as a partial safety factor applied to the characteristic cumulative fatigue damage, D_C, in order to obtain the design fatigue damage, D_D, as described by Equation 7.3.

Using Eqn. 7.1 and Eqn. 7.2,

$$D_C = \sum_{i=1}^{I} \left(\frac{n_i}{N_i}\right) \leq 1.0 \quad \text{Eqn. 7.3[a]}$$

And with,

$$D_D = DFF \times D_C \quad \text{Eqn. 7.3[b]}$$

Since the design criterion requires:

$$D_D \leq 1.0 \quad \text{Eqn. 7.3[c]}$$

$$\therefore D_C \leq \frac{1.0}{DFF} \quad \text{Eqn. 7.3[d]}$$

Similarly, ABS prescribes the application of DFFs in comparing the calculated fatigue life to the limiting criterion. This may be achieved in terms of the number of cycles or the cumulative damage by Equation 7.4 and Equation 7.5, respectively.

$$N_f \geq N_t \cdot \mathrm{DFF} \quad \text{Eqn. 7.4}$$

Where,

- N_f corresponds to the calculated number of cycles at the fatigue life for a particular structural detail; and,
- N_t is taken as the total number of stress cycles expected for the service life at that detail.

$$D \leq \frac{1.0}{DFF} \qquad \text{Eqn. 7.5}$$

Where,

- D corresponds to cumulative fatigue damage (by Equation 7.1).

Effectively, the design fatigue factor modifies the calculated fatigue life for each individual structural detail to account for uncertainties in the fatigue assessment and to further account for the consequences of failure. The overall calculated fatigue life for the OWTG is therefore given by the limiting fatigue critical detail.

As noted in Section 6.7.2, the design S-N curves (unmodified) may be associated with a probability of survival of approximately 97.6% (depending on the standard or guideline referenced). This corresponds therefore to a probability of failure of 2.4%. If all factors are considered equal, then the factored cumulative damage ratio as determined using a DFF taken equal to 1.0 would have a probability of failure equal to 2.4% at the design service life. Increased values of DFF therefore reduce the probability of failure in proportion. In the design of OWTG structures, particular components may have specific DFF values greater than 1.0 assigned during the design phase to ensure the service life requirements are met.

The ABS Guide for Offshore Wind Turbine Installations prescribes the values for fatigue design factors as listed in Table 7.1. As listed, the detail is considered only in terms of the criticality and in terms of the accessibility for inspection and repair. The uncertainty associated with inaccessible critical details is reflected in the assigned DFF taken as equal to 5.0. The towers of wind turbines as a rule contain components that are exclusively not fail-safe. Easily accessible components may include, for example, the bolts of ring flange connections and butt welds in the tower wall.

Table 7.1: Design Fatigue Factors by ABS

Inspection and Accessibility	Criticality	DFF (or, FDF)
Accessible for inspection and repair	Non-critical	1.0
	Critical	3.0
Not accessible for inspection and repair	Non-critical	3.0
	Critical	5.0

By comparison, DNV-OS-J101 applies the design fatigue factors as listed in Table 7.2. In addition to considering the accessibility of the component, DNV also takes into consideration the location of the component (reflective of the corrosion environment and coating conditions) and the type of design S-N curve that applies. Values for the DFF range between 1.0 and 3.0. As noted, DNV prescribes these values for structures having a low consequence of failure. By comparison, DNV prescribes a DFF equal to 10.0 for a manned offshore unit, which is assigned a high consequence of failure due to potential loss of life.

Table 7.2: Design Fatigue Factors by DNV-OS-J101

Inspection and Accessibility	Location	S-N Curve	DFF
Accessible for inspection and repair	Atmospheric Zone	"In air", for surfaces with coating; or, "Free corrosion", for surfaces protected by corrosion allowance only	1.0
	Splash Zone	Combination of curves marked "In air" and "Free Corrosion"	2.0
	Submerged Zone	"In seawater", for surfaces with cathodic protection; or, "Free corrosion", for surfaces protected by corrosion allowance only.	2.0
Not accessible for inspection and repair	All Zones	Any	3.0

For the OWTG tower structure, consisting of that portion at the waterline and above, the structure is considered as critical (i.e., not fail-safe) and generally accessible for inspection and repair of details or coatings. By ABS, the corresponding DFF is equal to 3.0; for DNV-OS-J101 the DFF is equal to 2.0. Therefore, the DFF values and hence the cumulative damages will differ when calculated in accordance with the different guidelines.

7.4.2 Material Factor for Fatigue

An alternative method, described by DNV-OS-J101 as Method (2) and that prescribed in the GL Guideline for the Certification of Offshore Wind Turbines, computes the design cumulative fatigue damage using the design stress range modified by a material factor for fatigue, γ_m.

As before, the cumulative damage ratio is calculated in accordance with Equation 7.6. However, the applied stress range used to determine the number of endured stress cycles is modified by a material factor, as given by Equation 7.7.

$$D = \sum_{i=1}^{J} \left(\frac{n_i}{N_i}\right) \leq 1.0 \qquad \text{Eqn. 7.6}$$

$$\Delta\sigma = \gamma_m \cdot \Delta\sigma_i \qquad \text{Eqn. 7.7}$$

Where,

- N_i is taken as the number of stress cycles associated with the applied stress range $\Delta\sigma_i$ and using the corresponding design S-N curve (modified, as appropriate);
- $\Delta\sigma_i$ corresponds to the i^{th} applied stress range
- γ_m is taken as a partial safety factor on the material for fatigue.

The material factor, γ_m, is a partial safety factor applied to all stress ranges before calculating the corresponding numbers of cycles to failure that are used to obtain the design fatigue damage. The material factors provided by DNV-OS-J101 and listed in Table 7.3 are related to the design fatigue factors and are valid when the applied number of load cycles during the design life is large, i.e. in excess of 10^7. DNV-OS-J101 does not specify DFF values for other conditions of the number of load cycles. By GL, the partial safety factors for material fatigue are determined according to Table 7.4.

In general, the values prescribed for the material factor are equivalent. However, it could be assumed that no part of the OWTG tower structure may be considered as non-fail safe such that the value of the material factor will not be taken as 1.0 (less conservative) for any condition.

Table 7.3: Material Factors for Fatigue by DNV-OS-J101

Inspection and Accessibility	Location	S-N Curve	DFF	γ_m
Accessible for inspection and repair	Atmospheric Zone	"In air", for surfaces with coating "Free corrosion", for surfaces protected by corrosion allowance only	1.0	1.00
	Splash Zone	Combination of curves marked "In air" and "Free Corrosion"	2.0	1.15
	Submerged Zone	"In seawater", for surfaces with cathodic protection. "Free corrosion", for surfaces protected by corrosion allowance only.	2.0	1.15
Not accessible for inspection and repair	All Zones	Any	3.0	1.25

Table 7.4: Material Factors for Fatigue by GL

Inspection and Accessibility	Location	γ_m
Periodic monitoring and maintenance; good accessibility; manufacturing and installation surveillance.	Part of a non-fail safe structure	1.15
	Part of a fail-safe structure	1.00
No periodic monitoring and maintenance possible or poor accessibility (e.g. under water or subsoil)	Part of a non-fail safe structure	1.25
	Part of a fail-safe structure	1.15

By factoring the applied stress range (for $\gamma_m > 1.0$), the number of stress cycles, N, obtained from the design S-N curve will be reduced. The degree to which N is changed depends on the slope of the design S-N curve. The general form of the S-N curve was described by Equation 7.1, rewritten in Equation 7.8 corresponding to the definition provided in the ABS Guide for the Fatigue Assessment of Offshore Structures.

For N is less than N_Q:

$$N = A \cdot S^{-m} \qquad \text{Eqn. 7.8[a]}$$

For N greater than N_Q:

$$N = C \cdot S^{-r} \qquad \text{Eqn. 7.8[b]}$$

Where,

- N corresponds to the number of cycles to failure;
- A and C are fatigue strength coefficients determined empirically;
- m and r are exponents corresponding to both slopes of the design S-N curve, also determined empirically;
- N_Q is the limiting number of cycles at which the slop of the design S-N curve changes; and,
- S is the applied stress range or, $\Delta\sigma$.

With reference to Equation 7.7, the expression in Equation 7.8 can be re-written as:

For N less than N_Q:

$$N = A \cdot (\gamma_m \cdot \Delta\sigma_i)^{-m} \qquad \text{Eqn. 7.9[a]}$$

And,

$$N = A \cdot (\gamma_m)^{-m} \cdot (\Delta\sigma_i)^{-m} \qquad \text{Eqn. 7.9[b]}$$

Such that,

$$(\gamma_m)^m \cdot N = A \cdot (\Delta\sigma_i)^{-m} \qquad \text{Eqn.7.9[c]}$$

Thus, for all applied stress ranges acting within the design S-N curve defined by the slope, m, the material factor may be considered as a form of a design fatigue factor. A factored cumulative damage ratio, D_f, could be expressed by Equation 7.10 as follows.

$$D_f = \sum_{i=1}^{J} \left(\frac{n_i}{A \cdot (\gamma_m)^{-m} \cdot (\Delta\sigma_i)^{-m}} \right) \leq 1.0 \qquad \text{Eqn. 7.10[a]}$$

$$D_f = (\gamma_m)^m \sum_{i=1}^{J} \left(\frac{n_i}{A \cdot (\Delta\sigma_i)^{-m}} \right) \leq 1.0 \qquad \text{Eqn. 7.10[b]}$$

$$D_f = (\gamma_m)^m \times D \leq 1.0 \qquad \text{Eqn. 7.10[c]}$$

$$D \leq \frac{1.0}{(\gamma_m)^m} \qquad \text{Eqn. 7.10[d]}$$

It should be noted that the total cumulative damage occurring at a detail depends on the effects of all applied stress ranges and the definition of the appropriate design S-N curves. Equation 7.10[d] illustrates the application of the material factor for the component of damage accumulated for stress ranges relating to the design S-N curve having the slope m.

For example, the detail category 'E' by the ABS Guide for the Fatigue Assessment of Offshore Structures is considered having a slope, m, at the first segment (for which N is less than N_Q) with a value equal to 3.0. If the detail under consideration is considered accessible for inspection and is part of a non-fail safe structure, the assigned material factor is equal to 1.15. Then, the factored cumulative design, D_f, is given as by Equation 7.11:

$$D_f = (1.15)^{3.0} \times D = 1.52 \times D \qquad \text{Eqn. 7.11}$$

The effect of the material factor on the cumulative fatigue ratio can thus be compared to the design fatigue factor, which for this example would be assigned a value of 2.0 (with reference to Table 7.2).

The selection to apply either the DFF or the partial material factor depends on the design philosophy. Both of these approaches help to ensure the fatigue service life is achieved. The level of conservatism introduced reflects the level of safety inherent in the design philosophy and the approach defined by the standard or guideline. Different standards will yield different fatigue lives and the desired level of conservatism must be assessed based on the local requirements. The design philosophy, safety factors, construction quality and inspection program must all be consistent and cannot be considered independently in order that the service life is achieved.

7.5 Fatigue Life Improvement

An absolute fatigue life cannot be known with any certainty for any steel structure. Since many factors are based on probabilities and assumptions, the fatigue life calculated during design is an estimate only. However, performance of the OWTG structure with respect to fatigue can be achieved for the service life by employing good design practice, including the following. A more detailed discussion is provided in Section 10.

- Selecting the appropriate details for design, with special consideration of plate thickness, stress concentrations, weld shapes, corrosion protection, and tubular members and connections (as are related to jacket or similar structures);
- Specifying details and workmanship (i.e., post-welding treatments like grinding) such that fatigue critical details are avoided or reduced; and,
- Establishing an inspection and maintenance program to ensure the performance over the life of the OWTG.

Some aspects of the structural design that may contribute to effective fatigue performance and to achieve the design service life include:

- Adjusting the tower diameter or wall thickness to achieve a weight-optimized and effective design;
- Employing post-tensioning in the tower design, or similar concepts, to contribute to minimising the cyclic tensile stresses, which will reduce the life at fatigue critical details and connections;
- Improving the structural damping by, for example, incorporating mass tuning to dampen the structural vibrations and to avoid exciting natural frequencies; and,
- Designing the appropriate structure for the water depth and environmental conditions, including investigating more recent design concepts such as multipods, TLP, or floating spars.

8 STRESS ANALYSIS DEMONSTRATION

8.1 Introduction

A fatigue and fracture assessment may be completed on the basis of the appropriate stress analysis according to the applications and techniques listed in Table 8.1. Local nominal stresses are those that would be calculated at the location of interest in the absence of the structural detail and weld stress concentrations. Stress concentrations resulting from the global or gross shape of the structure surrounding the detail are included in the local nominal stresses. Notch stresses are those that consider the effect of global structure; local structure; weld toe profiles and or existing cracks. They are used in notch stress fatigue initiation, crack growth or fracture analyses.

Each type of analysis may be completed using either analytic or numeric techniques. Analytic techniques correspond generally to formulae and parameters established on the basis of engineering mechanics (simple beam theory, for example) and empirical data. Numeric techniques typically include more complexity and involve finite element analysis to calculate directly the behavior.

Table 8.1: Fatigue and Fracture Assessment Stress Analysis

Application	Basis of Analysis	Analysis Technique	
		Analytic	Numeric
Fatigue	Global nominal stress	Closed-form beam or plate/shell theory	Finite element analysis
	Local nominal stress	Parametric formulae and global stress concentration factors to account for gross structural geometry and misalignment effects.	Coarse mesh global FEA is used to estimate local nominal stresses.
	Detail (Hot-spot or notch) stress	Parametric formulae for stress concentration factors at details.	Detailed local fine mesh finite element stress analysis. Extrapolate detail local maximum principal stress data to estimate hot-spot stress. The weld notch SCF is inherent in the S-N curve so does not appear in the calculations.
Fatigue and Fracture	Notch stress	Parametric formulae and notch stress concentration factors to account for welds or cracks.	Weld toe or crack tip finite element models used to estimate local SCFs or stress intensity factors.

8.2 Stress Components and Categories

Figure 6.1 illustrated the four components of the total stress field acting at a structural detail, including the distributions of membrane stresses; bending stresses; peak stresses; and, residual stresses.

- The membrane stress, σ_m, is uniformly distributed and equal to the average stress across the section thickness, such that $\sigma_m = P/A$.
- The bending stress, σ_b, is the component of nominal stress due to applied loading which varies linearly across the section thickness, such that $\sigma_b = My/I$.
- The peak stress, σ_p, includes the stress component due to local discontinuities in the vicinity of point of interest (crack initiation site) and represents the highest local stress value, usually at surface at a notch (e.g., weld toe).
- The residual stress, σ_r, corresponds to the local self-equilibrating stress from fabrication and welding and includes components of membrane and bending. Residual stresses add to the tensile stress field in the crack vicinity and should be considered in residual strength assessments, however they are not considered in the fatigue design process since their effect is accounted for in the S-N curves (being derived from testing of welded connections and components).

The peak stresses arise from stress concentrations due to:

- Geometric stress concentrations (K_g): gross detail geometry;
- Notch stress concentrations (K_w): local (weld toe) notch geometry; and,
- Misalignment stress concentrations (K_{te}, $K_{t\alpha}$): eccentric and angular misalignment, respectively.

The total stress field is therefore described by the sum of the contributing components of stress. The maximum value of the total stress at a crack location thus can be evaluated by Equation 8.1.

$$\sigma_1 = \sigma_m + \sigma_b + \sigma_p + \sigma_r = K_g \cdot K_w \cdot (K_{te} \cdot K_{t\alpha} \cdot \sigma_m + \sigma_b) + \sigma_r \qquad \text{Eqn. 8.1}$$

The stress categories are introduced in Section 6.2 and are summarized in Table 8.2 with the associated components of stress.

Table 8.2: Summary of Stress Categories and Components

Category	Stress Components	Description
Nominal Stress	$\sigma_{nominal} = \sigma_m + \sigma_b$	Global stress concentration (K_G) effects are included. Local detail; alignment; weld toe; or, geometry notch effects (K_w) and residual stresses are included in the S-N curve.
Hot-Spot Stress	$\sigma_1 = K_g \cdot (K_{te} \cdot K_{t\alpha} \cdot \sigma_m + \sigma_b)$	Weld toe or geometry notch effects (K_w) and residual stresses are included in the S-N curve.
Notch Stress	$\sigma_r = K_g \cdot K_w \cdot (K_{te} \cdot K_{t\alpha} \cdot \sigma_m + \sigma_b)$	Residual stress effects are included in the S-N curve.

8.3 Nominal Stress Analysis Techniques

8.3.1 Nominal Stress (Analytic)

Development of the nominal stresses using analytic techniques draws from experience in the design of ships by applying a hull girder analogy, which is based on simple beam theory and the following assumptions:

- Plane cross-sections remain plane;
- Stresses remain in the elastic range and thus allow superposition;
- The beam is essentially prismatic; and,
- There is no interaction between bending and other response modes.

The primary bending stresses are determined from the results of global ‘hull girder’ bending, while the local nominal stress corresponds to the sum of primary, secondary and tertiary effects. Secondary and tertiary stresses are associated with bending of the local structural members from lateral pressures.

The primary bending stresses associated with the hull girder bending, follow from Equation 8.2.

$$\sigma_{m,v} = K_G \cdot \frac{Mz}{I} \qquad \text{Eqn. 8.2}$$

Where,

- M is the moment amplitude;
- z is the distance from the neutral axis;
- I corresponds to the moment of inertia about the neutral axis; and,
- K_G is the global stress concentration factor to account for gross structural geometry (e.g., openings, shear lag, etc...), illustrated in Figure 8.1.

The geometric section properties (I and z) are based on the continuous structure effective in supporting the transverse loads, including:

- Primary tower structure;
- Continuous internal vertical stiffening elements; and,
- OWTG transition piece.

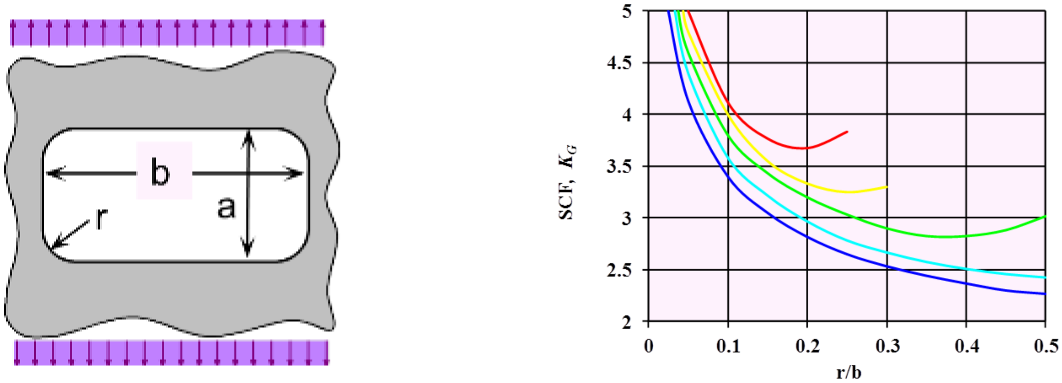


Figure 8.1: Global Stress Concentration for Cutouts or Openings

Computer programs have been developed to facilitate stress analysis by idealizing a stiffened structure using plate or beam elements accounting for the contribution of the stiffeners in the section properties for the simplified element.

8.3.2 Nominal Stress (Numeric)

For critical structural elements, or complex structures, finite element analysis (FEA) is used to estimate the global structure stress and displacement distribution. This can be completed generally using a relatively coarse finite element mesh, often using plate and beam elements to represent stiffened sections. The extent of the model included in the representation may be limited depending on the type of response to be analyzed; i.e., a limited global model may be used, unless the complexity of the OWTG internal structure dictates that the entire system shall be included.

The global finite element model is then loaded using the internal and external loads associated with the long-term distribution of the external conditions.

8.4 Detail (Hot-Spot or Notch) Stress Analysis Techniques

8.4.1 Detail Stress (Analytic)

Parametric SCF's for typical structural details (K_g) and misalignment effects (K_{te}, $K_{t\alpha}$) have been developed for local elements and stiffeners welded to plates; and, joints having angular mismatch at butt welds and stiffeners. SCFs are also defined for weld types such as butt welds and fillet welds (K_w is taken as 1.0 for non-welded components). These SCFs have in general been adopted for the design of OWTG structures and are identified in the standards and guidelines.

8.4.2 Detail Stress (Numeric)

If stress concentration factors are not available, detail stresses are calculated by local finite element analysis. For ships, the International Association of Classification Societies (IACS) rules define the conditions requiring FEA. For this application, a fine mesh finite element model is required, derived as a sub-model (of the global model) large enough to ensure that results are not affected by the boundary conditions and applied loads. The local model thus is essentially a substructure cut from the global model and loaded with displacements and/or forces from the global model. The standards and guidelines prescribe various considerations for defining the fine mesh local model, including the use of constraint equations; the transition from coarse to fine mesh; and, the use of quadrilateral or triangular elements, including aspect ratios.

8.4.3 Hot-Spot Stress Extrapolation

Hot-spot stresses can be considered as a weld location nominal stress and include all SCFs except those associated with the weld toe. Using the calculated stresses at the surface and perpendicular to a crack direction, the stress data is interpreted from the model as (i) a single value; or, (ii) as an extrapolation of multiple point values to the weld toe. The principal stresses are then estimated based on the stress components extrapolated to the hot-spot location.

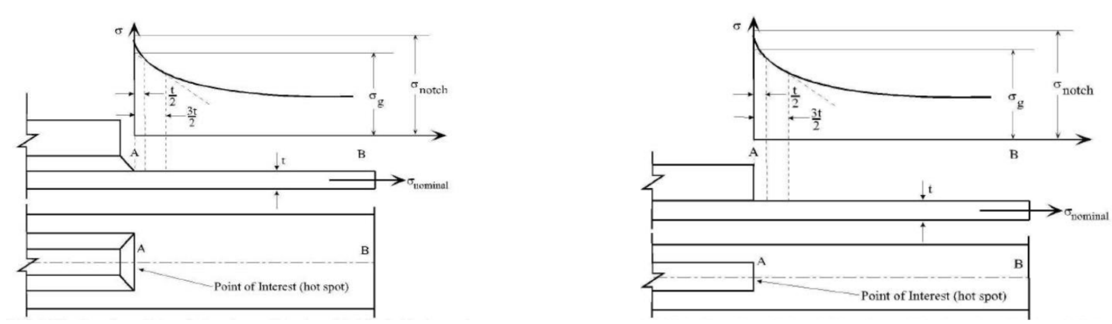
Figure 8.2 illustrates the extrapolation for the FEA hot-spot stresses using solid and shell elements. As a demonstration, using the case of the solid elements, the hot-spot stress may be defined according to Equation 8.3[a] using a single point assessment; or, Equation 8.3[b] using a linear extrapolation. A quadratic extrapolation option is also available, requiring three points near the weld toe.

Single point assessment:

$$\sigma_{Hot-Spot} = \sigma_A \qquad \text{Eqn. 8.3[a]}$$

Linear extrapolation:

$$\sigma_{Hot-Spot} = \sigma_A + \frac{(\sigma_A - \sigma_B)}{T} \times \frac{T}{2} = \sigma_A + \frac{(\sigma_A - \sigma_B)}{2} \qquad \text{Eqn. 8.3[b]}$$

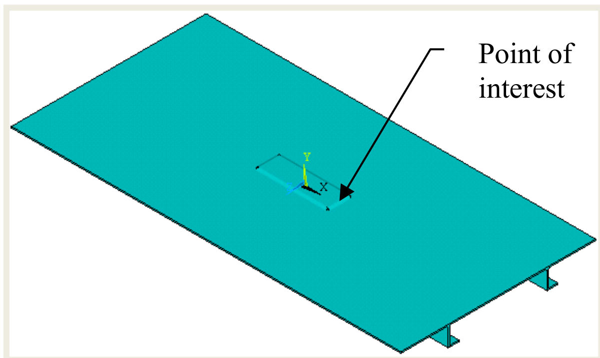


(a) Solid elements **(b) Shell elements**

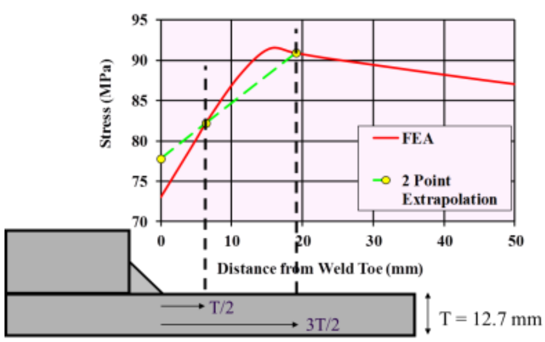
Figure 8.2: Detail Local Finite Element Model Hot-Spot Stress Extrapolation

Figure 8.2 illustrates the two point extrapolation (linear) approach for solid and shell element models. In each case, the extrapolation points are located at distances of *t*/2 and 3*t*/2 from the hot-spot location. Here, the net thickness (accounting for a corrosion margin) is used for *t*. When the weld toe is modeled explicitly, the hot-spot is located at the weld toe itself. When the weld toe is not modeled directly, the hot-spot is taken as the intersection of the finite elements.

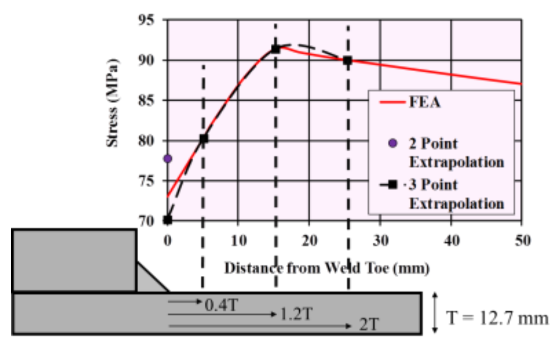
Figure 8.3(a) illustrates an example case of a doubler plate repair welded to a stiffened steel panel. The hot-spot stress extrapolations in areas of significant bending local to the doubler plate are shown in Figure 8.3(b) and Figure 8.3(c) for the linear extrapolation and quadratic extrapolation (Hobbacher 2003).



(a) Doubler plate repair



(a) Linear extrapolation



(b) Quadratic extrapolation

Figure 8.3: Hot-Spot Stress Extrapolation Example

The suitability of the extrapolation method must be confirmed for the detail under consideration to ensure that the value obtained does not significantly underestimate or overestimate the stress. Variability in the calculated results for the hot-spot stress may lead to significantly different fatigue lives using the design S-N curves and the calculated damage accumulation.

8.5 Notch Stress (Fracture) Analysis Techniques

8.5.1 Notch Stress (Analytic)

In a notch stress fatigue initiation analysis the detail (hot-spot) stress is modified by including a notch stress concentration factor (K_w). These notch stress concentration factors for welds, K_w, may be taken as equal to 1.5, or as determined using Equation 8.4. Further, stress intensity factors (K_I) are calculated using Equation 8.5 to represent non-linear crack stress/strains.

$$K_w = 1 + 0.27 \tan(\theta)^{0.25} \left(\frac{t}{r}\right)^{0.25} \qquad \text{Eqn. 8.4}$$

Where,

- t is plate thickness;
- r is weld toe radius; and,
- θ is the angle between the base plate and the weld face, illustrated in Figure 8.4.

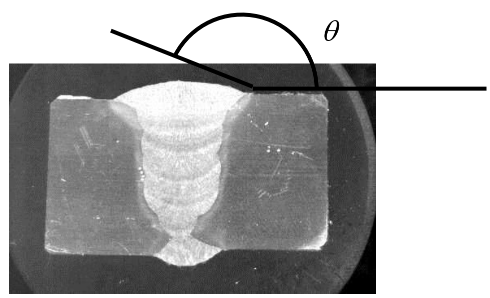


Figure 8.4: Angle between the Base Plate and Weld Face

$$K_I = Y_m \cdot \sigma_1 \cdot \left(\sqrt{\pi a}\right) \cdot f_w \qquad \text{Eqn. 8.5}$$

Where,

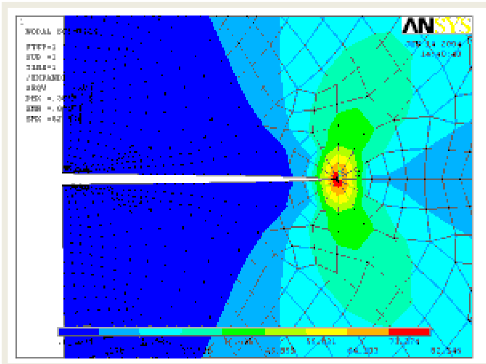
- f_w is the finite width correction (from handbooks);
- Y_m corresponds to the flaw shape factor (from handbooks); and,
- σ_1 is given by Equation 8.6.

$$\sigma_1 = K_G \cdot K_g \cdot K_W \cdot (K_{te} \cdot K_{t\alpha} \cdot \sigma_m + \sigma_b) \qquad \text{Eqn. 8.6}$$

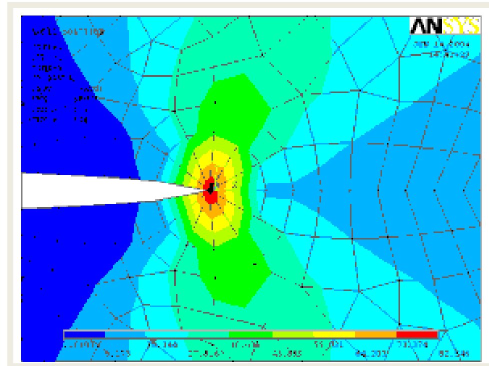
8.5.2 Notch Stress (Numeric)

A numerical solution to the determination of the notch stress concentration factor (used for a notch stress fatigue crack growth approach) requires an extremely fine finite element mesh. Typically, to ensure accurate stresses are calculated, the notch radius is of the order of 1.0 mm and at least one element per 15 to 30 degrees of radius. The finite elements at the notch tip are taken as the lesser of 1/8th of the crack length, or 1.0 mm. Figure 8.5 illustrates a fine mesh used for a notch stress finite element model. The notch stress concentration is localized and only extends 10% of plate thickness. Typically the model would accommodate at least one plate thickness of the fine mesh elements. The mesh density and element type requirement are evolving with the advancement of finite element codes and would need to consider the software being used.

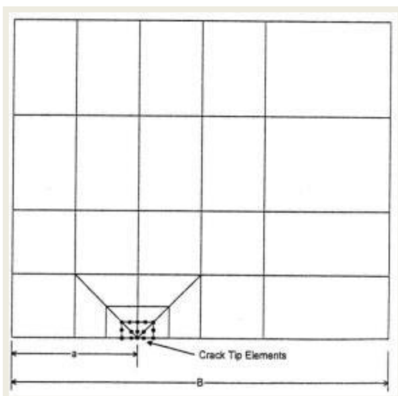
Figure 8.5(a) and (b) illustrate the model at the crack tip (i.e., the location of high stresses). Where published solutions are not available for a prescribed detail, finite element analysis can be used to model the crack tip singularity. Initially, hundreds of conventional elements were used to model crack tips; however, more modern finite element formulations have been developed to account for the singularity for many commercial FEM programs. Typically, this corresponds to modified triangular or rectangular quadratic iso-parametric elements. Figure 8.5(c) and (d) illustrate a 2D and 3D finite element mesh at crack locations.



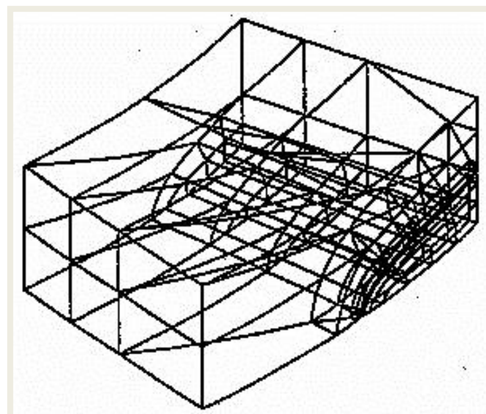
(a) Solid elements



(b) Shell elements



(c) 2D model of edge cracked plate



(d) 3D model of semi-elliptical surface crack

Figure 8.5: Notch Stress Finite Element Mesh

9 ENVIRONMENTAL EFFECTS ON FATIGUE

9.1 Introduction

Steel structures operating in marine environments are highly susceptible to corrosive attack, which influences significantly the fatigue performance and the design for fatigue. Corrosion will occur in the structure for two basic conditions:

(i) free corrosion; or

(ii) corrosion in areas with cathodic protection.

Naturally, as a structure becomes corroded and the thickness is reduced, the residual fatigue life will decrease. However, a distinction is made between the fatigue of corroded structures and corrosion fatigue. A general discussion of corrosion fatigue and fatigue of corroded structures is provided (Semiga and Tiku April 2007).

9.2 Corrosion

The areas commonly affected by localized corrosion attack in ship structures are illustrated in Figure 9.1, for which most corrosion occurs at cutouts, complex structure in bilges, machinery seatings, and ballast tanks. Figure 9.2 illustrates the result of a corrosive attack at a plate-to-stiffener connection, termed the corrosive necking effect due to the reduced section of steel. The causes of this necking can include dissimilar material enhanced corrosion; coating application deficiencies; and, local mechanical actions (i.e., strains) that promote holidays (i.e., breaks or discontinuities) in the coating and corrosion product to accelerate local corrosion.

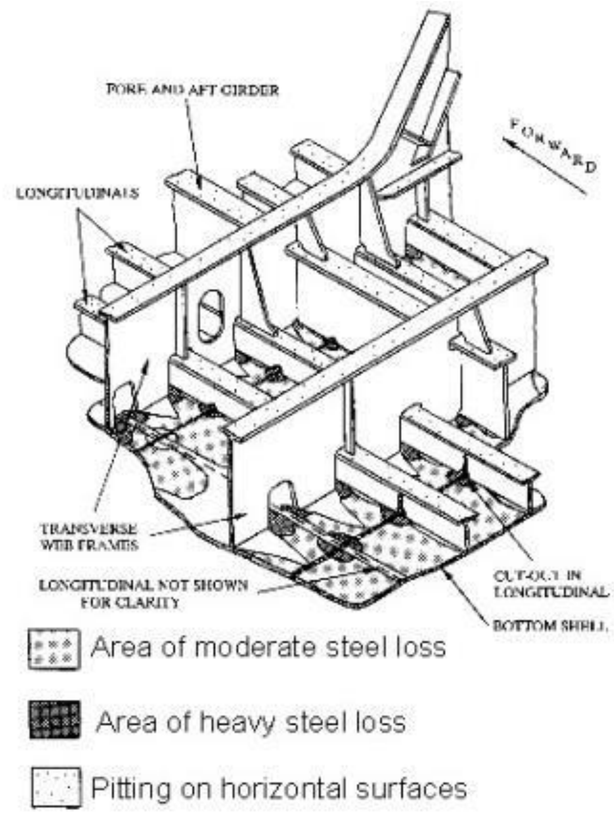


Figure 9.1: Typical Areas of Localized Corrosion Attack in Ship Structures

Figure 9.2: Corrosion Necking Effect

There are many methods by which the effects of corrosion are prevented (or mitigated), including protective coatings; cathodic protection; and, material selection. Protective coatings are often the simplest method, but the effectiveness is governed by the quality of the application and the behavior of the environment. Any break in the protective coating leads to rapid localized corrosive attack.

Cathodic protection involves suppressing the anodic reaction by subjecting the structure to an electric field. Variations in the electric potential are setup depending on the location within the structure. This can be achieved using sacrificial anodes or applied potentials. The optimum cathodic protection may occur generally between -0.8 V and -0.9 V (with a standard calomel electrode, SCE).

Material selection will influence the susceptibility to corrosion and charts, such as that shown in Figure 9.3, are developed to provide guidance.

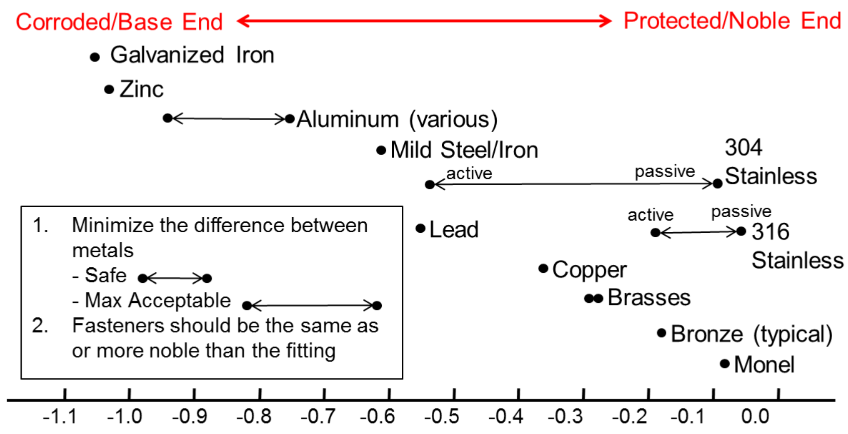


Figure 9.3: Galvanic Chart for Material Selection

9.3 Overview of Fracture Mechanics Approach to Fatigue Life Assessment

Fracture mechanics assumes that a crack or a crack-like flaw already exists in a component. Therefore, the approach considers only the crack propagation portion of a component's fatigue life. The total life of a component is estimated by calculating the number of cycles required to propagate a crack from a sub-critical size to a critical size. Crack growth is calculated based on the geometry of the component; the crack size; the applied loading; and, an experimentally determined crack growth rate (*da*/*dN*). The latter is determined using Equation 9.1 (Paris and Erdogan 1963):

$$\left(\frac{da}{dN}\right) = C(\Delta K^m) \qquad \text{Eqn. 9.1}$$

Where,

- *da*/*dN* is the crack extension per load cycle;
- *C* and *m* are material constants derived from experimental crack growth data considering only one crack growth region (slope), as illustrated in Figure 9.4; and,
- *ΔK* corresponds to the stress intensity factor range.

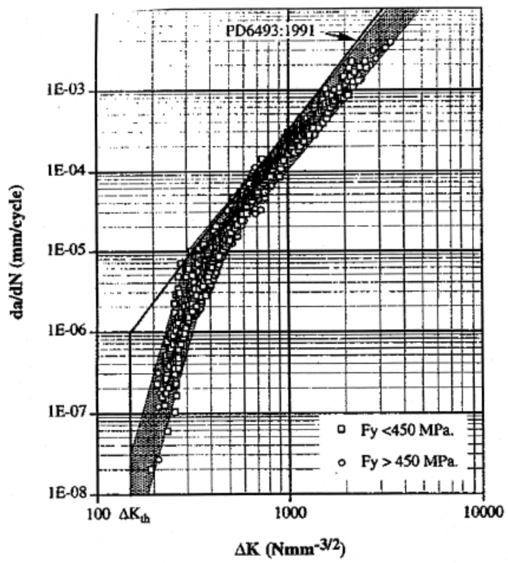


Figure 9.4: Experimental Crack Growth (*da*/*dN*) Data

9.4 Corrosion and Fatigue

In this section, a general discussion is provided regarding the fatigue of corroded structures; and, corrosion fatigue.

9.4.1 Fatigue of Corroding Structures

Typically, the standards and guidelines for OWTG design or offshore structures, adopt a stress life (S-N) approach to fatigue life assessment. The applied stress levels are calculated based on the net scantling thickness, which represents the minimum allowable section for a given structural component and thus ensures that the highest permitted applied stress will be used during the fatigue assessment. The applied stress levels are then used in conjunction with design S-N curves to estimate the fatigue life of the component. The design S-N curves, depending on the standard and guideline, are defined (in part) based on the operating environment (i.e., in air or in water) and the availability of cathodic protection.

The fatigue evaluation of corroding structures is a simplified approach to address the effects of corrosion on the component by using the reduced (i.e., net) section scantlings. However, the applied stress levels so determined and the use of the design S-N curves with the stress-life approach do not address the effects of a corrosive environment with respect to crack initiation or crack propagation. Since a corrosive environment is generally detrimental to both crack initiation and crack propagation, this approach may be un-conservative when estimating the fatigue life of a component.

9.4.2 Corrosion Fatigue

Corrosion fatigue refers to the effects of the interaction between cyclic mechanical loading and a corrosive environment. The effects of corrosion fatigue can manifest themselves in both the crack initiation and the crack propagation stages of crack growth.

As noted previously, corrosion decreases the net effective section of a structural component, which has the effect of increasing the applied stress levels and reducing the overall life of a component. Further, the interaction between the corrosive environment and the cyclic mechanical loading generally results in a significant increase in the rate of crack growth, as compared to that in a non-corrosive environment, which further reduces the overall fatigue life. The effect of the corrosive environment on the fatigue behavior of a structural component is a function of several factors including:

- Geometry of the structural component;
- Environment chemistry (water and seawater);
- Length of exposure;
- Frequency of loading;
- Stress ratio;
- Stress range;
- Electrode potential; and,
- Cathodic protection.

Corrosive environments may affect the crack initiation behavior of an initially crack free component through two mechanisms (Suresh 1998). In the first mechanism, corrosion pits, as shown in Figure 9.5, act as micro-notches to increase locally the stress level. Cracks may initiate at the root of the corrosion pit. In the second mechanism, corrosive environments (such as water and seawater) may introduce hydrogen into the metal by the dissociation of hydrogen molecules into atomic hydrogen. Under cyclic loading conditions, the resulting hydrogen embrittlement of the material may accelerate the initiation of surface flaws.

Figure 9.5: Corrosion Pits

Figure 9.6 illustrates the hydrogen embrittlement process (or, hydrogen assisted cracking) involving the absorption and transport of atomic hydrogen from the corrosive environment into the highly stressed regions at the crack tip (plastic zone) where it causes localized damage and increases the fatigue crack growth rate. The process illustrated in Figure 9.6 is sequential and therefore the rate of corrosion fatigue crack growth is controlled by the slowest process in the sequence (Pao 1996).

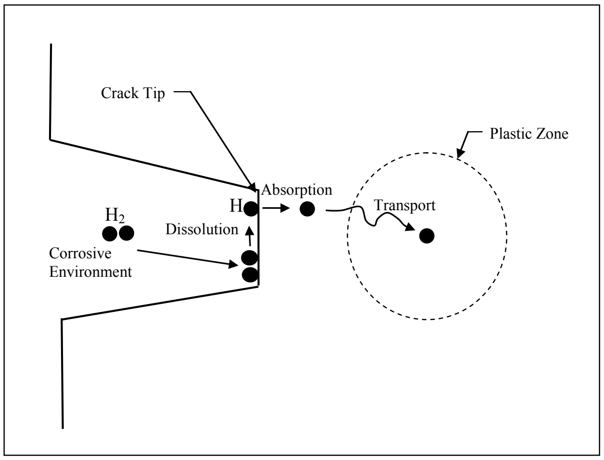


Figure 9.6: Hydrogen Embrittlement Process

Hydrogen embrittlement affects the behavior of a material by various mechanisms, as follows:

- Pressure mechanism: Atomic hydrogen recombines to form molecular hydrogen at molecular discontinuities in the area of the crack tip and builds up high internal pressures;
- Lattice decohesion mechanism: Hydrogen decreases the cohesive bonding forces between adjacent metal atoms;
- Adsorption mechanism: Adsorbed hydrogen at the surface lowers the surface energy of the metal and therefore facilitates crack extension and increases the crack growth rate;
- Hydride mechanism: Adsorbed hydrogen forms brittle hydrides, which assist cracking; or,
- Hydrogen enhanced plasticity: Hydrogen assists plastic flow by reducing the stress required to generate or move dislocations through the metal.

In corrosion fatigue, anodic dissolution can occur at the crack tip, as illustrated in Figure 9.6. When subjected to an applied cyclic stress, the protective oxide layer (i.e., passive film), which forms on the exposed surfaces at a crack tip, can rupture and new metal is exposed to the environment allowing for additional dissolution. The rate of corrosion fatigue associated with anodic dissolution depends on several variables including the oxide formation rate (repassivation rate); the oxide rupture rate (loading frequency); the bare metal dissolution rate; and, the mass transport rate of reactants to the dissolving surface.

9.5 Variables Influencing Corrosion Fatigue Rates

Some of the primary factors influencing the corrosion fatigue rate are described in the sections that follow.

9.5.1 Fatigue Loading Frequency

Corrosion fatigue involves a number of rate dependent processes, including mass transport rate; oxide rupture (formation) rate; repassivation rate; and, dissolution rate. Therefore, corrosion fatigue generally is a highly time dependent process, unlike that which occurs in an inert environment. The fatigue loading frequency therefore has a significant effect on the corrosion fatigue crack growth rate, as illustrated in Figure 9.7, which presents the crack growth rate (da/dN) versus the stress intensity factor range (ΔK) for AISI 4340 steel in water vapor at various loading frequencies. As shown, with increasing frequency the crack growth rate decreases until reaching 20 Hz, and the corrosion fatigue crack growth rate approaches the fatigue crack growth rate in an inert argon environment.

Although the sample data illustrated in Figure 9.7 was developed for an atypical steel, the information indicates that corrosion fatigue has the potential to contribute to an increase in fatigue crack growth rate, but this effect is a function of the frequency of loading.

9.5.2 Fatigue Loading Ratio

Figure 9.8 illustrates the crack growth rate data for MF-80 HSLA steel in a 3.5% NaCl solution. In general, higher fatigue loading ratios (R = $\sigma_{min}/\sigma_{max}$) result in higher corrosion fatigue crack growth rates and lower corrosion fatigue crack growth thresholds. For cases of fully reversed loading, the fatigue loading ratio may have a value equal to approximately -1.0. However, factors such as weld residual stresses may promote a positive mean stress state. It is therefore possible that high fatigue loading ratios could promote higher corrosion fatigue crack growth rates, as illustrated in Figure 9.8.

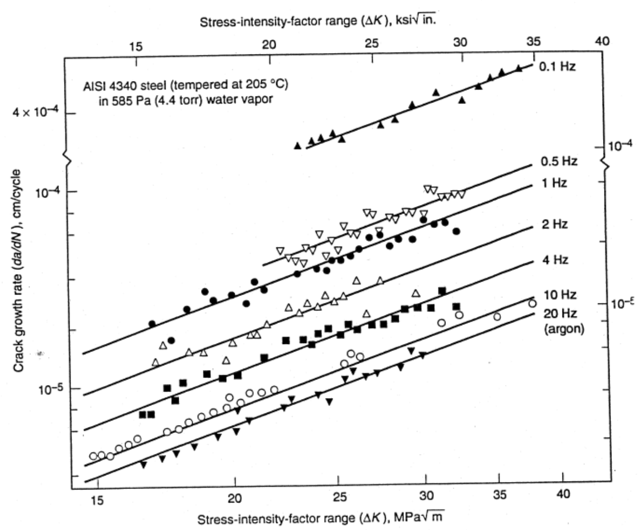


Figure 9.7: Effect of Loading Frequency on Corrosion Fatigue Crack Growth Rates (Pao 1996)

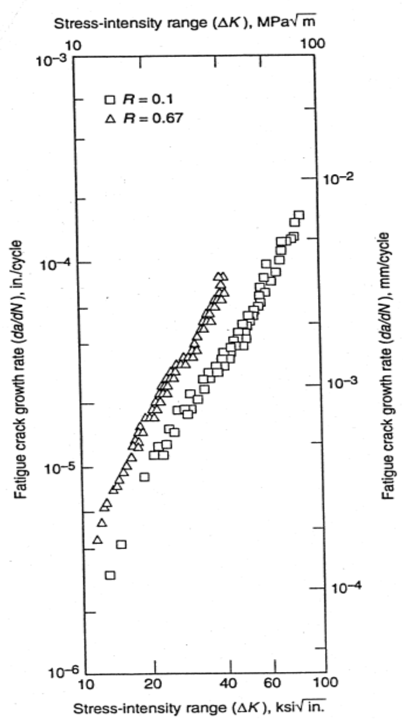


Figure 9.8: Effect of Load Ratio on Corrosion Fatigue Crack Growth Rates (Pao 1996)

9.5.3 Fatigue Load Range

The horizontal axes of Figure 9.7 and Figure 9.8 reference the stress intensity factor range, ΔK, which corresponds to the fatigue load range and affects the crack growth rate. As illustrated in Figure 9.9, in a corrosive environment, the crack growth rate curves can exhibit distinct types of behavior. For lower values of ΔK, the crack growth rate is influenced by a threshold stress intensity factor below which corrosion fatigue does not occur. This is represented generally as the Stress Corrosion Cracking Threshold, or K_{ISCC}. Note that K_{ISCC} is generally below the ΔK_{th} value associated with fatigue crack growth in an inert environment. Above the value of K_{ISCC}, the crack growth rate increases rapidly with increasing ΔK, followed by a reduction in crack growth rate with increasing ΔK. In general, the result is that a corrosive environment results in higher crack growth rates for most of the ΔK range of interest in typical marine structural assessments.

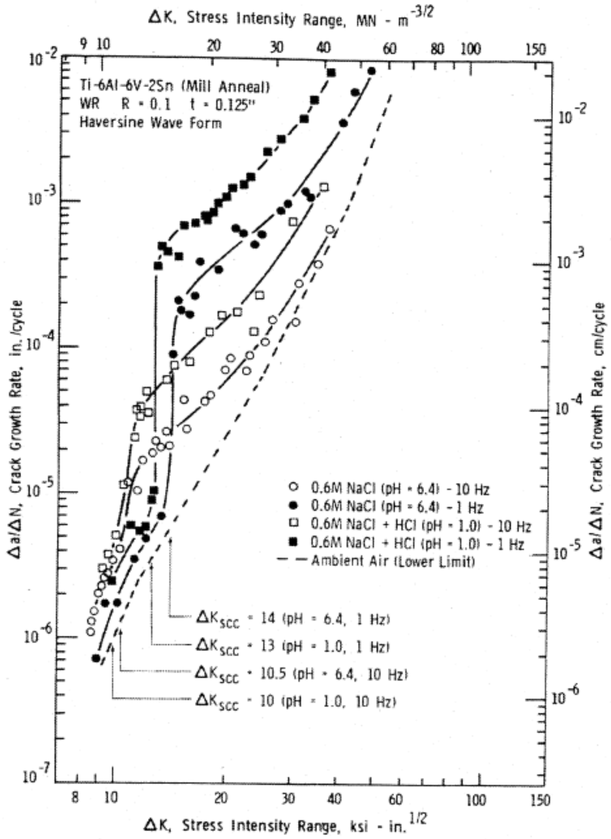


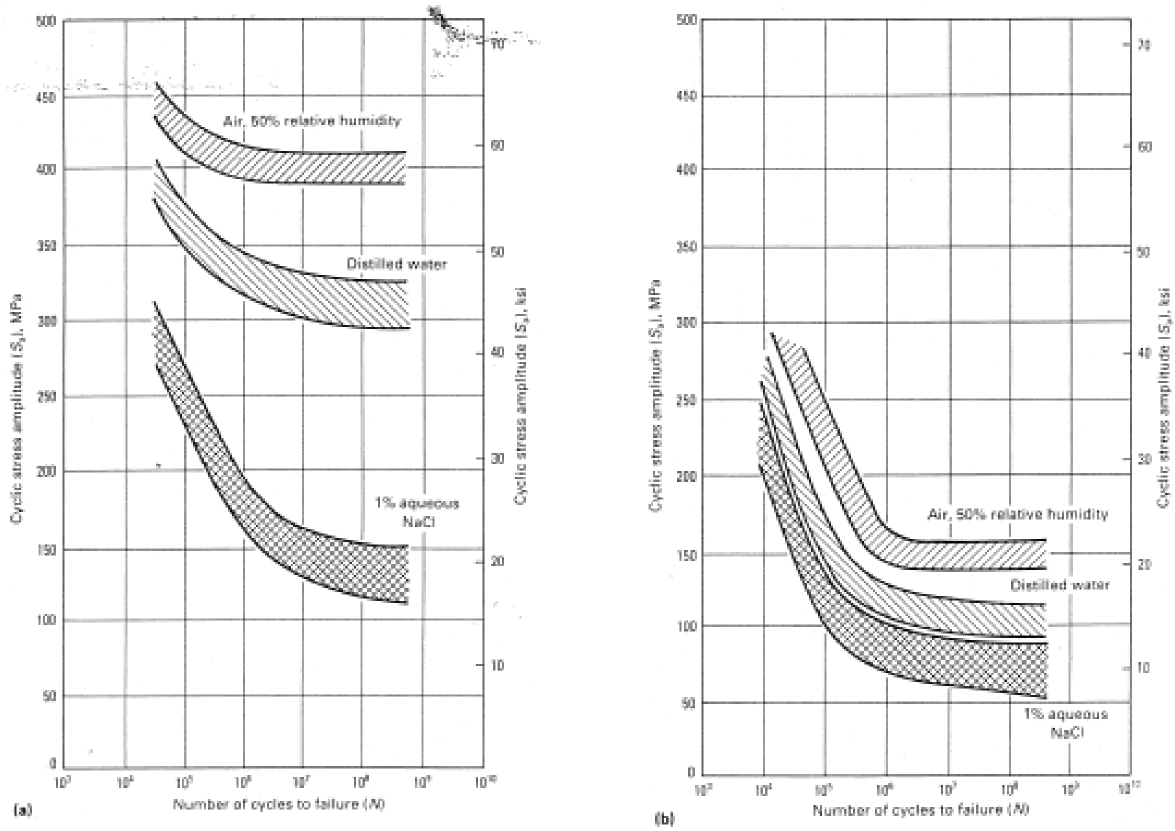
Figure 9.9: Effect of Load Range on Corrosion Fatigue Crack Growth Rate (Gangloff 2005)

9.5.4 Environment Chemistry

Corrosion fatigue rates are highly dependent on the chemistry of the operating environment. In general, more aggressive environments yield more pronounced corrosion fatigue rates. For example, in distilled water crack growth rates are generally lower than those occurring in seawater (containing NaCl). Similarly, environments containing H_2S gas (such as for sour crude oil) generally result in higher crack growth rates than do either distilled water or seawater. The higher the concentration of H_2S gas, the higher the crack growth rate.

9.5.5 Component Geometry

The geometry of the structural component, including specifically any notches that exist, has a critical effect on crack initiation and thus on fatigue life. Figure 9.10 illustrates stress-life (S-N) data for 13% Cr steel for different environments for both smooth and notched bar fatigue specimens. As shown, the effect of a corrosive environment is more pronounced in smooth specimens than in notched specimens. In inert environments, crack initiation generally occurs more quickly in notched specimens than in smooth specimens due to the stress concentrating effects of the notch. Once a macro crack has formed in the specimen, the effect of a notch is considered negligible and the crack propagation stage for smooth and notched specimens are considered to be similar. However, in a corrosive environment, corrosion pits formed on the surface of a smooth specimen act as micro notches and accelerate fatigue crack initiation and reduce the overall fatigue life of the component.



(a) Smooth bar specimen **(b) Notched bar specimen**

Figure 9.10: Effect of Specimen Geometry on Corrosion Fatigue Life (Gangloff 2005)

10 EFFECT OF FABRICATION QUALITY ON FATIGUE

10.1 Introduction

Fatigue cracks in steel structures are commonly initiated at weld toes due to the presence of an initial defect or a change in geometry. Further, local structural geometry and material considerations can play a role in promoting fatigue. However, in many instances, fatigue cracking may be avoided through the application of proper detailing and fabrication practice. The effects of fabrication (weld) faults and detailing practices are discussed with respect to the effects on fatigue life.

10.2 Dimensional Weld Faults

Defects associated with dimensional weld faults can be considered with respect to those prior to welding and those which occur after welding, described as follows:

- Dimensional faults (prior to welding): Dimensional faults prior to welding are related to poor workmanship and indicate a lack of quality control, as listed in Table 10.1. Dimensional fault limits can be in fabricator quality standard or in design specification.
- Dimensional faults (after welding): Dimensional faults after welding are related to poor workmanship and quality control, including those listed in Table 10.2. Distortion limits are specified in applicable design code or standard. Figure 10.1 illustrates different effects of distortion associated with welded steel construction.

Table 10.1: Dimensional Weld Faults (Prior to Welding)

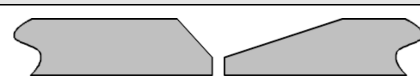
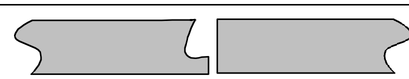
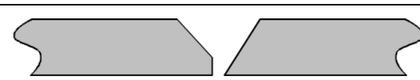
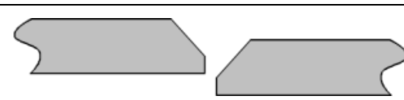
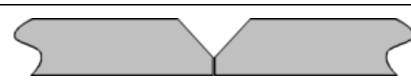
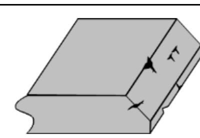
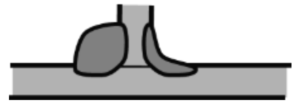
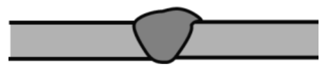
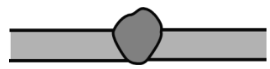
Type	Example
Incorrect bevel angle	
Incorrect J-groove radii	
Incorrect root face	
Incorrect fit-up	
Incorrect root opening	
Irregular joint preparation surfaces	

Table 10.2: Dimensional Weld Faults (After Welding)

Type	Example
Convexity or Concavity	
Internal & External Undercut	
Insufficient throat or leg	
Overlap	
Excessive reinforcement	
Out of line weld beads	

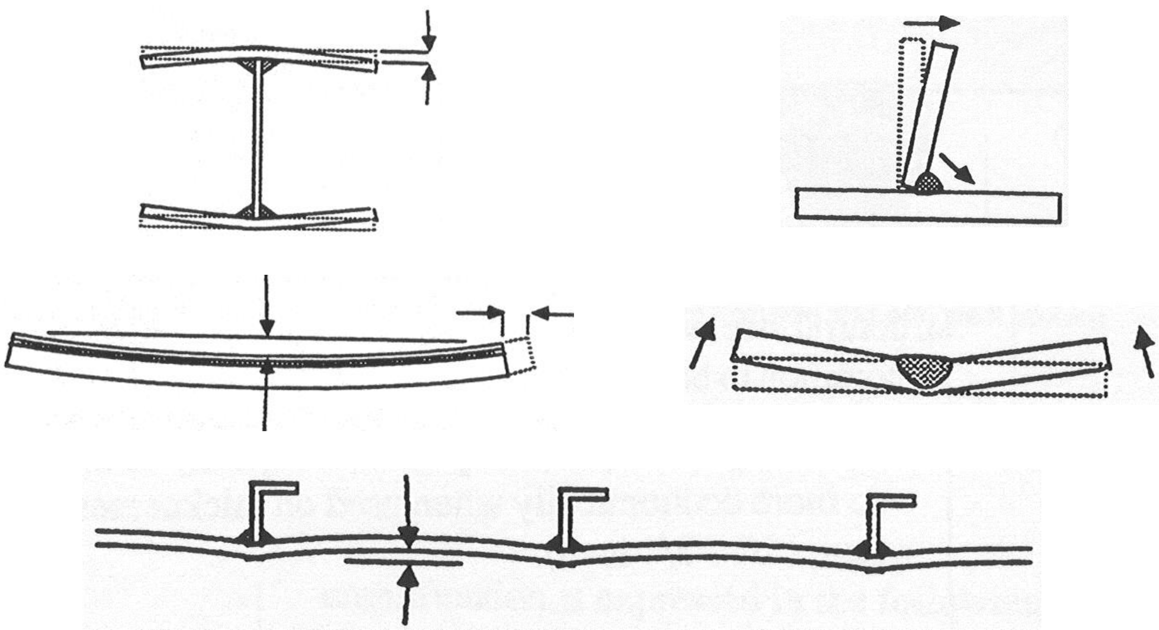
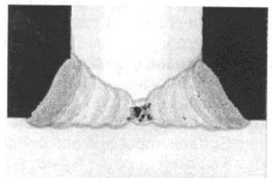
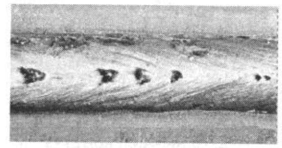
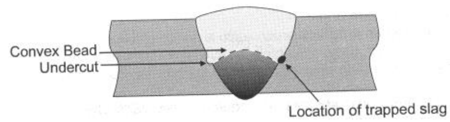
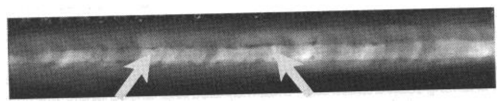
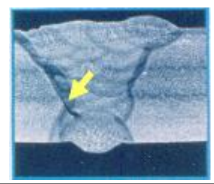
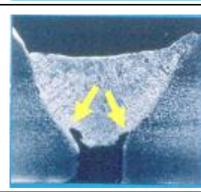
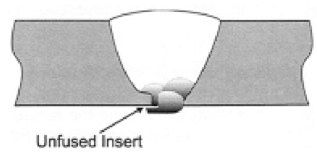
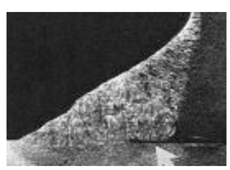
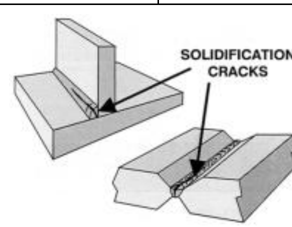
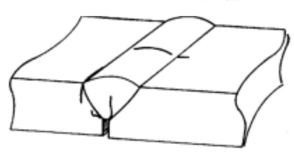
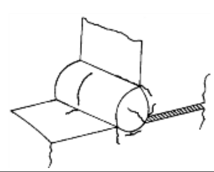


Figure 10.1: Welded Steel Plate Distortion

10.3 Structural Weld Faults

Weld structural discontinuities and weld metal discontinuities are related to poor workmanship and quality control, but may also be related to the welding procedures. Table 10.3 provides a list of example structural weld faults.

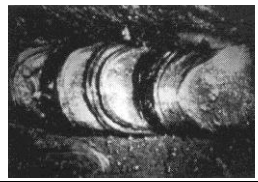
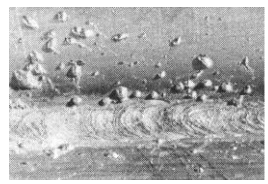
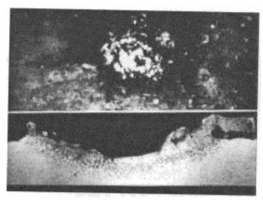
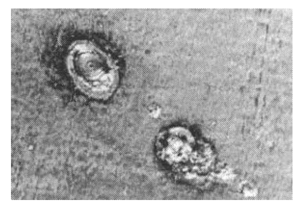
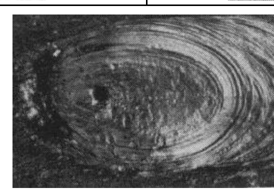
Table 10.3: Structural Weld Faults

Type	Example
Gas inclusions (porosity) including gas holes and piping	 
Inclusions such as: • Slag and slag lines • Slag missed in back gouging • Copper • Tungsten	 
Lack of fusion • Root • Side wall • Cold lap • Un-fused insert/backing	   
Incomplete penetration	 
Solidification cracks, hot cracks or cold cracks	  

10.4 Surface Weld Faults

Surface defects are associated with the welding process and may occur away from the weld area and include those listed in Table 10.4.

Table 10.4: Surface Weld Faults

Type	Example	
Unsatisfactory surface		
Spatter		
Stray arc strikes or flashes		
Craters		

10.5 Weld Faults Related to Material Properties

Specific material properties are required of a weld and are checked in the development of welding procedures to avoid weld faults attributable to improper welding conditions or technique and use of improper base or weld metal materials. The mechanical properties of specific interest include the material strength (yield and ultimate tensile); material hardness; toughness; and, ductility. Chemical properties regarding the weldability, hardenability and corrosion resistance are also important considerations.

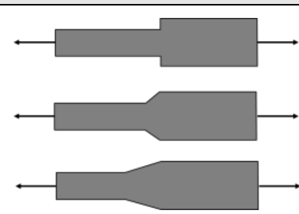
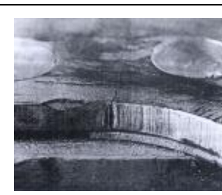
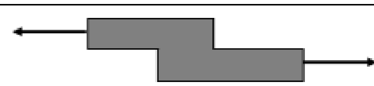
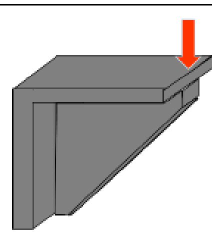
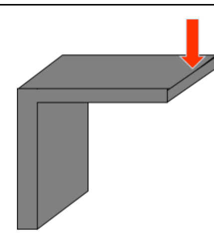
10.6 Fabrication and Detailing to Reduce Weld Faults

The fatigue life of welded structures may be improved through attention to detail in terms of:

- Load path continuity, as illustrated by the examples listed in Table 10.5.
- Weld location and loading;

- Selection and location of materials can affect structural fatigue life, and requires consideration of:
    - the loading direction relative to the direction of steel rolling;
    - implications of higher strength steel;
    - higher stress levels;
    - higher weld residual stress; and,
    - reduction in toughness in the heat affected zone.
- Fabrication particulars, which can affect structural fatigue life, including:
    - the ease of welding (working environment and adequate welder access);
    - fabrication tolerances and instructions; and,
    - weld sequencing to control residual stress and weld faults.
- Repair and modification approaches.

Table 10.5: Load Path Continuity

Type	Example
Abrupt cross section changes and sharp corners	
Rough surface finishes (e.g. flame cut edges that have not been ground flush)	
Eccentric loading	
Load path direction changes	 vs. 

10.7 Survey of Practice

Based on the results of a survey of typical shipyard welding practice, the effect of variability in workmanship is demonstrated. Understanding that the quality of typical fabrication practices follows a probability distribution, the effects on the fatigue life can be evaluated. Figure 10.2 illustrates a result of the survey indicating the normal distribution associated with the stress concentration factor due to the angle between the base plate and weld face.

Figure 10.3(a) illustrates the effect that the angle of the weld profile, θ, has on the fatigue life (given in terms of the number of cycles). As shown, as the angle approaches 90° (meaning the weld face becomes steeper) then the fatigue life decreases significantly. This effect is a result of the increased stresses that are associated with the concentrations occurring at the weld toe (at the notch). The importance of proper weld quality control for the fatigue life is demonstrated. Interpreting the probability distribution shown in Figure 10.2 with this data will provide an estimate for the probability of achieving the fatigue life for a detail.

Figure 10.3(a) illustrates the effects on the fatigue life associated with misalignment, arising in an eccentric in-plane loading condition. With increasing value of the eccentricity, the fatigue life decreases significantly. For example, the fatigue life is reduced by a factor of 10 when the eccentricity increases from 0.5 mm to approximately 5.0 mm. Similarly, the effect of angular misalignment, or weld shrinkage, on the fatigue life is illustrated in Figure 10.3(c). Again, the increasing eccentricity, which increases the bending stresses through the component, results in a decreased fatigue life.

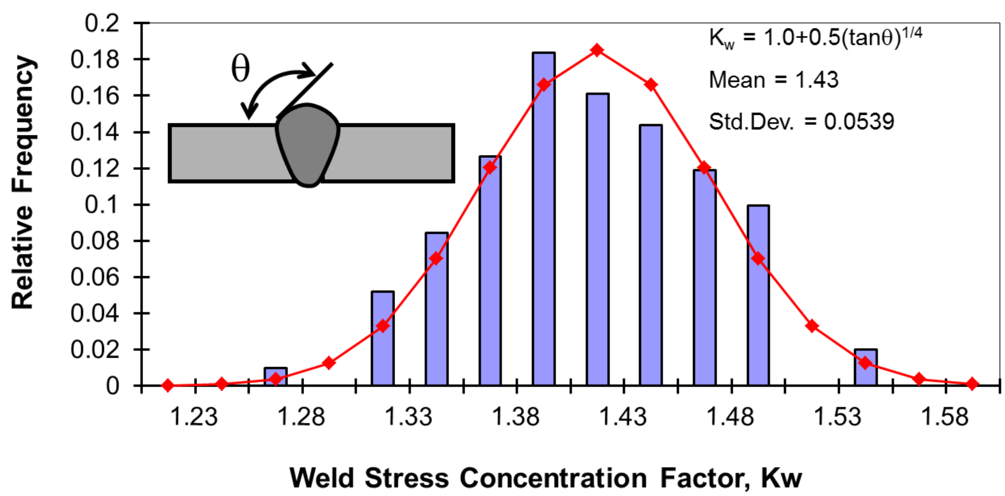
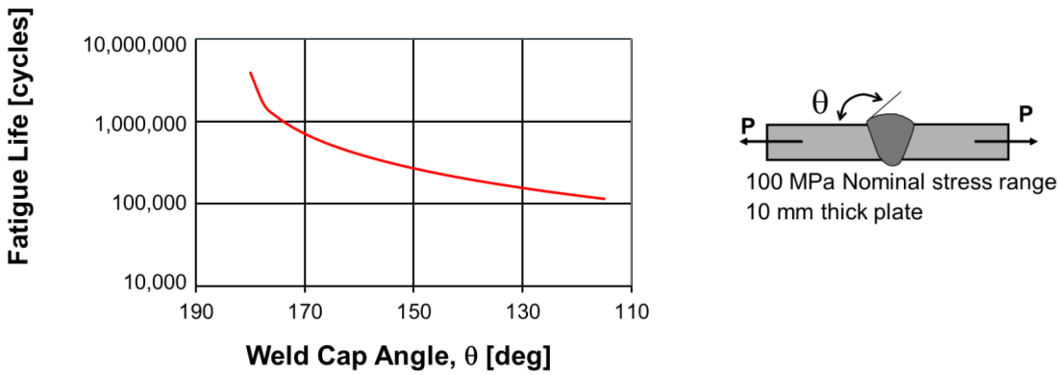
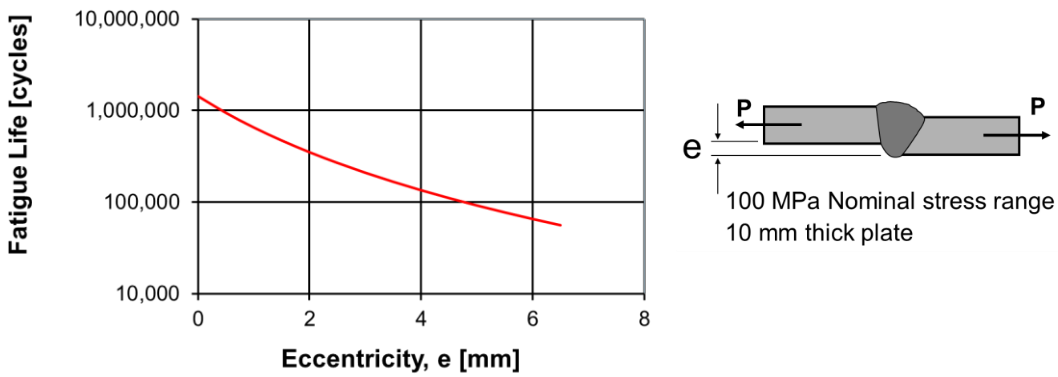


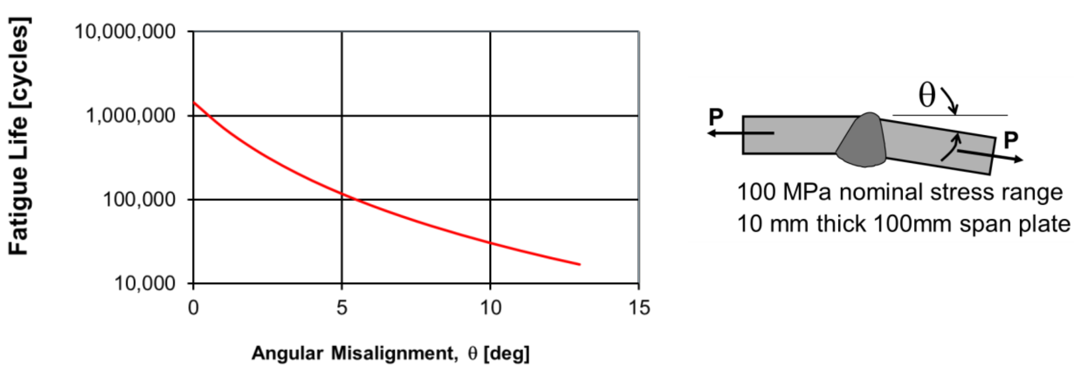
Figure 10.2: Observed Frequency of Weld Stress Concentration due to Angle between Base Plate and Weld Face



(a) Effect of weld profile



(b) Effect of Incorrect Fit-up or Misalignment



(c) Effect of angular misalignment or weld shrinkage

Figure 10.3: Effect of Workmanship

11 SUMMARY AND CONCLUSIONS

A review of the standards and guidelines relevant to the fatigue design of OWTG tower structures has been completed and the methodologies by which the estimation of fatigue life is completed are summarized. The relevant standards and guidelines are based on established principles of design, defining a comprehensive methodology for estimating and achieving the desired service life for fatigue. The design philosophies and the approaches taken (mainly, the limit states design approach) provide a consistent method by which a safe, reliable structural design can be achieved. The key aspects of each step in the fatigue assessment process are summarized, as follows.

11.1 Design Guidelines and Certification

The basic approaches applied in the relevant standards and guidelines include:

- The certification is based largely on monopile type installations, or other fixed-types, which are valid (or, feasible) only up to some maximum water depth. Floating types, which are more recently under development, reference offshore structures design standards, mainly the API and ISO guides for offshore platforms in the oil and gas industry.
- The limit states design approach (sometimes referred to as load and resistance factor design) is accepted increasingly as the preferred design philosophy, for which partial factors are applied to ensure that the probability of exceeding a defined limit state is maintained at an acceptably small value. The determination of the partial factors for design are based on probabilistic evaluations and design studies with consideration for the requirements of the structure to function throughout the design service life most appropriate for the class of structure and the prescribed safety index.
- In order that the service life of the OWTG asset is achieved, it is critical that the whole approach considers both the design philosophy and the inspection regime together. The inspection and maintenance schedule and the level of inspection (visual versus non-destructive examination) should be developed with consideration for a safe life, fail-safe or damage tolerant design philosophy. Subsequent life extension assessments can then be completed consistent with the original design approach to estimate residual fatigue life as the end of service life approaches.
- For the fatigue limit state, the partial factors for loads are uniformly defined in the standards and guidelines with a value equal to 1.0. Regarding the partial factors for calculated fatigue life, the standards and guidelines offer two approaches with the application of partial material factors or design fatigue factors (DFFs). The values for these latter factors are selected based on determinations regarding uncertainties associated with the characterization of the loads; the determination of the material and component resistances; and, accessibility for inspection and maintenance.
- With respect to the fatigue design of OWTG tower support structures, a safe life philosophy is applied generally. The development of the partial resistance factors and the limit states design approach are consistent with safe life, based on a prescribed service life for design. Components may be designed as damage tolerant, where appropriate, using fracture mechanics and a defined inspection program. For the

tower structures, a fail-safe approach is limited since the towers are not redundant structures. However, designs for support structures borrowed from the offshore oil and gas industry, including jackets and semi-submersibles, can be designed with the appropriate redundancies.

11.2 Conditions Associated with External Loads

The standards and guidelines define methods by which the loads are characterized based on the effects associated with the environment and operating conditions. These are considered in terms of the external conditions and include permanent, environmental, operational and other conditions.

- Typically, the fatigue loads are based on the effects of “normal” environmental conditions, corresponding to those having a probability of being exceeded at least once a year.
- In general, similar or equivalent methodologies are applied to describe the environmental conditions. Details associated with the application of different theories may arise, but the approaches taken effectively develop an analytical representation of the naturally occurring, random conditions.
- The action of the wind turbine operating within the environment creates significant aerodynamic load effects that must be considered in the evaluation and may require aero-elastic analyses.
- Regarding fixed structures, a dynamic analysis is necessary to ensure the natural frequency of the OWTG support structure does not approach the excitation frequency and its higher harmonics of functional loads or the frequency of energy-rich environmental loads.

11.3 Calculated Structural Response

Important considerations relevant to the modeling and simulation for fatigue design, and using the load cases associated with the fatigue limit state, are identified in the standards and guidelines and include:

- The external conditions, and the corresponding characteristic loads, are combined in accordance with the standards and guidelines to develop the design load cases for the fatigue limit state. The fatigue load cases as prescribed include the set of normal and fault design operating situations; and, the normal and extreme external conditions. Further, consideration is made for design situations associated with transportation, erection and maintenance.

- The modeling of welds, specifically at the weld toe regions as relevant to the evaluation of hot-spot stress ranges, requires special consideration. Guidance is provided in DNV-RP-C203 regarding the modeling and analysis by finite element methods of the stress distributions local at hot-spot regions.

- The response of the structure is most effectively considered using an integrated approach (which considers the aerodynamic and structural damping).

- For fixed type structures, consideration of the elastic behavior of the support structure is required. The behavior of the support structure and the natural frequencies will be influenced by the elastic foundation stiffness. Thus, these fixed type OWTGs have been categorized as soft-soft, soft-stiff or stiff-stiff based on the response. The transient behavior of the soil-stiffness interaction for the life of the structure may be an important design consideration. Changes in the soil-stiffness over time may affect the resonant frequencies of fixed structures.

- Stress concentrations associated with the geometry or other aspects:

    a. Including sufficient detail in finite element models so as to capture the localised effects;
    b. Applying stress concentration factors, as commonly used for tubular joints with Efthymiou parametric equations;
    c. S-N curves with implicit stress concentration factors.

- Structural analysis of OWTGs is a complex endeavor and, as for the design of offshore platforms in the oil and gas industry, analyses may be completed in the frequency domain. However, time domain solutions have become more reasonable due to advances in the software and computing power. Structural analysis in the time domain calculates directly the effects of fatigue loading resulting from the wind and wave combinations and is the preferred approach.

11.4 Characterization of Ice Loads for Fatigue

The methods and approaches are compared for evaluating dynamic ice loads and their contribution to the fatigue of an OWTG tower foundation using the existing standards and guidelines and with application of a test case.

- Following an evaluation of the different methodologies adopted for characterizing the ice loads, it was determined that the use of existing standards and guidelines is considered to be the preferred approach.

- Following a review of the three standards and guidelines (including, IEC 61400, DNV-OS-J101 and ISO 19906), it was determined that ISO 19906 provides a more complete treatment of dynamic ice loadings than do IEC 61400 or DNV-OS-J101. In order to apply ISO 19906, more inputs must be specified and more analyses must be carried out. Recognizing the uncertainties and issues in defining the ice input requirements, it is not clear whether or not this will lead to a more accurate assessment of dynamic ice loadings.

- It was concluded that the required ice inputs can be defined for both IEC 61400 and DNV-OS-J101, although significant investigation would be required for an engineering design project, including probably, the collection of site-specific data. It was concluded the availability of ice information requirements will not limit the application of IEC 61400 and DNV-OS-J101.
- Rafted ice events produced larger load reversals although there were fewer cycles. The sheet ice events produced lower load reversals with a larger number of cycles. This trend is considered to be realistic.
- The resulting S-N curve from IEC 61400 spanned a wide range due to the inclusion of both rafted ice and sheet ice. That for DNV-OS-J101 also indicated that the number of cycles was inversely related to the magnitude of the load reversal.
- The S-N curve for DNV-OS-J101 spanned a narrower range of load reversal magnitudes than did the one from IEC 61400. This is due to the fact that only certain cases for sheet ice needed to be included for DNV-OS-J101, as the others did not meet the tuning criterion. The variation is mainly due to differences with respect to rafted ice, these cases generated the highest ice loads and largest load reversals. Furthermore, DNV-OS-J101 indicated significantly lower load reversal magnitudes were associated with a given number of cycles than did IEC 61400. Again, this variation is principally due to the fact that the rafted ice cases did not meet the tuning criterion in DNV-OS-J101, and thus they were excluded.
- As an overall observation, the results demonstrated the significance of differences between the methodologies prescribed in IEC 61400 and DNV-OS-J101.

11.5 Stress Analysis

The calculated structural response and the design S-N curves are used to determine the load effects (or, stress history) used in the fatigue life estimation. The load effects will be defined in terms of a transient load history (from analysis in the time domain) or a power spectral density, PSD (from analysis in the frequency domain). For each detail under evaluation, the joint and weld profile may be correspond to prescribed detail classifications and specific design S-N curves. In such cases, the nominal stress approach may be adopted. If the joint or weld is not represented adequately, or if the component corresponds to a tubular structure, the hot-spot stress approach should be used. The notch stress approach is used less often and requires detailed analysis including representation of the weld notch.

- The nominal stress approach uses the calculated results from generalized structural models, in which welds and other connections are not represented in detail. The effects associated with weld profiles and other limited geometric effects are addressed by

  (i) application of the design (nominal) S-N curves; and,

  (ii) the application of geometric stress concentration factors.

The design S-N curves must correspond to those determined from testing on the basis of nominal stresses. A stress concentration factor may be calculated for a detail in the assessment to reflect the influence of holes, tapers, cutouts or other gross geometric influences not implicitly included in the determination of the nominal design S-N curve.

- The hot-spot stress approach applies to situations for which the local stress and the geometry of a connection does not correspond adequately to a detail classification, including connections having a welded attachment such that local stress concentrations will occur at the weld toe; or, arrangements of tubular joints, as for a jacket structure. Detailed finite element models are used to calculate the stress ranges at the local detail and the manner by which the hot-spot stress is extracted linearly from the adjacent reference stresses is defined in the guides for different weld geometries. The calculated stress ranges by the hot-spot stress approach must be used in conjunction with the corresponding design S-N curves in order that the desired safety level is achieved.
- Cycle counting is used develop the stress history of constant amplitude loadings resulting from the variable stress history and may apply the rainflow counting method or the reservoir method in the time-domain. Other methods of cycle counting in the frequency domain may be used and include a formulations by Rayleigh, Rice and Dirlik.
- The design S-N curves and the material considerations are applied in the damage accumulation calculations for the fatigue life assessment as follows:
    - The majority of the S-N curves applied for design correspond to component S-N curves, which have been developed from the results of experimental tests for specific components and welded connections. Therefore, the effects of weld residual stresses and stress concentrations are included in the design curves.
    - Some standards and guidance documents apply material S-N curves, which have been determined only from tests using base materials. This approach is non-conservative and stress concentration factors must be applied subsequently to account for the effects of weld residual stresses. Therefore, when applying an S-N curve for design it is important to understand the basis on which the characteristic curve has been derived.
    - In addition to the different factors affecting the classification of design S-N curves, the calculated stress ranges may need to be adjusted to reflect the effects on the number of cycles with consideration of material thickness (or size) effects; mean stress effects; and, the effects of any surface treatments applied.

11.6 Fatigue Life Estimation

With the response history defined, the estimation of fatigue life by the method of stress-life, strain-life or fracture mechanics can be completed.

- A Miner-Palmgren linear damage summation is used typically and, together with the corresponding design S-N curves, the fatigue life is estimated. Calculated fatigue lives represent conservative design estimates and should be considered as indicating relative performance, rather than being relied upon as an absolute life.
- A stress-life approach is used to calculate the design cumulative damage as determined from the number of cycles for each interval of applied stress range, taken from the stress range histogram, and then compared to the permissible number of cycles for that applied stress range.
- A fracture mechanics approach to damage tolerance estimation is based on the assumption of a pre-existing crack (of some dimension), which then propagates through the material thickness. A crack growth rate can be characterized and the structure or component can be designed with a fail-safe approach such that the performance can be monitored between inspection intervals.
- Since the stress-life method assumes elastic behavior, the true nature of fatigue crack initiation, which is caused by plastic deformation, is ignored. This simplifying assumption is reasonable as long as the design of the structure or component is made such that elastic behavior is maintained.
- Design fatigue factors (DFFs) are provided in the standards and guidelines to modify the calculated fatigue life for individual structural details to account for uncertainties in the fatigue assessment and to further account for the consequences of failure. Effectively, the DFF (or, FDF) modifies the calculated fatigue life for each individual structural detail to account for uncertainties in the fatigue assessment and to further account for the consequences of failure. The overall calculated fatigue life for the OWTG is therefore given by the limiting fatigue critical detail.
- Alternatively, DNV-OS-J101 and GL define a material factor for fatigue. In this case, the applied stress range used to determine the number of endured stress cycles is modified.
- The application of either the DFF or the partial material factor must be made consistent with the design philosophy applied in the standard or guideline to ensure the fatigue service life is achieved. The level of conservatism introduced reflects the level of safety inherent in the design philosophy, which takes into consideration the local site requirements, safety factors, construction quality and inspection program.
- The fatigue performance of an OWTG tower structure can be improved over the service life by employing good design practice, including selecting the appropriate details for design (considering plate thickness, stress concentrations, weld details, etc…) and by specifying the appropriate fabrication procedures (i.e. post-welding treatments like grinding). A comprehensive inspection and maintenance program is required also to ensure performance for the service life.

11.7 Stress Analysis Demonstration

A fatigue and fracture assessment may be completed in any of several ways. The various analytical techniques, corresponding to nominal stress; hot-spot stress and notch stress, and the basis on which each is undertaken, are described.

- The application of the nominal stress approach is straightforward, being based on simple beam theory and adoption of a hull-girder analogy. Simplified finite elements have been developed to facilitate stress analysis by idealizing a stiffened structure using plates or beams that account for the contribution of the stiffeners in the section properties. Otherwise, fairly coarse mesh finite element modeling can be used to develop the nominal stresses.
- For the hot-spot stress approach, parametric stress concentration factors (SCFs) have been developed for typical structural details. Alternatively, fine mesh finite element models may be used to derive the SCFs, with application of the appropriate extrapolation techniques.
- In a notch stress fatigue initiation analysis the hot-spot stress is modified with the application of a notch stress concentration factor, determined analytically or numerically. For the latter, extremely fine mesh finite element models are required to model the crack tip singularity.

11.8 Environmental Effects on Fatigue

The fatigue performance and the design for fatigue is influenced significantly by corrosion. Corrosion and the fatigue of corroding structures as compared to corrosion fatigue are discussed.

- In the stress life (S-N) approach to fatigue life assessment, calculations are based on the net scantling thickness to ensure that the highest applied stress will be used during the fatigue assessment. However, this simplified approach does not address the effects of a corrosive environment with respect to crack initiation or crack propagation. Since a corrosive environment is generally detrimental to both crack initiation and crack propagation, this approach may be un-conservative when estimating the fatigue life of a component.
- Corrosion fatigue refers to the effects of the interaction between cyclic mechanical loading and a corrosive environment. The effects of corrosion fatigue can manifest themselves in both the crack initiation and the crack propagation stages of crack growth. The interaction between the corrosive environment and the cyclic mechanical loading generally results in a significant increase in the rate of crack growth, as compared to that in a non-corrosive environment, which further reduces the overall fatigue life.

11.9 Effect of Fabrication Quality on Fatigue

The quality of fabrication involved in welded structures and connections has a significant effect on the reliability of the structure with respect to achieving the design service life. The effects of weld faults and detailing practices are described.

- Dimensional weld faults can be classified in terms those that may occur prior to or after welding is completed. Both types are related to poor workmanship and indicate a lack of quality control. Prior to welding, edge preparation and fit-up are important considerations. Following (or during) welding, weld defects must be avoided, including undercut, concavity, overlap, etc. Further, plate distortion needs to be controlled using the appropriate weld procedures and restraint.
- Poor weld quality and control is also manifested in structural weld faults, which may include porosity; slag or other inclusions; lack of fusion; incomplete penetration; and, cracking. Suitable welder training and weld procedures are a requirement. Surface weld faults may also occur, at or away from the weld area. These types of faults include spatter and arc strikes, any of which will reduce the fatigue life of a cyclically loaded structure.
- The results of a survey of shipyard welding practice demonstrated the probabilistic nature of the workmanship. Illustrated for the example of the weld profile angle, the variability is shown to follow a normal distribution. Therefore, the effect of the weld profile in practice deviating from the design value will similarly be probabilistic. The reliability of the welded detail to achieve the design fatigue life may be evaluated with an assessment of the probability of achieving a defined weld quality.

11.10 Conclusions

Overall, this project has evaluated the different standards and guidelines available for the design and certification of OWTGs and has concluded that fatigue design methodologies presented are suitable. Each standard and guideline reviewed provides the direction required to characterize the environmental conditions from which the loads are developed; the methods by which the stress analysis is completed; a stress life method to fatigue life estimation (with allowance for a fracture mechanics approach); and, guidance for proper weld detailing and design for fatigue. In this respect, the standards and guidelines are equivalent in general.

The detailed characterization of ice loads may be an area for which additional research and development is required. The review indicated that different approaches have been described by IEC 61400, DNV-OS-J101 and ISO 19906, and that the results can be significant.

The application of either design fatigue factors (DFFs) or partial material safety factors represent another issue which distinguishes the different standards and guidelines. The fatigue life calculated in accordance with the different requirements will produce different results. Further, the values assigned for the DFFs may be open to interpretation depending on the assumed levels of the consequence of failure and criticality.

12 REFERENCES

12.1 Bibliography

Al-Bahadly, Ibrahim, ed. *Wind Turbines.* Rijeka, Croatia: InTech, 2011.

Andersen, Ulrik Vølcker. *Load Reduction of Support Structures.* Masters Thesis, Technical University of Denmark, 2008.

Bannantine, Julie A., Jess J. Comer, and James L. Handrock. *Fundamentals of Metal Fatigue Analysis.* Englewood Cliffs, New Jersey: Prentice-Hall Inc., 1990.

Bhat, S., and Cox. "Ice Loads on Multi-Legged Structures in Cook Inlet." POAC Conference, 1995.

Blenkarn, K. "Measurement and Analysis of Ice Forces on Cook Inlet Structures." *OTC Paper 1261.* Houston, TX: Offshore Technology Conference, 1970.

Brower, W., R. Baldwin, C. Williams, J. Wise, and L. Leslie. *Climatic Atlas of the Outer Continental Shelf Waters and Coastal Regions of Alaska: Vol. I – Gulf of Alaska, Vol. II – Bering Sea; and Vol. III – Chukchi-Beaufort Sea.* Anchorage, Alaska, 1988.

CIS. *MANICE Manual – Manual of Standard Procedures for Observing and Reporting Ice Conditions.* Canadian Ice Service (www.ec.gc.ca), 2005.

Dalhoff, Peter, Kimon Argyriadis, and Marcus Klose. *Load and Strength Analysis for Offshore Wind Turbines with Jacket Structures.* Hamburg, Germany: Germanischer Lloyd (GL) Industrial Services GmbH, Business Segment Wind Energy, 2007.

Eranti, E., H. Pukkila, E. Lehtonen, and L. Rantala. "A Novel Offshore Windmill Foundation for Heavy Ice Conditions." *OMAE.* Rotterdam, Netherlands, 2011.

Gangloff, R.P. "Corrosion Fatigue Cracking." In *Corrosion Tests and Standards: Application and Interpretation, 2nd Edition*, by ed. R. Baboian, pp. 302-321. ASTM International, 2005.

Gerven, Frank van. *Optimising the Design of a Steel Substructure for Offshore Wind Turbines in Deeper Waters.* MSc Thesis, Delft, The Netherlands: Delft University of Technology, 2011.

Glen, I.F., A. Dinovitzer, R.B. Paterson, L. Luznik, and C. Bayley. *Fatigue-Resistant Detail Design Guide for Ship Structures.* Washington, DC: Ship Structure Committee, 1999.

Henderson, A.R., and M.B. Zaaijer. "Hydrodynamic Loading on Offshore Wind Turbines (Paper No. 2004-ARH-01)." *ISOPE Conference.* Toulon, France, 2004.

Jeffries, M., T. Karna, and S. Loset. "Field Data on the Magnification of Ice Loads on Vertical Structures." Vancouver: IAHR Ice Symposium, 2008.

Karna, T., H. Gravesen, L. Fransson, and S. Loset. "Simulation of Multi-Modal Vibrations due to Ice Actions." *Proc. IAHR Ice Symposium.* Lahti, Finland, 2010.

Kühn, Martin Johannes. *Dynamics and Design Optimisation of Offshore Wind Energy Conversion Systems.* PhD Thesis, Delft, The Netherlands: Delft University Wind energy research institute (DUWind), 2001.

Mulherin, N., T. Tucker, O. Smith, and W. Lee. "Marine Ice Atlas for Cook Inlet, Alaska." *CRREL report TR-01-10.* US Army Corps of Engineers, 2001.

Musial, W., and S. Butterfield. "Energy from Offshore Wind." Houston, TX: OTC Conference, 2006.

Nelson, W. "Sea Ice Formation in Cook Inlet, Alaska: A High Energy Environment." *Proc. OMAE Conference.* Copenhagen, Denmark, 1995.

Pao, P.S. *Mechanisms of Corrosion Fatigue, ASM Handbook Volume 19, Fatigue And Fracture.* ASM International, 1996.

Paris, P.C., and F. Erdogan. "A Critical Analysis of Crack Propagation Laws", ." *Trans. ASME, J. Basic Eng., Vol. D85*, 1963: pp. 528-534.

Ralston, T.D. "Ice Force Design Considerations for Conical Offshore Structures." *Proc.4th Int. POAC Conf.* St. Johns, Newfoundland, 1977. vol.2, pp.741-752.

Rogers, N. "Structural Dynamics of Offshore Wind Turbines subject to Extreme Wave Loading." *Proceedings of the 20th BWEA Annual Conference.* UK, 1998. 8.

Semiga, V., and S. Tiku. *Evaluation of the Difference between Corrosion Fatigue and Fatigue of Corroding Structures.* Report for American Bureau of Shipping, BMT Project 5959, April 2007.

Suresh, S. *Fatigue of Materials, Second Edition.* Cambridge University Press, 1998.

Tempel, Jan Van Der. *Design of Support Structures for Offshore Wind Turbines.* PhD Thesis, Delft, The Netherlands: Delft University Wind energy research institute (DUWind), 2006.

Visser, R. "A Retrospective of Platform Development in Cook Inlet Alaska." Houston, TX: OTC Conference, 1992.

Whitney, J. *HAZMAT Report 2003-01.* Anchorage, Alaska: National Ocean Services, Office of Response and Restoration - Hazardous Materials Response and Assessment Division, 2002.

Wizelius, T, and Frykberg. "Understanding Impacts on Great Lakes Fishery and other Aquatic Resources." *Workshop held by the GLERL on Offshore Wind Energy.* 2012. http://www.glc.org/energy/wind/fishimpact/pdf/CaseStudy1-LakeVanern-AnnArbor-121128.pdf.

12.2 Offshore Wind Turbine Standards and Guidelines

1. Det Norske Veritas (DNV), "Design of Offshore Wind Turbine Structures". DNV Standard DNV-OS-J101 (September 2011).

2. Det Norske Veritas (DNV), "Offshore Substations for Wind Farms". Offshore Standard DNV-OS-J201" (October 2009).

3. Det Norske Veritas (DNV) and Risø National Laboratory, "Guidelines for Design of Wind Turbines", Second Edition. Denmark, 2002.

4. Det Norske Veritas (DNV), "Standard for Classification of Wind Turbine Installation Units". DNV Standard DNV-OS-J301 (April 2011).

5. International Electrotechnical Commission (IEC), "Wind turbines - Part 1: Design Requirements". IEC 61400-1 ed.3.0 (2005-08-31).

6. International Electrotechnical Commission (IEC), "Amendment 1 - Wind turbines - Part 1: Design requirements". IEC 61400-1-am1 ed.3.0 (2010-10-13).

7. International Electrotechnical Commission (IEC), "Wind turbines - Part 3: Design Requirements for Offshore Wind Turbines". IEC 61400-3 ed.1.0 (2009-02-11).

8. Germanischer Lloyd (GL) WindEnergie, Rules and Guidelines – Industrial Services "Guideline for the Certification of Offshore Wind Turbines" (2005).

9. American Bureau of Shipping (ABS), "Guide for Building and Classing: Offshore Wind Turbine Installations". December 2010.

10. American Bureau of Shipping (ABS), "Design Standards for Offshore Wind Farms". Submitted to U.S. Department of the Interior Bureau of Ocean Energy Management, Regulation and Enforcement, September 2011.

11. American Bureau of Shipping (ABS), "Floating Wind Turbines". May 2012.

12. American Bureau of Shipping (ABS), "Guide for Building and Classing: Bottom-Founded Offshore Wind Turbine Installations". January 2013.

12.3 Design of Offshore Structures

1. American Bureau of Shipping (ABS), "Guide for the Fatigue Assessment of Offshore Structures". ABS-115. April 2003 (updated November 2010).

2. American Bureau of Shipping (ABS), "Guidance Notes on Spectral-Based Fatigue Analysis for Floating Offshore Structures". ABS-104. March 2005.

3. American Petroleum Institute (API), "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms (API RP-2A) – Working Stress Design".

4. American Petroleum Institute (API), "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms (API RP-2A) – LRFD Design".

5. American Petroleum Institute (API), "Recommended Practice for Planning, Designing and Constructing Structures and Pipelines for Arctic Conditions (API RP-2N)". Second Edition, December 1995.

6. American Petroleum Institute (API), "Interim Guidance on Hurricane Conditions in the Gulf of Mexico (API Bulletin 2INT-MET)". May 2007.

7. Det Norske Veritas (DNV), "Design of Offshore Steel Structures, General (LRFD Method)". Offshore Standard DNV-OS-C101 (April 2011).

8. Det Norske Veritas (DNV), "Fatigue Design of Offshore Steel Structures". DNV Recommended Practice DNV-RP-C203. October 2011.

9. International Organization for Standardization, ISO 4354:2009 "Wind Actions on Structures".

10. International Organization for Standardization, ISO 19900:2002 "Petroleum and natural gas industries -- General requirements for offshore structures".

11. International Organization for Standardization, ISO 19901 "Petroleum and natural gas industries -- Specific requirements for offshore structures".

12. International Organization for Standardization, ISO 19902:2007 "Petroleum and natural gas industries -- Fixed steel offshore structures".

13. International Organization for Standardization, ISO 19903:2006 "Petroleum and natural gas industries -- Fixed concrete offshore structures".

14. International Organization for Standardization, ISO 19904:2006 "Petroleum and natural gas industries -- Floating offshore structures -- Part 1: Monohulls, semi-submersibles and spars".

15. International Organization for Standardization, ISO 19905:2012 "Petroleum and natural gas industries -- Site-specific assessment of mobile offshore units -- Part 1: Jack-ups".

16. International Organization for Standardization, ISO 19906:2012 "Petroleum and natural gas industries -- Arctic offshore structures".

17. International Organization for Standardization, ISO 2394:1998 "General principles on reliability for structures".

18. Standards Norway (NORSOK), "Integrity of Offshore Structures". NORSOK Standard N-001, Edition 8. September 2012.

19. Standards Norway (NORSOK), "Actions and Action Effects". NORSOK Standard N-003, Edition 2. September 2007.

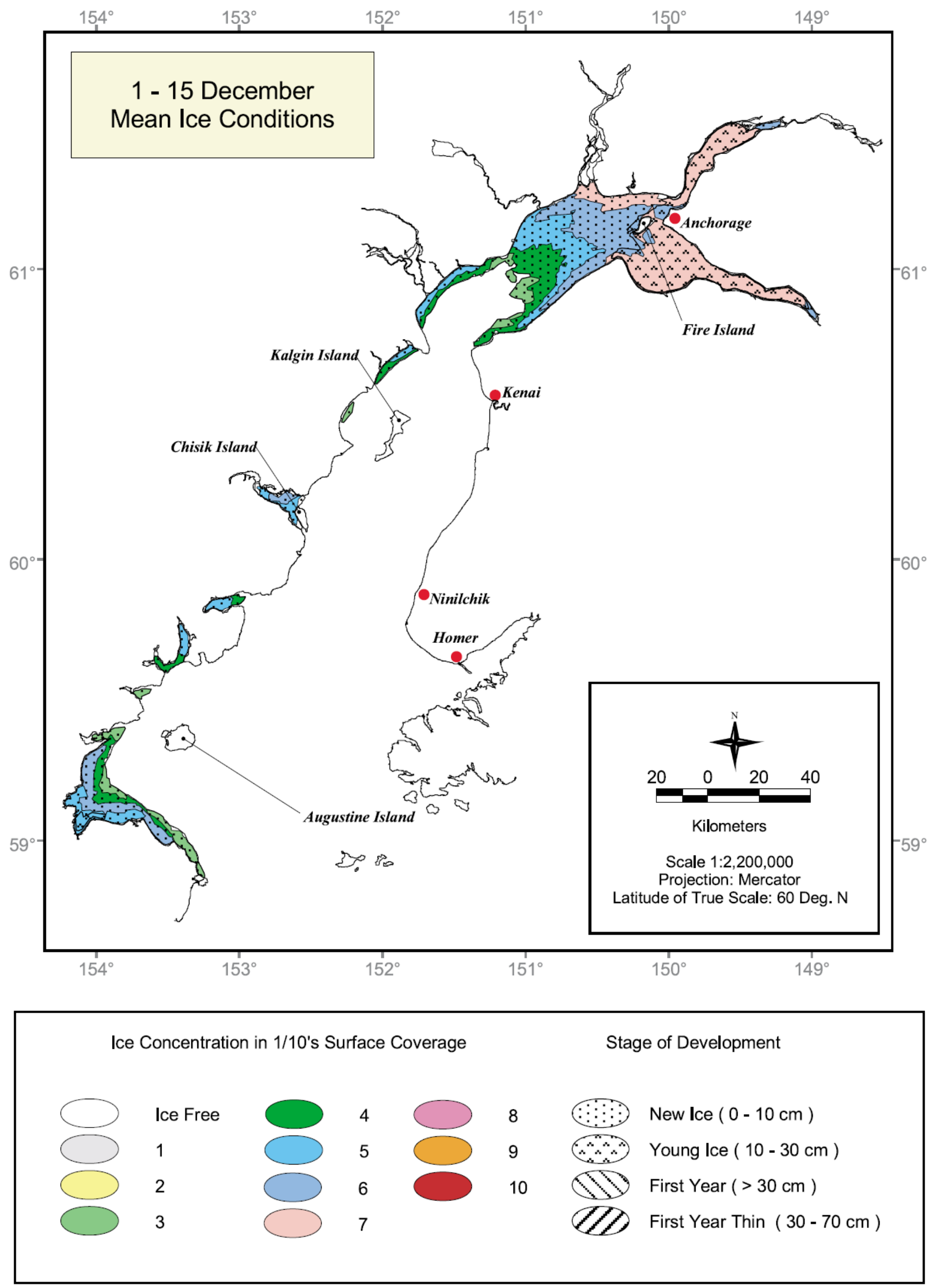
20. Standards Norway (NORSOK), "Design of Steel Structures". NORSOK Standard N-004, Rev. 2. October 2004.

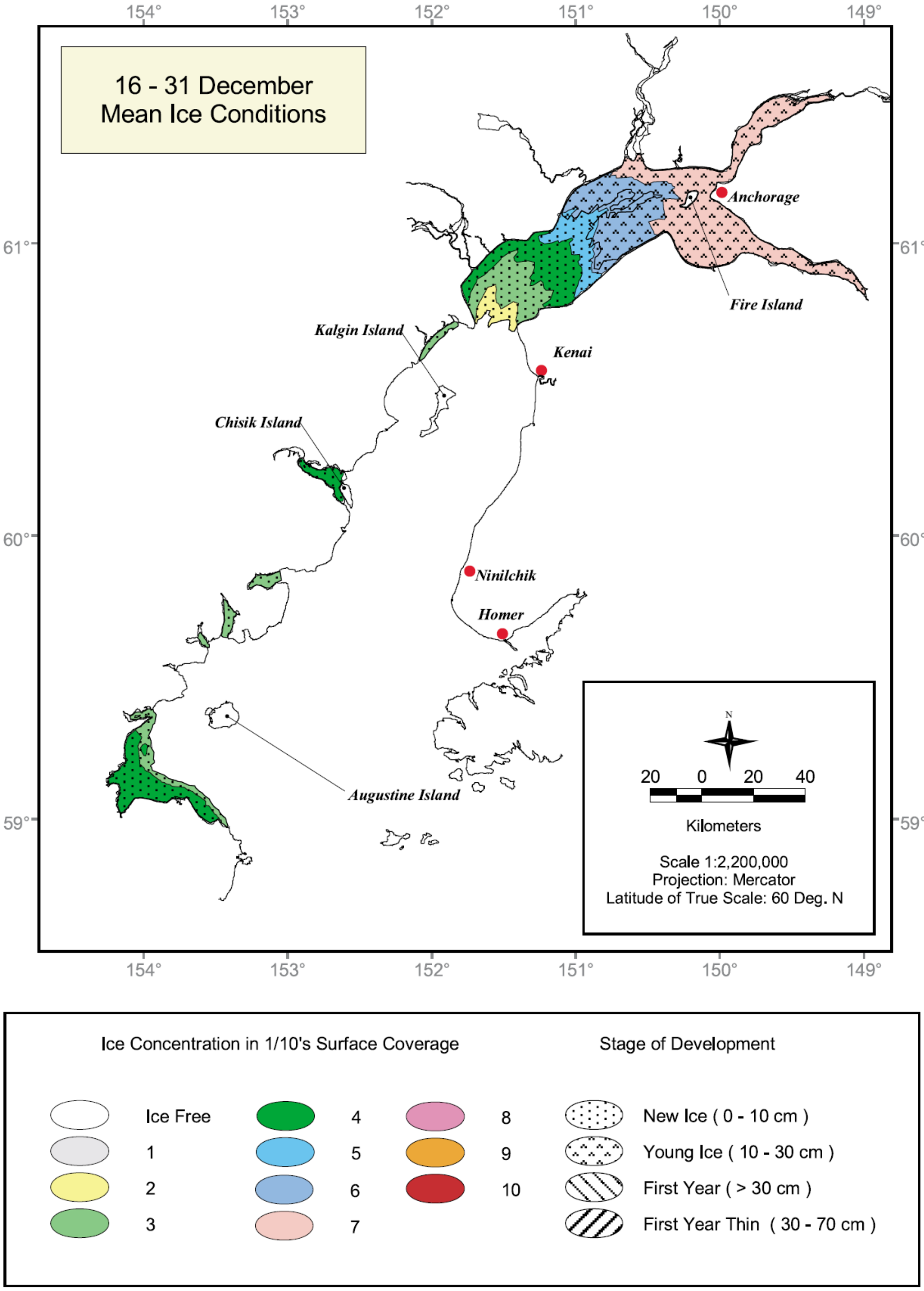
21. Standards Norway (NORSOK), "Condition Monitoring of Loadbearing Structures". NORSOK Standard N-005, Rev. 1. December 1997.

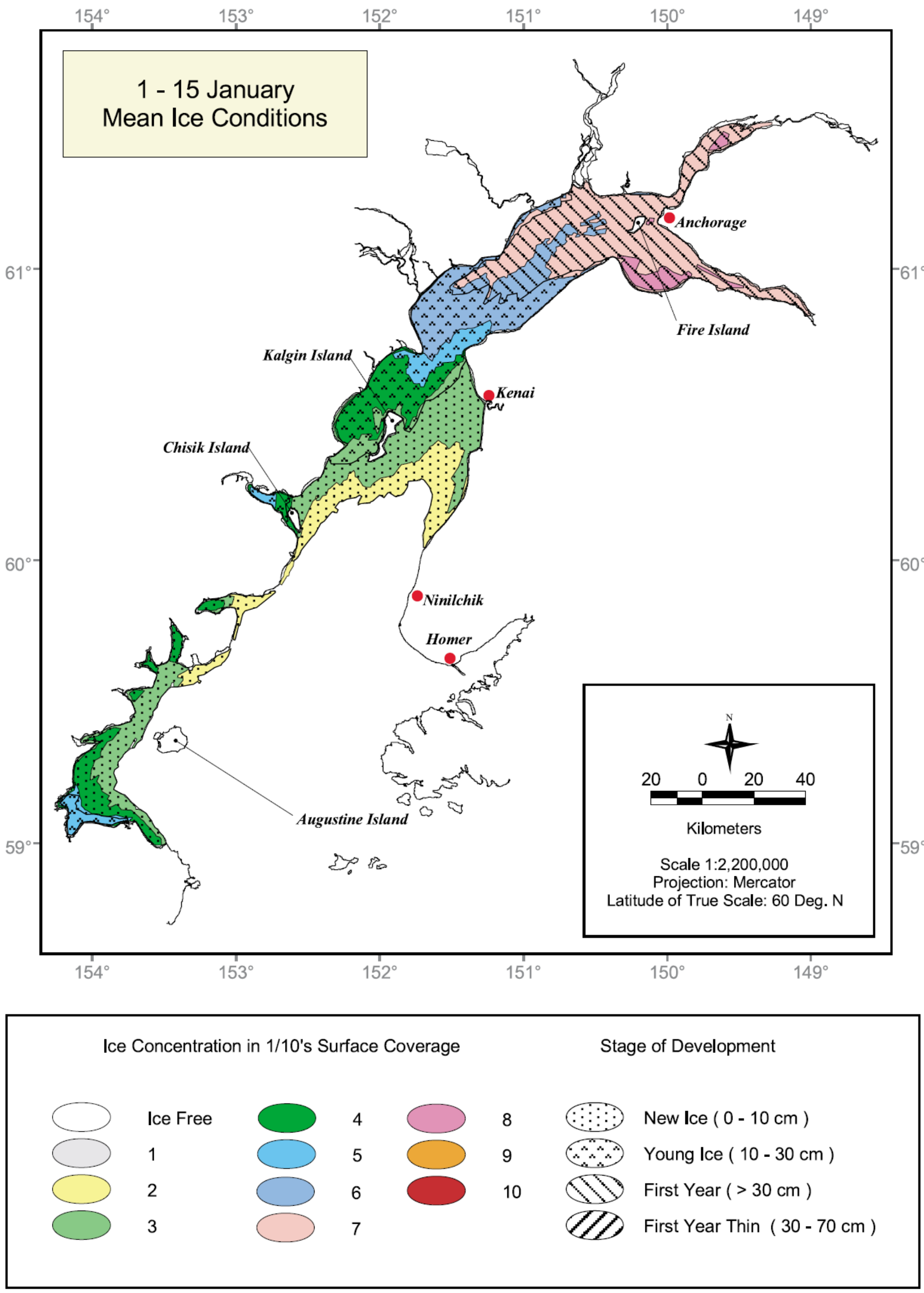
22. Standards Norway (NORSOK), "Assessment of structural integrity for existing offshore load-bearing structures". NORSOK Standard N-006, Edition 1. March 2009.

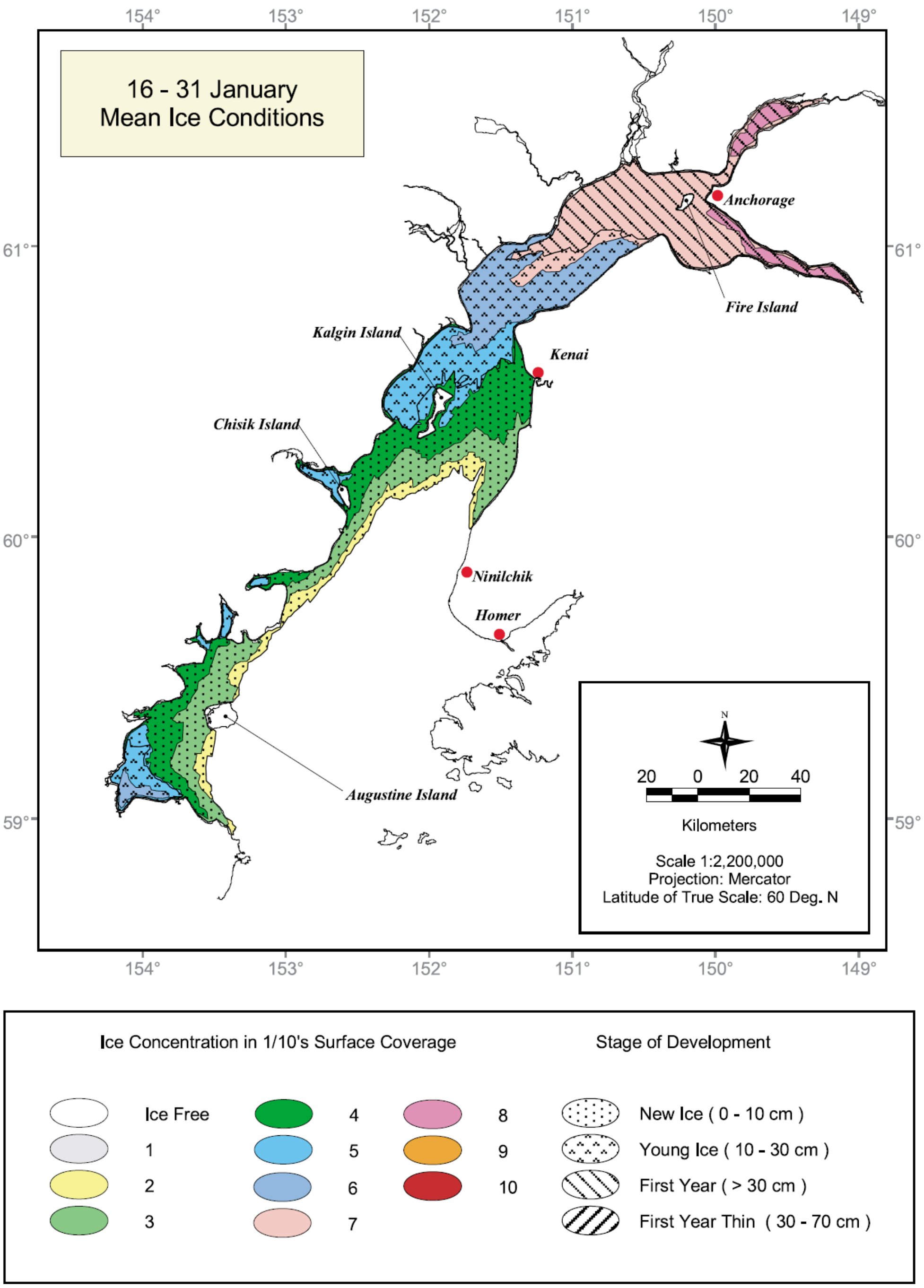
ANNEX A
MEAN ICE COVER IN COOK INLET BY CONCENTRATION AND STAGE OF DEVELOPMENT AFTER (MULHERIN, ET AL. 2001)

A.1 MEAN ICE COVER IN COOK INLET BY CONCENTRATION AND STAGE OF DEVELOPMENT

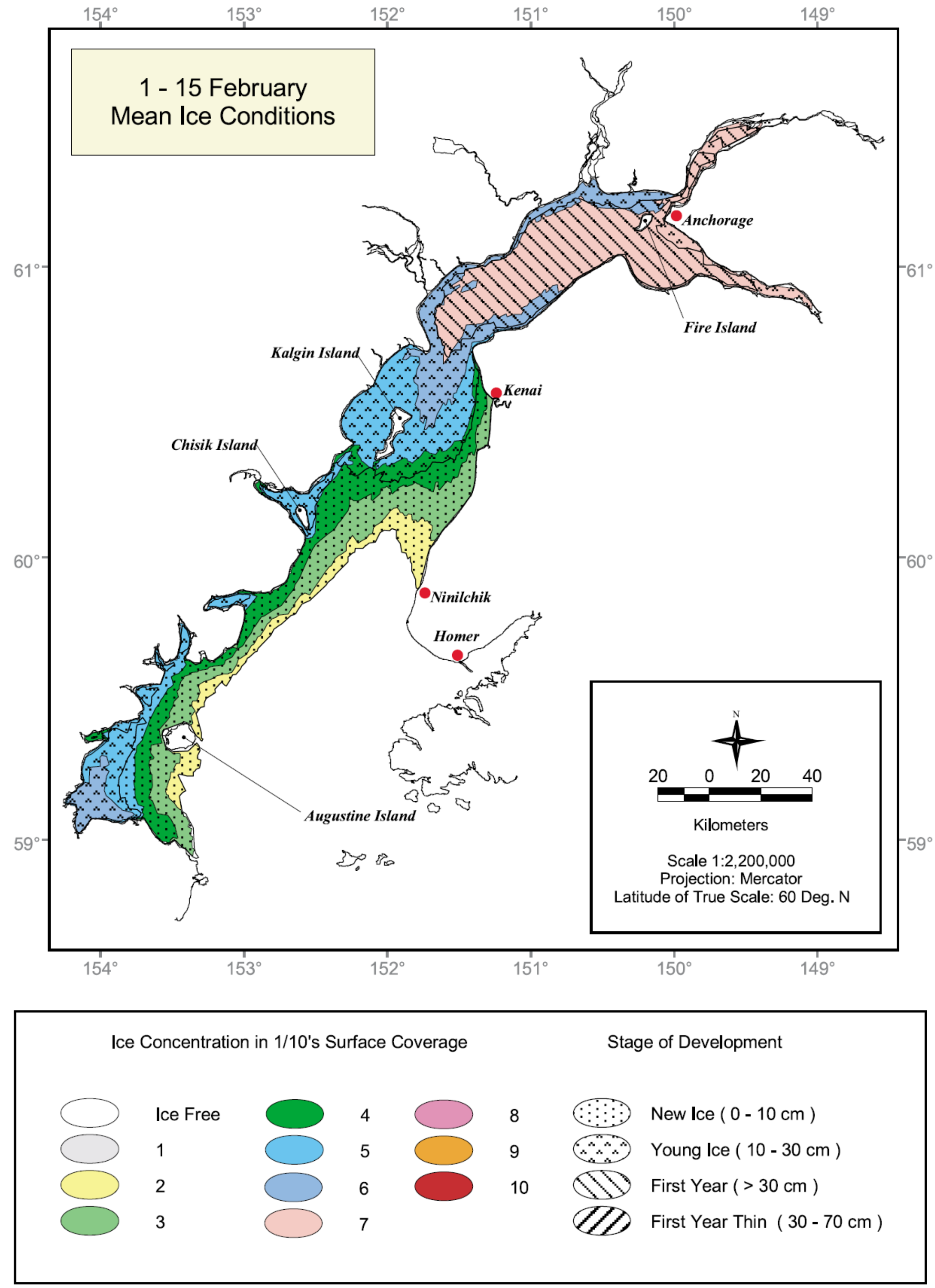


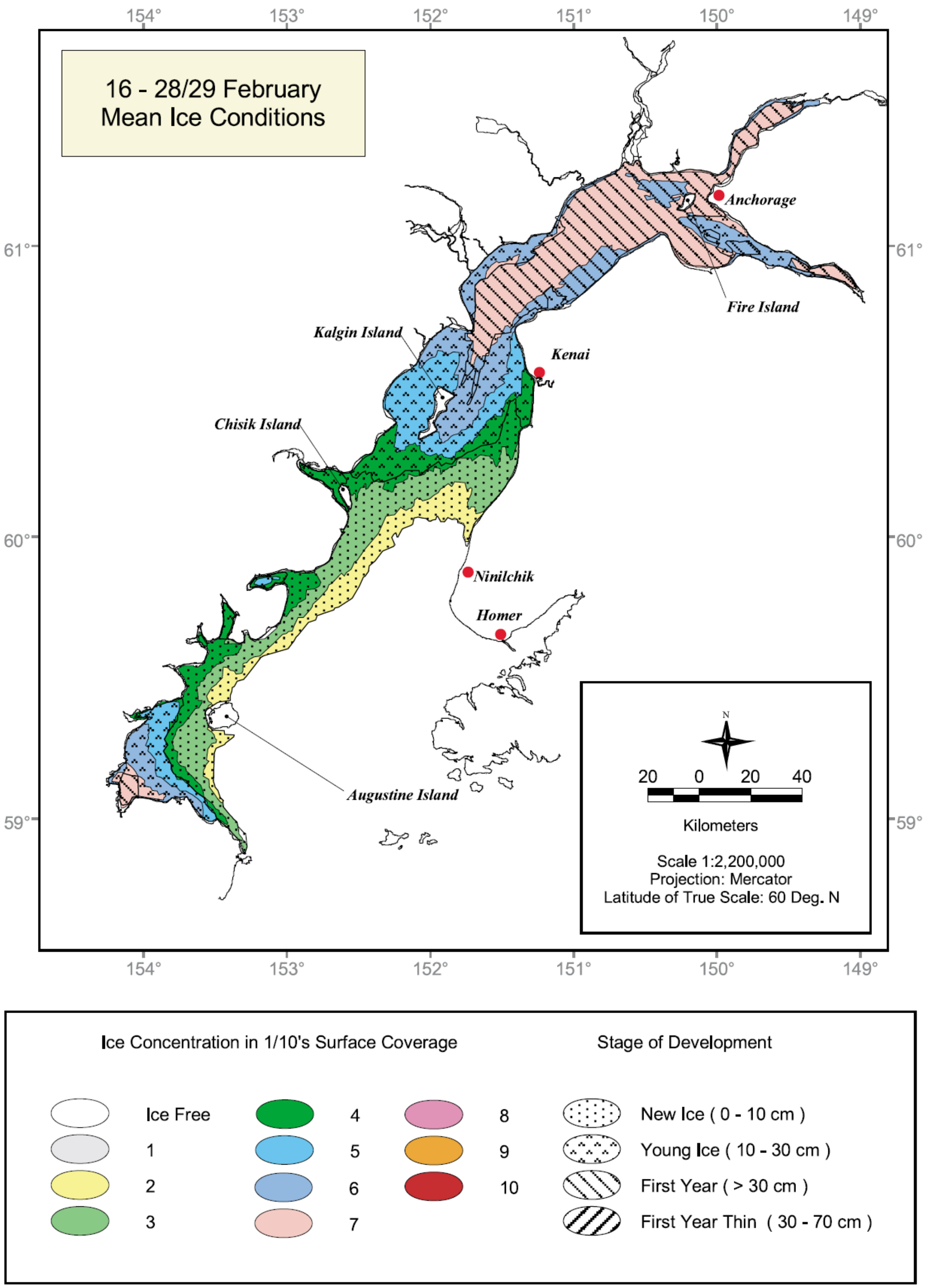

16 - 31 December
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)

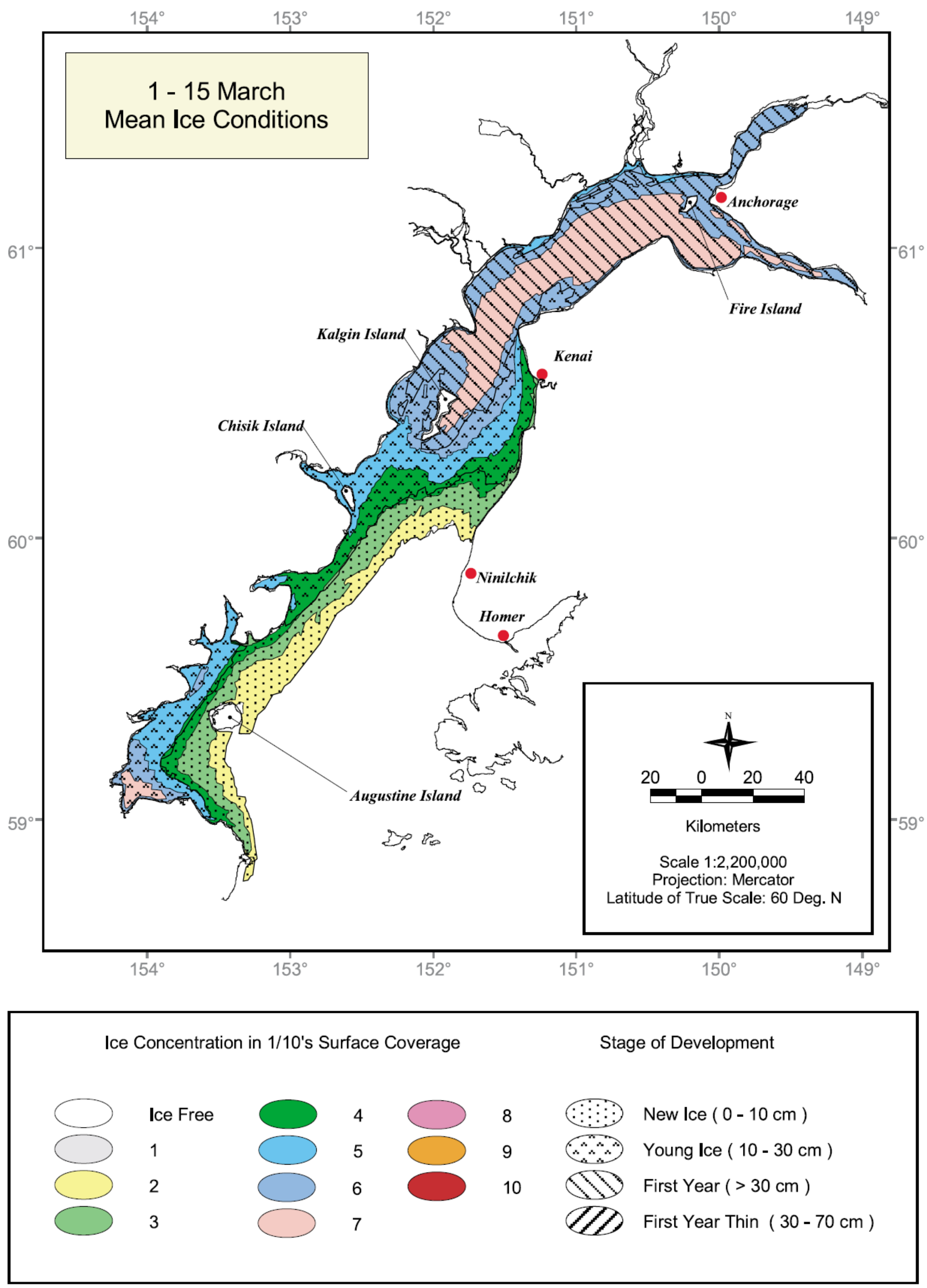

1 - 15 January
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)

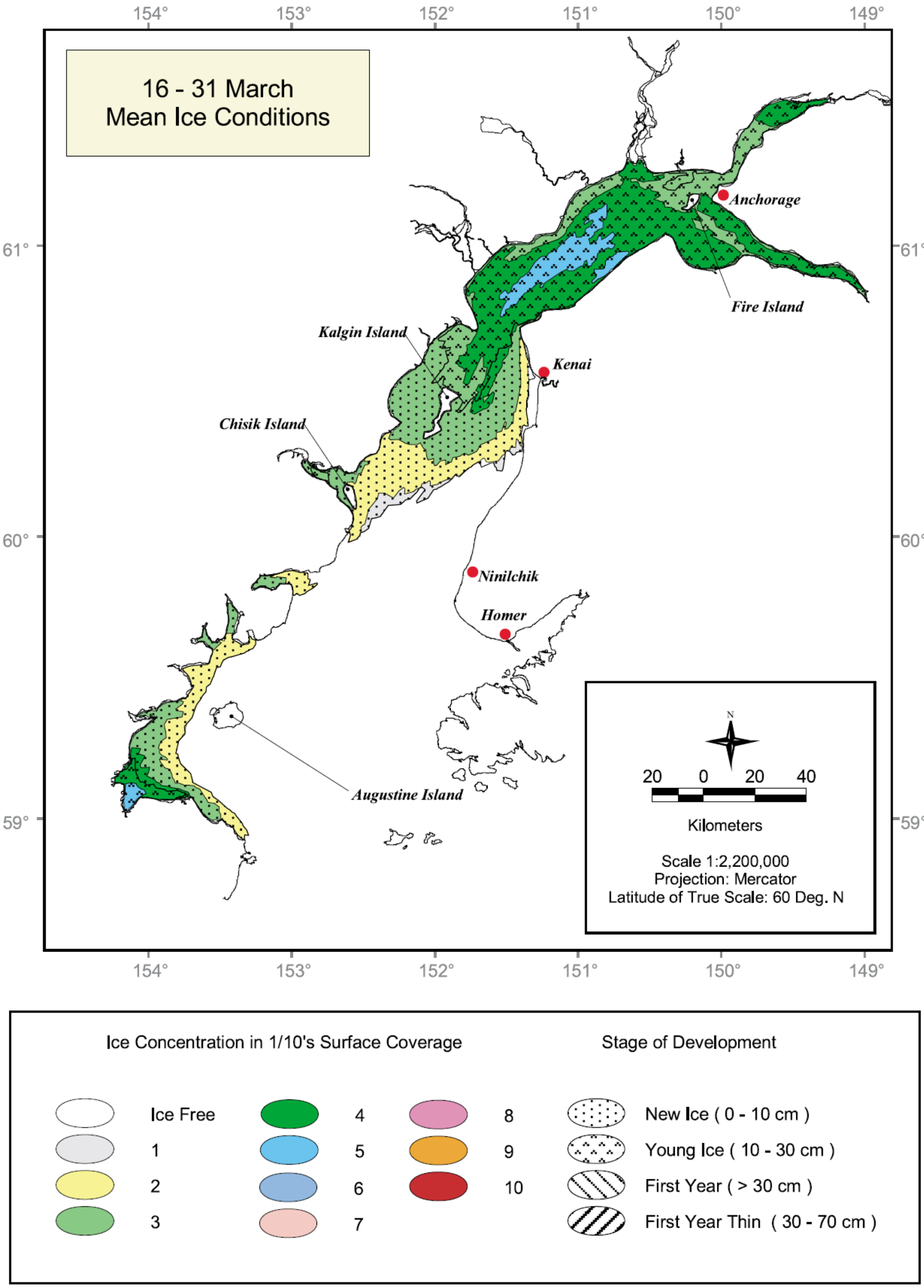


16 - 31 January
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)


1 - 15 February
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)


16 - 28/29 February
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)


1 - 15 March
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)



16 - 31 March
Mean Ice Conditions
154°
153°
152°
151°
150°
149°
61°
60°
59°
Anchorage
Fire Island
Kalgin Island
Kenai
Chisik Island
Ninilchik
Homer
Augustine Island
N
20 0 20 40
Kilometers
Scale 1:2,200,000
Projection: Mercator
Latitude of True Scale: 60 Deg. N
Ice Concentration in 1/10's Surface Coverage
Ice Free
1
2
3
4
5
6
7
8
9
10
Stage of Development
New Ice (0 - 10 cm)
Young Ice (10 - 30 cm)
First Year (> 30 cm)
First Year Thin (30 - 70 cm)

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Head Office - Ottawa
311 Legget Drive
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611 Alexander Street, Suite 412
Vancouver, BC, Canada, V6A 1E1
Tel: 604-253-0955

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